

# Appendix P. 2015 Hydrology Master Plan and Preliminary WQMP

## Appendix

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**MILL CREEK SPA HYDROLOGY MASTER PLAN**  
**CITY OF CHINO**  
**CHINO AGRICULTURAL PRESERVE AREA**

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March 12, 2015

**Prepared under the Supervision of:**

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**Date**

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## ***INTRODUCTION***

The Study Area is comprised of an approximately 273-acre area, bounded by Chino Corona Road on the north, Cucamonga Avenue on the west, Prado Reservoir to the south, and Mill Creek to the east. The site will be re-graded to create building pads for future single family residential home sites. The proposed Mill Creek Farming SPA is proposed to be graded in a manner so as to “raise” the development portion of the study area above the Prado Flood Control Basin’s “High Water Elevation” of 566.

The purpose of this report is to update the hydrology from a previous report prepared by Huitt-Zollars in January of 2008. This report examines the preliminary hydrology for the proposed development and will be used for planning purposes prior to final site design. Preliminary alignments, and sizes for backbone storm facilities are recommended herein for planning purposes. We have included preliminary storm drain hydraulics using WSPG for the backbone storm drains lines.

The results compare the runoff of the built-out condition of the study area based on the Chino General Plan and the Mill Creek Farming SPA development and the runoff of the existing undeveloped condition of the developed area of the site. This site will not be accepting any offsite flows from neighboring developments to the north as was previously planned. All offsite flows from the north are now directed to water quality basins east of the site.

## ***METHODOLOGY***

The hydrology calculations presented in this study are based upon the San Bernardino County Hydrology Manual. Subareas were determined based on the proposed grading of the site. A link-node model was created for each subarea, with flow path length and elevations shown for the upstream and downstream nodes for the subarea. Peak 100-year flowrates were determined for each subarea using the CivilDesign Corporation rational method hydrology software. The results of those calculations are shown on the site hydrology map included with this report. The developed and existing condition hydrology maps are included with this report.

Rational method hydrology calculations have been prepared for the development. Soil Type “C” was used in the models for the existing and proposed conditions. A saturated soil condition was assumed for 100 year calculations. The models also use a one hour intensity of 1.22” for the 100-year event and 0.5” for the 2 year event.

The existing condition is an undeveloped area with average to poor cover. It is made of 4 main basins within the site boundary. Basins E1, E2, and E3 are directed to ponding areas within the site. In a large storm event these basins will spill out of the site boundaries to the west and south of the site. Basin E4 drains to Mill Creek that runs along the east side of the property boundary.

The developed basins have been delineated into 3 main tributary areas Basins A, B, and C. The majority of the runoff will be directed to a retention basin on the southwest side of the project (Basin A) and to a basin (Basin B) on the east side of the project and will then be discharged to the east side of the project into Mill Creek running along the East side of the Property.

**Model Results**

The rational method flows presented in this report are undetained flows. In final design, the detention basin will be sized and outlet peak flows at or below peak flow levels that would be generated from the site in the undeveloped condition. The peak 100-year and 2-year routed flows for the developed condition are summarized below:

**Rational Method Routed Peak Flows**

Basin A Design Flows					Basin B Design Flows					Condition Flows	
Design Point	Elevation (Ft)	Tc100 (Min.)	100-year Flow (CFS)	2-year Flow (CFS)	Design Point	Elevation (Ft)	Tc100 (Min.)	100-year Flow (CFS)	2-year Flow (CFS)	Design Point	100-year Flow (CFS)
90-100	530	19.7	72.2	n/a	198	541	12.31	93.44	32.4	201	110
101	535	19.7	30.4	10.0	300	540	29.77	47.50	13.4	101	86.7
120	535	12.0	52.8	17.8	400	540	10.17	23.16	8.5	300	51
140	555	15.2	95.2	31.9	500	540	12.18	14.82	5.2	510	172.6
160-150	535	23.3	7.0	2.2						505	198.7
151-150	535	11.5	7.3	2.6						600	211.9

## ***PRADO BASIN DISCUSSION***

A major feature of the development of the project area is the relationship of the proposed development area both horizontally and vertically in relationship to the Army Corps of Engineers (ACOE) – Prado Basin area. In the early 1940's, the ACOE obtained an inundation easement to the 556' contour elevation across the Mill Creek property. As a part of the ongoing Prado Dam enhancement project, the ACOE has determined they require additional storage volume based on the existing 566' contour elevation in the project area. We have calculated the existing storage volume on a one-foot incremental basis on the Mill Creek property from an elevation of 509' up to 566'. Based on our proposed grading for TTM 18846, we have calculated the proposed storage volume on a one-foot incremental basis on the Mill Creek property from an elevation of 509' up to 566'. We increase the total potential storage volume on the Mill Creek property to 106% of the baseline existing condition storage volume number. We have included a table that compares the storage volume for the existing and developed condition on an incremental foot basis in Appendix J. Neither the storage volume for the WQMP basins or the dead storage volume in Basin A are included in the net storage volume for the proposed condition.

## ***BASIN A & B DISCUSSION***

Basin A on the west side of the property has a total storage volume of 204.7 acre-feet from a bottom elevation of 530' to 545'. The 100-year 24-hour unit hydrograph calculations for Basin A show a total volume of 47.5 acre-feet., which is 23% of the available storage volume. A summary of the basin volumetrics is included in Appendix H: "Basin A -Unit Hydrograph Calculations".

Basin B is currently an outlet location for basins B-1 and B-2 that are sized for water quality purposes. Basin B is currently graded to maximize the volume needed for ACOE storage purposes. Should detention be required at this location, it will be sized and designed in final design.

## ***CONCLUSION***

Based on the calculations and proposed improvements, onsite flows can be conveyed to suitable points of disposal, and the proposed TTM development will not impact offsite property. The proposed grading for TTM 18846 will result in a net increase of approximately 5% in the ACOE storage volume for Prado Basin over the existing base line condition for the site. Basin A has sufficient dead storage capacity to handle the 100-year storm flows from the design tributary area. Basin B has sufficient storage capacity to handle the 100-year storm flows from the design tributary area, and will drain out to Basin C and thence Mill Creek.

***APPENDIX A: 100-YEAR DEVELOPED CONDITION MODEL OUTPUT***

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/11/15

100 YEAR  
RANCHO MIRAMONTE  
BASIN A100

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 105.000 to Point/Station 104.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Initial subarea data:  
Initial area flow distance = 575.000(Ft.)  
Top (of initial area) elevation = 571.500(Ft.)  
Bottom (of initial area) elevation = 570.500(Ft.)  
Difference in elevation = 1.000(Ft.)  
Slope = 0.00174 s(%)= 0.17  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 14.667 min.  
Rainfall intensity = 2.841(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.883  
Subarea runoff = 7.629(CFS)  
Total initial stream area = 3.040(Ac.)  
Pervious area fraction = 0.200

Initial area Fm value = 0.052(In/Hr)

+++++  
Process from Point/Station 104.000 to Point/Station 103.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 570.500(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 820.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 14.000(Ft.)  
Distance from crown to crossfall grade break = 12.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [1] side(s) of the street  
Distance from curb to property line = 4.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 1.500(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150  
Estimated mean flow rate at midpoint of street = 16.340(CFS)  
Depth of flow = 0.601(Ft.), Average velocity = 2.917(Ft/s)  
Warning: depth of flow exceeds top of curb  
Note: depth of flow exceeds top of street crown.  
Distance that curb overflow reaches into property = 5.07(Ft.)  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 14.000(Ft.)  
Flow velocity = 2.92(Ft/s)  
Travel time = 4.68 min. TC = 19.35 min.  
Adding area flow to street  
RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Rainfall intensity = 2.406(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.866  
Subarea runoff = 17.323(CFS) for 8.940(Ac.)  
Total runoff = 24.952(CFS)  
Effective area this stream = 11.98(Ac.)  
Total Study Area (Main Stream No. 1) = 11.98(Ac.)  
Area averaged Fm value = 0.091(In/Hr)  
Street flow at end of street = 24.952(CFS)  
Half street flow at end of street = 24.952(CFS)  
Depth of flow = 0.691(Ft.), Average velocity = 3.456(Ft/s)  
Warning: depth of flow exceeds top of curb  
Note: depth of flow exceeds top of street crown.  
Distance that curb overflow reaches into property = 9.57(Ft.)  
Flow width (from curb towards crown)= 14.000(Ft.)

\*\*\*\*\*  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 567.000(Ft.)  
Downstream point/station elevation = 562.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 24.952(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 24.952(CFS)  
Normal flow depth in pipe = 13.00(In.)  
Flow top width inside pipe = 20.40(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 15.96(Ft/s)  
Travel time through pipe = 0.10 min.  
Time of concentration (TC) = 19.46 min.

\*\*\*\*\*  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Time of concentration = 19.46 min.  
Rainfall intensity = 2.398(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.870  
Subarea runoff = 5.423(CFS) for 2.580(Ac.)  
Total runoff = 30.376(CFS)  
Effective area this stream = 14.56(Ac.)  
Total Study Area (Main Stream No. 1) = 14.56(Ac.)  
Area averaged Fm value = 0.080(In/Hr)

\*\*\*\*\*  
Process from Point/Station 102.000 to Point/Station 101.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 562.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 300.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 30.376(CFS)  
Nearest computed pipe diameter = 18.00(In.)  
Calculated individual pipe flow = 30.376(CFS)  
Normal flow depth in pipe = 14.20(In.)  
Flow top width inside pipe = 14.69(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 20.31(Ft/s)  
Travel time through pipe = 0.25 min.

Time of concentration (TC) = 19.70 min.

++++  
Process from Point/Station 101.000 to Point/Station 90.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 19.70 min.  
Rainfall intensity = 2.380(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.863  
Subarea runoff = 18.383(CFS) for 9.190(Ac.)  
Total runoff = 48.759(CFS)  
Effective area this stream = 23.75(Ac.)  
Total Study Area (Main Stream No. 1) = 23.75(Ac.)  
Area averaged Fm value = 0.099(In/Hr)

++++  
Process from Point/Station 90.000 to Point/Station 100.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

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PARK subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.8500 Max loss rate(Fm)= 0.222(In/Hr)  
Time of concentration = 19.70 min.  
Rainfall intensity = 2.380(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.847  
Subarea runoff = 23.437(CFS) for 12.070(Ac.)  
Total runoff = 72.196(CFS)  
Effective area this stream = 35.82(Ac.)  
Total Study Area (Main Stream No. 1) = 35.82(Ac.)  
Area averaged Fm value = 0.140(In/Hr)  
End of computations, Total Study Area = 35.82 (Ac.)

The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.667  
Area averaged SCS curve number = 71.6



San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/08/15

BASIN A200  
100 YEAR  
RANCO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 124.000 to Point/Station 123.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 344.000(Ft.)  
Top (of initial area) elevation = 570.500(Ft.)  
Bottom (of initial area) elevation = 567.000(Ft.)  
Difference in elevation = 3.500(Ft.)  
Slope = 0.01017 s(%)= 1.02  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.683 min.  
Rainfall intensity = 3.645(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.874  
Subarea runoff = 9.622(CFS)  
Total initial stream area = 3.020(Ac.)  
Pervious area fraction = 0.400

Initial area Fm value = 0.105(In/Hr)

+++++  
Process from Point/Station 123.000 to Point/Station 121.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 567.000(Ft.)  
End of street segment elevation = 566.000(Ft.)  
Length of street segment = 344.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 14.000(Ft.)  
Distance from crown to crossfall grade break = 12.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [1] side(s) of the street  
Distance from curb to property line = 4.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 1.500(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150  
Estimated mean flow rate at midpoint of street = 18.530(CFS)  
Depth of flow = 0.667(Ft.), Average velocity = 2.734(Ft/s)  
Warning: depth of flow exceeds top of curb  
Note: depth of flow exceeds top of street crown.  
Distance that curb overflow reaches into property = 8.34(Ft.)  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 14.000(Ft.)  
Flow velocity = 2.73(Ft/s)  
Travel time = 2.10 min. TC = 11.78 min.  
Adding area flow to street  
RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Rainfall intensity = 3.240(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 17.752(CFS) for 6.680(Ac.)  
Total runoff = 27.374(CFS)  
Effective area this stream = 9.70(Ac.)  
Total Study Area (Main Stream No. 1) = 9.70(Ac.)  
Area averaged Fm value = 0.105(In/Hr)  
Street flow at end of street = 27.374(CFS)  
Half street flow at end of street = 27.374(CFS)  
Depth of flow = 0.766(Ft.), Average velocity = 3.196(Ft/s)  
Warning: depth of flow exceeds top of curb  
Note: depth of flow exceeds top of street crown.  
Distance that curb overflow reaches into property = 13.31(Ft.)  
Flow width (from curb towards crown)= 14.000(Ft.)

\*\*\*\*\*  
Process from Point/Station 122.000 to Point/Station 121.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Time of concentration = 11.78 min.  
Rainfall intensity = 3.240(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 11.853(CFS) for 4.200(Ac.)  
Total runoff = 39.226(CFS)  
Effective area this stream = 13.90(Ac.)  
Total Study Area (Main Stream No. 1) = 13.90(Ac.)  
Area averaged Fm value = 0.105(In/Hr)

\*\*\*\*\*  
Process from Point/Station 122.000 to Point/Station 121.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Time of concentration = 11.78 min.  
Rainfall intensity = 3.240(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 13.546(CFS) for 4.800(Ac.)  
Total runoff = 52.772(CFS)  
Effective area this stream = 18.70(Ac.)  
Total Study Area (Main Stream No. 1) = 18.70(Ac.)  
Area averaged Fm value = 0.105(In/Hr)

\*\*\*\*\*  
Process from Point/Station 121.000 to Point/Station 120.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 315.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 52.772(CFS)  
Nearest computed pipe diameter = 24.00(In.)  
Calculated individual pipe flow = 52.772(CFS)

Normal flow depth in pipe = 15.42(In.)  
Flow top width inside pipe = 23.00(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 24.75(Ft/s)  
Travel time through pipe = 0.21 min.  
Time of concentration (TC) = 11.99 min.  
End of computations, Total Study Area = 18.70 (Ac.)

The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.400  
Area averaged SCS curve number = 69.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/11/15

SUBAREA A300  
100 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 148.000 to Point/Station 145.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 524.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00763 s(%)= 0.76  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 12.136 min.  
Rainfall intensity = 3.183(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.870  
Subarea runoff = 13.824(CFS)  
Total initial stream area = 4.990(Ac.)  
Pervious area fraction = 0.400

Initial area Fm value = 0.105(In/Hr)

++++  
Process from Point/Station 150.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

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COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 3.183(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.874  
Subarea runoff = 2.358(CFS) for 0.830(Ac.)  
Total runoff = 16.182(CFS)  
Effective area this stream = 5.82(Ac.)  
Total Study Area (Main Stream No. 1) = 5.82(Ac.)  
Area averaged Fm value = 0.093(In/Hr)

++++  
Process from Point/Station 150.000 to Point/Station 145.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 1  
Stream flow area = 5.820(Ac.)  
Runoff from this stream = 16.182(CFS)  
Time of concentration = 12.14 min.  
Rainfall intensity = 3.183(In/Hr)  
Area averaged loss rate (Fm) = 0.0934(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3572  
Program is now starting with Main Stream No. 2

++++  
Process from Point/Station 149.000 to Point/Station 145.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 634.000(Ft.)  
Top (of initial area) elevation = 573.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)

Difference in elevation = 5.000(Ft.)  
 Slope = 0.00789 s(%)= 0.79  
 $TC = k(0.374)*[(length^3)/(elevation\ change)]^{0.2}$   
 Initial area time of concentration = 13.012 min.  
 Rainfall intensity = 3.052(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.869  
 Subarea runoff = 22.392(CFS)  
 Total initial stream area = 8.440(Ac.)  
 Pervious area fraction = 0.400  
 Initial area Fm value = 0.105(In/Hr)

++++++  
 Process from Point/Station 149.000 to Point/Station 145.000  
 \*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

The following data inside Main Stream is listed:

In Main Stream number: 2  
 Stream flow area = 8.440(Ac.)  
 Runoff from this stream = 22.392(CFS)  
 Time of concentration = 13.01 min.  
 Rainfall intensity = 3.052(In/Hr)  
 Area averaged loss rate (Fm) = 0.1046(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.4000  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	16.18	5.820	12.14	0.093	3.183
2	22.39	8.440	13.01	0.105	3.052

Qmax(1) =  
 1.000 \* 1.000 \* 16.182) +  
 1.044 \* 0.933 \* 22.392) + = 37.990  
 Qmax(2) =  
 0.958 \* 1.000 \* 16.182) +  
 1.000 \* 1.000 \* 22.392) + = 37.892

Total of 2 main streams to confluence:  
 Flow rates before confluence point:  
 17.182      23.392  
 Maximum flow rates at confluence using above data:  
 37.990      37.892  
 Area of streams before confluence:  
 5.820      8.440  
 Effective area values after confluence:  
 13.692      14.260

Results of confluence:  
 Total flow rate = 37.990(CFS)  
 Time of concentration = 12.136 min.  
 Effective stream area after confluence = 13.692(Ac.)  
 Study area average Pervious fraction(Ap) = 0.383  
 Study area average soil loss rate(Fm) = 0.100(In/Hr)  
 Study area total = 14.26(Ac.)

+++++  
Process from Point/Station 147.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 3.183(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 11.135(CFS) for 4.020(Ac.)  
Total runoff = 49.125(CFS)  
Effective area this stream = 17.71(Ac.)  
Total Study Area (Main Stream No. 1) = 18.28(Ac.)  
Area averaged Fm value = 0.101(In/Hr)

+++++  
Process from Point/Station 146.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 3.183(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 13.437(CFS) for 4.850(Ac.)  
Total runoff = 62.562(CFS)  
Effective area this stream = 22.56(Ac.)  
Total Study Area (Main Stream No. 1) = 23.13(Ac.)  
Area averaged Fm value = 0.102(In/Hr)

+++++  
Process from Point/Station 145.000 to Point/Station 144.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.000(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 747.000(Ft.)  
Height of curb above gutter flowline = 8.0(In.)  
Width of half street (curb to crown) = 22.000(Ft.)

Distance from crown to crossfall grade break = 18.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 62.562(CFS)  
Half street flow before street inlet = 31.281(CFS)  
Existing pipe flow before street inlet = 0.000(CFS)  
Number of street inlets = 2  
Depth of flow = 0.941(Ft.), Average velocity = 2.128(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
Street flow half width at start of inlet = 22.000(Ft.)  
Flow rate in gutter section of street =  $Q_w = 4.336$ (CFS)  
Ratio of frontal flow to total flow =  $E_0 = 0.1386$   
Given curb inlet length  $L = 14.000$ (Ft.)  
Street slope is less than .5% , depth of flow indicates an orifice flow condition exists for an opening height of 6.00(In.)  
Using equation  $Q_i = .67hL(2gd_0)^{.5}$   
Total inlet flow capacity= 31.290(CFS)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Gutter depression depth = 0.000(In.)  
Gutter depression width = 2.000(Ft.)  
Efficiency =  $1 - (1-L/Lt)^{1.8} = 1.0000$

Note: Single inlet capacity is greater than 1/2 street flow

Pipe calculations for under street flow rate of 62.562(CFS)  
Using a pipe slope = 0.500 %  
Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 747.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 62.562(CFS)  
Nearest computed pipe diameter = 42.00(In.)  
Calculated individual pipe flow = 62.562(CFS)  
Normal flow depth in pipe = 30.56(In.)  
Flow top width inside pipe = 37.39(In.)  
Critical Depth = 29.76(In.)  
Pipe flow velocity = 8.34(Ft/s)  
Travel time through pipe = 1.49 min.  
Time of concentration (TC) = 13.63 min.  
Maximum flow rate of street inlet(s) = 62.562(CFS)  
Maximum pipe flow capacity = 62.562(CFS)

Remaining flow in street below inlet = 0.000(CFS)  
 Adding area flow to street  
 COMMERCIAL subarea type  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Adjusted SCS curve number for AMC 3 = 86.20  
 Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
 Rainfall intensity = 2.969(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.871  
 Subarea runoff = 2.038(CFS) for 2.410(Ac.)  
 Total runoff = 64.600(CFS)  
 Effective area this stream = 24.97(Ac.)  
 Total Study Area (Main Stream No. 1) = 25.54(Ac.)  
 Area averaged Fm value = 0.095(In/Hr)  
 Street flow at end of street = 2.038(CFS)  
 Half street flow at end of street = 1.019(CFS)  
 Depth of flow = 0.335(Ft.), Average velocity = 0.844(Ft/s)  
 Flow width (from curb towards crown)= 10.394(Ft.)

++++++  
 Process from Point/Station 144.000 to Point/Station 143.000  
 \*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 567.000(Ft.)  
 End of street segment elevation = 566.500(Ft.)  
 Length of street segment = 300.000(Ft.)  
 Height of curb above gutter flowline = 8.0(In.)  
 Width of half street (curb to crown) = 22.000(Ft.)  
 Distance from crown to crossfall grade break = 18.000(Ft.)  
 Slope from gutter to grade break (v/hz) = 0.020  
 Slope from grade break to crown (v/hz) = 0.020  
 Street flow is on [2] side(s) of the street  
 Distance from curb to property line = 10.000(Ft.)  
 Slope from curb to property line (v/hz) = 0.025  
 Gutter width = 2.000(Ft.)  
 Gutter hike from flowline = 2.000(In.)  
 Manning's N in gutter = 0.0150  
 Manning's N from gutter to grade break = 0.0150  
 Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 2.038(CFS)  
 Half street flow before street inlet = 1.019(CFS)  
 Existing pipe flow before street inlet = 62.562(CFS)  
 Number of street inlets = 2  
 Depth of flow = 0.325(Ft.), Average velocity = 0.919(Ft/s)  
 U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
 Street flow half width at start of inlet = 9.913(Ft.)  
 Flow rate in gutter section of street = Qw = 0.582(CFS)  
 Ratio of frontal flow to total flow = E0 = 0.5713  
 Given curb inlet length L = 14.000(Ft.)  
 Street slope is less than .5% , depth of flow indicates a weir flow

condition exists for an opening height of 8.00(In.)  
Using equation  $Q_{weir} = 2.3(1.25 \text{ for SI}) (L + 1.8W)d^{1.5}$   
Total inlet flow capacity= 7.497(CFS)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Gutter depression depth = 0.000(In.)

Gutter depression width = 2.000(Ft.)

Efficiency =  $1 - (1-L/Lt)^{1.8} = 1.0000$

Note: Single inlet capacity is greater than 1/2 street flow

Pipe calculations for under street flow rate of 64.600(CFS)

Using a pipe slope = 0.500 %

Upstream point/station elevation = 567.000(Ft.)

Downstream point/station elevation = 566.500(Ft.)

Pipe length = 300.00(Ft.) Manning's N = 0.013

No. of pipes = 1 Required pipe flow = 64.600(CFS)

Nearest computed pipe diameter = 42.00(In.)

Calculated individual pipe flow = 64.600(CFS)

Normal flow depth in pipe = 31.41(In.)

Flow top width inside pipe = 36.48(In.)

Critical Depth = 30.22(In.)

Pipe flow velocity = 8.38(Ft/s)

Travel time through pipe = 0.60 min.

Time of concentration (TC) = 14.22 min.

Maximum flow rate of street inlet(s) = 2.038(CFS)

Maximum pipe flow capacity = 64.600(CFS)

Remaining flow in street below inlet = 0.000(CFS)

Adding area flow to street

RESIDENTIAL(8 - 10 dwl/acre)

Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000

Decimal fraction soil group C = 1.000

Decimal fraction soil group D = 0.000

SCS curve number for soil(AMC 2) = 69.00

Adjusted SCS curve number for AMC 3 = 86.20

Pervious ratio( $A_p$ ) = 0.4000 Max loss rate( $F_m$ )= 0.105(In/Hr)

Rainfall intensity = 2.894(In/Hr) for a 100.0 year storm

Effective runoff coefficient used for area,(total area with modified rational method)( $Q=KCIA$ ) is  $C = 0.870$

Subarea runoff = 6.137(CFS) for 3.120(Ac.)

Total runoff = 70.737(CFS)

Effective area this stream = 28.09(Ac.)

Total Study Area (Main Stream No. 1) = 28.66(Ac.)

Area averaged  $F_m$  value = 0.096(In/Hr)

Street flow at end of street = 6.137(CFS)

Half street flow at end of street = 3.069(CFS)

Depth of flow = 0.440(Ft.), Average velocity = 1.192(Ft/s)

Flow width (from curb towards crown)= 15.646(Ft.)

\*\*\*\*\*  
Process from Point/Station 143.000 to Point/Station 142.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.000(Ft.)  
End of street segment elevation = 566.000(Ft.)  
Length of street segment = 400.000(Ft.)  
Height of curb above gutter flowline = 8.0(In.)  
Width of half street (curb to crown) = 22.000(Ft.)  
Distance from crown to crossfall grade break = 18.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 6.137(CFS)  
Half street flow before street inlet = 3.069(CFS)  
Existing pipe flow before street inlet = 64.600(CFS)  
Number of street inlets = 2  
Depth of flow = 0.377(Ft.), Average velocity = 1.810(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
Street flow half width at start of inlet = 12.525(Ft.)  
Flow rate in gutter section of street =  $Q_w = 1.424$ (CFS)  
Ratio of frontal flow to total flow =  $E_0 = 0.4640$   
Given curb inlet length  $L = 27.000$ (Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Length required for total flow interception =  $L_t$   
 $L_t = .6 * Q^{0.42} * Slope^{.3} * (1/(n*Se))^{.6} = 25.473$ (Ft.)  
where Manning's  $n = 0.0150$  and Slope = street slope = 0.0050  
 $Se =$  Equivalent Street x-slope including depression = 0.0200  
Gutter depression depth = 0.000(In.)  
Gutter depression width = 2.000(Ft.)  
Efficiency =  $1 - (1-L/L_t)^{1.8} = 1.0000$

Pipe calculations for under street flow rate of 70.737(CFS)  
Using a pipe slope = 1.000 %  
Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 566.000(Ft.)  
Pipe length = 400.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 70.737(CFS)  
Nearest computed pipe diameter = 39.00(In.)  
Calculated individual pipe flow = 70.737(CFS)

Normal flow depth in pipe = 27.80(In.)  
 Flow top width inside pipe = 35.29(In.)  
 Critical Depth = 32.02(In.)  
 Pipe flow velocity = 11.19(Ft/s)  
 Travel time through pipe = 0.60 min.  
 Time of concentration (TC) = 14.82 min.  
 Maximum flow rate of street inlet(s) = 6.137(CFS)  
 Maximum pipe flow capacity = 70.737(CFS)  
 Remaining flow in street below inlet = 0.000(CFS)  
 Adding area flow to street  
 RESIDENTIAL(5 - 7 dwl/acre)  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Adjusted SCS curve number for AMC 3 = 86.20  
 Pervious ratio(Ap) = 0.5000 Max loss rate(Fm)= 0.131(In/Hr)  
 Rainfall intensity = 2.823(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.867  
 Subarea runoff = 15.086(CFS) for 6.960(Ac.)  
 Total runoff = 85.823(CFS)  
 Effective area this stream = 35.05(Ac.)  
 Total Study Area (Main Stream No. 1) = 35.62(Ac.)  
 Area averaged Fm value = 0.103(In/Hr)  
 Street flow at end of street = 15.086(CFS)  
 Half street flow at end of street = 7.543(CFS)  
 Depth of flow = 0.486(Ft.), Average velocity = 2.248(Ft/s)  
 Flow width (from curb towards crown)= 17.970(Ft.)

++++++  
 Process from Point/Station 142.000 to Point/Station 141.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 561.000(Ft.)  
 Downstream point/station elevation = 560.000(Ft.)  
 Pipe length = 100.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 85.823(CFS)  
 Nearest computed pipe diameter = 42.00(In.)  
 Calculated individual pipe flow = 85.823(CFS)  
 Normal flow depth in pipe = 29.81(In.)  
 Flow top width inside pipe = 38.12(In.)  
 Critical Depth = 34.62(In.)  
 Pipe flow velocity = 11.74(Ft/s)  
 Travel time through pipe = 0.14 min.  
 Time of concentration (TC) = 14.96 min.

++++++  
 Process from Point/Station 142.000 to Point/Station 141.000  
 \*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:  
 In Main Stream number: 1  
 Stream flow area = 35.052(Ac.)

Runoff from this stream = 85.823(CFS)  
Time of concentration = 14.96 min.  
Rainfall intensity = 2.807(In/Hr)  
Area averaged loss rate (Fm) = 0.1026(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3924  
Program is now starting with Main Stream No. 2

\*\*\*\*\*  
Process from Point/Station 152.000 to Point/Station 151.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 690.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 567.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00580 s(%)= 0.58  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 14.314 min.  
Rainfall intensity = 2.883(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.867  
Subarea runoff = 9.601(CFS)  
Total initial stream area = 3.840(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.105(In/Hr)

\*\*\*\*\*  
Process from Point/Station 151.000 to Point/Station 141.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 562.000(Ft.)  
Downstream point/station elevation = 561.000(Ft.)  
Pipe length = 400.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 9.601(CFS)  
Nearest computed pipe diameter = 24.00(In.)  
Calculated individual pipe flow = 9.601(CFS)  
Normal flow depth in pipe = 16.97(In.)  
Flow top width inside pipe = 21.85(In.)  
Critical Depth = 13.29(In.)  
Pipe flow velocity = 4.04(Ft/s)  
Travel time through pipe = 1.65 min.  
Time of concentration (TC) = 15.96 min.

\*\*\*\*\*  
Process from Point/Station 151.000 to Point/Station 141.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 2  
Stream flow area = 3.840(Ac.)  
Runoff from this stream = 9.601(CFS)  
Time of concentration = 15.96 min.  
Rainfall intensity = 2.700(In/Hr)  
Area averaged loss rate (Fm) = 0.1046(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.4000  
Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
------------	-----------------	------------	----------	------------	----------------------------

1	85.82	35.052	14.96	0.103	2.807
2	9.60	3.840	15.96	0.105	2.700

Qmax(1) =  
1.000 \* 1.000 \* 85.823) +  
1.041 \* 0.937 \* 9.601) + = 95.192

Qmax(2) =  
0.960 \* 1.000 \* 85.823) +  
1.000 \* 1.000 \* 9.601) + = 92.026

Total of 2 main streams to confluence:

Flow rates before confluence point:  
86.823 10.601

Maximum flow rates at confluence using above data:  
95.192 92.026

Area of streams before confluence:  
35.052 3.840

Effective area values after confluence:  
38.651 38.892

Results of confluence:

Total flow rate = 95.192(CFS)  
Time of concentration = 14.963 min.  
Effective stream area after confluence = 38.651(Ac.)  
Study area average Pervious fraction(Ap) = 0.393  
Study area average soil loss rate(Fm) = 0.103(In/Hr)  
Study area total = 38.89(Ac.)

\*\*\*\*\*  
Process from Point/Station 141.000 to Point/Station 140.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 561.000(Ft.)  
Downstream point/station elevation = 555.000(Ft.)  
Pipe length = 250.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 95.192(CFS)  
Nearest computed pipe diameter = 36.00(In.)  
Calculated individual pipe flow = 95.192(CFS)  
Normal flow depth in pipe = 27.23(In.)  
Flow top width inside pipe = 30.90(In.)  
Critical depth could not be calculated.

Pipe flow velocity = 16.59(Ft/s)  
Travel time through pipe = 0.25 min.  
Time of concentration (TC) = 15.21 min.

++++  
Process from Point/Station 141.000 to Point/Station 140.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio( $A_p$ ) = 1.0000 Max loss rate( $F_m$ )= 0.129(In/Hr)  
The area added to the existing stream causes a  
a lower flow rate of  $Q = 95.125$ (CFS)  
therefore the upstream flow rate of  $Q = 95.192$ (CFS) is being used  
Time of concentration = 15.21 min.  
Rainfall intensity = 2.779(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)( $Q=KCIA$ ) is  $C = 0.867$   
Subarea runoff = 0.000(CFS) for 0.850(Ac.)  
Total runoff = 95.192(CFS)  
Effective area this stream = 39.50(Ac.)  
Total Study Area (Main Stream No. 1) = 40.31(Ac.)  
Area averaged  $F_m$  value = 0.103(In/Hr)  
End of computations, Total Study Area = 40.31 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.406  
Area averaged SCS curve number = 69.2

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005  
Version 7.1

Rational Hydrology Study Date: 03/08/15

-----  
SUBAREA BASIN A400  
100 YEAR  
RANCHO MIRAMONTE  
-----

-----  
Program License Serial Number 6288  
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-----  
\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*  
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Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3  
-----

+++++  
+++++  
Process from Point/Station 161.000 to Point/Station 162.000  
\*\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*\*  
-----

-----  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)=  
0.129(In/Hr)  
Initial subarea data:  
Initial area flow distance = 1000.000(Ft.)  
Top (of initial area) elevation = 575.000(Ft.)

Bottom (of initial area) elevation = 549.000(Ft.)  
Difference in elevation = 26.000(Ft.)  
Slope = 0.02600 s(%)= 2.60  
TC =  $k(0.706) * [(length^3)/(elevation\ change)]^{0.2}$   
Initial area time of concentration = 23.217 min.  
Rainfall intensity = 2.157(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.846  
Subarea runoff = 2.920(CFS)  
Total initial stream area = 1.600(Ac.)  
Pervious area fraction = 1.000  
Initial area Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 161.000 to Point/Station  
160.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)=  
0.026(In/Hr)  
Time of concentration = 23.22 min.  
Rainfall intensity = 2.157(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with  
modified  
rational method)(Q=KCIA) is C = 0.871  
Subarea runoff = 4.122(CFS) for 2.150(Ac.)  
Total runoff = 7.043(CFS)  
Effective area this stream = 3.75(Ac.)  
Total Study Area (Main Stream No. 1) = 3.75(Ac.)  
Area averaged Fm value = 0.070(In/Hr)

+++++  
Process from Point/Station 160.000 to Point/Station  
150.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 545.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 7.043(CFS)

Nearest computed pipe diameter = 9.00(In.)  
Calculated individual pipe flow = 7.043(CFS)  
Normal flow depth in pipe = 7.02(In.)  
Flow top width inside pipe = 7.46(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 19.06(Ft/s)  
Travel time through pipe = 0.04 min.  
Time of concentration (TC) = 23.26 min.  
End of computations, Total Study Area = 3.75 (Ac.)

The following figures may

be used for a unit hydrograph study of the same area.

Note: These figures do not consider reduced effective area effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.484

Area averaged SCS curve number = 73.3

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/11/15

SUBAREA A500  
100 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

+++++  
Process from Point/Station 171.000 to Point/Station 170.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Initial subarea data:  
Initial area flow distance = 1000.000(Ft.)  
Top (of initial area) elevation = 566.000(Ft.)  
Bottom (of initial area) elevation = 553.000(Ft.)  
Difference in elevation = 13.000(Ft.)  
Slope = 0.01300 s(%)= 1.30  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 11.484 min.  
Rainfall intensity = 3.290(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.893  
Subarea runoff = 3.760(CFS)  
Total initial stream area = 1.280(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.026(In/Hr)

+++++  
Process from Point/Station 170.000 to Point/Station 150.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 553.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 3.760(CFS)  
Nearest computed pipe diameter = 9.00(In.)  
Calculated individual pipe flow = 3.760(CFS)  
Normal flow depth in pipe = 3.84(In.)  
Flow top width inside pipe = 8.90(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 20.91(Ft/s)  
Travel time through pipe = 0.04 min.  
Time of concentration (TC) = 11.52 min.

+++++  
Process from Point/Station 151.000 to Point/Station 150.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 11.52 min.  
Rainfall intensity = 3.283(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.879  
Subarea runoff = 3.569(CFS) for 1.260(Ac.)  
Total runoff = 7.329(CFS)  
Effective area this stream = 2.54(Ac.)  
Total Study Area (Main Stream No. 1) = 2.54(Ac.)  
Area averaged Fm value = 0.077(In/Hr)  
End of computations, Total Study Area = 2.54 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.546  
Area averaged SCS curve number = 74.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version

7.1

Rational Hydrology Study Date: 03/09/15

-----  
SUBAREA BASIN B100  
100 YEAR  
RANCHO MIRAMONTE  
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-----  
Program License Serial Number 6288  
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\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*  
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-----  
Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

+++++  
+++++  
Process from Point/Station 205.000 to Point/Station  
204.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*  
-----

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 436.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 568.500(Ft.)

Difference in elevation = 3.500(Ft.)  
Slope = 0.00803 s(%)= 0.80  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 11.162 min.  
Rainfall intensity = 3.347(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.872  
Subarea runoff = 8.899(CFS)  
Total initial stream area = 3.050(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.105(In/Hr)

+++++  
\*\*\*\*\*  
Process from Point/Station 206.000 to Point/Station  
204.000  
\*\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Time of concentration = 11.16 min.  
Rainfall intensity = 3.347(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.872  
Subarea runoff = 9.133(CFS) for 3.130(Ac.)  
Total runoff = 18.032(CFS)  
Effective area this stream = 6.18(Ac.)  
Total Study Area (Main Stream No. 1) = 6.18(Ac.)  
Area averaged Fm value = 0.105(In/Hr)

+++++  
\*\*\*\*\*  
Process from Point/Station 150.000 to Point/Station  
204.000  
\*\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20

Pervious ratio(Ap) = 0.1000      Max loss rate(Fm)=      0.026(In/Hr)  
 Time of concentration =      11.16 min.  
 Rainfall intensity =      3.347(In/Hr) for a      100.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.874  
 Subarea runoff =      1.733(CFS) for      0.580(Ac.)  
 Total runoff =      19.765(CFS)  
 Effective area this stream =      6.76(Ac.)  
 Total Study Area (Main Stream No. 1) =      6.76(Ac.)  
 Area averaged Fm value =      0.098(In/Hr)

++++++  
 \*\*\*\*\*  
 Process from Point/Station      204.000 to Point/Station  
 203.000  
 \*\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*\*

---

Top of street segment elevation =      568.500(Ft.)  
 End of street segment elevation =      567.000(Ft.)  
 Length of street segment =      100.000(Ft.)  
 Height of curb above gutter flowline =      8.0(In.)  
 Width of half street (curb to crown) =      22.000(Ft.)  
 Distance from crown to crossfall grade break =      18.000(Ft.)  
 Slope from gutter to grade break (v/hz) =      0.020  
 Slope from grade break to crown (v/hz) =      0.020  
 Street flow is on [2] side(s) of the street  
 Distance from curb to property line =      10.000(Ft.)  
 Slope from curb to property line (v/hz) =      0.025  
 Gutter width =      2.000(Ft.)  
 Gutter hike from flowline =      2.000(In.)  
 Manning's N in gutter =      0.0150  
 Manning's N from gutter to grade break =      0.0150  
 Manning's N from grade break to crown =      0.0150

Street Inlet Calculations:

Street flow before street inlet =      19.765(CFS)  
 Half street flow before street inlet =      9.883(CFS)  
 Existing pipe flow before street inlet =      0.000(CFS)  
 Number of street inlets =      2  
 Depth of flow =      0.680(Ft.), Average velocity =      3.699(Ft/s)  
 U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet

calculations:

Street flow half width at start of inlet =      15.150(Ft.)  
 Flow rate in gutter section of street = Qw =      4.876(CFS)  
 Ratio of frontal flow to total flow = E0 =      0.4934  
 Given curb inlet length L =      14.000(Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	1.1667 right of way

10.0000	0.9167 top of curb
10.0000	0.0000 flow line
12.0000	0.4167 gutter/depression end
14.0000	0.4567 grade break
32.0000	0.8167 crown

Length required for total flow interception = Lt  
 $Lt = .6 * Q^{0.42} * Slope^{.3} * (1/(n*Se))^{.6} = 24.883(\text{Ft.})$   
 where Manning's n = 0.0150 and Slope = street slope = 0.0150  
 Se = Equivalent Street x-slope including depression = 0.0817  
 Gutter depression depth = 3.000(In.)  
 Gutter depression width = 2.000(Ft.)  
 Efficiency =  $1 - (1-L/Lt)^{1.8} = 0.7743$

Pipe calculations for under street flow rate of 15.304(CFS)  
 Using a pipe slope = 1.500 %  
 Upstream point/station elevation = 568.500(Ft.)  
 Downstream point/station elevation = 567.000(Ft.)  
 Pipe length = 100.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 15.304(CFS)  
 Nearest computed pipe diameter = 21.00(In.)  
 Calculated individual pipe flow = 15.304(CFS)  
 Normal flow depth in pipe = 14.06(In.)  
 Flow top width inside pipe = 19.75(In.)  
 Critical Depth = 17.37(In.)  
 Pipe flow velocity = 8.94(Ft/s)  
 Travel time through pipe = 0.19 min.  
 Time of concentration (TC) = 11.35 min.  
 Maximum flow rate of street inlet(s) = 15.304(CFS)  
 Maximum pipe flow capacity = 15.304(CFS)  
 Remaining flow in street below inlet = 4.461(CFS)

Adding area flow to street  
 COMMERCIAL subarea type  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Adjusted SCS curve number for AMC 3 = 86.20  
 Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
 Rainfall intensity = 3.313(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area, (total area with modified  
 rational method)(Q=KCIA) is C = 0.875  
 Subarea runoff = 0.982(CFS) for 0.400(Ac.)  
 Total runoff = 20.747(CFS)  
 Effective area this stream = 7.16(Ac.)  
 Total Study Area (Main Stream No. 1) = 7.16(Ac.)  
 Area averaged Fm value = 0.094(In/Hr)  
 Street flow at end of street = 5.443(CFS)  
 Half street flow at end of street = 2.721(CFS)  
 Depth of flow = 0.315(Ft.), Average velocity = 2.684(Ft/s)  
 Flow width (from curb towards crown)= 9.419(Ft.)

+++++  
Process from Point/Station 203.000 to Point/Station  
202.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 1  
Stream flow area = 7.160(Ac.)  
Runoff from this stream = 20.747(CFS)  
Time of concentration = 11.35 min.  
Rainfall intensity = 3.313(In/Hr)  
Area averaged loss rate (Fm) = 0.0939(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3589

+++++  
Process from Point/Station 207.000 to Point/Station  
203.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 394.000(Ft.)  
Top (of initial area) elevation = 571.200(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 3.200(Ft.)  
Slope = 0.00812 s(%)= 0.81  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 10.694 min.  
Rainfall intensity = 3.434(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.873  
Subarea runoff = 6.472(CFS)  
Total initial stream area = 2.160(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.105(In/Hr)

+++++  
Process from Point/Station 203.000 to Point/Station  
202.000

\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 567.500(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 6.472(CFS)  
Nearest computed pipe diameter = 18.00(In.)  
Calculated individual pipe flow = 6.472(CFS)  
Normal flow depth in pipe = 10.22(In.)  
Flow top width inside pipe = 17.83(In.)  
Critical Depth = 11.80(In.)  
Pipe flow velocity = 6.25(Ft/s)  
Travel time through pipe = 0.13 min.  
Time of concentration (TC) = 10.83 min.

+++++

Process from Point/Station 203.000 to Point/Station  
202.000

\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 2  
Stream flow area = 2.160(Ac.)  
Runoff from this stream = 6.472(CFS)  
Time of concentration = 10.83 min.  
Rainfall intensity = 3.408(In/Hr)  
Area averaged loss rate (Fm) = 0.1046(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.4000

+++++

Process from Point/Station 208.000 to Point/Station  
203.000

\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.105(In/Hr)  
Initial subarea data:  
Initial area flow distance = 540.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)

Difference in elevation = 3.000(Ft.)  
 Slope = 0.00556 s(%)= 0.56  
 TC =  $k(0.374)*[(\text{length}^3)/(\text{elevation change})]^{0.2}$   
 Initial area time of concentration = 13.088 min.  
 Rainfall intensity = 3.042(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.869  
 Subarea runoff = 7.454(CFS)  
 Total initial stream area = 2.820(Ac.)  
 Pervious area fraction = 0.400  
 Initial area Fm value = 0.105(In/Hr)

++++++  
 ++++++ Process from Point/Station 203.000 to Point/Station  
 202.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 569.000(Ft.)  
 Downstream point/station elevation = 568.500(Ft.)  
 Pipe length = 50.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 7.454(CFS)  
 Nearest computed pipe diameter = 18.00(In.)  
 Calculated individual pipe flow = 7.454(CFS)  
 Normal flow depth in pipe = 11.20(In.)  
 Flow top width inside pipe = 17.45(In.)  
 Critical Depth = 12.70(In.)  
 Pipe flow velocity = 6.45(Ft/s)  
 Travel time through pipe = 0.13 min.  
 Time of concentration (TC) = 13.22 min.

++++++  
 ++++++ Process from Point/Station 203.000 to Point/Station  
 202.000  
 \*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 3  
 Stream flow area = 2.820(Ac.)  
 Runoff from this stream = 7.454(CFS)  
 Time of concentration = 13.22 min.  
 Rainfall intensity = 3.024(In/Hr)  
 Area averaged loss rate (Fm) = 0.1046(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.4000  
 Summary of stream data:

Stream Flow rate No.	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
-------------------------	---------------	-------------	---------------	-------------------------------

1	20.75	7.160	11.35	0.094	3.313
2	6.47	2.160	10.83	0.105	3.408
3	7.45	2.820	13.22	0.105	3.024

Qmax(1) =

1.000 *	1.000 *	20.747)	+	
0.971 *	1.000 *	6.472)	+	
1.099 *	0.859 *	7.454)	+	= 34.069

Qmax(2) =

1.029 *	0.954 *	20.747)	+	
1.000 *	1.000 *	6.472)	+	
1.132 *	0.819 *	7.454)	+	= 33.759

Qmax(3) =

0.910 *	1.000 *	20.747)	+	
0.884 *	1.000 *	6.472)	+	
1.000 *	1.000 *	7.454)	+	= 32.054

Total of 3 streams to confluence:  
Flow rates before confluence point:  
20.747            6.472            7.454  
Maximum flow rates at confluence using above data:  
34.069            33.759            32.054  
Area of streams before confluence:  
7.160            2.160            2.820  
Effective area values after confluence:  
11.741            11.301            12.140  
Results of confluence:  
Total flow rate =        34.069(CFS)  
Time of concentration =    11.349 min.  
Effective stream area after confluence =        11.741(Ac.)  
Study area average Pervious fraction(Ap) =    0.376  
Study area average soil loss rate(Fm) =        0.098(In/Hr)  
Study area total (this main stream) =        12.14(Ac.)

+++++

Process from Point/Station            202.000 to Point/Station  
201.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation =    568.500(Ft.)  
End of street segment elevation =    567.000(Ft.)  
Length of street segment =    180.000(Ft.)  
Height of curb above gutter flowline =    8.0(In.)  
Width of half street (curb to crown) =    22.000(Ft.)  
Distance from crown to crossfall grade break =    18.000(Ft.)  
Slope from gutter to grade break (v/hz) =    0.020  
Slope from grade break to crown (v/hz) =    0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line =    10.000(Ft.)

Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 4.838(CFS)  
Half street flow before street inlet = 2.419(CFS)  
Existing pipe flow before street inlet = 29.230(CFS)  
Number of street inlets = 2  
Depth of flow = 0.546(Ft.), Average velocity = 2.210(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet

calculations:

Street flow half width at start of inlet = 8.474(Ft.)  
Flow rate in gutter section of street =  $Q_w = 1.900$ (CFS)  
Ratio of frontal flow to total flow =  $E_0 = 0.7855$   
Given curb inlet length  $L = 14.000$ (Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	1.1667 right of way
10.0000	0.9167 top of curb
10.0000	0.0000 flow line
12.0000	0.4167 gutter/depression end
14.0000	0.4567 grade break
32.0000	0.8167 crown

Length required for total flow interception =  $L_t$   
 $L_t = .6 * Q^{0.42} * \text{Slope}^{.3} * (1/(n*Se))^{.6} = 9.254$ (Ft.)  
where Manning's  $n = 0.0150$  and Slope = street slope = 0.0083  
 $Se = \text{Equivalent Street x-slope including depression} = 0.1182$   
Gutter depression depth = 3.000(In.)  
Gutter depression width = 2.000(Ft.)  
Efficiency =  $1 - (1-L/L_t)^{1.8} = 1.0000$

Pipe calculations for under street flow rate of 34.069(CFS)  
Using a pipe slope = 0.833 %  
Upstream point/station elevation = 568.500(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 180.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 34.069(CFS)  
Nearest computed pipe diameter = 30.00(In.)  
Calculated individual pipe flow = 34.069(CFS)  
Normal flow depth in pipe = 22.45(In.)  
Flow top width inside pipe = 26.03(In.)  
Critical Depth = 23.81(In.)  
Pipe flow velocity = 8.64(Ft/s)  
Travel time through pipe = 0.35 min.  
Time of concentration (TC) = 11.70 min.  
Maximum flow rate of street inlet(s) = 4.838(CFS)  
Maximum pipe flow capacity = 34.069(CFS)

Remaining flow in street below inlet = 0.000(CFS)  
Adding area flow to street  
COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Rainfall intensity = 3.254(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.874  
Subarea runoff = 0.616(CFS) for 0.460(Ac.)  
Total runoff = 34.685(CFS)  
Effective area this stream = 12.20(Ac.)  
Total Study Area (Main Stream No. 1) = 12.60(Ac.)  
Area averaged Fm value = 0.096(In/Hr)  
Street flow at end of street = 0.616(CFS)  
Half street flow at end of street = 0.308(CFS)  
Depth of flow = 0.182(Ft.), Average velocity = 1.518(Ft/s)  
Flow width (from curb towards crown)= 2.760(Ft.)

++++  
Process from Point/Station 201.000 to Point/Station  
200.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 567.000(Ft.)  
Downstream point/station elevation = 566.500(Ft.)  
Pipe length = 60.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 34.685(CFS)  
Nearest computed pipe diameter = 30.00(In.)  
Calculated individual pipe flow = 34.685(CFS)  
Normal flow depth in pipe = 22.83(In.)  
Flow top width inside pipe = 25.59(In.)  
Critical Depth = 24.02(In.)  
Pipe flow velocity = 8.66(Ft/s)  
Travel time through pipe = 0.12 min.  
Time of concentration (TC) = 11.81 min.

++++  
Process from Point/Station 210.000 to Point/Station  
200.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000      Max loss rate(Fm)=      0.105(In/Hr)  
Time of concentration =    11.81 min.  
Rainfall intensity =      3.235(In/Hr) for a    100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.873  
Subarea runoff =      10.468(CFS) for      3.790(Ac.)  
Total runoff =      45.152(CFS)  
Effective area this stream =      15.99(Ac.)  
Total Study Area (Main Stream No. 1) =      16.39(Ac.)  
Area averaged Fm value =      0.098(In/Hr)

++++  
Process from Point/Station      209.000 to Point/Station  
200.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000      Max loss rate(Fm)=      0.105(In/Hr)  
Time of concentration =    11.81 min.  
Rainfall intensity =      3.235(In/Hr) for a    100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.873  
Subarea runoff =      8.621(CFS) for      3.060(Ac.)  
Total runoff =      53.774(CFS)  
Effective area this stream =      19.05(Ac.)  
Total Study Area (Main Stream No. 1) =      19.45(Ac.)  
Area averaged Fm value =      0.099(In/Hr)

++++  
Process from Point/Station      200.000 to Point/Station  
199.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.500(Ft.)  
Downstream point/station elevation = 565.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 53.774(CFS)  
Nearest computed pipe diameter = 33.00(In.)  
Calculated individual pipe flow = 53.774(CFS)  
Normal flow depth in pipe = 22.97(In.)  
Flow top width inside pipe = 30.36(In.)  
Critical Depth = 28.75(In.)  
Pipe flow velocity = 12.20(Ft/s)  
Travel time through pipe = 0.14 min.  
Time of concentration (TC) = 11.95 min.

+++++  
+++++  
Process from Point/Station 200.000 to Point/Station  
199.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:  
In Main Stream number: 1  
Stream flow area = 19.051(Ac.)  
Runoff from this stream = 53.774(CFS)  
Time of concentration = 11.95 min.  
Rainfall intensity = 3.213(In/Hr)  
Area averaged loss rate (Fm) = 0.0988(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3778  
Program is now starting with Main Stream No. 2

+++++  
+++++  
Process from Point/Station 222.000 to Point/Station  
221.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Initial subarea data:  
Initial area flow distance = 690.000(Ft.)  
Top (of initial area) elevation = 570.500(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 2.500(Ft.)

Slope = 0.00362 s(%)= 0.36  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 13.623 min.  
Rainfall intensity = 2.970(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.884  
Subarea runoff = 10.607(CFS)  
Total initial stream area = 4.040(Ac.)  
Pervious area fraction = 0.200  
Initial area Fm value = 0.052(In/Hr)

+++++  
Process from Point/Station 221.000 to Point/Station  
214.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.500(Ft.)  
Downstream point/station elevation = 568.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 10.607(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 10.607(CFS)  
Normal flow depth in pipe = 12.56(In.)  
Flow top width inside pipe = 20.59(In.)  
Critical Depth = 14.55(In.)  
Pipe flow velocity = 7.06(Ft/s)  
Travel time through pipe = 0.12 min.  
Time of concentration (TC) = 13.74 min.

+++++  
Process from Point/Station 215.000 to Point/Station  
214.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Time of concentration = 13.74 min.  
Rainfall intensity = 2.954(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.885  
Subarea runoff = 2.131(CFS) for 0.830(Ac.)

Total runoff = 12.739(CFS)  
Effective area this stream = 4.87(Ac.)  
Total Study Area (Main Stream No. 2) = 24.32(Ac.)  
Area averaged Fm value = 0.048(In/Hr)

+++++  
Process from Point/Station 214.000 to Point/Station  
212.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.200(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 400.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 12.739(CFS)  
Nearest computed pipe diameter = 27.00(In.)  
Calculated individual pipe flow = 12.739(CFS)  
Normal flow depth in pipe = 17.46(In.)  
Flow top width inside pipe = 25.81(In.)  
Critical Depth = 14.87(In.)  
Pipe flow velocity = 4.68(Ft/s)  
Travel time through pipe = 1.42 min.  
Time of concentration (TC) = 15.16 min.

+++++  
Process from Point/Station 213.000 to Point/Station  
212.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 2 in normal stream number 1  
Stream flow area = 4.870(Ac.)  
Runoff from this stream = 12.739(CFS)  
Time of concentration = 15.16 min.  
Rainfall intensity = 2.785(In/Hr)  
Area averaged loss rate (Fm) = 0.0478(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.1830

+++++  
Process from Point/Station 218.000 to Point/Station  
217.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000      Max loss rate(Fm)=      0.026(In/Hr)  
Initial subarea data:  
Initial area flow distance =    489.000(Ft.)  
Top (of initial area) elevation =    568.000(Ft.)  
Bottom (of initial area) elevation =    566.000(Ft.)  
Difference in elevation =      2.000(Ft.)  
Slope =    0.00409    s(%)=      0.41  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration =    10.871 min.  
Rainfall intensity =      3.400(In/Hr) for a    100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.893  
Subarea runoff =      3.128(CFS)  
Total initial stream area =      1.030(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value =      0.026(In/Hr)

+++++  
Process from Point/Station      217.000 to Point/Station  
216.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation =    566.000(Ft.)  
Downstream point/station elevation =    565.800(Ft.)  
Pipe length =    30.00(Ft.)    Manning's N = 0.013  
No. of pipes = 1    Required pipe flow =      3.128(CFS)  
Nearest computed pipe diameter =      15.00(In.)  
Calculated individual pipe flow =      3.128(CFS)  
Normal flow depth in pipe =      8.31(In.)  
Flow top width inside pipe =    14.91(In.)  
Critical Depth =      8.54(In.)  
Pipe flow velocity =      4.48(Ft/s)  
Travel time through pipe =      0.11 min.  
Time of concentration (TC) =      10.98 min.

+++++  
Process from Point/Station      220.000 to Point/Station  
216.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.4000      Max loss rate(Fm)=      0.105(In/Hr)  
Time of concentration =      10.98 min.  
Rainfall intensity =      3.379(In/Hr) for a      100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.875  
Subarea runoff =      16.957(CFS) for      5.760(Ac.)  
Total runoff =      20.085(CFS)  
Effective area this stream =      6.79(Ac.)  
Total Study Area (Main Stream No. 2) =      31.11(Ac.)  
Area averaged Fm value =      0.093(In/Hr)

++++  
Process from Point/Station      216.000 to Point/Station  
212.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation =      565.800(Ft.)  
Downstream point/station elevation =      565.000(Ft.)  
Pipe length =      100.00(Ft.)      Manning's N = 0.013  
No. of pipes = 1      Required pipe flow =      20.085(CFS)  
Nearest computed pipe diameter =      24.00(In.)  
Calculated individual pipe flow =      20.085(CFS)  
Normal flow depth in pipe =      19.50(In.)  
Flow top width inside pipe =      18.73(In.)  
Critical Depth =      19.31(In.)  
Pipe flow velocity =      7.34(Ft/s)  
Travel time through pipe =      0.23 min.  
Time of concentration (TC) =      11.21 min.

++++  
Process from Point/Station      216.000 to Point/Station  
212.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 2 in normal stream number 2  
Stream flow area =      6.790(Ac.)  
Runoff from this stream =      20.085(CFS)  
Time of concentration =      11.21 min.  
Rainfall intensity =      3.338(In/Hr)  
Area averaged loss rate (Fm) =      0.0927(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3545

Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	12.74	4.870	15.16	0.048	2.785
2	20.08	6.790	11.21	0.093	3.338
Qmax(1) =					
	1.000 *	1.000 *	12.739)	+	
	0.829 *	1.000 *	20.085)	+	29.397
Qmax(2) =					
	1.202 *	0.739 *	12.739)	+	
	1.000 *	1.000 *	20.085)	+	31.406

Total of 2 streams to confluence:

Flow rates before confluence point:

12.739          20.085

Maximum flow rates at confluence using above data:

29.397          31.406

Area of streams before confluence:

4.870          6.790

Effective area values after confluence:

11.660          10.390

Results of confluence:

Total flow rate = 31.406(CFS)

Time of concentration = 11.209 min.

Effective stream area after confluence = 10.390(Ac.)

Study area average Pervious fraction(Ap) = 0.283

Study area average soil loss rate(Fm) = 0.074(In/Hr)

Study area total (this main stream) = 11.66(Ac.)

+++++

Process from Point/Station 212.000 to Point/Station 211.000

\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

COMMERCIAL subarea type

Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000

Decimal fraction soil group C = 1.000

Decimal fraction soil group D = 0.000

SCS curve number for soil(AMC 2) = 69.00

Adjusted SCS curve number for AMC 3 = 86.20

Pervious ratio(Ap) = 0.1000          Max loss rate(Fm)= 0.026(In/Hr)

Time of concentration = 11.21 min.

Rainfall intensity = 3.338(In/Hr) for a 100.0 year storm

Effective runoff coefficient used for area,(total area with modified rational method)(Q=KCIA) is C = 0.881

Subarea runoff = 1.263(CFS) for 0.720(Ac.)  
 Total runoff = 32.669(CFS)  
 Effective area this stream = 11.11(Ac.)  
 Total Study Area (Main Stream No. 2) = 31.83(Ac.)  
 Area averaged Fm value = 0.071(In/Hr)

+++++  
 Process from Point/Station 212.000 to Point/Station  
 211.000

\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 565.000(Ft.)  
 Downstream point/station elevation = 563.000(Ft.)  
 Pipe length = 400.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 32.669(CFS)  
 Nearest computed pipe diameter = 33.00(In.)  
 Calculated individual pipe flow = 32.669(CFS)  
 Normal flow depth in pipe = 23.88(In.)  
 Flow top width inside pipe = 29.51(In.)  
 Critical Depth = 22.82(In.)  
 Pipe flow velocity = 7.10(Ft/s)  
 Travel time through pipe = 0.94 min.  
 Time of concentration (TC) = 12.15 min.

+++++  
 Process from Point/Station 211.000 to Point/Station  
 199.000

\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 2  
 Stream flow area = 11.110(Ac.)  
 Runoff from this stream = 32.669(CFS)  
 Time of concentration = 12.15 min.  
 Rainfall intensity = 3.181(In/Hr)  
 Area averaged loss rate (Fm) = 0.0709(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.2710  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
------------	-----------------	------------	----------	------------	----------------------------

1	53.77	19.051	11.95	0.099	3.213
2	32.67	11.110	12.15	0.071	3.181

Qmax(1) =

	1.000 *	1.000 *	53.774) +	
	1.010 *	0.983 *	32.669) + =	86.233
Qmax(2) =				
	0.990 *	1.000 *	53.774) +	
	1.000 *	1.000 *	32.669) + =	85.891

Total of 2 main streams to confluence:

Flow rates before confluence point:

54.774          33.669

Maximum flow rates at confluence using above data:

86.233          85.891

Area of streams before confluence:

19.051          11.110

Effective area values after confluence:

29.978          30.161

Results of confluence:

Total flow rate =          86.233(CFS)

Time of concentration =      11.948 min.

Effective stream area after confluence =      29.978(Ac.)

Study area average Pervious fraction(Ap) = 0.338

Study area average soil loss rate(Fm) =      0.089(In/Hr)

Study area total =          30.16(Ac.)

+++++

Process from Point/Station          199.000 to Point/Station

223.000

\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation =    563.000(Ft.)

Downstream point/station elevation =    560.000(Ft.)

Pipe length =    200.00(Ft.)    Manning's N = 0.013

No. of pipes = 1    Required pipe flow =      86.233(CFS)

Nearest computed pipe diameter =      39.00(In.)

Calculated individual pipe flow =      86.233(CFS)

Normal flow depth in pipe =    27.70(In.)

Flow top width inside pipe =    35.38(In.)

Critical Depth =    34.64(In.)

Pipe flow velocity =      13.69(Ft/s)

Travel time through pipe =      0.24 min.

Time of concentration (TC) =      12.19 min.

+++++

Process from Point/Station          224.000 to Point/Station

223.000

\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000      Max loss rate(Fm)=      0.129(In/Hr)  
Time of concentration =      12.19 min.  
Rainfall intensity =      3.174(In/Hr) for a      100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.874  
Subarea runoff =      7.211(CFS) for      3.720(Ac.)  
Total runoff =      93.444(CFS)  
Effective area this stream =      33.70(Ac.)  
Total Study Area (Main Stream No. 1) =      35.55(Ac.)  
Area averaged Fm value =      0.093(In/Hr)

++++  
++++  
Process from Point/Station      223.000 to Point/Station  
198.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation =      560.000(Ft.)  
Downstream point/station elevation =      541.000(Ft.)  
Pipe length =      200.00(Ft.)      Manning's N = 0.013  
No. of pipes = 1      Required pipe flow =      93.444(CFS)  
Nearest computed pipe diameter =      27.00(In.)  
Calculated individual pipe flow =      93.444(CFS)  
Normal flow depth in pipe =      21.63(In.)  
Flow top width inside pipe =      21.55(In.)  
Critical depth could not be calculated.  
Pipe flow velocity =      27.36(Ft/s)  
Travel time through pipe =      0.12 min.  
Time of concentration (TC) =      12.31 min.  
End of computations, Total Study Area =      35.55 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.406  
Area averaged SCS curve number = 70.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B200  
100 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 310.000 to Point/Station 309.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(11+ dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Initial subarea data:  
Initial area flow distance = 460.000(Ft.)  
Top (of initial area) elevation = 577.000(Ft.)  
Bottom (of initial area) elevation = 573.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00870 s(%)= 0.87  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.723 min.  
Rainfall intensity = 3.636(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.887  
Subarea runoff = 19.898(CFS)  
Total initial stream area = 6.170(Ac.)  
Pervious area fraction = 0.200

Initial area Fm value = 0.052(In/Hr)

++++  
Process from Point/Station 309.000 to Point/Station 307.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 573.000(Ft.)  
Downstream point elevation = 568.000(Ft.)  
Channel length thru subarea = 700.000(Ft.)  
Channel base width = 200.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 26.289(CFS)  
Manning's 'N' = 0.035  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 26.289(CFS)  
Depth of flow = 0.137(Ft.), Average velocity = 0.953(Ft/s)  
Channel flow top width = 201.649(Ft.)  
Flow Velocity = 0.95(Ft/s)  
Travel time = 12.25 min.  
Time of concentration = 21.97 min.  
Critical depth = 0.081(Ft.)

Adding area flow to channel  
APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Rainfall intensity = 2.229(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.879  
Subarea runoff = 12.705(CFS) for 10.470(Ac.)  
Total runoff = 32.603(CFS)  
Effective area this stream = 16.64(Ac.)  
Total Study Area (Main Stream No. 1) = 16.64(Ac.)  
Area averaged Fm value = 0.052(In/Hr)  
Depth of flow = 0.156(Ft.), Average velocity = 1.038(Ft/s)  
Critical depth = 0.094(Ft.)

++++  
Process from Point/Station 306.000 to Point/Station 303.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 572.000(Ft.)  
Downstream point elevation = 566.500(Ft.)  
Channel length thru subarea = 800.000(Ft.)  
Channel base width = 200.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 36.391(CFS)  
Manning's 'N' = 0.015  
Maximum depth of channel = 5.000(Ft.)

Flow(q) thru subarea = 36.391(CFS)  
Depth of flow = 0.102(Ft.), Average velocity = 1.785(Ft/s)  
Channel flow top width = 201.219(Ft.)  
Flow Velocity = 1.79(Ft/s)  
Travel time = 7.47 min.  
Time of concentration = 29.44 min.  
Critical depth = 0.101(Ft.)  
Adding area flow to channel  
RESIDENTIAL(11+ dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Rainfall intensity = 1.870(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.875  
Subarea runoff = 7.500(CFS) for 7.870(Ac.)  
Total runoff = 40.103(CFS)  
Effective area this stream = 24.51(Ac.)  
Total Study Area (Main Stream No. 1) = 24.51(Ac.)  
Area averaged Fm value = 0.052(In/Hr)  
Depth of flow = 0.108(Ft.), Average velocity = 1.855(Ft/s)  
Critical depth = 0.107(Ft.)

\*\*\*\*\*  
Process from Point/Station 303.000 to Point/Station 302.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.500(Ft.)  
Downstream point/station elevation = 566.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 40.103(CFS)  
Nearest computed pipe diameter = 30.00(In.)  
Calculated individual pipe flow = 40.103(CFS)  
Normal flow depth in pipe = 24.00(In.)  
Flow top width inside pipe = 24.00(In.)  
Critical Depth = 25.57(In.)  
Pipe flow velocity = 9.52(Ft/s)  
Travel time through pipe = 0.09 min.  
Time of concentration (TC) = 29.52 min.

\*\*\*\*\*  
Process from Point/Station 303.000 to Point/Station 302.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 1  
Stream flow area = 24.510(Ac.)  
Runoff from this stream = 40.103(CFS)  
Time of concentration = 29.52 min.  
Rainfall intensity = 1.867(In/Hr)  
Area averaged loss rate (Fm) = 0.0523(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.2000

+++++  
Process from Point/Station 305.000 to Point/Station 304.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Initial subarea data:  
Initial area flow distance = 550.000(Ft.)  
Top (of initial area) elevation = 574.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 6.000(Ft.)  
Slope = 0.01091 s(%)= 1.09  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.364 min.  
Rainfall intensity = 3.719(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.894  
Subarea runoff = 3.988(CFS)  
Total initial stream area = 1.200(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.026(In/Hr)

+++++  
Process from Point/Station 304.000 to Point/Station 302.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 566.000(Ft.)  
Pipe length = 310.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 3.988(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 3.988(CFS)  
Normal flow depth in pipe = 9.86(In.)  
Flow top width inside pipe = 14.24(In.)  
Critical Depth = 9.69(In.)  
Pipe flow velocity = 4.66(Ft/s)  
Travel time through pipe = 1.11 min.  
Time of concentration (TC) = 10.47 min.

+++++  
Process from Point/Station 304.000 to Point/Station 302.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 2  
Stream flow area = 1.200(Ac.)  
Runoff from this stream = 3.988(CFS)  
Time of concentration = 10.47 min.  
Rainfall intensity = 3.477(In/Hr)

Area averaged loss rate (Fm) = 0.0262(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.1000

\*\*\*\*\*  
 Process from Point/Station 311.000 to Point/Station 302.000  
 \*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

COMMERCIAL subarea type  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Adjusted SCS curve number for AMC 3 = 86.20  
 Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
 Initial subarea data:  
 Initial area flow distance = 745.000(Ft.)  
 Top (of initial area) elevation = 569.000(Ft.)  
 Bottom (of initial area) elevation = 566.000(Ft.)  
 Difference in elevation = 3.000(Ft.)  
 Slope = 0.00403 s(%)= 0.40  
 TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
 Initial area time of concentration = 12.905 min.  
 Rainfall intensity = 3.068(In/Hr) for a 100.0 year storm  
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.892  
 Subarea runoff = 5.748(CFS)  
 Total initial stream area = 2.100(Ac.)  
 Pervious area fraction = 0.100  
 Initial area Fm value = 0.026(In/Hr)

\*\*\*\*\*  
 Process from Point/Station 311.000 to Point/Station 302.000  
 \*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 3  
 Stream flow area = 2.100(Ac.)  
 Runoff from this stream = 5.748(CFS)  
 Time of concentration = 12.90 min.  
 Rainfall intensity = 3.068(In/Hr)  
 Area averaged loss rate (Fm) = 0.0262(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.1000  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	40.10	24.510	29.52	0.052	1.867
2	3.99	1.200	10.47	0.026	3.477
3	5.75	2.100	12.90	0.026	3.068

Qmax(1) =  
 1.000 \* 1.000 \* 40.103) +  
 0.533 \* 1.000 \* 3.988) +  
 0.605 \* 1.000 \* 5.748) + = 45.709  
 Qmax(2) =

	1.887 *	0.355 *	40.103)	+	
	1.000 *	1.000 *	3.988)	+	
	1.135 *	0.812 *	5.748)	+	= 36.127
Qmax(3) =					
	1.662 *	0.437 *	40.103)	+	
	0.881 *	1.000 *	3.988)	+	
	1.000 *	1.000 *	5.748)	+	= 38.388

Total of 3 streams to confluence:  
Flow rates before confluence point:  
40.103            3.988            5.748  
Maximum flow rates at confluence using above data:  
45.709            36.127            38.388  
Area of streams before confluence:  
24.510            1.200            2.100  
Effective area values after confluence:  
27.810            11.598            14.013

Results of confluence:  
Total flow rate = 45.709(CFS)  
Time of concentration = 29.525 min.  
Effective stream area after confluence = 27.810(Ac.)  
Study area average Pervious fraction(Ap) = 0.188  
Study area average soil loss rate(Fm) = 0.049(In/Hr)  
Study area total (this main stream) = 27.81(Ac.)

\*\*\*\*\*  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.000(Ft.)  
Downstream point/station elevation = 550.000(Ft.)  
Pipe length = 220.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 45.709(CFS)  
Nearest computed pipe diameter = 24.00(In.)  
Calculated individual pipe flow = 45.709(CFS)  
Normal flow depth in pipe = 15.49(In.)  
Flow top width inside pipe = 22.96(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 21.31(Ft/s)  
Travel time through pipe = 0.17 min.  
Time of concentration (TC) = 29.70 min.

\*\*\*\*\*  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000      Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 29.70 min.

Rainfall intensity = 1.860(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.874  
Subarea runoff = 1.791(CFS) for 1.390(Ac.)  
Total runoff = 47.501(CFS)  
Effective area this stream = 29.20(Ac.)  
Total Study Area (Main Stream No. 1) = 29.20(Ac.)  
Area averaged Fm value = 0.053(In/Hr)

\*\*\*\*\*  
Process from Point/Station 301.000 to Point/Station 300.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 550.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 47.501(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 47.501(CFS)  
Normal flow depth in pipe = 16.31(In.)  
Flow top width inside pipe = 17.49(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 23.70(Ft/s)  
Travel time through pipe = 0.07 min.  
Time of concentration (TC) = 29.77 min.

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN A300  
100 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 403.000 to Point/Station 402.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Initial subarea data:  
Initial area flow distance = 205.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)  
Difference in elevation = 2.000(Ft.)  
Slope = 0.00976 s(%)= 0.98  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 6.877 min.  
Rainfall intensity = 4.475(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.889  
Subarea runoff = 9.036(CFS)  
Total initial stream area = 2.270(Ac.)  
Pervious area fraction = 0.200

Initial area Fm value = 0.052(In/Hr)

++++  
Process from Point/Station 402.000 to Point/Station 401.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 569.000(Ft.)  
End of street segment elevation = 566.000(Ft.)  
Length of street segment = 520.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 20.000(Ft.)  
Distance from crown to crossfall grade break = 0.500(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0130  
Manning's N from gutter to grade break = 0.0130  
Manning's N from grade break to crown = 0.0130  
Estimated mean flow rate at midpoint of street = 16.154(CFS)  
Depth of flow = 0.466(Ft.), Average velocity = 2.688(Ft/s)  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 16.964(Ft.)  
Flow velocity = 2.69(Ft/s)  
Travel time = 3.22 min. TC = 10.10 min.  
Adding area flow to street  
APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.052(In/Hr)  
Rainfall intensity = 3.553(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.887  
Subarea runoff = 14.123(CFS) for 5.080(Ac.)  
Total runoff = 23.159(CFS)  
Effective area this stream = 7.35(Ac.)  
Total Study Area (Main Stream No. 1) = 7.35(Ac.)  
Area averaged Fm value = 0.052(In/Hr)  
Street flow at end of street = 23.159(CFS)  
Half street flow at end of street = 11.580(CFS)  
Depth of flow = 0.522(Ft.), Average velocity = 2.862(Ft/s)  
Warning: depth of flow exceeds top of curb  
Distance that curb overflow reaches into property = 1.10(Ft.)  
Flow width (from curb towards crown)= 19.766(Ft.)

++++  
Process from Point/Station 401.000 to Point/Station 400.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 569.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 120.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 23.159(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 23.159(CFS)  
Normal flow depth in pipe = 9.50(In.)  
Flow top width inside pipe = 14.45(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 28.24(Ft/s)  
Travel time through pipe = 0.07 min.  
Time of concentration (TC) = 10.17 min.  
End of computations, Total Study Area = 7.35 (Ac.)

The following figures may

be used for a unit hydrograph study of the same area.

Note: These figures do not consider reduced effective area effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.200

Area averaged SCS curve number = 69.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B400  
100 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.220 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 503.000 to Point/Station 502.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Initial subarea data:  
Initial area flow distance = 205.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)  
Difference in elevation = 2.000(Ft.)  
Slope = 0.00976 s(%)= 0.98  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 6.452 min.  
Rainfall intensity = 4.650(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.895  
Subarea runoff = 6.991(CFS)  
Total initial stream area = 1.680(Ac.)  
Pervious area fraction = 0.100

Initial area Fm value = 0.026(In/Hr)

++++  
Process from Point/Station 502.000 to Point/Station 501.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 569.000(Ft.)  
Downstream point elevation = 566.000(Ft.)  
Channel length thru subarea = 480.000(Ft.)  
Channel base width = 100.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 10.946(CFS)  
Manning's 'N' = 0.015  
Maximum depth of channel = 3.000(Ft.)  
Flow(q) thru subarea = 10.946(CFS)  
Depth of flow = 0.077(Ft.), Average velocity = 1.414(Ft/s)  
Channel flow top width = 100.925(Ft.)  
Flow Velocity = 1.41(Ft/s)  
Travel time = 5.66 min.  
Time of concentration = 12.11 min.  
Critical depth = 0.072(Ft.)  
Adding area flow to channel  
COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Adjusted SCS curve number for AMC 3 = 86.20  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.026(In/Hr)  
Rainfall intensity = 3.187(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.893  
Subarea runoff = 7.829(CFS) for 3.530(Ac.)  
Total runoff = 14.820(CFS)  
Effective area this stream = 5.21(Ac.)  
Total Study Area (Main Stream No. 1) = 5.21(Ac.)  
Area averaged Fm value = 0.026(In/Hr)  
Depth of flow = 0.092(Ft.), Average velocity = 1.595(Ft/s)  
Critical depth = 0.088(Ft.)

++++  
Process from Point/Station 501.000 to Point/Station 500.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 560.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 14.820(CFS)  
Nearest computed pipe diameter = 12.00(In.)  
Calculated individual pipe flow = 14.820(CFS)  
Normal flow depth in pipe = 9.16(In.)  
Flow top width inside pipe = 10.20(In.)  
Critical depth could not be calculated.

Pipe flow velocity = 23.05(Ft/s)  
Travel time through pipe = 0.07 min.  
Time of concentration (TC) = 12.18 min.  
End of computations, Total Study Area = 5.21 (Ac.)

The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.100  
Area averaged SCS curve number = 69.0

***APPENDIX B: 2-YEAR DEVELOPED CONDITION MODEL OUTPUT***

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/08/15

BASIN A100  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 105.000 to Point/Station 104.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Initial subarea data:  
Initial area flow distance = 575.000(Ft.)  
Top (of initial area) elevation = 571.500(Ft.)  
Bottom (of initial area) elevation = 570.500(Ft.)  
Difference in elevation = 1.000(Ft.)  
Slope = 0.00174 s(%)= 0.17  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 14.667 min.  
Rainfall intensity = 1.164(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.815  
Subarea runoff = 2.886(CFS)  
Total initial stream area = 3.040(Ac.)  
Pervious area fraction = 0.200  
Initial area Fm value = 0.110(In/Hr)

+++++  
Process from Point/Station 104.000 to Point/Station 103.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 570.500(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 820.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 14.000(Ft.)  
Distance from crown to crossfall grade break = 12.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [1] side(s) of the street  
Distance from curb to property line = 4.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 1.500(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150  
Estimated mean flow rate at midpoint of street = 5.474(CFS)  
Depth of flow = 0.407(Ft.), Average velocity = 2.082(Ft/s)  
Note: depth of flow exceeds top of street crown.  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 14.000(Ft.)  
Flow velocity = 2.08(Ft/s)  
Travel time = 6.56 min. TC = 21.23 min.  
Adding area flow to street  
RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Rainfall intensity = 0.933(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.715  
Subarea runoff = 5.106(CFS) for 8.940(Ac.)  
Total runoff = 7.992(CFS)  
Effective area this stream = 11.98(Ac.)  
Total Study Area (Main Stream No. 1) = 11.98(Ac.)  
Area averaged Fm value = 0.191(In/Hr)  
Street flow at end of street = 7.992(CFS)  
Half street flow at end of street = 7.992(CFS)  
Depth of flow = 0.455(Ft.), Average velocity = 2.419(Ft/s)  
Note: depth of flow exceeds top of street crown.  
Flow width (from curb towards crown)= 14.000(Ft.)

+++++  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 567.000(Ft.)

Downstream point/station elevation = 562.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 7.992(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 7.992(CFS)  
Normal flow depth in pipe = 7.97(In.)  
Flow top width inside pipe = 14.97(In.)  
Critical Depth = 13.37(In.)  
Pipe flow velocity = 12.07(Ft/s)  
Travel time through pipe = 0.14 min.  
Time of concentration (TC) = 21.37 min.

+++++  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Time of concentration = 21.37 min.  
Rainfall intensity = 0.929(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.738  
Subarea runoff = 1.991(CFS) for 2.580(Ac.)  
Total runoff = 9.982(CFS)  
Effective area this stream = 14.56(Ac.)  
Total Study Area (Main Stream No. 1) = 14.56(Ac.)  
Area averaged Fm value = 0.167(In/Hr)

+++++  
Process from Point/Station 102.000 to Point/Station 101.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 562.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 300.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 9.982(CFS)  
Nearest computed pipe diameter = 12.00(In.)  
Calculated individual pipe flow = 9.982(CFS)  
Normal flow depth in pipe = 9.19(In.)  
Flow top width inside pipe = 10.17(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 15.46(Ft/s)  
Travel time through pipe = 0.32 min.  
Time of concentration (TC) = 21.69 min.  
End of computations, Total Study Area = 14.56 (Ac.)

The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.305  
Area averaged SCS curve number = 69.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/08/15

BASIN A200  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 124.000 to Point/Station 123.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 344.000(Ft.)  
Top (of initial area) elevation = 570.500(Ft.)  
Bottom (of initial area) elevation = 567.000(Ft.)  
Difference in elevation = 3.500(Ft.)  
Slope = 0.01017 s(%)= 1.02  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.683 min.  
Rainfall intensity = 1.494(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.768  
Subarea runoff = 3.464(CFS)  
Total initial stream area = 3.020(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.219(In/Hr)

+++++  
Process from Point/Station 123.000 to Point/Station 121.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 567.000(Ft.)  
End of street segment elevation = 566.000(Ft.)  
Length of street segment = 344.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 14.000(Ft.)  
Distance from crown to crossfall grade break = 12.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [1] side(s) of the street  
Distance from curb to property line = 4.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 1.500(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150  
Estimated mean flow rate at midpoint of street = 6.380(CFS)  
Depth of flow = 0.450(Ft.), Average velocity = 1.970(Ft/s)  
Note: depth of flow exceeds top of street crown.  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 14.000(Ft.)  
Flow velocity = 1.97(Ft/s)  
Travel time = 2.91 min. TC = 12.59 min.  
Adding area flow to street  
RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Rainfall intensity = 1.276(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.745  
Subarea runoff = 5.760(CFS) for 6.680(Ac.)  
Total runoff = 9.225(CFS)  
Effective area this stream = 9.70(Ac.)  
Total Study Area (Main Stream No. 1) = 9.70(Ac.)  
Area averaged Fm value = 0.219(In/Hr)  
Street flow at end of street = 9.225(CFS)  
Half street flow at end of street = 9.225(CFS)  
Depth of flow = 0.513(Ft.), Average velocity = 2.242(Ft/s)  
Warning: depth of flow exceeds top of curb  
Note: depth of flow exceeds top of street crown.  
Distance that curb overflow reaches into property = 0.63(Ft.)  
Flow width (from curb towards crown)= 14.000(Ft.)

+++++  
Process from Point/Station 122.000 to Point/Station 121.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 12.59 min.  
Rainfall intensity = 1.276(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.745  
Subarea runoff = 3.994(CFS) for 4.200(Ac.)  
Total runoff = 13.219(CFS)  
Effective area this stream = 13.90(Ac.)  
Total Study Area (Main Stream No. 1) = 13.90(Ac.)  
Area averaged Fm value = 0.219(In/Hr)

+++++  
Process from Point/Station 122.000 to Point/Station 121.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 12.59 min.  
Rainfall intensity = 1.276(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.745  
Subarea runoff = 4.565(CFS) for 4.800(Ac.)  
Total runoff = 17.783(CFS)  
Effective area this stream = 18.70(Ac.)  
Total Study Area (Main Stream No. 1) = 18.70(Ac.)  
Area averaged Fm value = 0.219(In/Hr)

+++++  
Process from Point/Station 121.000 to Point/Station 120.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 315.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 17.783(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 17.783(CFS)  
Normal flow depth in pipe = 10.90(In.)  
Flow top width inside pipe = 13.37(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 18.63(Ft/s)  
Travel time through pipe = 0.28 min.  
Time of concentration (TC) = 12.87 min.

End of computations, Total Study Area = 18.70 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.400  
Area averaged SCS curve number = 69.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/11/15

SUBAREA A300  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 148.000 to Point/Station 145.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 524.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00763 s(%)= 0.76  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 12.136 min.  
Rainfall intensity = 1.304(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.749  
Subarea runoff = 4.874(CFS)  
Total initial stream area = 4.990(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.219(In/Hr)

+++++  
Process from Point/Station 150.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 1.304(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.765  
Subarea runoff = 0.933(CFS) for 0.830(Ac.)  
Total runoff = 5.807(CFS)  
Effective area this stream = 5.82(Ac.)  
Total Study Area (Main Stream No. 1) = 5.82(Ac.)  
Area averaged Fm value = 0.196(In/Hr)

+++++  
Process from Point/Station 150.000 to Point/Station 145.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 1  
Stream flow area = 5.820(Ac.)  
Runoff from this stream = 5.807(CFS)  
Time of concentration = 12.14 min.  
Rainfall intensity = 1.304(In/Hr)  
Area averaged loss rate (Fm) = 0.1957(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3572  
Program is now starting with Main Stream No. 2

+++++  
Process from Point/Station 149.000 to Point/Station 145.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 634.000(Ft.)  
Top (of initial area) elevation = 573.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 5.000(Ft.)  
Slope = 0.00789 s(%)= 0.79  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2

Initial area time of concentration = 13.012 min.  
 Rainfall intensity = 1.251(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.742  
 Subarea runoff = 7.838(CFS)  
 Total initial stream area = 8.440(Ac.)  
 Pervious area fraction = 0.400  
 Initial area Fm value = 0.219(In/Hr)

Process from Point/Station 149.000 to Point/Station 145.000  
 \*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

The following data inside Main Stream is listed:

In Main Stream number: 2  
 Stream flow area = 8.440(Ac.)  
 Runoff from this stream = 7.838(CFS)  
 Time of concentration = 13.01 min.  
 Rainfall intensity = 1.251(In/Hr)  
 Area averaged loss rate (Fm) = 0.2192(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.4000  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	5.81	5.820	12.14	0.196	1.304
2	7.84	8.440	13.01	0.219	1.251

Qmax(1) =  
 1.000 \* 1.000 \* 5.807) +  
 1.052 \* 0.933 \* 7.838) + = 13.496  
 Qmax(2) =  
 0.952 \* 1.000 \* 5.807) +  
 1.000 \* 1.000 \* 7.838) + = 13.366

Total of 2 main streams to confluence:  
 Flow rates before confluence point:  
 6.807 8.838  
 Maximum flow rates at confluence using above data:  
 13.496 13.366  
 Area of streams before confluence:  
 5.820 8.440  
 Effective area values after confluence:  
 13.692 14.260

Results of confluence:  
 Total flow rate = 13.496(CFS)  
 Time of concentration = 12.136 min.  
 Effective stream area after confluence = 13.692(Ac.)  
 Study area average Pervious fraction(Ap) = 0.383  
 Study area average soil loss rate(Fm) = 0.210(In/Hr)  
 Study area total = 14.26(Ac.)

\*\*\*\*\*

Process from Point/Station 147.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 1.304(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.754  
Subarea runoff = 3.922(CFS) for 4.020(Ac.)  
Total runoff = 17.418(CFS)  
Effective area this stream = 17.71(Ac.)  
Total Study Area (Main Stream No. 1) = 18.28(Ac.)  
Area averaged Fm value = 0.212(In/Hr)

++++  
Process from Point/Station 146.000 to Point/Station 145.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 12.14 min.  
Rainfall intensity = 1.304(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.753  
Subarea runoff = 4.737(CFS) for 4.850(Ac.)  
Total runoff = 22.155(CFS)  
Effective area this stream = 22.56(Ac.)  
Total Study Area (Main Stream No. 1) = 23.13(Ac.)  
Area averaged Fm value = 0.213(In/Hr)

++++  
Process from Point/Station 145.000 to Point/Station 144.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.000(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 747.000(Ft.)  
Height of curb above gutter flowline = 8.0(In.)  
Width of half street (curb to crown) = 22.000(Ft.)  
Distance from crown to crossfall grade break = 18.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)

Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 22.155(CFS)  
Half street flow before street inlet = 11.078(CFS)  
Existing pipe flow before street inlet = 0.000(CFS)  
Number of street inlets = 2  
Depth of flow = 0.651(Ft.), Average velocity = 1.626(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
Street flow half width at start of inlet = 22.000(Ft.)  
Flow rate in gutter section of street =  $Q_w = 2.260$ (CFS)  
Ratio of frontal flow to total flow =  $E_0 = 0.2040$   
Given curb inlet length  $L = 14.000$ (Ft.)  
Street slope is less than .5% , depth of flow indicates an orifice flow condition exists for an opening height of 6.00(In.)  
Using equation  $Q_i = .67hL(2gd_0)^{.5}$   
Total inlet flow capacity = 23.819(CFS)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Gutter depression depth = 0.000(In.)  
Gutter depression width = 2.000(Ft.)  
Efficiency =  $1 - (1-L/L_t)^{1.8} = 1.0000$

Note: Single inlet capacity is greater than 1/2 street flow

Pipe calculations for under street flow rate of 22.155(CFS)  
Using a pipe slope = 0.500 %  
Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 747.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 22.155(CFS)  
Nearest computed pipe diameter = 30.00(In.)  
Calculated individual pipe flow = 22.155(CFS)  
Normal flow depth in pipe = 19.64(In.)  
Flow top width inside pipe = 28.53(In.)  
Critical Depth = 19.20(In.)  
Pipe flow velocity = 6.51(Ft/s)  
Travel time through pipe = 1.91 min.  
Time of concentration (TC) = 14.05 min.  
Maximum flow rate of street inlet(s) = 22.155(CFS)  
Maximum pipe flow capacity = 22.155(CFS)  
Remaining flow in street below inlet = 0.000(CFS)  
Adding area flow to street  
COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000

Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Pervious ratio(Ap) = 0.1000      Max loss rate(Fm)=      0.055(In/Hr)  
 Rainfall intensity =      1.195(In/Hr) for a      2.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.751  
 Subarea runoff =      0.246(CFS) for      2.410(Ac.)  
 Total runoff =      22.401(CFS)  
 Effective area this stream =      24.97(Ac.)  
 Total Study Area (Main Stream No. 1) =      25.54(Ac.)  
 Area averaged Fm value =      0.198(In/Hr)  
 Street flow at end of street =      0.246(CFS)  
 Half street flow at end of street =      0.123(CFS)  
 Depth of flow =      0.181(Ft.), Average velocity =      0.609(Ft/s)  
 Flow width (from curb towards crown)=      2.736(Ft.)

++++++  
 Process from Point/Station      144.000 to Point/Station      143.000  
 \*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation =      567.000(Ft.)  
 End of street segment elevation =      566.500(Ft.)  
 Length of street segment =      300.000(Ft.)  
 Height of curb above gutter flowline =      8.0(In.)  
 Width of half street (curb to crown) =      22.000(Ft.)  
 Distance from crown to crossfall grade break =      18.000(Ft.)  
 Slope from gutter to grade break (v/hz) =      0.020  
 Slope from grade break to crown (v/hz) =      0.020  
 Street flow is on [2] side(s) of the street  
 Distance from curb to property line =      10.000(Ft.)  
 Slope from curb to property line (v/hz) =      0.025  
 Gutter width =      2.000(Ft.)  
 Gutter hike from flowline =      2.000(In.)  
 Manning's N in gutter =      0.0150  
 Manning's N from gutter to grade break =      0.0150  
 Manning's N from grade break to crown =      0.0150

Street Inlet Calculations:

Street flow before street inlet =      0.246(CFS)  
 Half street flow before street inlet =      0.123(CFS)  
 Existing pipe flow before street inlet =      22.155(CFS)  
 Number of street inlets = 2  
 Depth of flow =      0.169(Ft.), Average velocity =      0.720(Ft/s)  
 U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
 Street flow half width at start of inlet =      2.092(Ft.)  
 Flow rate in gutter section of street = Qw =      0.123(CFS)  
 Ratio of frontal flow to total flow = E0 = 1.0000  
 Given curb inlet length L =      14.000(Ft.)  
 Street slope is less than .5% , depth of flow indicates a weir flow  
 condition exists for an opening height of      8.00(In.)  
 Using equation  $Q_{weir} = 2.3(1.25 \text{ for SI}) (L + 1.8W)d^{1.5}$   
 Total inlet flow capacity=      2.800(CFS)

Half street cross section data points at curb inlet:  
    X-coordinate (Ft.)      Y-coordinate (Ft.)

0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Gutter depression depth = 0.000(In.)  
 Gutter depression width = 2.000(Ft.)  
 Efficiency =  $1 - (1-L/Lt)^{1.8} = 1.0000$

Note: Single inlet capacity is greater than 1/2 street flow

Pipe calculations for under street flow rate of 22.401(CFS)

Using a pipe slope = 0.500 %  
 Upstream point/station elevation = 567.000(Ft.)  
 Downstream point/station elevation = 566.500(Ft.)  
 Pipe length = 300.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 22.401(CFS)  
 Nearest computed pipe diameter = 30.00(In.)  
 Calculated individual pipe flow = 22.401(CFS)  
 Normal flow depth in pipe = 19.78(In.)  
 Flow top width inside pipe = 28.44(In.)  
 Critical Depth = 19.34(In.)  
 Pipe flow velocity = 6.52(Ft/s)  
 Travel time through pipe = 0.77 min.  
 Time of concentration (TC) = 14.82 min.  
 Maximum flow rate of street inlet(s) = 0.246(CFS)  
 Maximum pipe flow capacity = 22.401(CFS)  
 Remaining flow in street below inlet = 0.000(CFS)

Adding area flow to street

RESIDENTIAL(8 - 10 dwl/acre)  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
 Rainfall intensity = 1.157(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.744  
 Subarea runoff = 1.792(CFS) for 3.120(Ac.)  
 Total runoff = 24.192(CFS)  
 Effective area this stream = 28.09(Ac.)  
 Total Study Area (Main Stream No. 1) = 28.66(Ac.)  
 Area averaged Fm value = 0.200(In/Hr)  
 Street flow at end of street = 1.792(CFS)  
 Half street flow at end of street = 0.896(CFS)  
 Depth of flow = 0.314(Ft.), Average velocity = 0.892(Ft/s)  
 Flow width (from curb towards crown)= 9.366(Ft.)

\*\*\*\*\*  
 Process from Point/Station 143.000 to Point/Station 142.000  
 \*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.000(Ft.)  
 End of street segment elevation = 566.000(Ft.)  
 Length of street segment = 400.000(Ft.)

Height of curb above gutter flowline = 8.0(In.)  
 Width of half street (curb to crown) = 22.000(Ft.)  
 Distance from crown to crossfall grade break = 18.000(Ft.)  
 Slope from gutter to grade break (v/hz) = 0.020  
 Slope from grade break to crown (v/hz) = 0.020  
 Street flow is on [2] side(s) of the street  
 Distance from curb to property line = 10.000(Ft.)  
 Slope from curb to property line (v/hz) = 0.025  
 Gutter width = 2.000(Ft.)  
 Gutter hike from flowline = 2.000(In.)  
 Manning's N in gutter = 0.0150  
 Manning's N from gutter to grade break = 0.0150  
 Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 1.792(CFS)  
 Half street flow before street inlet = 0.896(CFS)  
 Existing pipe flow before street inlet = 22.401(CFS)  
 Number of street inlets = 2  
 Depth of flow = 0.272(Ft.), Average velocity = 1.372(Ft/s)  
 U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
 Street flow half width at start of inlet = 7.255(Ft.)  
 Flow rate in gutter section of street =  $Q_w$  = 0.647(CFS)  
 Ratio of frontal flow to total flow =  $E_0$  = 0.7220  
 Given curb inlet length  $L$  = 27.000(Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	0.9167 right of way
10.0000	0.6667 top of curb
10.0000	0.0000 flow line
12.0000	0.1667 gutter/depression end
14.0000	0.2067 grade break
32.0000	0.5667 crown

Length required for total flow interception =  $L_t$   
 $L_t = .6 * Q^{0.42} * Slope^{.3} * (1/(n*Se))^{.6} = 15.188(Ft.)$   
 where Manning's  $n = 0.0150$  and Slope = street slope = 0.0050  
 $Se =$  Equivalent Street x-slope including depression = 0.0200  
 Gutter depression depth = 0.000(In.)  
 Gutter depression width = 2.000(Ft.)  
 Efficiency =  $1 - (1-L/L_t)^{1.8} = 1.0000$

Pipe calculations for under street flow rate of 24.192(CFS)  
 Using a pipe slope = 1.000 %  
 Upstream point/station elevation = 568.000(Ft.)  
 Downstream point/station elevation = 566.000(Ft.)  
 Pipe length = 400.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 24.192(CFS)  
 Nearest computed pipe diameter = 27.00(In.)  
 Calculated individual pipe flow = 24.192(CFS)  
 Normal flow depth in pipe = 17.95(In.)  
 Flow top width inside pipe = 25.49(In.)  
 Critical Depth = 20.65(In.)  
 Pipe flow velocity = 8.62(Ft/s)  
 Travel time through pipe = 0.77 min.  
 Time of concentration (TC) = 15.59 min.  
 Maximum flow rate of street inlet(s) = 1.792(CFS)

Maximum pipe flow capacity = 24.192(CFS)  
 Remaining flow in street below inlet = 0.000(CFS)  
 Adding area flow to street  
 RESIDENTIAL(5 - 7 dwl/acre)  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Pervious ratio(Ap) = 0.5000 Max loss rate(Fm)= 0.274(In/Hr)  
 Rainfall intensity = 1.122(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.728  
 Subarea runoff = 4.435(CFS) for 6.960(Ac.)  
 Total runoff = 28.627(CFS)  
 Effective area this stream = 35.05(Ac.)  
 Total Study Area (Main Stream No. 1) = 35.62(Ac.)  
 Area averaged Fm value = 0.215(In/Hr)  
 Street flow at end of street = 4.435(CFS)  
 Half street flow at end of street = 2.217(CFS)  
 Depth of flow = 0.345(Ft.), Average velocity = 1.677(Ft/s)  
 Flow width (from curb towards crown)= 10.934(Ft.)

++++++  
 Process from Point/Station 142.000 to Point/Station 141.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 561.000(Ft.)  
 Downstream point/station elevation = 560.000(Ft.)  
 Pipe length = 100.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 28.627(CFS)  
 Nearest computed pipe diameter = 27.00(In.)  
 Calculated individual pipe flow = 28.627(CFS)  
 Normal flow depth in pipe = 20.48(In.)  
 Flow top width inside pipe = 23.11(In.)  
 Critical Depth = 22.30(In.)  
 Pipe flow velocity = 8.84(Ft/s)  
 Travel time through pipe = 0.19 min.  
 Time of concentration (TC) = 15.78 min.

++++++  
 Process from Point/Station 142.000 to Point/Station 141.000  
 \*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 1  
 Stream flow area = 35.052(Ac.)  
 Runoff from this stream = 28.627(CFS)  
 Time of concentration = 15.78 min.  
 Rainfall intensity = 1.114(In/Hr)  
 Area averaged loss rate (Fm) = 0.2150(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.3924  
 Program is now starting with Main Stream No. 2

+++++  
Process from Point/Station 152.000 to Point/Station 151.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 690.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 567.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00580 s(%)= 0.58  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 14.314 min.  
Rainfall intensity = 1.181(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.733  
Subarea runoff = 3.326(CFS)  
Total initial stream area = 3.840(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.219(In/Hr)

+++++  
Process from Point/Station 151.000 to Point/Station 141.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 562.000(Ft.)  
Downstream point/station elevation = 561.000(Ft.)  
Pipe length = 400.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 3.326(CFS)  
Nearest computed pipe diameter = 18.00(In.)  
Calculated individual pipe flow = 3.326(CFS)  
Normal flow depth in pipe = 10.39(In.)  
Flow top width inside pipe = 17.78(In.)  
Critical Depth = 8.34(In.)  
Pipe flow velocity = 3.15(Ft/s)  
Travel time through pipe = 2.12 min.  
Time of concentration (TC) = 16.43 min.

+++++  
Process from Point/Station 151.000 to Point/Station 141.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:  
In Main Stream number: 2  
Stream flow area = 3.840(Ac.)  
Runoff from this stream = 3.326(CFS)  
Time of concentration = 16.43 min.  
Rainfall intensity = 1.087(In/Hr)  
Area averaged loss rate (Fm) = 0.2192(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.4000

Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	28.63	35.052	15.78	0.215	1.114
2	3.33	3.840	16.43	0.219	1.087
Qmax(1) =					
	1.000 *	1.000 *	28.627)	+	
	1.031 *	0.960 *	3.326)	+ =	31.919
Qmax(2) =					
	0.970 *	1.000 *	28.627)	+	
	1.000 *	1.000 *	3.326)	+ =	31.096

Total of 2 main streams to confluence:

Flow rates before confluence point:

29.627            4.326

Maximum flow rates at confluence using above data:

31.919            31.096

Area of streams before confluence:

35.052            3.840

Effective area values after confluence:

38.738            38.892

Results of confluence:

Total flow rate = 31.919(CFS)

Time of concentration = 15.778 min.

Effective stream area after confluence = 38.738(Ac.)

Study area average Pervious fraction(Ap) = 0.393

Study area average soil loss rate(Fm) = 0.215(In/Hr)

Study area total = 38.89(Ac.)

++++  
 Process from Point/Station 141.000 to Point/Station 140.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 561.000(Ft.)  
 Downstream point/station elevation = 555.000(Ft.)  
 Pipe length = 250.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 31.919(CFS)  
 Nearest computed pipe diameter = 24.00(In.)  
 Calculated individual pipe flow = 31.919(CFS)  
 Normal flow depth in pipe = 17.98(In.)  
 Flow top width inside pipe = 20.81(In.)  
 Critical Depth = 22.67(In.)  
 Pipe flow velocity = 12.64(Ft/s)  
 Travel time through pipe = 0.33 min.  
 Time of concentration (TC) = 16.11 min.

++++  
 Process from Point/Station 141.000 to Point/Station 140.000  
 \*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio( $A_p$ ) = 1.0000      Max loss rate( $F_m$ )=      0.387(In/Hr)  
The area added to the existing stream causes a  
a lower flow rate of  $Q = 31.410$ (CFS)  
therefore the upstream flow rate of  $Q = 31.919$ (CFS) is being used  
Time of concentration = 16.11 min.  
Rainfall intensity = 1.101(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)( $Q=KCIA$ ) is  $C = 0.721$   
Subarea runoff = 0.000(CFS) for 0.850(Ac.)  
Total runoff = 31.919(CFS)  
Effective area this stream = 39.59(Ac.)  
Total Study Area (Main Stream No. 1) = 40.31(Ac.)  
Area averaged  $F_m$  value = 0.219(In/Hr)  
End of computations, Total Study Area = 40.31 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.406  
Area averaged SCS curve number = 69.2

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/08/15

SUBAREA BASIN A400  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 162.000 to Point/Station 161.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Initial subarea data:  
Initial area flow distance = 1000.000(Ft.)  
Top (of initial area) elevation = 570.000(Ft.)  
Bottom (of initial area) elevation = 549.000(Ft.)  
Difference in elevation = 21.000(Ft.)  
Slope = 0.02100 s(%)= 2.10  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 24.230 min.  
Rainfall intensity = 0.861(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.496  
Subarea runoff = 0.684(CFS)  
Total initial stream area = 1.600(Ac.)  
Pervious area fraction = 1.000  
Initial area Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 161.000 to Point/Station 160.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Time of concentration = 24.23 min.  
Rainfall intensity = 0.861(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.695  
Subarea runoff = 1.561(CFS) for 2.150(Ac.)  
Total runoff = 2.245(CFS)  
Effective area this stream = 3.75(Ac.)  
Total Study Area (Main Stream No. 1) = 3.75(Ac.)  
Area averaged Fm value = 0.196(In/Hr)

+++++  
Process from Point/Station 160.000 to Point/Station 150.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 544.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 2.245(CFS)  
Nearest computed pipe diameter = 6.00(In.)  
Calculated individual pipe flow = 2.245(CFS)  
Normal flow depth in pipe = 4.63(In.)  
Flow top width inside pipe = 5.03(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 13.79(Ft/s)  
Travel time through pipe = 0.06 min.  
Time of concentration (TC) = 24.29 min.  
End of computations, Total Study Area = 3.75 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.484  
Area averaged SCS curve number = 73.3

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/11/15

SUBAREA A500  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 171.000 to Point/Station 170.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Initial subarea data:  
Initial area flow distance = 1000.000(Ft.)  
Top (of initial area) elevation = 566.000(Ft.)  
Bottom (of initial area) elevation = 553.000(Ft.)  
Difference in elevation = 13.000(Ft.)  
Slope = 0.01300 s(%)= 1.30  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 11.484 min.  
Rainfall intensity = 1.348(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.863  
Subarea runoff = 1.490(CFS)  
Total initial stream area = 1.280(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.055(In/Hr)

+++++  
Process from Point/Station 170.000 to Point/Station 150.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 553.000(Ft.)  
Downstream point/station elevation = 535.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 1.490(CFS)  
Nearest computed pipe diameter = 6.00(In.)  
Calculated individual pipe flow = 1.490(CFS)  
Normal flow depth in pipe = 2.79(In.)  
Flow top width inside pipe = 5.99(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 16.62(Ft/s)  
Travel time through pipe = 0.05 min.  
Time of concentration (TC) = 11.53 min.

+++++  
Process from Point/Station 151.000 to Point/Station 150.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 11.53 min.  
Rainfall intensity = 1.345(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.753  
Subarea runoff = 1.082(CFS) for 1.260(Ac.)  
Total runoff = 2.573(CFS)  
Effective area this stream = 2.54(Ac.)  
Total Study Area (Main Stream No. 1) = 2.54(Ac.)  
Area averaged Fm value = 0.219(In/Hr)  
End of computations, Total Study Area = 2.54 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 0.546  
Area averaged SCS curve number = 74.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B100  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 205.000 to Point/Station 204.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 436.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 568.500(Ft.)  
Difference in elevation = 3.500(Ft.)  
Slope = 0.00803 s(%)= 0.80  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 11.162 min.  
Rainfall intensity = 1.372(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.756  
Subarea runoff = 3.163(CFS)  
Total initial stream area = 3.050(Ac.)  
Pervious area fraction = 0.400  
Initial area Fm value = 0.219(In/Hr)

++++  
Process from Point/Station 206.000 to Point/Station 204.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 11.16 min.  
Rainfall intensity = 1.372(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.756  
Subarea runoff = 3.246(CFS) for 3.130(Ac.)  
Total runoff = 6.410(CFS)  
Effective area this stream = 6.18(Ac.)  
Total Study Area (Main Stream No. 1) = 6.18(Ac.)  
Area averaged Fm value = 0.219(In/Hr)

++++  
Process from Point/Station 150.000 to Point/Station 204.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Time of concentration = 11.16 min.  
Rainfall intensity = 1.372(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.765  
Subarea runoff = 0.687(CFS) for 0.580(Ac.)  
Total runoff = 7.097(CFS)  
Effective area this stream = 6.76(Ac.)  
Total Study Area (Main Stream No. 1) = 6.76(Ac.)  
Area averaged Fm value = 0.205(In/Hr)

++++  
Process from Point/Station 204.000 to Point/Station 203.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.500(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 100.000(Ft.)  
Height of curb above gutter flowline = 8.0(In.)  
Width of half street (curb to crown) = 22.000(Ft.)  
Distance from crown to crossfall grade break = 18.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020

Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 7.097(CFS)  
Half street flow before street inlet = 3.548(CFS)  
Existing pipe flow before street inlet = 0.000(CFS)  
Number of street inlets = 2  
Depth of flow = 0.556(Ft.), Average velocity = 3.007(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
Street flow half width at start of inlet = 8.963(Ft.)  
Flow rate in gutter section of street =  $Q_w$  = 2.698(CFS)  
Ratio of frontal flow to total flow =  $E_0$  = 0.7602  
Given curb inlet length  $L$  = 14.000(Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	1.1667 right of way
10.0000	0.9167 top of curb
10.0000	0.0000 flow line
12.0000	0.4167 gutter/depression end
14.0000	0.4567 grade break
32.0000	0.8167 crown

Length required for total flow interception =  $L_t$   
 $L_t = .6 * Q^{0.42} * \text{Slope}^{.3} * (1/(n*Se))^{.6} = 13.178(\text{Ft.})$   
where Manning's  $n = 0.0150$  and Slope = street slope = 0.0150  
 $Se = \text{Equivalent Street x-slope including depression} = 0.1150$   
Gutter depression depth = 3.000(In.)  
Gutter depression width = 2.000(Ft.)  
Efficiency =  $1 - (1-L/L_t)^{1.8} = 1.0000$

Pipe calculations for under street flow rate of 7.097(CFS)  
Using a pipe slope = 1.500 %  
Upstream point/station elevation = 568.500(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 7.097(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 7.097(CFS)  
Normal flow depth in pipe = 11.09(In.)  
Flow top width inside pipe = 13.17(In.)  
Critical Depth = 12.79(In.)  
Pipe flow velocity = 7.29(Ft/s)  
Travel time through pipe = 0.23 min.  
Time of concentration (TC) = 11.39 min.  
Maximum flow rate of street inlet(s) = 7.097(CFS)  
Maximum pipe flow capacity = 7.097(CFS)  
Remaining flow in street below inlet = 0.000(CFS)  
Adding area flow to street  
COMMERCIAL subarea type

Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Rainfall intensity = 1.355(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.769  
Subarea runoff = 0.367(CFS) for 0.400(Ac.)  
Total runoff = 7.464(CFS)  
Effective area this stream = 7.16(Ac.)  
Total Study Area (Main Stream No. 1) = 7.16(Ac.)  
Area averaged Fm value = 0.197(In/Hr)  
Street flow at end of street = 0.367(CFS)  
Half street flow at end of street = 0.184(CFS)  
Depth of flow = 0.129(Ft.), Average velocity = 1.844(Ft/s)  
Flow width (from curb towards crown)= 2.000(Ft.)

++++  
Process from Point/Station 203.000 to Point/Station 202.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 1  
Stream flow area = 7.160(Ac.)  
Runoff from this stream = 7.464(CFS)  
Time of concentration = 11.39 min.  
Rainfall intensity = 1.355(In/Hr)  
Area averaged loss rate (Fm) = 0.1967(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3589

++++  
Process from Point/Station 207.000 to Point/Station 203.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 394.000(Ft.)  
Top (of initial area) elevation = 571.200(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 3.200(Ft.)  
Slope = 0.00812 s(%)= 0.81  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 10.694 min.  
Rainfall intensity = 1.407(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.760  
Subarea runoff = 2.310(CFS)  
Total initial stream area = 2.160(Ac.)  
Pervious area fraction = 0.400

Initial area Fm value = 0.219(In/Hr)

++++  
Process from Point/Station 203.000 to Point/Station 202.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 567.500(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 2.310(CFS)  
Nearest computed pipe diameter = 12.00(In.)  
Calculated individual pipe flow = 2.310(CFS)  
Normal flow depth in pipe = 7.03(In.)  
Flow top width inside pipe = 11.82(In.)  
Critical Depth = 7.81(In.)  
Pipe flow velocity = 4.83(Ft/s)  
Travel time through pipe = 0.17 min.  
Time of concentration (TC) = 10.87 min.

++++  
Process from Point/Station 203.000 to Point/Station 202.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 2  
Stream flow area = 2.160(Ac.)  
Runoff from this stream = 2.310(CFS)  
Time of concentration = 10.87 min.  
Rainfall intensity = 1.394(In/Hr)  
Area averaged loss rate (Fm) = 0.2192(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.4000

++++  
Process from Point/Station 208.000 to Point/Station 203.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Initial subarea data:  
Initial area flow distance = 540.000(Ft.)  
Top (of initial area) elevation = 572.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)  
Difference in elevation = 3.000(Ft.)  
Slope = 0.00556 s(%)= 0.56  
TC = k(0.374)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 13.088 min.  
Rainfall intensity = 1.247(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.742  
Subarea runoff = 2.608(CFS)  
Total initial stream area = 2.820(Ac.)

Pervious area fraction = 0.400  
 Initial area Fm value = 0.219(In/Hr)

\*\*\*\*\*  
 Process from Point/Station 203.000 to Point/Station 202.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 569.000(Ft.)  
 Downstream point/station elevation = 568.500(Ft.)  
 Pipe length = 50.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 2.608(CFS)  
 Nearest computed pipe diameter = 12.00(In.)  
 Calculated individual pipe flow = 2.608(CFS)  
 Normal flow depth in pipe = 7.63(In.)  
 Flow top width inside pipe = 11.55(In.)  
 Critical Depth = 8.30(In.)  
 Pipe flow velocity = 4.95(Ft/s)  
 Travel time through pipe = 0.17 min.  
 Time of concentration (TC) = 13.26 min.

\*\*\*\*\*  
 Process from Point/Station 203.000 to Point/Station 202.000  
 \*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 3  
 Stream flow area = 2.820(Ac.)  
 Runoff from this stream = 2.608(CFS)  
 Time of concentration = 13.26 min.  
 Rainfall intensity = 1.237(In/Hr)  
 Area averaged loss rate (Fm) = 0.2192(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.4000  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	7.46	7.160	11.39	0.197	1.355
2	2.31	2.160	10.87	0.219	1.394
3	2.61	2.820	13.26	0.219	1.237

Qmax(1) =  
 1.000 \* 1.000 \* 7.464) +  
 0.967 \* 1.000 \* 2.310) +  
 1.116 \* 0.859 \* 2.608) + = 12.198  
 Qmax(2) =  
 1.034 \* 0.954 \* 7.464) +  
 1.000 \* 1.000 \* 2.310) +  
 1.154 \* 0.820 \* 2.608) + = 12.136  
 Qmax(3) =  
 0.898 \* 1.000 \* 7.464) +  
 0.867 \* 1.000 \* 2.310) +  
 1.000 \* 1.000 \* 2.608) + = 11.314

Total of 3 streams to confluence:  
 Flow rates before confluence point:

7.464 2.310 2.608  
Maximum flow rates at confluence using above data:  
12.198 12.136 11.314

Area of streams before confluence:  
7.160 2.160 2.820

Effective area values after confluence:  
11.743 11.302 12.140

Results of confluence:  
Total flow rate = 12.198(CFS)  
Time of concentration = 11.391 min.  
Effective stream area after confluence = 11.743(Ac.)  
Study area average Pervious fraction(Ap) = 0.376  
Study area average soil loss rate(Fm) = 0.206(In/Hr)  
Study area total (this main stream) = 12.14(Ac.)

+++++  
Process from Point/Station 202.000 to Point/Station 201.000  
\*\*\*\* STREET INLET + AREA + PIPE TRAVEL TIME \*\*\*\*

---

Top of street segment elevation = 568.500(Ft.)  
End of street segment elevation = 567.000(Ft.)  
Length of street segment = 180.000(Ft.)  
Height of curb above gutter flowline = 8.0(In.)  
Width of half street (curb to crown) = 22.000(Ft.)  
Distance from crown to crossfall grade break = 18.000(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.025  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0150  
Manning's N from gutter to grade break = 0.0150  
Manning's N from grade break to crown = 0.0150

Street Inlet Calculations:

Street flow before street inlet = 0.183(CFS)  
Half street flow before street inlet = 0.092(CFS)  
Existing pipe flow before street inlet = 12.014(CFS)  
Number of street inlets = 2  
Depth of flow = 0.161(Ft.), Average velocity = 1.470(Ft/s)  
U.S. DOT Hydraulic Engineering Circular No. 12 curb inlet calculations:  
Street flow half width at start of inlet = 2.000(Ft.)  
Flow rate in gutter section of street = Qw = 0.092(CFS)  
Ratio of frontal flow to total flow = E0 = 1.0000  
Given curb inlet length L = 14.000(Ft.)

Half street cross section data points at curb inlet:

X-coordinate (Ft.)	Y-coordinate (Ft.)
0.0000	1.1667 right of way
10.0000	0.9167 top of curb
10.0000	0.0000 flow line
12.0000	0.4167 gutter/depression end
14.0000	0.4567 grade break
32.0000	0.8167 crown

Length required for total flow interception = Lt  
 $Lt = .6 * Q^{0.42} * Slope^{.3} * (1/(n*Se))^{.6} = 2.070(\text{Ft.})$   
 where Manning's n = 0.0150 and Slope = street slope = 0.0083  
 Se = Equivalent Street x-slope including depression = 0.1450  
 Gutter depression depth = 3.000(In.)  
 Gutter depression width = 2.000(Ft.)  
 Efficiency =  $1 - (1-L/Lt)^{1.8} = 1.0000$

Pipe calculations for under street flow rate of 12.198(CFS)  
 Using a pipe slope = 0.833 %  
 Upstream point/station elevation = 568.500(Ft.)  
 Downstream point/station elevation = 567.000(Ft.)  
 Pipe length = 180.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 12.198(CFS)  
 Nearest computed pipe diameter = 21.00(In.)  
 Calculated individual pipe flow = 12.198(CFS)  
 Normal flow depth in pipe = 14.79(In.)  
 Flow top width inside pipe = 19.17(In.)  
 Critical Depth = 15.62(In.)  
 Pipe flow velocity = 6.74(Ft/s)  
 Travel time through pipe = 0.45 min.  
 Time of concentration (TC) = 11.84 min.  
 Maximum flow rate of street inlet(s) = 0.183(CFS)  
 Maximum pipe flow capacity = 12.198(CFS)  
 Remaining flow in street below inlet = 0.000(CFS)

Adding area flow to street  
 COMMERCIAL subarea type  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
 Rainfall intensity = 1.324(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area,(total area with modified  
 rational method)(Q=KCIA) is C = 0.764  
 Subarea runoff = 0.147(CFS) for 0.460(Ac.)  
 Total runoff = 12.344(CFS)  
 Effective area this stream = 12.20(Ac.)  
 Total Study Area (Main Stream No. 1) = 12.60(Ac.)  
 Area averaged Fm value = 0.200(In/Hr)  
 Street flow at end of street = 0.147(CFS)  
 Half street flow at end of street = 0.073(CFS)  
 Depth of flow = 0.102(Ft.), Average velocity = 1.176(Ft/s)  
 Flow width (from curb towards crown)= 2.000(Ft.)

\*\*\*\*\*  
 Process from Point/Station 201.000 to Point/Station 200.000  
 \*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 567.000(Ft.)  
 Downstream point/station elevation = 566.500(Ft.)  
 Pipe length = 60.00(Ft.) Manning's N = 0.013  
 No. of pipes = 1 Required pipe flow = 12.344(CFS)  
 Nearest computed pipe diameter = 21.00(In.)  
 Calculated individual pipe flow = 12.344(CFS)

Normal flow depth in pipe = 14.93(In.)  
Flow top width inside pipe = 19.04(In.)  
Critical Depth = 15.70(In.)  
Pipe flow velocity = 6.76(Ft/s)  
Travel time through pipe = 0.15 min.  
Time of concentration (TC) = 11.98 min.

\*\*\*\*\*  
Process from Point/Station 210.000 to Point/Station 200.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 11.98 min.  
Rainfall intensity = 1.314(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.760  
Subarea runoff = 3.628(CFS) for 3.790(Ac.)  
Total runoff = 15.972(CFS)  
Effective area this stream = 15.99(Ac.)  
Total Study Area (Main Stream No. 1) = 16.39(Ac.)  
Area averaged Fm value = 0.205(In/Hr)

\*\*\*\*\*  
Process from Point/Station 209.000 to Point/Station 200.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 11.98 min.  
Rainfall intensity = 1.314(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.758  
Subarea runoff = 3.016(CFS) for 3.060(Ac.)  
Total runoff = 18.988(CFS)  
Effective area this stream = 19.05(Ac.)  
Total Study Area (Main Stream No. 1) = 19.45(Ac.)  
Area averaged Fm value = 0.207(In/Hr)

\*\*\*\*\*  
Process from Point/Station 200.000 to Point/Station 199.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.500(Ft.)

Downstream point/station elevation = 565.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 18.988(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 18.988(CFS)  
Normal flow depth in pipe = 16.83(In.)  
Flow top width inside pipe = 16.76(In.)  
Critical Depth = 18.88(In.)  
Pipe flow velocity = 9.20(Ft/s)  
Travel time through pipe = 0.18 min.  
Time of concentration (TC) = 12.17 min.

\*\*\*\*\*  
Process from Point/Station 200.000 to Point/Station 199.000  
\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*

---

The following data inside Main Stream is listed:

In Main Stream number: 1  
Stream flow area = 19.053(Ac.)  
Runoff from this stream = 18.988(CFS)  
Time of concentration = 12.17 min.  
Rainfall intensity = 1.303(In/Hr)  
Area averaged loss rate (Fm) = 0.2070(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3778  
Program is now starting with Main Stream No. 2

\*\*\*\*\*  
Process from Point/Station 222.000 to Point/Station 221.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Initial subarea data:  
Initial area flow distance = 690.000(Ft.)  
Top (of initial area) elevation = 570.500(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 2.500(Ft.)  
Slope = 0.00362 s(%)= 0.36  
TC =  $k(0.324)*[(length^3)/(elevation\ change)]^{0.2}$   
Initial area time of concentration = 13.623 min.  
Rainfall intensity = 1.217(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.819  
Subarea runoff = 4.027(CFS)  
Total initial stream area = 4.040(Ac.)  
Pervious area fraction = 0.200  
Initial area Fm value = 0.110(In/Hr)

\*\*\*\*\*  
Process from Point/Station 221.000 to Point/Station 214.000

\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.500(Ft.)  
Downstream point/station elevation = 568.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 4.027(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 4.027(CFS)  
Normal flow depth in pipe = 8.58(In.)  
Flow top width inside pipe = 14.84(In.)  
Critical Depth = 9.74(In.)  
Pipe flow velocity = 5.55(Ft/s)  
Travel time through pipe = 0.15 min.  
Time of concentration (TC) = 13.77 min.

+++++  
Process from Point/Station 215.000 to Point/Station 214.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Time of concentration = 13.77 min.  
Rainfall intensity = 1.209(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.825  
Subarea runoff = 0.833(CFS) for 0.830(Ac.)  
Total runoff = 4.860(CFS)  
Effective area this stream = 4.87(Ac.)  
Total Study Area (Main Stream No. 2) = 24.32(Ac.)  
Area averaged Fm value = 0.100(In/Hr)

+++++  
Process from Point/Station 214.000 to Point/Station 212.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.200(Ft.)  
Downstream point/station elevation = 567.000(Ft.)  
Pipe length = 400.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 4.860(CFS)  
Nearest computed pipe diameter = 18.00(In.)  
Calculated individual pipe flow = 4.860(CFS)  
Normal flow depth in pipe = 12.68(In.)  
Flow top width inside pipe = 16.43(In.)  
Critical Depth = 10.17(In.)  
Pipe flow velocity = 3.65(Ft/s)  
Travel time through pipe = 1.83 min.  
Time of concentration (TC) = 15.60 min.

+++++

Process from Point/Station 213.000 to Point/Station 212.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 2 in normal stream number 1  
Stream flow area = 4.870(Ac.)  
Runoff from this stream = 4.860(CFS)  
Time of concentration = 15.60 min.  
Rainfall intensity = 1.122(In/Hr)  
Area averaged loss rate (Fm) = 0.1002(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.1830

+++++  
Process from Point/Station 218.000 to Point/Station 217.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Initial subarea data:  
Initial area flow distance = 489.000(Ft.)  
Top (of initial area) elevation = 568.000(Ft.)  
Bottom (of initial area) elevation = 566.000(Ft.)  
Difference in elevation = 2.000(Ft.)  
Slope = 0.00409 s(%)= 0.41  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 10.871 min.  
Rainfall intensity = 1.394(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.865  
Subarea runoff = 1.241(CFS)  
Total initial stream area = 1.030(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.055(In/Hr)

+++++  
Process from Point/Station 217.000 to Point/Station 216.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.000(Ft.)  
Downstream point/station elevation = 565.800(Ft.)  
Pipe length = 30.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 1.241(CFS)  
Nearest computed pipe diameter = 9.00(In.)  
Calculated individual pipe flow = 1.241(CFS)  
Normal flow depth in pipe = 6.80(In.)  
Flow top width inside pipe = 7.74(In.)  
Critical Depth = 6.15(In.)  
Pipe flow velocity = 3.47(Ft/s)  
Travel time through pipe = 0.14 min.  
Time of concentration (TC) = 11.01 min.

Process from Point/Station 220.000 to Point/Station 216.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

RESIDENTIAL(8 - 10 dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.4000 Max loss rate(Fm)= 0.219(In/Hr)  
Time of concentration = 11.01 min.  
Rainfall intensity = 1.383(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.774  
Subarea runoff = 6.021(CFS) for 5.760(Ac.)  
Total runoff = 7.262(CFS)  
Effective area this stream = 6.79(Ac.)  
Total Study Area (Main Stream No. 2) = 31.11(Ac.)  
Area averaged Fm value = 0.194(In/Hr)

Process from Point/Station 216.000 to Point/Station 212.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

Upstream point/station elevation = 565.800(Ft.)  
Downstream point/station elevation = 565.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 7.262(CFS)  
Nearest computed pipe diameter = 18.00(In.)  
Calculated individual pipe flow = 7.262(CFS)  
Normal flow depth in pipe = 11.88(In.)  
Flow top width inside pipe = 17.05(In.)  
Critical Depth = 12.53(In.)  
Pipe flow velocity = 5.87(Ft/s)  
Travel time through pipe = 0.28 min.  
Time of concentration (TC) = 11.30 min.

Process from Point/Station 216.000 to Point/Station 212.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

Along Main Stream number: 2 in normal stream number 2  
Stream flow area = 6.790(Ac.)  
Runoff from this stream = 7.262(CFS)  
Time of concentration = 11.30 min.  
Rainfall intensity = 1.362(In/Hr)  
Area averaged loss rate (Fm) = 0.1942(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.3545  
Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
------------	-----------------	------------	----------	------------	----------------------------

1	4.86	4.870	15.60	0.100	1.122
2	7.26	6.790	11.30	0.194	1.362

Qmax(1) =

1.000 *	1.000 *	4.860) +	
0.795 *	1.000 *	7.262) + =	10.631

Qmax(2) =

1.234 *	0.724 *	4.860) +	
1.000 *	1.000 *	7.262) + =	11.607

Total of 2 streams to confluence:  
Flow rates before confluence point:

4.860            7.262

Maximum flow rates at confluence using above data:

10.631            11.607

Area of streams before confluence:

4.870            6.790

Effective area values after confluence:

11.660            10.317

Results of confluence:

Total flow rate =        11.607(CFS)  
Time of concentration =    11.299 min.  
Effective stream area after confluence =    10.317(Ac.)  
Study area average Pervious fraction(Ap) = 0.283  
Study area average soil loss rate(Fm) =    0.155(In/Hr)  
Study area total (this main stream) =        11.66(Ac.)

\*\*\*\*\*  
Process from Point/Station        212.000 to Point/Station        211.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000        Max loss rate(Fm)=        0.055(In/Hr)  
Time of concentration =    11.30 min.  
Rainfall intensity =        1.362(In/Hr) for a        2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.802  
Subarea runoff =        0.444(CFS) for        0.720(Ac.)  
Total runoff =        12.051(CFS)  
Effective area this stream =        11.04(Ac.)  
Total Study Area (Main Stream No. 2) =        31.83(Ac.)  
Area averaged Fm value =    0.148(In/Hr)

\*\*\*\*\*  
Process from Point/Station        212.000 to Point/Station        211.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation =    565.000(Ft.)  
Downstream point/station elevation =    563.000(Ft.)  
Pipe length =    400.00(Ft.)        Manning's N = 0.013  
No. of pipes = 1    Required pipe flow =        12.051(CFS)

Nearest computed pipe diameter = 24.00(In.)  
 Calculated individual pipe flow = 12.051(CFS)  
 Normal flow depth in pipe = 15.56(In.)  
 Flow top width inside pipe = 22.92(In.)  
 Critical Depth = 14.96(In.)  
 Pipe flow velocity = 5.59(Ft/s)  
 Travel time through pipe = 1.19 min.  
 Time of concentration (TC) = 12.49 min.

++++++  
 Process from Point/Station 211.000 to Point/Station 199.000  
 \*\*\*\*\* CONFLUENCE OF MAIN STREAMS \*\*\*\*\*

The following data inside Main Stream is listed:

In Main Stream number: 2  
 Stream flow area = 11.037(Ac.)  
 Runoff from this stream = 12.051(CFS)  
 Time of concentration = 12.49 min.  
 Rainfall intensity = 1.282(In/Hr)  
 Area averaged loss rate (Fm) = 0.1484(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.2709  
 Summary of stream data:

Stream No.	Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	18.99	19.053	12.17	0.207	1.303
2	12.05	11.037	12.49	0.148	1.282

Qmax(1) =  
 1.000 \* 1.000 \* 18.988) +  
 1.018 \* 0.974 \* 12.051) + = 30.937  
 Qmax(2) =  
 0.981 \* 1.000 \* 18.988) +  
 1.000 \* 1.000 \* 12.051) + = 30.684

Total of 2 main streams to confluence:  
 Flow rates before confluence point:  
 19.988 13.051  
 Maximum flow rates at confluence using above data:  
 30.937 30.684  
 Area of streams before confluence:  
 19.053 11.037  
 Effective area values after confluence:  
 29.803 30.091

Results of confluence:  
 Total flow rate = 30.937(CFS)  
 Time of concentration = 12.165 min.  
 Effective stream area after confluence = 29.803(Ac.)  
 Study area average Pervious fraction(Ap) = 0.339  
 Study area average soil loss rate(Fm) = 0.186(In/Hr)  
 Study area total = 30.09(Ac.)

\*\*\*\*\*  
Process from Point/Station 199.000 to Point/Station 223.000  
\*\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*\*

---

Upstream point/station elevation = 563.000(Ft.)  
Downstream point/station elevation = 560.000(Ft.)  
Pipe length = 200.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 30.937(CFS)  
Nearest computed pipe diameter = 27.00(In.)  
Calculated individual pipe flow = 30.937(CFS)  
Normal flow depth in pipe = 18.54(In.)  
Flow top width inside pipe = 25.05(In.)  
Critical Depth = 23.06(In.)  
Pipe flow velocity = 10.64(Ft/s)  
Travel time through pipe = 0.31 min.  
Time of concentration (TC) = 12.48 min.

\*\*\*\*\*  
Process from Point/Station 224.000 to Point/Station 223.000  
\*\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 12.48 min.  
Rainfall intensity = 1.283(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.754  
Subarea runoff = 1.495(CFS) for 3.720(Ac.)  
Total runoff = 32.432(CFS)  
Effective area this stream = 33.52(Ac.)  
Total Study Area (Main Stream No. 1) = 35.55(Ac.)  
Area averaged Fm value = 0.208(In/Hr)

\*\*\*\*\*  
Process from Point/Station 223.000 to Point/Station 198.000  
\*\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*\*

---

Upstream point/station elevation = 560.000(Ft.)  
Downstream point/station elevation = 541.000(Ft.)  
Pipe length = 200.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 32.432(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 32.432(CFS)  
Normal flow depth in pipe = 12.50(In.)  
Flow top width inside pipe = 20.61(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 21.72(Ft/s)  
Travel time through pipe = 0.15 min.  
Time of concentration (TC) = 12.63 min.  
End of computations, Total Study Area = 35.55 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.

Note: These figures do not consider reduced effective area effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.406

Area averaged SCS curve number = 70.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B200  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 310.000 to Point/Station 309.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

RESIDENTIAL(11+ dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Initial subarea data:  
Initial area flow distance = 460.000(Ft.)  
Top (of initial area) elevation = 577.000(Ft.)  
Bottom (of initial area) elevation = 573.000(Ft.)  
Difference in elevation = 4.000(Ft.)  
Slope = 0.00870 s(%)= 0.87  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.723 min.  
Rainfall intensity = 1.490(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.834  
Subarea runoff = 7.665(CFS)  
Total initial stream area = 6.170(Ac.)  
Pervious area fraction = 0.200  
Initial area Fm value = 0.110(In/Hr)

+++++  
Process from Point/Station 309.000 to Point/Station 307.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 573.000(Ft.)  
Downstream point elevation = 568.000(Ft.)  
Channel length thru subarea = 700.000(Ft.)  
Channel base width = 200.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 8.893(CFS)  
Manning's 'N' = 0.035  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 8.893(CFS)  
Depth of flow = 0.072(Ft.), Average velocity = 0.619(Ft/s)  
Channel flow top width = 200.861(Ft.)  
Flow Velocity = 0.62(Ft/s)  
Travel time = 18.86 min.  
Time of concentration = 28.58 min.  
Critical depth = 0.040(Ft.)  
Adding area flow to channel  
APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Rainfall intensity = 0.780(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.774  
Subarea runoff = 2.377(CFS) for 10.470(Ac.)  
Total runoff = 10.043(CFS)  
Effective area this stream = 16.64(Ac.)  
Total Study Area (Main Stream No. 1) = 16.64(Ac.)  
Area averaged Fm value = 0.110(In/Hr)  
Depth of flow = 0.077(Ft.), Average velocity = 0.649(Ft/s)  
Critical depth = 0.043(Ft.)

+++++  
Process from Point/Station 306.000 to Point/Station 303.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 572.000(Ft.)  
Downstream point elevation = 566.500(Ft.)  
Channel length thru subarea = 800.000(Ft.)  
Channel base width = 200.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 10.805(CFS)  
Manning's 'N' = 0.015  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 10.805(CFS)  
Depth of flow = 0.049(Ft.), Average velocity = 1.100(Ft/s)

Channel flow top width = 200.589(Ft.)  
Flow Velocity = 1.10(Ft/s)  
Travel time = 12.13 min.  
Time of concentration = 40.71 min.  
Critical depth = 0.045(Ft.)  
Adding area flow to channel  
RESIDENTIAL(11+ dwl/acre)  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Rainfall intensity = 0.631(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.744  
Subarea runoff = 1.460(CFS) for 7.870(Ac.)  
Total runoff = 11.502(CFS)  
Effective area this stream = 24.51(Ac.)  
Total Study Area (Main Stream No. 1) = 24.51(Ac.)  
Area averaged Fm value = 0.110(In/Hr)  
Depth of flow = 0.051(Ft.), Average velocity = 1.127(Ft/s)  
Critical depth = 0.047(Ft.)

+++++  
Process from Point/Station 303.000 to Point/Station 302.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.500(Ft.)  
Downstream point/station elevation = 566.000(Ft.)  
Pipe length = 50.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 11.502(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 11.502(CFS)  
Normal flow depth in pipe = 13.27(In.)  
Flow top width inside pipe = 20.26(In.)  
Critical Depth = 15.18(In.)  
Pipe flow velocity = 7.18(Ft/s)  
Travel time through pipe = 0.12 min.  
Time of concentration (TC) = 40.83 min.

+++++  
Process from Point/Station 303.000 to Point/Station 302.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 1  
Stream flow area = 24.510(Ac.)  
Runoff from this stream = 11.502(CFS)  
Time of concentration = 40.83 min.  
Rainfall intensity = 0.630(In/Hr)  
Area averaged loss rate (Fm) = 0.1096(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.2000

+++++

Process from Point/Station 305.000 to Point/Station 304.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

---

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Initial subarea data:  
Initial area flow distance = 550.000(Ft.)  
Top (of initial area) elevation = 574.000(Ft.)  
Bottom (of initial area) elevation = 568.000(Ft.)  
Difference in elevation = 6.000(Ft.)  
Slope = 0.01091 s(%)= 1.09  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 9.364 min.  
Rainfall intensity = 1.524(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.868  
Subarea runoff = 1.587(CFS)  
Total initial stream area = 1.200(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.055(In/Hr)

\*\*\*\*\*  
Process from Point/Station 304.000 to Point/Station 302.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 568.000(Ft.)  
Downstream point/station elevation = 566.000(Ft.)  
Pipe length = 310.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 1.587(CFS)  
Nearest computed pipe diameter = 12.00(In.)  
Calculated individual pipe flow = 1.587(CFS)  
Normal flow depth in pipe = 6.38(In.)  
Flow top width inside pipe = 11.98(In.)  
Critical Depth = 6.42(In.)  
Pipe flow velocity = 3.74(Ft/s)  
Travel time through pipe = 1.38 min.  
Time of concentration (TC) = 10.75 min.

\*\*\*\*\*  
Process from Point/Station 304.000 to Point/Station 302.000  
\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*

---

Along Main Stream number: 1 in normal stream number 2  
Stream flow area = 1.200(Ac.)  
Runoff from this stream = 1.587(CFS)  
Time of concentration = 10.75 min.  
Rainfall intensity = 1.403(In/Hr)  
Area averaged loss rate (Fm) = 0.0548(In/Hr)  
Area averaged Pervious ratio (Ap) = 0.1000

\*\*\*\*\*  
 Process from Point/Station 311.000 to Point/Station 302.000  
 \*\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*\*

---

COMMERCIAL subarea type  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 69.00  
 Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
 Initial subarea data:  
 Initial area flow distance = 745.000(Ft.)  
 Top (of initial area) elevation = 569.000(Ft.)  
 Bottom (of initial area) elevation = 566.000(Ft.)  
 Difference in elevation = 3.000(Ft.)  
 Slope = 0.00403 s(%)= 0.40  
 $TC = k(0.304)*[(length^3)/(elevation\ change)]^{0.2}$   
 Initial area time of concentration = 12.905 min.  
 Rainfall intensity = 1.257(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.861  
 Subarea runoff = 2.273(CFS)  
 Total initial stream area = 2.100(Ac.)  
 Pervious area fraction = 0.100  
 Initial area Fm value = 0.055(In/Hr)

\*\*\*\*\*  
 Process from Point/Station 311.000 to Point/Station 302.000  
 \*\*\*\*\* CONFLUENCE OF MINOR STREAMS \*\*\*\*\*

---

Along Main Stream number: 1 in normal stream number 3  
 Stream flow area = 2.100(Ac.)  
 Runoff from this stream = 2.273(CFS)  
 Time of concentration = 12.90 min.  
 Rainfall intensity = 1.257(In/Hr)  
 Area averaged loss rate (Fm) = 0.0548(In/Hr)  
 Area averaged Pervious ratio (Ap) = 0.1000  
 Summary of stream data:

Stream No.	Stream Flow rate (CFS)	Area (Ac.)	TC (min)	Fm (In/Hr)	Rainfall Intensity (In/Hr)
1	11.50	24.510	40.83	0.110	0.630
2	1.59	1.200	10.75	0.055	1.403
3	2.27	2.100	12.90	0.055	1.257
Qmax(1) =					
	1.000 *	1.000 *	11.502)	+	
	0.427 *	1.000 *	1.587)	+	
	0.478 *	1.000 *	2.273)	+	= 13.266
Qmax(2) =					
	2.486 *	0.263 *	11.502)	+	
	1.000 *	1.000 *	1.587)	+	
	1.121 *	0.833 *	2.273)	+	= 11.236
Qmax(3) =					
	2.205 *	0.316 *	11.502)	+	

0.892 \* 1.000 \* 1.587) +  
1.000 \* 1.000 \* 2.273) + = 11.706

Total of 3 streams to confluence:

Flow rates before confluence point:

11.502 1.587 2.273

Maximum flow rates at confluence using above data:

13.266 11.236 11.706

Area of streams before confluence:

24.510 1.200 2.100

Effective area values after confluence:

27.810 9.401 11.047

Results of confluence:

Total flow rate = 13.266(CFS)

Time of concentration = 40.826 min.

Effective stream area after confluence = 27.810(Ac.)

Study area average Pervious fraction(Ap) = 0.188

Study area average soil loss rate(Fm) = 0.103(In/Hr)

Study area total (this main stream) = 27.81(Ac.)

++++  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 566.000(Ft.)  
Downstream point/station elevation = 550.000(Ft.)  
Pipe length = 220.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 13.266(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 13.266(CFS)  
Normal flow depth in pipe = 9.80(In.)  
Flow top width inside pipe = 14.28(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 15.63(Ft/s)  
Travel time through pipe = 0.23 min.  
Time of concentration (TC) = 41.06 min.

++++  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 41.06 min.  
Rainfall intensity = 0.628(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.733  
Subarea runoff = 0.168(CFS) for 1.390(Ac.)  
Total runoff = 13.434(CFS)  
Effective area this stream = 29.20(Ac.)

Total Study Area (Main Stream No. 1) = 29.20(Ac.)  
Area averaged Fm value = 0.117(In/Hr)

+++++  
Process from Point/Station 301.000 to Point/Station 300.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 550.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 13.434(CFS)  
Nearest computed pipe diameter = 15.00(In.)  
Calculated individual pipe flow = 13.434(CFS)  
Normal flow depth in pipe = 8.87(In.)  
Flow top width inside pipe = 14.75(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 17.76(Ft/s)  
Travel time through pipe = 0.09 min.  
Time of concentration (TC) = 41.15 min.

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B300  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 403.000 to Point/Station 402.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Initial subarea data:  
Initial area flow distance = 205.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)  
Difference in elevation = 2.000(Ft.)  
Slope = 0.00976 s(%)= 0.98  
TC = k(0.324)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 6.877 min.  
Rainfall intensity = 1.834(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.846  
Subarea runoff = 3.523(CFS)  
Total initial stream area = 2.270(Ac.)  
Pervious area fraction = 0.200  
Initial area Fm value = 0.110(In/Hr)

+++++  
Process from Point/Station 402.000 to Point/Station 401.000  
\*\*\*\* STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION \*\*\*\*

---

Top of street segment elevation = 569.000(Ft.)  
End of street segment elevation = 566.000(Ft.)  
Length of street segment = 520.000(Ft.)  
Height of curb above gutter flowline = 6.0(In.)  
Width of half street (curb to crown) = 20.000(Ft.)  
Distance from crown to crossfall grade break = 0.500(Ft.)  
Slope from gutter to grade break (v/hz) = 0.020  
Slope from grade break to crown (v/hz) = 0.020  
Street flow is on [2] side(s) of the street  
Distance from curb to property line = 10.000(Ft.)  
Slope from curb to property line (v/hz) = 0.020  
Gutter width = 2.000(Ft.)  
Gutter hike from flowline = 2.000(In.)  
Manning's N in gutter = 0.0130  
Manning's N from gutter to grade break = 0.0130  
Manning's N from grade break to crown = 0.0130  
Estimated mean flow rate at midpoint of street = 6.040(CFS)  
Depth of flow = 0.354(Ft.), Average velocity = 2.125(Ft/s)  
Streetflow hydraulics at midpoint of street travel:  
Halfstreet flow width = 11.379(Ft.)  
Flow velocity = 2.12(Ft/s)  
Travel time = 4.08 min. TC = 10.96 min.  
Adding area flow to street  
APARTMENT subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.2000 Max loss rate(Fm)= 0.110(In/Hr)  
Rainfall intensity = 1.387(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.829  
Subarea runoff = 4.927(CFS) for 5.080(Ac.)  
Total runoff = 8.450(CFS)  
Effective area this stream = 7.35(Ac.)  
Total Study Area (Main Stream No. 1) = 7.35(Ac.)  
Area averaged Fm value = 0.110(In/Hr)  
Street flow at end of street = 8.450(CFS)  
Half street flow at end of street = 4.225(CFS)  
Depth of flow = 0.388(Ft.), Average velocity = 2.300(Ft/s)  
Flow width (from curb towards crown)= 13.078(Ft.)

+++++  
Process from Point/Station 401.000 to Point/Station 400.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 569.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 120.00(Ft.) Manning's N = 0.013

No. of pipes = 1 Required pipe flow = 8.450(CFS)  
Nearest computed pipe diameter = 12.00(In.)  
Calculated individual pipe flow = 8.450(CFS)  
Normal flow depth in pipe = 5.88(In.)  
Flow top width inside pipe = 12.00(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 22.10(Ft/s)  
Travel time through pipe = 0.09 min.  
Time of concentration (TC) = 11.05 min.  
End of computations, Total Study Area = 7.35 (Ac.)

The following figures may

be used for a unit hydrograph study of the same area.

Note: These figures do not consider reduced effective area effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.200

Area averaged SCS curve number = 69.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 03/09/15

SUBAREA BASIN B400  
2 YEAR  
RANCHO MIRAMONTE

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 503.000 to Point/Station 502.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Initial subarea data:  
Initial area flow distance = 205.000(Ft.)  
Top (of initial area) elevation = 571.000(Ft.)  
Bottom (of initial area) elevation = 569.000(Ft.)  
Difference in elevation = 2.000(Ft.)  
Slope = 0.00976 s(%)= 0.98  
TC = k(0.304)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 6.452 min.  
Rainfall intensity = 1.906(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.874  
Subarea runoff = 2.798(CFS)  
Total initial stream area = 1.680(Ac.)  
Pervious area fraction = 0.100  
Initial area Fm value = 0.055(In/Hr)

+++++  
Process from Point/Station 502.000 to Point/Station 501.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 569.000(Ft.)  
Downstream point elevation = 566.000(Ft.)  
Channel length thru subarea = 480.000(Ft.)  
Channel base width = 100.000(Ft.)  
Slope or 'Z' of left channel bank = 6.000  
Slope or 'Z' of right channel bank = 6.000  
Estimated mean flow rate at midpoint of channel = 4.017(CFS)  
Manning's 'N' = 0.015  
Maximum depth of channel = 3.000(Ft.)  
Flow(q) thru subarea = 4.017(CFS)  
Depth of flow = 0.042(Ft.), Average velocity = 0.948(Ft/s)  
Channel flow top width = 100.507(Ft.)  
Flow Velocity = 0.95(Ft/s)  
Travel time = 8.44 min.  
Time of concentration = 14.89 min.  
Critical depth = 0.037(Ft.)  
Adding area flow to channel  
COMMERCIAL subarea type  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 69.00  
Pervious ratio(Ap) = 0.1000 Max loss rate(Fm)= 0.055(In/Hr)  
Rainfall intensity = 1.154(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area,(total area with modified  
rational method)(Q=KCIA) is C = 0.857  
Subarea runoff = 2.355(CFS) for 3.530(Ac.)  
Total runoff = 5.154(CFS)  
Effective area this stream = 5.21(Ac.)  
Total Study Area (Main Stream No. 1) = 5.21(Ac.)  
Area averaged Fm value = 0.055(In/Hr)  
Depth of flow = 0.049(Ft.), Average velocity = 1.047(Ft/s)  
Critical depth = 0.043(Ft.)

+++++  
Process from Point/Station 501.000 to Point/Station 500.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 560.000(Ft.)  
Downstream point/station elevation = 540.000(Ft.)  
Pipe length = 100.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 5.154(CFS)  
Nearest computed pipe diameter = 9.00(In.)  
Calculated individual pipe flow = 5.154(CFS)  
Normal flow depth in pipe = 5.53(In.)  
Flow top width inside pipe = 8.76(In.)  
Critical depth could not be calculated.  
Pipe flow velocity = 18.10(Ft/s)  
Travel time through pipe = 0.09 min.

Time of concentration (TC) = 14.98 min.  
End of computations, Total Study Area = 5.21 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 0.100  
Area averaged SCS curve number = 69.0

***APPENDIX C: 100-YEAR EXISTING CONDITION MODEL OUTPUT***

Existing Cond  
100-year  
Basins E1  
E2  
E3  
E4

San B

l Hydrology Program

August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

Chino Hydrology  
Basin E1 - Existing Condition  
100-year flows

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.600 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

\*\*\*\*\*  
Process from Point/Station 204.000 to Point/Station 203.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Initial subarea data:  
Initial area flow distance = 773.000(Ft.)  
Top (of initial area) elevation = 576.500(Ft.)  
Bottom (of initial area) elevation = 555.000(Ft.)  
Difference in elevation = 21.500(Ft.)  
Slope = 0.02781 s(%)= 2.78  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 20.664 min.  
Rainfall intensity = 3.033(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.862  
Subarea runoff = 19.239(CFS)  
Total initial stream area = 7.360(Ac.)  
Pervious area fraction = 1.000

Initial area Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 203.000 to Point/Station 201.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 555.000(Ft.)  
Downstream point elevation = 544.000(Ft.)  
Channel length thru subarea = 983.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 52.555(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 52.555(CFS)  
Depth of flow = 0.638(Ft.), Average velocity = 3.552(Ft/s)  
Channel flow top width = 26.380(Ft.)  
Flow Velocity = 3.55(Ft/s)  
Travel time = 4.61 min.  
Time of concentration = 25.28 min.  
Critical depth = 0.570(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 2.688(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.857  
Subarea runoff = 66.576(CFS) for 29.900(Ac.)  
Total runoff = 85.815(CFS)  
Effective area this stream = 37.26(Ac.)  
Total Study Area (Main Stream No. 1) = 37.26(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 0.845(Ft.), Average velocity = 4.191(Ft/s)  
Critical depth = 0.773(Ft.)  
End of computations, Total Study Area = 37.26 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 1.000

Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

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Chino Hydrology - Basin E2  
Existing Condition  
100-year flows  
-----

-----  
Program License Serial Number 6288  
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-----  
\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*  
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-----  
Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.600 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

-----  
+++++  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*  
-----

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Initial subarea data:  
Initial area flow distance = 939.000(Ft.)  
Top (of initial area) elevation = 564.000(Ft.)  
Bottom (of initial area) elevation = 549.000(Ft.)  
Difference in elevation = 15.000(Ft.)  
Slope = 0.01597 s(%)= 1.60  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 24.957 min.  
Rainfall intensity = 2.708(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.857  
Subarea runoff = 12.514(CFS)  
Total initial stream area = 5.390(Ac.)  
Pervious area fraction = 1.000

Initial area Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 102.000 to Point/Station 101.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 549.000(Ft.)  
Downstream point elevation = 545.700(Ft.)  
Channel length thru subarea = 752.000(Ft.)  
Channel base width = 25.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 42.359(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 6.000(Ft.)  
Flow(q) thru subarea = 42.359(CFS)  
Depth of flow = 0.633(Ft.), Average velocity = 2.137(Ft/s)  
Channel flow top width = 37.655(Ft.)  
Flow Velocity = 2.14(Ft/s)  
Travel time = 5.87 min.  
Time of concentration = 30.82 min.  
Critical depth = 0.422(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 2.386(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.851  
Subarea runoff = 59.592(CFS) for 30.100(Ac.)  
Total runoff = 72.106(CFS)  
Effective area this stream = 35.49(Ac.)  
Total Study Area (Main Stream No. 1) = 35.49(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 0.851(Ft.), Average velocity = 2.530(Ft/s)  
Critical depth = 0.586(Ft.)  
End of computations, Total Study Area = 35.49 (Ac.)

The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 1.000  
Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

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Chino Hydrology - Basin E3  
Existing Condition  
100-year flows  
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Program License Serial Number 6288  
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\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*  
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Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.600 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3  
-----

\*\*\*\*\*  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*  
-----

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Initial subarea data:  
Initial area flow distance = 700.000(Ft.)  
Top (of initial area) elevation = 579.500(Ft.)  
Bottom (of initial area) elevation = 556.000(Ft.)  
Difference in elevation = 23.500(Ft.)  
Slope = 0.03357 s(%)= 3.36  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 19.127 min.  
Rainfall intensity = 3.177(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.864  
Subarea runoff = 19.232(CFS)  
Total initial stream area = 7.010(Ac.)  
Pervious area fraction = 1.000

Initial area Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 301.000 to Point/Station 300.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 19.13 min.  
Rainfall intensity = 3.177(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.864  
Subarea runoff = 31.880(CFS) for 11.620(Ac.)  
Total runoff = 51.112(CFS)  
Effective area this stream = 18.63(Ac.)  
Total Study Area (Main Stream No. 1) = 18.63(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
End of computations, Total Study Area = 18.63 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 1.000  
Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

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Chino Hydrology - Basin E4  
Existing Condition  
100-year flows  
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Program License Serial Number 6288  
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\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*  
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-----  
Rational hydrology study storm event year is 100.0  
Computed rainfall intensity:  
Storm year = 100.00 1 hour rainfall = 1.600 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 3

-----  
+++++  
Process from Point/Station 406.000 to Point/Station 405.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*  
-----

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Initial subarea data:  
Initial area flow distance = 542.000(Ft.)  
Top (of initial area) elevation = 564.000(Ft.)  
Bottom (of initial area) elevation = 563.000(Ft.)  
Difference in elevation = 1.000(Ft.)  
Slope = 0.00185 s(%)= 0.18  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 30.846 min.  
Rainfall intensity = 2.385(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.851  
Subarea runoff = 6.742(CFS)  
Total initial stream area = 3.320(Ac.)  
Pervious area fraction = 1.000

Initial area Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 405.000 to Point/Station 404.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 563.000(Ft.)  
Downstream point elevation = 546.000(Ft.)  
Channel length thru subarea = 833.000(Ft.)  
Channel base width = 10.000(Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 16.374(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 16.374(CFS)  
Depth of flow = 0.397(Ft.), Average velocity = 3.439(Ft/s)  
Channel flow top width = 13.972(Ft.)  
Flow Velocity = 3.44(Ft/s)  
Travel time = 4.04 min.  
Time of concentration = 34.88 min.  
Critical depth = 0.406(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 2.215(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.848  
Subarea runoff = 19.194(CFS) for 10.490(Ac.)  
Total runoff = 25.936(CFS)  
Effective area this stream = 13.81(Ac.)  
Total Study Area (Main Stream No. 1) = 13.81(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 0.516(Ft.), Average velocity = 4.000(Ft/s)  
Critical depth = 0.539(Ft.)

+++++  
Process from Point/Station 410.000 to Point/Station 404.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 34.88 min.

Rainfall intensity = 2.215(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.848  
Subarea runoff = 28.208(CFS) for 15.020(Ac.)  
Total runoff = 54.143(CFS)  
Effective area this stream = 28.83(Ac.)  
Total Study Area (Main Stream No. 1) = 28.83(Ac.)  
Area averaged Fm value = 0.129(In/Hr)

+++++  
Process from Point/Station 404.000 to Point/Station 403.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 546.000(Ft.)  
Downstream point elevation = 538.000(Ft.)  
Channel length thru subarea = 724.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 77.489(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 77.489(CFS)  
Depth of flow = 0.761(Ft.), Average velocity = 3.686(Ft/s)  
Channel flow top width = 35.228(Ft.)  
Flow Velocity = 3.69(Ft/s)  
Travel time = 3.27 min.  
Time of concentration = 38.16 min.

Critical depth = 0.688(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 2.099(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.845  
Subarea runoff = 46.594(CFS) for 27.970(Ac.)  
Total runoff = 100.737(CFS)  
Effective area this stream = 56.80(Ac.)  
Total Study Area (Main Stream No. 1) = 56.80(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 0.877(Ft.), Average velocity = 3.990(Ft/s)  
Critical depth = 0.805(Ft.)

+++++  
Process from Point/Station 415.000 to Point/Station 403.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Time of concentration = 38.16 min.  
Rainfall intensity = 2.099(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.845  
Subarea runoff = 29.441(CFS) for 16.600(Ac.)  
Total runoff = 130.178(CFS)  
Effective area this stream = 73.40(Ac.)  
Total Study Area (Main Stream No. 1) = 73.40(Ac.)  
Area averaged Fm value = 0.129(In/Hr)

\*\*\*\*\*  
Process from Point/Station 403.000 to Point/Station 402.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 538.000(Ft.)  
Downstream point/station elevation = 537.000(Ft.)  
Pipe length = 66.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 130.178(CFS)  
Nearest computed pipe diameter = 45.00(In.)  
Calculated individual pipe flow = 130.178(CFS)  
Normal flow depth in pipe = 32.58(In.)  
Flow top width inside pipe = 40.23(In.)  
Critical Depth = 40.68(In.)  
Pipe flow velocity = 15.19(Ft/s)  
Travel time through pipe = 0.07 min.  
Time of concentration (TC) = 38.23 min.

\*\*\*\*\*  
Process from Point/Station 402.000 to Point/Station 401.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 537.000(Ft.)  
Downstream point elevation = 535.500(Ft.)  
Channel length thru subarea = 370.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 134.639(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 134.639(CFS)  
Depth of flow = 1.329(Ft.), Average velocity = 3.042(Ft/s)  
Channel flow top width = 46.587(Ft.)  
Flow Velocity = 3.04(Ft/s)  
Travel time = 2.03 min.  
Time of concentration = 40.26 min.  
Critical depth = 0.953(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea

Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 79.00  
 Adjusted SCS curve number for AMC 3 = 93.40  
 Pervious ratio(Ap) = 1.0000      Max loss rate(Fm)=      0.129(In/Hr)  
 Rainfall intensity =      2.033(In/Hr) for a      100.0 year storm  
 Effective runoff coefficient used for area, (total area with modified  
 rational method)(Q=KCIA) is C = 0.843  
 Subarea runoff =      8.845(CFS) for      7.720(Ac.)  
 Total runoff =      139.023(CFS)  
 Effective area this stream =      81.12(Ac.)  
 Total Study Area (Main Stream No. 1) =      81.12(Ac.)  
 Area averaged Fm value =      0.129(In/Hr)  
 Depth of flow =      1.351(Ft.), Average velocity =      3.070(Ft/s)  
 Critical depth =      0.969(Ft.)

++++++  
 Process from Point/Station      420.000 to Point/Station      401.000  
 \*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 79.00  
 Adjusted SCS curve number for AMC 3 = 93.40  
 Pervious ratio(Ap) = 1.0000      Max loss rate(Fm)=      0.129(In/Hr)  
 Time of concentration =      40.26 min.  
 Rainfall intensity =      2.033(In/Hr) for a      100.0 year storm  
 Effective runoff coefficient used for area, (total area with modified  
 rational method)(Q=KCIA) is C = 0.843  
 Subarea runoff =      32.922(CFS) for      19.210(Ac.)  
 Total runoff =      171.945(CFS)  
 Effective area this stream =      100.33(Ac.)  
 Total Study Area (Main Stream No. 1) =      100.33(Ac.)  
 Area averaged Fm value =      0.129(In/Hr)

++++++  
 Process from Point/Station      401.000 to Point/Station      510.000  
 \*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation =      535.500(Ft.)  
 Downstream point elevation =      520.000(Ft.)  
 Channel length thru subarea =      794.000(Ft.)  
 Channel base width      =      20.000(Ft.)  
 Slope or 'Z' of left channel bank =      10.000  
 Slope or 'Z' of right channel bank =      10.000  
 Estimated mean flow rate at midpoint of channel =      179.939(CFS)  
 Manning's 'N'      =      0.030  
 Maximum depth of channel =      5.000(Ft.)  
 Flow(q) thru subarea =      179.939(CFS)  
 Depth of flow =      1.027(Ft.), Average velocity =      5.788(Ft/s)

Channel flow top width = 40.540(Ft.)  
Flow Velocity = 5.79(Ft/s)  
Travel time = 2.29 min.  
Time of concentration = 42.54 min.  
Critical depth = 1.125(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 1.967(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.841  
Subarea runoff = 15.934(CFS) for 13.250(Ac.)  
Total runoff = 187.878(CFS)  
Effective area this stream = 113.58(Ac.)  
Total Study Area (Main Stream No. 1) = 113.58(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 1.051(Ft.), Average velocity = 5.862(Ft/s)  
Critical depth = 1.148(Ft.)

\*\*\*\*\*  
Process from Point/Station 510.000 to Point/Station 505.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 520.000(Ft.)  
Downstream point elevation = 515.000(Ft.)  
Channel length thru subarea = 1051.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 198.333(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 10.000(Ft.)  
Flow(q) thru subarea = 198.333(CFS)  
Depth of flow = 1.554(Ft.), Average velocity = 3.592(Ft/s)  
Channel flow top width = 51.077(Ft.)  
Flow Velocity = 3.59(Ft/s)  
Travel time = 4.88 min.  
Time of concentration = 47.42 min.  
Critical depth = 1.188(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 1.843(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified

rational method)(Q=KCIA) is C = 0.837  
Subarea runoff = 20.846(CFS) for 21.730(Ac.)  
Total runoff = 208.725(CFS)  
Effective area this stream = 135.31(Ac.)  
Total Study Area (Main Stream No. 1) = 135.31(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 1.594(Ft.), Average velocity = 3.643(Ft/s)  
Critical depth = 1.219(Ft.)

\*\*\*\*\*  
Process from Point/Station 505.000 to Point/Station 600.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 515.000(Ft.)  
Downstream point elevation = 507.000(Ft.)  
Channel length thru subarea = 1009.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 214.079(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 10.000(Ft.)  
Flow(q) thru subarea = 214.079(CFS)  
Depth of flow = 1.549(Ft.), Average velocity = 4.982(Ft/s)  
Channel flow top width = 35.488(Ft.)  
Flow Velocity = 4.98(Ft/s)  
Travel time = 3.38 min.  
Time of concentration = 50.80 min.  
Critical depth = 1.359(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Adjusted SCS curve number for AMC 3 = 93.40  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.129(In/Hr)  
Rainfall intensity = 1.768(In/Hr) for a 100.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.835  
Subarea runoff = 10.629(CFS) for 13.350(Ac.)  
Total runoff = 219.353(CFS)  
Effective area this stream = 148.66(Ac.)  
Total Study Area (Main Stream No. 1) = 148.66(Ac.)  
Area averaged Fm value = 0.129(In/Hr)  
Depth of flow = 1.569(Ft.), Average velocity = 5.019(Ft/s)  
Critical depth = 1.375(Ft.)  
End of computations, Total Study Area = 148.66 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 1.000  
Area averaged SCS curve number = 79.0

***APPENDIX D: 2-YEAR EXISTING CONDITION MODEL***

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

Chino Hydrology  
Basin E1 - Existing Condition  
2-year flow rates

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

+++++  
Process from Point/Station 204.000 to Point/Station 203.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Initial subarea data:  
Initial area flow distance = 773.000(Ft.)  
Top (of initial area) elevation = 576.500(Ft.)  
Bottom (of initial area) elevation = 555.000(Ft.)  
Difference in elevation = 21.500(Ft.)  
Slope = 0.02781 s(%)= 2.78  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 20.664 min.  
Rainfall intensity = 0.948(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.533  
Subarea runoff = 3.717(CFS)  
Total initial stream area = 7.360(Ac.)  
Pervious area fraction = 1.000  
Initial area Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 203.000 to Point/Station 201.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 555.000 (Ft.)  
Downstream point elevation = 544.000 (Ft.)  
Channel length thru subarea = 983.000 (Ft.)  
Channel base width = 20.000 (Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 8.202 (CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000 (Ft.)  
Flow(q) thru subarea = 8.202 (CFS)  
Depth of flow = 0.215 (Ft.), Average velocity = 1.814 (Ft/s)  
Channel flow top width = 22.145 (Ft.)  
Flow Velocity = 1.81 (Ft/s)  
Travel time = 9.03 min.  
Time of concentration = 29.69 min.  
Critical depth = 0.172 (Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil (AMC 2) = 79.00  
Pervious ratio (Ap) = 1.0000 Max loss rate (Fm) = 0.387 (In/Hr)  
Rainfall intensity = 0.763 (In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.444  
Subarea runoff = 8.885 (CFS) for 29.900 (Ac.)  
Total runoff = 12.602 (CFS)  
Effective area this stream = 37.26 (Ac.)  
Total Study Area (Main Stream No. 1) = 37.26 (Ac.)  
Area averaged Fm value = 0.387 (In/Hr)  
Depth of flow = 0.277 (Ft.), Average velocity = 2.130 (Ft/s)  
Critical depth = 0.227 (Ft.)  
End of computations, Total Study Area = 37.26 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction (Ap) = 1.000  
Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

Chino Hydrology - Basin E2  
Existing Condition  
2-year flow rates

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 103.000 to Point/Station 102.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Initial subarea data:  
Initial area flow distance = 939.000(Ft.)  
Top (of initial area) elevation = 564.000(Ft.)  
Bottom (of initial area) elevation = 549.000(Ft.)  
Difference in elevation = 15.000(Ft.)  
Slope = 0.01597 s(%)= 1.60  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 24.957 min.  
Rainfall intensity = 0.846(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.489  
Subarea runoff = 2.230(CFS)  
Total initial stream area = 5.390(Ac.)  
Pervious area fraction = 1.000  
Initial area Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 102.000 to Point/Station 101.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 549.000(Ft.)  
Downstream point elevation = 545.700(Ft.)  
Channel length thru subarea = 752.000(Ft.)  
Channel base width = 25.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 5.701(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 6.000(Ft.)  
Flow(q) thru subarea = 5.701(CFS)  
Depth of flow = 0.198(Ft.), Average velocity = 1.064(Ft/s)  
Channel flow top width = 28.969(Ft.)  
Flow Velocity = 1.06(Ft/s)  
Travel time = 11.78 min.  
Time of concentration = 36.73 min.  
Critical depth = 0.115(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Rainfall intensity = 0.671(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.381  
Subarea runoff = 6.856(CFS) for 30.100(Ac.)  
Total runoff = 9.086(CFS)  
Effective area this stream = 35.49(Ac.)  
Total Study Area (Main Stream No. 1) = 35.49(Ac.)  
Area averaged Fm value = 0.387(In/Hr)  
Depth of flow = 0.261(Ft.), Average velocity = 1.261(Ft/s)  
Critical depth = 0.156(Ft.)  
End of computations, Total Study Area = 35.49 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction(Ap) = 1.000  
Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

Chino Hydrology - Basin E3  
Existing Condition  
2-year flows

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

\*\*\*\*\*  
Process from Point/Station 302.000 to Point/Station 301.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Initial subarea data:  
Initial area flow distance = 700.000(Ft.)  
Top (of initial area) elevation = 579.500(Ft.)  
Bottom (of initial area) elevation = 556.000(Ft.)  
Difference in elevation = 23.500(Ft.)  
Slope = 0.03357 s(%)= 3.36  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 19.127 min.  
Rainfall intensity = 0.993(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.549  
Subarea runoff = 3.824(CFS)  
Total initial stream area = 7.010(Ac.)  
Pervious area fraction = 1.000  
Initial area Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 301.000 to Point/Station 300.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio( $A_p$ ) = 1.0000 Max loss rate( $F_m$ )= 0.387(In/Hr)  
Time of concentration = 19.13 min.  
Rainfall intensity = 0.993(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)( $Q=KCIA$ ) is  $C = 0.549$   
Subarea runoff = 6.339(CFS) for 11.620(Ac.)  
Total runoff = 10.162(CFS)  
Effective area this stream = 18.63(Ac.)  
Total Study Area (Main Stream No. 1) = 18.63(Ac.)  
Area averaged  $F_m$  value = 0.387(In/Hr)  
End of computations, Total Study Area = 18.63 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area  
effects caused by confluences in the rational equation.

Area averaged pervious area fraction( $A_p$ ) = 1.000  
Area averaged SCS curve number = 79.0

San Bernardino County Rational Hydrology Program

(Hydrology Manual Date - August 1986)

CIVILCADD/CIVILDESIGN Engineering Software, (c) 1989-2005 Version 7.1  
Rational Hydrology Study Date: 06/04/13

Chino Hydrology - Basin E4  
Existing Condition  
2-year flows

Program License Serial Number 6288

\*\*\*\*\* Hydrology Study Control Information \*\*\*\*\*

Rational hydrology study storm event year is 2.0  
Computed rainfall intensity:  
Storm year = 2.00 1 hour rainfall = 0.500 (In.)  
Slope used for rainfall intensity curve b = 0.6000  
Soil antecedent moisture condition (AMC) = 2

+++++  
Process from Point/Station 406.000 to Point/Station 405.000  
\*\*\*\* INITIAL AREA EVALUATION \*\*\*\*

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Initial subarea data:  
Initial area flow distance = 542.000(Ft.)  
Top (of initial area) elevation = 564.000(Ft.)  
Bottom (of initial area) elevation = 563.000(Ft.)  
Difference in elevation = 1.000(Ft.)  
Slope = 0.00185 s(%)= 0.18  
TC = k(0.706)\*[(length^3)/(elevation change)]^0.2  
Initial area time of concentration = 30.846 min.  
Rainfall intensity = 0.745(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area (Q=KCIA) is C = 0.433  
Subarea runoff = 1.071(CFS)  
Total initial stream area = 3.320(Ac.)  
Pervious area fraction = 1.000

Initial area Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 405.000 to Point/Station 404.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 563.000(Ft.)  
Downstream point elevation = 546.000(Ft.)  
Channel length thru subarea = 833.000(Ft.)  
Channel base width = 10.000(Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 2.199(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 2.199(CFS)  
Depth of flow = 0.123(Ft.), Average velocity = 1.683(Ft/s)  
Channel flow top width = 11.230(Ft.)  
Flow Velocity = 1.68(Ft/s)  
Travel time = 8.25 min.  
Time of concentration = 39.09 min.  
Critical depth = 0.112(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Rainfall intensity = 0.647(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method)(Q=KCIA) is C = 0.362  
Subarea runoff = 2.158(CFS) for 10.490(Ac.)  
Total runoff = 3.229(CFS)  
Effective area this stream = 13.81(Ac.)  
Total Study Area (Main Stream No. 1) = 13.81(Ac.)  
Area averaged Fm value = 0.387(In/Hr)  
Depth of flow = 0.154(Ft.), Average velocity = 1.941(Ft/s)  
Critical depth = 0.145(Ft.)

+++++  
Process from Point/Station 410.000 to Point/Station 404.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 39.09 min.  
Rainfall intensity = 0.647(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified

rational method) (Q=KCIA) is C = 0.362  
Subarea runoff = 3.512(CFS) for 15.020(Ac.)  
Total runoff = 6.742(CFS)  
Effective area this stream = 28.83(Ac.)  
Total Study Area (Main Stream No. 1) = 28.83(Ac.)  
Area averaged Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 404.000 to Point/Station 403.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 546.000(Ft.)  
Downstream point elevation = 538.000(Ft.)  
Channel length thru subarea = 724.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 8.522(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 8.522(CFS)  
Depth of flow = 0.217(Ft.), Average velocity = 1.768(Ft/s)  
Channel flow top width = 24.348(Ft.)  
Flow Velocity = 1.77(Ft/s)  
Travel time = 6.83 min.  
Time of concentration = 45.92 min.  
Critical depth = 0.172(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Rainfall intensity = 0.587(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.307  
Subarea runoff = 3.498(CFS) for 27.970(Ac.)  
Total runoff = 10.240(CFS)  
Effective area this stream = 56.80(Ac.)  
Total Study Area (Main Stream No. 1) = 56.80(Ac.)  
Area averaged Fm value = 0.387(In/Hr)  
Depth of flow = 0.242(Ft.), Average velocity = 1.887(Ft/s)  
Critical depth = 0.195(Ft.)

+++++  
Process from Point/Station 415.000 to Point/Station 403.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000

SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 45.92 min.  
Rainfall intensity = 0.587(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.307  
Subarea runoff = 2.993(CFS) for 16.600(Ac.)  
Total runoff = 13.232(CFS)  
Effective area this stream = 73.40(Ac.)  
Total Study Area (Main Stream No. 1) = 73.40(Ac.)  
Area averaged Fm value = 0.387(In/Hr)

+++++  
Process from Point/Station 403.000 to Point/Station 402.000  
\*\*\*\* PIPEFLOW TRAVEL TIME (Program estimated size) \*\*\*\*

---

Upstream point/station elevation = 538.000(Ft.)  
Downstream point/station elevation = 537.000(Ft.)  
Pipe length = 66.00(Ft.) Manning's N = 0.013  
No. of pipes = 1 Required pipe flow = 13.232(CFS)  
Nearest computed pipe diameter = 21.00(In.)  
Calculated individual pipe flow = 13.232(CFS)  
Normal flow depth in pipe = 12.68(In.)  
Flow top width inside pipe = 20.54(In.)  
Critical Depth = 16.26(In.)  
Pipe flow velocity = 8.71(Ft/s)  
Travel time through pipe = 0.13 min.  
Time of concentration (TC) = 46.05 min.

+++++  
Process from Point/Station 402.000 to Point/Station 401.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 537.000(Ft.)  
Downstream point elevation = 535.500(Ft.)  
Channel length thru subarea = 370.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 13.276(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 13.276(CFS)  
Depth of flow = 0.376(Ft.), Average velocity = 1.488(Ft/s)  
Channel flow top width = 27.513(Ft.)  
Flow Velocity = 1.49(Ft/s)  
Travel time = 4.15 min.  
Time of concentration = 50.19 min.  
Critical depth = 0.230(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000

SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
The area added to the existing stream causes a  
a lower flow rate of Q = 12.397(CFS)  
therefore the upstream flow rate of Q = 13.232(CFS) is being used  
Rainfall intensity = 0.557(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.275  
Subarea runoff = 0.000(CFS) for 7.720(Ac.)  
Total runoff = 13.232(CFS)  
Effective area this stream = 81.12(Ac.)  
Total Study Area (Main Stream No. 1) = 81.12(Ac.)  
Area averaged Fm value = 0.387(In/Hr)  
Depth of flow = 0.375(Ft.), Average velocity = 1.486(Ft/s)  
Critical depth = 0.230(Ft.)

++++  
Process from Point/Station 420.000 to Point/Station 401.000  
\*\*\*\* SUBAREA FLOW ADDITION \*\*\*\*

---

UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
Time of concentration = 50.19 min.  
Rainfall intensity = 0.557(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified  
rational method) (Q=KCIA) is C = 0.275  
Subarea runoff = 2.100(CFS) for 19.210(Ac.)  
Total runoff = 15.332(CFS)  
Effective area this stream = 100.33(Ac.)  
Total Study Area (Main Stream No. 1) = 100.33(Ac.)  
Area averaged Fm value = 0.387(In/Hr)

++++  
Process from Point/Station 401.000 to Point/Station 510.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 535.500(Ft.)  
Downstream point elevation = 520.000(Ft.)  
Channel length thru subarea = 794.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 10.000  
Slope or 'Z' of right channel bank = 10.000  
Estimated mean flow rate at midpoint of channel = 15.364(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 5.000(Ft.)  
Flow(q) thru subarea = 15.364(CFS)  
Depth of flow = 0.260(Ft.), Average velocity = 2.618(Ft/s)  
Channel flow top width = 25.194(Ft.)  
Flow Velocity = 2.62(Ft/s)  
Travel time = 5.05 min.

Time of concentration = 55.25 min.  
 Critical depth = 0.254(Ft.)  
 Adding area flow to channel  
 UNDEVELOPED (average cover) subarea  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 79.00  
 Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
 The area added to the existing stream causes a  
 a lower flow rate of Q = 14.175(CFS)  
 therefore the upstream flow rate of Q = 15.332(CFS) is being used  
 Rainfall intensity = 0.525(In/Hr) for a 2.0 year storm  
 Effective runoff coefficient used for area, (total area with modified  
 rational method)(Q=KCIA) is C = 0.238  
 Subarea runoff = 0.000(CFS) for 13.250(Ac.)  
 Total runoff = 15.332(CFS)  
 Effective area this stream = 113.58(Ac.)  
 Total Study Area (Main Stream No. 1) = 113.58(Ac.)  
 Area averaged Fm value = 0.387(In/Hr)  
 Depth of flow = 0.259(Ft.), Average velocity = 2.616(Ft/s)  
 Critical depth = 0.252(Ft.)

++++++  
 Process from Point/Station 510.000 to Point/Station 505.000  
 \*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 520.000(Ft.)  
 Downstream point elevation = 515.000(Ft.)  
 Channel length thru subarea = 1051.000(Ft.)  
 Channel base width = 20.000(Ft.)  
 Slope or 'Z' of left channel bank = 10.000  
 Slope or 'Z' of right channel bank = 10.000  
 Estimated mean flow rate at midpoint of channel = 15.378(CFS)  
 Manning's 'N' = 0.030  
 Maximum depth of channel = 10.000(Ft.)  
 Flow(q) thru subarea = 15.378(CFS)  
 Depth of flow = 0.390(Ft.), Average velocity = 1.648(Ft/s)  
 Channel flow top width = 27.807(Ft.)  
 Flow Velocity = 1.65(Ft/s)  
 Travel time = 10.63 min.  
 Time of concentration = 65.87 min.  
 Critical depth = 0.254(Ft.)  
 Adding area flow to channel  
 UNDEVELOPED (average cover) subarea  
 Decimal fraction soil group A = 0.000  
 Decimal fraction soil group B = 0.000  
 Decimal fraction soil group C = 1.000  
 Decimal fraction soil group D = 0.000  
 SCS curve number for soil(AMC 2) = 79.00  
 Pervious ratio(Ap) = 1.0000 Max loss rate(Fm)= 0.387(In/Hr)  
 The area added to the existing stream causes a  
 a lower flow rate of Q = 10.476(CFS)  
 therefore the upstream flow rate of Q = 15.332(CFS) is being used  
 Rainfall intensity = 0.473(In/Hr) for a 2.0 year storm

Effective runoff coefficient used for area, (total area with modified rational method) ( $Q=KCIA$ ) is  $C = 0.164$   
Subarea runoff = 0.000(CFS) for 21.730(Ac.)  
Total runoff = 15.332(CFS)  
Effective area this stream = 135.31(Ac.)  
Total Study Area (Main Stream No. 1) = 135.31(Ac.)  
Area averaged  $F_m$  value = 0.387(In/Hr)  
Depth of flow = 0.390(Ft.), Average velocity = 1.647(Ft/s)  
Critical depth = 0.252(Ft.)

+++++  
Process from Point/Station 505.000 to Point/Station 600.000  
\*\*\*\* IMPROVED CHANNEL TRAVEL TIME \*\*\*\*

---

Upstream point elevation = 515.000(Ft.)  
Downstream point elevation = 507.000(Ft.)  
Channel length thru subarea = 1009.000(Ft.)  
Channel base width = 20.000(Ft.)  
Slope or 'Z' of left channel bank = 5.000  
Slope or 'Z' of right channel bank = 5.000  
Estimated mean flow rate at midpoint of channel = 15.380(CFS)  
Manning's 'N' = 0.030  
Maximum depth of channel = 10.000(Ft.)  
Flow( $q$ ) thru subarea = 15.380(CFS)  
Depth of flow = 0.344(Ft.), Average velocity = 2.056(Ft/s)  
Channel flow top width = 23.444(Ft.)  
Flow Velocity = 2.06(Ft/s)  
Travel time = 8.18 min.  
Time of concentration = 74.05 min.  
Critical depth = 0.258(Ft.)  
Adding area flow to channel  
UNDEVELOPED (average cover) subarea  
Decimal fraction soil group A = 0.000  
Decimal fraction soil group B = 0.000  
Decimal fraction soil group C = 1.000  
Decimal fraction soil group D = 0.000  
SCS curve number for soil(AMC 2) = 79.00  
Pervious ratio( $A_p$ ) = 1.0000 Max loss rate( $F_m$ ) = 0.387(In/Hr)  
The area added to the existing stream causes a  
a lower flow rate of  $Q = 7.220$ (CFS)  
therefore the upstream flow rate of  $Q = 15.332$ (CFS) is being used  
Rainfall intensity = 0.441(In/Hr) for a 2.0 year storm  
Effective runoff coefficient used for area, (total area with modified rational method) ( $Q=KCIA$ ) is  $C = 0.110$   
Subarea runoff = 0.000(CFS) for 13.350(Ac.)  
Total runoff = 15.332(CFS)  
Effective area this stream = 148.66(Ac.)  
Total Study Area (Main Stream No. 1) = 148.66(Ac.)  
Area averaged  $F_m$  value = 0.387(In/Hr)  
Depth of flow = 0.344(Ft.), Average velocity = 2.053(Ft/s)  
Critical depth = 0.258(Ft.)  
End of computations, Total Study Area = 148.66 (Ac.)  
The following figures may  
be used for a unit hydrograph study of the same area.  
Note: These figures do not consider reduced effective area effects caused by confluences in the rational equation.

)  
Area averaged pervious area fraction( $A_p$ ) = 1.000  
Area averaged SCS curve number = 79.0

Prepared for the City of Chino, CA

***APPENDIX E: DEVELOPED CONDITION HYDROLOGY MAP***

Prepared for the City of Chino, CA

***APPENDIX F: EXISTING HYDROLOGY MAP***

***APPENDIX G: BASIN A – UNIT HYDROGRAPH CALCULATIONS***

BASINA 2 YEAR UH

Unit Hydrograph Analysis

Copyright (c) CIVILCADD/CIVILDESIGN, 1989 - 2004, Version 7.0

Study date 03/11/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

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BASIN A  
2 YEAR 24 HR HYDROGRAPH  
RANCHO MIRAMONTE

-----

Storm Event Year = 2

Antecedent Moisture Condition = 2

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity	isohyetal data:	
Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 2		
35.82	1	0.48
18.70	1	0.48
40.31	1	0.48
3.75	1	0.48
2.54	1	0.48

-----

Rainfall data for year 2		
101.12	6	1.20

-----

Rainfall data for year 2		
101.12	24	2.00

-----

\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

(In/Hr)	SCS curve No. (AMCII)	SCS curve NO. (AMC 2)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm
	71.6	71.6	35.82	0.354	0.507	0.667	0.338
	69.0	69.0	18.70	0.185	0.548	0.400	0.219
	69.2	69.2	40.31	0.399	0.545	0.406	0.221

		BASINA 2 YEAR UH				
73.3	73.3	3.75	0.037	0.480	0.484	0.233
74.0	74.0	2.54	0.025	0.469	0.546	0.256

Area-averaged adjusted loss rate Fm (In/Hr) = 0.264

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area (Ac.)	Area Fract	SCS CN (AMC2)	SCS CN (AMC2)	S	Pervious Yield Fr
23.89	0.236	71.6	71.6	3.97	0.141
11.93	0.118	98.0	98.0	0.20	0.887
7.48	0.074	69.0	69.0	4.49	0.108
11.22	0.111	98.0	98.0	0.20	0.887
16.37	0.162	69.2	69.2	4.45	0.111
23.94	0.237	98.0	98.0	0.20	0.887
1.81	0.018	73.3	73.3	3.64	0.164
1.94	0.019	98.0	98.0	0.20	0.887
1.39	0.014	74.0	74.0	3.51	0.175
1.15	0.011	98.0	98.0	0.20	0.887

Area-averaged catchment yield fraction, Y = 0.505

Area-averaged low loss fraction, Yb = 0.495

User entry of time of concentration = 0.295 (hours)

+++++

Watershed area = 101.12(Ac.)

Catchment Lag time = 0.236 hours

Unit interval = 30.000 minutes

Unit interval percentage of lag time = 211.8644

Hydrograph baseflow = 0.00(CFS)

Average maximum watershed loss rate(Fm) = 0.264(In/Hr)

Average low loss rate fraction (Yb) = 0.495 (decimal)

DESERT S-Graph Selected

Computed peak 5-minute rainfall = 0.178(In)

Computed peak 30-minute rainfall = 0.364(In)

Specified peak 1-hour rainfall = 0.480(In)

Computed peak 3-hour rainfall = 0.842(In)

Specified peak 6-hour rainfall = 1.200(In)

Specified peak 24-hour rainfall = 2.000(In)

Rainfall depth area reduction factors:

Using a total area of 101.12(Ac.) (Ref: fig. E-4)

5-minute factor = 0.995 Adjusted rainfall = 0.177(In)

30-minute factor = 0.995 Adjusted rainfall = 0.362(In)

1-hour factor = 0.995 Adjusted rainfall = 0.478(In)

3-hour factor = 0.999 Adjusted rainfall = 0.841(In)

6-hour factor = 1.000 Adjusted rainfall = 1.200(In)

24-hour factor = 1.000 Adjusted rainfall = 2.000(In)

Unit Hydrograph

+++++

Interval Number	'S' Graph Mean values	Unit Hydrograph (CFS)
--------------------	--------------------------	--------------------------

(K = 203.82 (CFS))

1	44.268	90.228
2	88.969	91.109
3	97.179	16.732
4	100.000	5.750

BASINA 2 YEAR UH

Peak Unit Number	Adjusted mass rainfall (In)	Unit rainfall (In)		
1	0.3620	0.0255		
2	0.4777	0.0163		
3	0.5887	0.0171		
4	0.6827	0.0148		
5	0.7659	0.0133		
6	0.8413	0.0121		
7	0.9104	0.0112		
8	0.9748	0.0104		
9	1.0354	0.0099		
10	1.0927	0.0094		
11	1.1474	0.0089		
12	1.1996	0.0086		
13	1.2355	0.0059		
14	1.2698	0.0056		
15	1.3025	0.0054		
16	1.3338	0.0051		
17	1.3640	0.0049		
18	1.3930	0.0048		
19	1.4211	0.0046		
20	1.4482	0.0045		
21	1.4745	0.0043		
22	1.5000	0.0042		
23	1.5247	0.0041		
24	1.5489	0.0040		
25	1.5723	0.0039		
26	1.5952	0.0038		
27	1.6176	0.0037		
28	1.6394	0.0036		
29	1.6608	0.0035		
30	1.6816	0.0034		
31	1.7021	0.0034		
32	1.7221	0.0033		
33	1.7418	0.0032		
34	1.7610	0.0032		
35	1.7800	0.0031		
36	1.7985	0.0031		
37	1.8168	0.0030		
38	1.8347	0.0030		
39	1.8524	0.0029		
40	1.8698	0.0029		
41	1.8869	0.0028		
42	1.9037	0.0028		
43	1.9203	0.0027		
44	1.9366	0.0027		
45	1.9527	0.0027		
46	1.9686	0.0026		
47	1.9843	0.0026		
48	1.9997	0.0026		

Unit Period (number)	Unit Rainfall (In)	Unit Soil-Loss (In)	Effective Rainfall (In)
1	0.0155	0.0077	0.0078
2	0.0158	0.0078	0.0080
3	0.0161	0.0080	0.0082
4	0.0165	0.0082	0.0083
5	0.0169	0.0084	0.0085
6	0.0173	0.0086	0.0087
7	0.0177	0.0088	0.0089
8	0.0182	0.0090	0.0092
9	0.0186	0.0092	0.0094
10	0.0192	0.0095	0.0097

BASINA 2 YEAR UH

11	0.0197	0.0098	0.0099
12	0.0203	0.0101	0.0103
13	0.0210	0.0104	0.0106
14	0.0217	0.0107	0.0109
15	0.0224	0.0111	0.0113
16	0.0233	0.0115	0.0118
17	0.0242	0.0120	0.0122
18	0.0253	0.0125	0.0128
19	0.0264	0.0131	0.0133
20	0.0277	0.0137	0.0140
21	0.0292	0.0145	0.0148
22	0.0309	0.0153	0.0156
23	0.0330	0.0163	0.0166
24	0.0353	0.0175	0.0178
25	0.0526	0.0261	0.0266
26	0.0564	0.0279	0.0285
27	0.0612	0.0303	0.0309
28	0.0674	0.0334	0.0340
29	0.0767	0.0380	0.0387
30	0.0902	0.0446	0.0455
31	0.1063	0.0527	0.0537
32	0.1925	0.0953	0.0972
33	0.2899	0.1318	0.1580
34	0.0858	0.0425	0.0433
35	0.0654	0.0324	0.0330
36	0.0552	0.0273	0.0279
37	0.0346	0.0171	0.0174
38	0.0304	0.0150	0.0153
39	0.0273	0.0135	0.0138
40	0.0249	0.0123	0.0126
41	0.0230	0.0114	0.0116
42	0.0214	0.0106	0.0108
43	0.0201	0.0100	0.0102
44	0.0190	0.0094	0.0096
45	0.0180	0.0089	0.0091
46	0.0171	0.0085	0.0087
47	0.0164	0.0081	0.0083
48	0.0157	0.0078	0.0079

-----  
 Total soil rain loss = 0.98(In)  
 Total effective rainfall = 1.02(In)  
 Peak flow rate in flood hydrograph = 24.27(CFS)  
 -----

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 24 - H O U R S T O R M  
 R u n o f f H y d r o g r a p h  
 -----

Hydrograph in 30 Minute intervals ((CFS))

Time(h+m)	Volume Ac.Ft	Q(CFS)	0	7.5	15.0	22.5	30.0
0+30	0.0292	0.71	Q				
1+ 0	0.0884	1.43	VQ				
1+30	0.1542	1.59	V Q				
2+ 0	0.2234	1.67	VQ				
2+30	0.2940	1.71	VQ				
3+ 0	0.3663	1.75	VQ				
3+30	0.4403	1.79	Q				
4+ 0	0.5162	1.84	Q				
4+30	0.5940	1.88	Q				
5+ 0	0.6739	1.93	QV				
5+30	0.7561	1.99	QV				
6+ 0	0.8407	2.05	QV				
6+30	0.9280	2.11	Q V				

			BASINA	2	YEAR	UH				
7+ 0	1.0180	2.18	Q	V						
7+30	1.1112	2.25	Q	V						
8+ 0	1.2077	2.34	Q	V						
8+30	1.3080	2.43	Q	V						
9+ 0	1.4124	2.53	Q	V						
9+30	1.5214	2.64	Q	V						
10+ 0	1.6355	2.76	Q	V						
10+30	1.7555	2.90	Q	V						
11+ 0	1.8821	3.06	Q	V						
11+30	2.0165	3.25	Q	V						
12+ 0	2.1599	3.47	Q	V						
12+30	2.3414	4.39	Q	V						
13+ 0	2.5639	5.39	Q	V						
13+30	2.8089	5.93	Q	V						
14+ 0	3.0781	6.52	Q	V						
14+30	3.3787	7.27	Q	V						
15+ 0	3.7250	8.38	Q	V						
15+30	4.1313	9.83	Q	V						
16+ 0	4.7364	14.64	Q	V						
16+30	5.7394	24.27	Q	V						
17+ 0	6.5758	20.24	Q	V						
17+30	6.9944	10.13	Q	V						
18+ 0	7.2900	7.15	Q	V						
18+30	7.4931	4.91	Q	V						
19+ 0	7.6430	3.63	Q	V						
19+30	7.7708	3.09	Q	V						
20+ 0	7.8843	2.75	Q	V						
20+30	7.9881	2.51	Q	V						
21+ 0	8.0841	2.32	Q	V						
21+30	8.1737	2.17	Q	V						
22+ 0	8.2579	2.04	Q	V						
22+30	8.3375	1.93	Q	V						
23+ 0	8.4130	1.83	Q	V						
23+30	8.4850	1.74	Q	V						
24+ 0	8.5538	1.67	Q	V						

BASINA 100 YEAR UH

Unit Hydrograph Analysis

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Study date 03/12/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

-----  
BASIN A  
100 year unit hydrograph  
RANCHO MIRAMONTE  
-----

Storm Event Year = 100

Antecedent Moisture Condition = 3

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity	isohyetal data:	
Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 100		
35.82	1	1.22
18.70	1	1.22
40.31	1	1.22
3.75	1	1.22
2.54	1	1.22

-----  
Rainfall data for year 100  
101.12                      6                      1.20  
-----

Rainfall data for year 100  
101.12                      24                      1.22  
-----

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\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

(In/Hr)	SCS curve No. (AMCII)	SCS curve NO. (AMC 3)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm
71.6	88.3	88.3	35.82	0.354	0.224	0.667	0.149
69.0	86.2	86.2	18.70	0.185	0.262	0.400	0.105
69.2	86.4	86.4	40.31	0.399	0.259	0.406	0.105

BASINA 100 YEAR UH

73.3	89.6	3.75	0.037	0.199	0.484	0.096
74.0	90.2	2.54	0.025	0.189	0.546	0.103

Area-averaged adjusted loss rate Fm (In/Hr) = 0.120

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area (Ac.)	Area Fract	SCS CN (AMC2)	SCS CN (AMC3)	S	Pervious Yield Fr
23.89	0.236	71.6	88.3	1.33	0.791
11.93	0.118	98.0	98.0	0.20	0.963
7.48	0.074	69.0	86.2	1.60	0.755
11.22	0.111	98.0	98.0	0.20	0.963
16.37	0.162	69.2	86.4	1.58	0.758
23.94	0.237	98.0	98.0	0.20	0.963
1.81	0.018	73.3	89.6	1.16	0.814
1.94	0.019	98.0	98.0	0.20	0.963
1.39	0.014	74.0	90.2	1.09	0.824
1.15	0.011	98.0	98.0	0.20	0.963

Area-averaged catchment yield fraction, Y = 0.869

Area-averaged low loss fraction, Yb = 0.131

User entry of time of concentration = 0.295 (hours)

+++++

Watershed area = 101.12(Ac.)

Catchment Lag time = 0.236 hours

Unit interval = 30.000 minutes

Unit interval percentage of lag time = 211.8644

Hydrograph baseflow = 0.00(CFS)

Average maximum watershed loss rate(Fm) = 0.120(In/Hr)

Average low loss rate fraction (Yb) = 0.131 (decimal)

DESERT S-Graph Selected

Computed peak 5-minute rainfall = 0.452(In)

Computed peak 30-minute rainfall = 0.925(In)

Specified peak 1-hour rainfall = 1.220(In)

Computed peak 3-hour rainfall = 1.220(In)

Specified peak 6-hour rainfall = 3.000(In)

Specified peak 24-hour rainfall = 6.500(In)

Note: user specified rainfall values used.

Rainfall depth area reduction factors:

Using a total area of 101.12(Ac.) (Ref: fig. E-4)

5-minute factor = 0.995 Adjusted rainfall = 0.449(In)

30-minute factor = 0.995 Adjusted rainfall = 0.920(In)

1-hour factor = 0.995 Adjusted rainfall = 1.214(In)

3-hour factor = 0.999 Adjusted rainfall = 1.219(In)

6-hour factor = 1.000 Adjusted rainfall = 2.999(In)

24-hour factor = 1.000 Adjusted rainfall = 6.499(In)

U n i t H y d r o g r a p h

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Interval Number	'S' Graph Mean values	Unit Hydrograph (CFS)
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(K = 203.82 (CFS))

1	44.268	90.228
2	88.969	91.109
3	97.179	16.732

BASINA 100 YEAR UH  
 4                      100.000                      5.750

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Peak Unit Number	Adjusted mass rainfall (In)	Unit rainfall (In)
1	0.9202	0.0647
2	1.2142	0.0415
3	1.2161	0.0003
4	1.2174	0.0002
5	1.2184	0.0002
6	1.2193	0.0001
7	1.4895	0.0459
8	1.7715	0.0478
9	2.0642	0.0495
10	2.3668	0.0511
11	2.6786	0.0526
12	2.9991	0.0540
13	3.1360	0.0225
14	3.2684	0.0218
15	3.3966	0.0211
16	3.5211	0.0205
17	3.6423	0.0200
18	3.7603	0.0195
19	3.8754	0.0190
20	3.9879	0.0186
21	4.0980	0.0182
22	4.2057	0.0178
23	4.3113	0.0175
24	4.4149	0.0171
25	4.5166	0.0168
26	4.6165	0.0165
27	4.7147	0.0163
28	4.8114	0.0160
29	4.9065	0.0158
30	5.0002	0.0155
31	5.0925	0.0153
32	5.1835	0.0151
33	5.2732	0.0149
34	5.3618	0.0147
35	5.4492	0.0145
36	5.5355	0.0143
37	5.6208	0.0141
38	5.7050	0.0140
39	5.7883	0.0138
40	5.8706	0.0137
41	5.9521	0.0135
42	6.0326	0.0134
43	6.1123	0.0132
44	6.1912	0.0131
45	6.2693	0.0130
46	6.3467	0.0128
47	6.4233	0.0127
48	6.4992	0.0126

---

Unit Period (number)	Unit Rainfall (In)	Unit Soil-Loss (In)	Effective Rainfall (In)
1	0.0760	0.0099	0.0661
2	0.0771	0.0101	0.0670
3	0.0782	0.0102	0.0680
4	0.0794	0.0104	0.0691
5	0.0807	0.0105	0.0702
6	0.0820	0.0107	0.0713
7	0.0834	0.0109	0.0725
8	0.0849	0.0111	0.0738
9	0.0865	0.0113	0.0752

BASINA 100 YEAR UH

10	0.0882	0.0115	0.0766
11	0.0900	0.0118	0.0782
12	0.0919	0.0120	0.0799
13	0.0939	0.0123	0.0816
14	0.0961	0.0126	0.0836
15	0.0985	0.0129	0.0856
16	0.1011	0.0132	0.0879
17	0.1039	0.0136	0.0903
18	0.1070	0.0140	0.0930
19	0.1104	0.0144	0.0960
20	0.1143	0.0149	0.0993
21	0.1185	0.0155	0.1030
22	0.1234	0.0161	0.1072
23	0.1289	0.0169	0.1121
24	0.1354	0.0177	0.1177
25	0.3190	0.0417	0.2773
26	0.3057	0.0400	0.2658
27	0.2910	0.0380	0.2529
28	0.2742	0.0358	0.2384
29	0.0009	0.0001	0.0008
30	0.0012	0.0002	0.0011
31	0.0866	0.0113	0.0753
32	0.4893	0.0601	0.4291
33	0.6402	0.0601	0.5801
34	0.0011	0.0001	0.0010
35	0.2798	0.0366	0.2432
36	0.3101	0.0405	0.2696
37	0.1333	0.0174	0.1158
38	0.1218	0.0159	0.1058
39	0.1130	0.0148	0.0982
40	0.1060	0.0139	0.0921
41	0.1002	0.0131	0.0871
42	0.0954	0.0125	0.0829
43	0.0912	0.0119	0.0793
44	0.0876	0.0115	0.0762
45	0.0844	0.0110	0.0734
46	0.0816	0.0107	0.0709
47	0.0790	0.0103	0.0687
48	0.0767	0.0100	0.0667

-----  
 Total soil rain loss = 0.82(In)  
 Total effective rainfall = 5.68(In)  
 Peak flow rate in flood hydrograph = 92.70(CFS)  
 -----

+++++  
 24 - H O U R S T O R M  
 R u n o f f H y d r o g r a p h  
 -----

Hydrograph in 30 Minute intervals ((CFS))

Time(h+m)	Volume Ac.Ft	Q(CFS)	0	25.0	50.0	75.0	100.0
0+30	0.2463	5.96	V Q				
1+ 0	0.7450	12.07	V Q				
1+30	1.2966	13.35	V Q				
2+ 0	1.8721	13.93	V Q				
2+30	2.4566	14.14	V Q				
3+ 0	3.0505	14.37	V Q				
3+30	3.6543	14.61	V Q				
4+ 0	4.2686	14.87	V Q				
4+30	4.8940	15.13	V Q				
5+ 0	5.5311	15.42	V Q				
5+30	6.1807	15.72	V Q				
6+ 0	6.8438	16.05	V Q				

			BASINA 100 YEAR UH			
6+30	7.5211	16.39		Q		
7+ 0	8.2138	16.76		Q		
7+30	8.9231	17.16		QV		
8+ 0	9.6503	17.60		QV		
8+30	10.3971	18.07		QV		
9+ 0	11.1651	18.59		Q V		
9+30	11.9567	19.16		Q Q	V	
10+ 0	12.7742	19.78		Q Q	V	
10+30	13.6208	20.49		Q Q	V	
11+ 0	14.5000	21.28		Q Q	V	
11+30	15.4165	22.18		Q Q	V	
12+ 0	16.3759	23.22		Q	V	
12+30	17.9558	38.23			Q	V
13+ 0	20.0987	51.86			V	Q
13+30	22.2621	52.35			V	Q
14+ 0	24.3528	50.59			Q	Q
14+30	25.4910	27.55		Q	V	
15+ 0	25.7227	5.61	Q		V	
15+30	26.0645	8.27	Q		V	
16+ 0	27.9488	45.60			Q	V
16+30	31.7795	92.70			Q	V
17+ 0	34.2816	60.55			Q	V
17+30	35.6950	34.20		Q		V
18+ 0	37.7542	49.83		Q		V
18+30	39.3694	39.09		Q		V
19+ 0	40.4444	26.01		Q		V
19+30	41.3533	21.99		Q		V
20+ 0	42.1673	19.70		Q		V
20+30	42.9321	18.51		Q		V
21+ 0	43.6564	17.53		Q		V
21+30	44.3464	16.70		Q		V
22+ 0	45.0070	15.99		Q		V
22+30	45.6419	15.37		Q		V
23+ 0	46.2542	14.82		Q		V
23+30	46.8462	14.33		Q		V
24+ 0	47.4201	13.89		Q		V

Prepared for the City of Chino, CA

***APPENDIX H: BASIN B – UNIT HYDROGRAPHS***

BASINB-1 2 YEAR UH

Unit Hydrograph Analysis

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Study date 03/10/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

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BASIN B1  
2 YEAR UNIT HYDROGRAPH 24 HOUR STORM  
RANCHO MIRAMONTE  
-----

Storm Event Year = 2

Antecedent Moisture Condition = 2

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity isohyetal data:  

Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 2		
29.20	1	0.50
7.35	1	0.50
5.21	1	0.50

-----  

Rainfall data for year 2		
29.20	6	1.20
7.35	6	1.20
5.21	6	1.20

-----  

Rainfall data for year 2		
29.20	24	2.00
7.35	24	2.00
5.21	24	2.00

+++++

\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

(In/Hr)	SCS curve No. (AMCII)	SCS curve NO. (AMC 2)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm
	69.0	69.0	29.20	0.699	0.548	0.190	0.104

BASINB-1 2 YEAR UH

69.0	69.0	7.35	0.176	0.548	0.200	0.110
69.0	69.0	5.21	0.125	0.548	0.100	0.055

Area-averaged adjusted loss rate Fm (In/Hr) = 0.099

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area (Ac.)	Area Fract	SCS CN (AMC2)	SCS CN (AMC2)	S	Pervious Yield Fr
5.55	0.133	69.0	69.0	4.49	0.108
23.65	0.566	98.0	98.0	0.20	0.887
1.47	0.035	69.0	69.0	4.49	0.108
5.88	0.141	98.0	98.0	0.20	0.887
0.52	0.012	69.0	69.0	4.49	0.108
4.69	0.112	98.0	98.0	0.20	0.887

Area-averaged catchment yield fraction, Y = 0.747

Area-averaged low loss fraction, Yb = 0.253

User entry of time of concentration = 0.540 (hours)

+++++

watershed area = 41.76(Ac.)

Catchment Lag time = 0.432 hours

Unit interval = 5.000 minutes

Unit interval percentage of lag time = 19.2901

Hydrograph baseflow = 0.00(CFS)

Average maximum watershed loss rate(Fm) = 0.099(In/Hr)

Average low loss rate fraction (Yb) = 0.253 (decimal)

DESERT S-Graph Selected

Computed peak 5-minute rainfall = 0.237(In)

Computed peak 30-minute rainfall = 0.406(In)

Specified peak 1-hour rainfall = 0.500(In)

Computed peak 3-hour rainfall = 0.855(In)

Specified peak 6-hour rainfall = 1.200(In)

Specified peak 24-hour rainfall = 2.000(In)

Rainfall depth area reduction factors:

Using a total area of 41.76(Ac.) (Ref: fig. E-4)

5-minute factor = 0.998 Adjusted rainfall = 0.237(In)

30-minute factor = 0.998 Adjusted rainfall = 0.405(In)

1-hour factor = 0.998 Adjusted rainfall = 0.499(In)

3-hour factor = 1.000 Adjusted rainfall = 0.855(In)

6-hour factor = 1.000 Adjusted rainfall = 1.200(In)

24-hour factor = 1.000 Adjusted rainfall = 2.000(In)

U n i t H y d r o g r a p h

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Interval Number	'S' Graph Mean values	Unit Hydrograph (CFS))
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(K = 505.03 (CFS))

1	0.944	4.769
2	4.246	16.675
3	10.392	31.037
4	23.829	67.862
5	40.957	86.503
6	53.094	61.294
7	61.260	41.245
8	67.075	29.367
9	71.724	23.477
10	75.446	18.797

BASINB-1 2 YEAR UH

11	78.589	15.875
12	81.156	12.965
13	83.341	11.036
14	85.300	9.891
15	87.066	8.921
16	88.581	7.649
17	89.796	6.138
18	90.904	5.596
19	91.928	5.170
20	92.843	4.623
21	93.693	4.290
22	94.401	3.576
23	95.057	3.312
24	95.678	3.136
25	96.188	2.575
26	96.689	2.530
27	97.101	2.083
28	97.449	1.754
29	97.777	1.661
30	97.991	1.076
31	98.183	0.974
32	98.392	1.052
33	98.623	1.168
34	98.855	1.169
35	99.086	1.169
36	99.318	1.169
37	99.521	1.027
38	99.646	0.632
39	99.767	0.609
40	99.887	0.609
41	100.000	0.569

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 -----  
 Total soil rain loss = 0.45(In)  
 Total effective rainfall = 1.55(In)  
 Peak flow rate in flood hydrograph = 28.58(CFS)  
 -----

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 24 - H O U R S T O R M  
 R u n o f f H y d r o g r a p h  
 -----

Hydrograph in 5 Minute intervals ((CFS))

Time(h+m)	Volume Ac.Ft	Q(CFS)	0	7.5	15.0	22.5	30.0
0+ 5	0.0001	0.01	Q				
0+10	0.0003	0.04	Q				
0+15	0.0010	0.10	Q				
0+20	0.0026	0.23	Q				
0+25	0.0054	0.40	Q				
0+30	0.0089	0.52	Q				
0+35	0.0130	0.60	Q				
0+40	0.0175	0.65	Q				
0+45	0.0223	0.70	Q				
0+50	0.0274	0.74	Q				
0+55	0.0327	0.77	VQ				
1+ 0	0.0383	0.80	VQ				
1+ 5	0.0439	0.82	VQ				
1+10	0.0497	0.85	VQ				
1+15	0.0557	0.87	VQ				
1+20	0.0618	0.88	VQ				
1+25	0.0680	0.90	VQ				
1+30	0.0742	0.91	VQ				
1+35	0.0806	0.92	VQ				

BASINB-1 2 YEAR UH

1+40	0.0871	0.94	VQ			
1+45	0.0936	0.95	VQ			
1+50	0.1002	0.96	VQ			
1+55	0.1069	0.97	VQ			
2+ 0	0.1136	0.98	VQ			
2+ 5	0.1204	0.99	VQ			
2+10	0.1272	0.99	VQ			
2+15	0.1341	1.00	VQ			
2+20	0.1411	1.01	Q			
2+25	0.1481	1.02	Q			
2+30	0.1551	1.02	Q			
2+35	0.1622	1.03	Q			
2+40	0.1693	1.03	Q			
2+45	0.1765	1.04	Q			
2+50	0.1837	1.05	Q			
2+55	0.1909	1.05	Q			
3+ 0	0.1982	1.06	Q			
3+ 5	0.2055	1.06	Q			
3+10	0.2129	1.07	Q			
3+15	0.2203	1.07	Q			
3+20	0.2277	1.08	Q			
3+25	0.2352	1.09	Q			
3+30	0.2427	1.09	Q			
3+35	0.2502	1.09	Q			
3+40	0.2578	1.10	Q			
3+45	0.2654	1.10	Q			
3+50	0.2730	1.11	QV			
3+55	0.2807	1.11	QV			
4+ 0	0.2884	1.12	QV			
4+ 5	0.2961	1.12	QV			
4+10	0.3039	1.13	QV			
4+15	0.3117	1.13	QV			
4+20	0.3195	1.14	QV			
4+25	0.3273	1.14	QV			
4+30	0.3352	1.15	QV			
4+35	0.3432	1.15	QV			
4+40	0.3511	1.16	QV			
4+45	0.3591	1.16	QV			
4+50	0.3671	1.17	QV			
4+55	0.3752	1.17	QV			
5+ 0	0.3833	1.18	QV			
5+ 5	0.3915	1.18	QV			
5+10	0.3996	1.19	QV			
5+15	0.4078	1.19	Q V			
5+20	0.4161	1.20	Q V			
5+25	0.4244	1.20	Q V			
5+30	0.4327	1.21	Q V			
5+35	0.4411	1.22	Q V			
5+40	0.4495	1.22	Q V			
5+45	0.4579	1.23	Q V			
5+50	0.4664	1.23	Q V			
5+55	0.4750	1.24	Q V			
6+ 0	0.4835	1.24	Q V			
6+ 5	0.4922	1.25	Q V			
6+10	0.5008	1.26	Q V			
6+15	0.5095	1.26	Q V			
6+20	0.5183	1.27	Q V			
6+25	0.5271	1.28	Q V			
6+30	0.5359	1.28	Q V			
6+35	0.5448	1.29	Q V			
6+40	0.5537	1.30	Q V			
6+45	0.5627	1.30	Q V			
6+50	0.5717	1.31	Q V			
6+55	0.5808	1.32	Q V			
7+ 0	0.5899	1.32	Q V			
7+ 5	0.5991	1.33	Q V			

BASINB-1 2 YEAR UH

7+10	0.6083	1.34	Q	V			
7+15	0.6176	1.35	Q	V			
7+20	0.6269	1.35	Q	V			
7+25	0.6363	1.36	Q	V			
7+30	0.6457	1.37	Q	V			
7+35	0.6552	1.38	Q	V			
7+40	0.6648	1.39	Q	V			
7+45	0.6744	1.39	Q	V			
7+50	0.6840	1.40	Q	V			
7+55	0.6938	1.41	Q	V			
8+ 0	0.7035	1.42	Q	V			
8+ 5	0.7134	1.43	Q	V			
8+10	0.7233	1.44	Q	V			
8+15	0.7332	1.45	Q	V			
8+20	0.7432	1.46	Q	V			
8+25	0.7533	1.46	Q	V			
8+30	0.7635	1.47	Q	V			
8+35	0.7737	1.48	Q	V			
8+40	0.7840	1.49	Q	V			
8+45	0.7943	1.50	Q	V			
8+50	0.8048	1.51	Q	V			
8+55	0.8152	1.52	Q	V			
9+ 0	0.8258	1.53	Q	V			
9+ 5	0.8364	1.54	Q	V			
9+10	0.8472	1.56	Q	V			
9+15	0.8579	1.57	Q	V			
9+20	0.8688	1.58	Q	V			
9+25	0.8798	1.59	Q	V			
9+30	0.8908	1.60	Q	V			
9+35	0.9019	1.61	Q	V			
9+40	0.9131	1.62	Q	V			
9+45	0.9243	1.64	Q	V			
9+50	0.9357	1.65	Q	V			
9+55	0.9472	1.66	Q	V			
10+ 0	0.9587	1.68	Q	V			
10+ 5	0.9703	1.69	Q	V			
10+10	0.9821	1.70	Q	V			
10+15	0.9939	1.72	Q	V			
10+20	1.0058	1.73	Q	V			
10+25	1.0178	1.75	Q	V			
10+30	1.0300	1.76	Q	V			
10+35	1.0422	1.78	Q	V			
10+40	1.0545	1.79	Q	V			
10+45	1.0670	1.81	Q	V			
10+50	1.0795	1.82	Q	V			
10+55	1.0922	1.84	Q	V			
11+ 0	1.1050	1.86	Q	V			
11+ 5	1.1179	1.88	Q	V			
11+10	1.1310	1.89	Q	V			
11+15	1.1441	1.91	Q	V			
11+20	1.1574	1.93	Q	V			
11+25	1.1708	1.95	Q	V			
11+30	1.1844	1.97	Q	V			
11+35	1.1981	1.99	Q	V			
11+40	1.2120	2.01	Q	V			
11+45	1.2260	2.03	Q	V			
11+50	1.2401	2.06	Q	V			
11+55	1.2544	2.08	Q	V			
12+ 0	1.2689	2.10	Q	V			
12+ 5	1.2836	2.13	Q	V			
12+10	1.2986	2.18	Q	V			
12+15	1.3142	2.26	Q	V			
12+20	1.3306	2.38	Q	V			
12+25	1.3481	2.54	Q	V			
12+30	1.3664	2.66	Q	V			
12+35	1.3854	2.75	Q	V			

BASINB-1 2 YEAR UH

12+40	1.4049	2.83	Q	V		
12+45	1.4249	2.90	Q	V		
12+50	1.4452	2.96	Q	V		
12+55	1.4660	3.02	Q	V		
13+ 0	1.4872	3.08	Q	V		
13+ 5	1.5088	3.13	Q	V		
13+10	1.5307	3.19	Q	V		
13+15	1.5530	3.24	Q	V		
13+20	1.5757	3.29	Q	V		
13+25	1.5988	3.35	Q	V		
13+30	1.6222	3.40	Q	V		
13+35	1.6460	3.46	Q	V		
13+40	1.6703	3.52	Q	V		
13+45	1.6949	3.58	Q	V		
13+50	1.7199	3.64	Q	V		
13+55	1.7454	3.70	Q	V		
14+ 0	1.7713	3.76	Q	V		
14+ 5	1.7977	3.83	Q	V		
14+10	1.8246	3.90	Q	V		
14+15	1.8520	3.98	Q	V		
14+20	1.8800	4.06	Q	V		
14+25	1.9085	4.14	Q	V		
14+30	1.9376	4.23	Q	V		
14+35	1.9674	4.32	Q	V		
14+40	1.9979	4.42	Q	V		
14+45	2.0291	4.53	Q	V		
14+50	2.0610	4.64	Q	V		
14+55	2.0938	4.76	Q	V		
15+ 0	2.1275	4.89	Q	V		
15+ 5	2.1622	5.04	Q	V		
15+10	2.1979	5.19	Q	V		
15+15	2.2349	5.36	Q	V		
15+20	2.2731	5.55	Q	V		
15+25	2.3127	5.74	Q	V		
15+30	2.3532	5.88	Q	V		
15+35	2.3942	5.97	Q	V		
15+40	2.4347	5.88	Q	V		
15+45	2.4742	5.73	Q	V		
15+50	2.5141	5.80	Q	V		
15+55	2.5564	6.14	Q	V		
16+ 0	2.6035	6.84	Q	V		
16+ 5	2.6646	8.87	Q	V		
16+10	2.7518	12.66	Q	V	Q	
16+15	2.8707	17.26		V	Q	
16+20	3.0459	25.44		V	Q	Q
16+25	3.2427	28.58		V	Q	Q
16+30	3.3976	22.48		V	Q	Q
16+35	3.5174	17.40		V	Q	Q
16+40	3.6163	14.36		V	Q	Q
16+45	3.7045	12.81		V	Q	Q
16+50	3.7836	11.49		V	Q	Q
16+55	3.8559	10.50		V	Q	Q
17+ 0	3.9216	9.53		V	Q	Q
17+ 5	3.9823	8.81		V	Q	Q
17+10	4.0392	8.27		V	Q	Q
17+15	4.0928	7.78		V	Q	Q
17+20	4.1426	7.24		V	Q	Q
17+25	4.1887	6.69		V	Q	Q
17+30	4.2324	6.36		V	Q	Q
17+35	4.2743	6.07		V	Q	Q
17+40	4.3140	5.78		V	Q	Q
17+45	4.3521	5.53		V	Q	Q
17+50	4.3881	5.22		V	Q	Q
17+55	4.4227	5.02		V	Q	Q
18+ 0	4.4560	4.84		V	Q	Q
18+ 5	4.4876	4.59		V	Q	Q

BASINB-1 2 YEAR UH

18+10	4.5181	4.42	Q	V
18+15	4.5467	4.16	Q	V
18+20	4.5734	3.88	Q	V
18+25	4.5983	3.62	Q	V
18+30	4.6212	3.32	Q	V
18+35	4.6429	3.15	Q	V
18+40	4.6639	3.05	Q	V
18+45	4.6844	2.98	Q	V
18+50	4.7043	2.88	Q	V
18+55	4.7235	2.79	Q	V
19+ 0	4.7422	2.71	Q	V
19+ 5	4.7600	2.59	Q	V
19+10	4.7768	2.43	Q	V
19+15	4.7930	2.36	Q	V
19+20	4.8088	2.29	Q	V
19+25	4.8240	2.21	Q	V
19+30	4.8380	2.03	Q	V
19+35	4.8516	1.98	Q	V
19+40	4.8649	1.93	Q	V
19+45	4.8779	1.89	Q	V
19+50	4.8906	1.85	Q	V
19+55	4.9031	1.81	Q	V
20+ 0	4.9153	1.77	Q	V
20+ 5	4.9272	1.73	Q	V
20+10	4.9389	1.70	Q	V
20+15	4.9504	1.67	Q	V
20+20	4.9618	1.64	Q	V
20+25	4.9729	1.61	Q	V
20+30	4.9838	1.59	Q	V
20+35	4.9946	1.56	Q	V
20+40	5.0052	1.54	Q	V
20+45	5.0157	1.52	Q	V
20+50	5.0260	1.50	Q	V
20+55	5.0361	1.47	Q	V
21+ 0	5.0461	1.45	Q	V
21+ 5	5.0560	1.43	Q	V
21+10	5.0657	1.41	Q	V
21+15	5.0754	1.40	Q	V
21+20	5.0849	1.38	Q	V
21+25	5.0942	1.36	Q	V
21+30	5.1035	1.35	Q	V
21+35	5.1127	1.33	Q	V
21+40	5.1217	1.32	Q	V
21+45	5.1307	1.30	Q	V
21+50	5.1396	1.29	Q	V
21+55	5.1484	1.28	Q	V
22+ 0	5.1571	1.26	Q	V
22+ 5	5.1657	1.25	Q	V
22+10	5.1742	1.24	Q	V
22+15	5.1826	1.23	Q	V
22+20	5.1910	1.21	Q	V
22+25	5.1993	1.20	Q	V
22+30	5.2075	1.19	Q	V
22+35	5.2156	1.18	Q	V
22+40	5.2236	1.17	Q	V
22+45	5.2316	1.16	Q	V
22+50	5.2395	1.15	Q	V
22+55	5.2474	1.14	Q	V
23+ 0	5.2551	1.13	Q	V
23+ 5	5.2629	1.12	Q	V
23+10	5.2705	1.11	Q	V
23+15	5.2781	1.10	Q	V
23+20	5.2856	1.09	Q	V
23+25	5.2931	1.08	Q	V
23+30	5.3005	1.07	Q	V
23+35	5.3078	1.07	Q	V

BASINB-1 2 YEAR UH

23+40	5.3151	1.06	Q				V
23+45	5.3223	1.05	Q				V
23+50	5.3295	1.04	Q				V
23+55	5.3366	1.03	Q				V
24+ 0	5.3437	1.03	Q				V

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BASINB-2 2 YEAR UH

Unit Hydrograph Analysis

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Study date 03/10/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

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BASIN B-2 UNIT HYDROGRAPH  
2 YEAR 24 HR STORM  
RANCHO MIRAMONTE  
-----

Storm Event Year = 2

Antecedent Moisture Condition = 2

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity isohyetal data:	Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 2	35.55	1	0.48
Rainfall data for year 2	35.55	6	1.20
Rainfall data for year 2	35.55	24	2.00

+++++

\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

(In/Hr)	SCS curve No. (AMCII)	SCS curve NO. (AMC 2)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm
70.0	70.0	70.0	35.55	1.000	0.532	0.406	0.216

Area-averaged adjusted loss rate Fm (In/Hr) = 0.216

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area Area SCS CN SCS CN S Pervious

		BASINB-2 2 YEAR UH			
(Ac.)	Fract	(AMC2)	(AMC2)	Yield Fr	
14.43	0.406	70.0	70.0	4.29	0.120
21.12	0.594	98.0	98.0	0.20	0.887

Area-averaged catchment yield fraction, Y = 0.576  
 Area-averaged low loss fraction, Yb = 0.424  
 User entry of time of concentration = 0.210 (hours)  
 ++++++  
 Watershed area = 35.55(Ac.)  
 Catchment Lag time = 0.168 hours  
 Unit interval = 5.000 minutes  
 Unit interval percentage of lag time = 49.4854  
 Hydrograph baseflow = 0.00(CFS)  
 Average maximum watershed loss rate(Fm) = 0.216(In/Hr)  
 Average low loss rate fraction (Yb) = 0.424 (decimal)  
 DESERT S-Graph Selected  
 Computed peak 5-minute rainfall = 0.178(In)  
 Computed peak 30-minute rainfall = 0.364(In)  
 Specified peak 1-hour rainfall = 0.480(In)  
 Computed peak 3-hour rainfall = 0.842(In)  
 Specified peak 6-hour rainfall = 1.200(In)  
 Specified peak 24-hour rainfall = 2.000(In)

Rainfall depth area reduction factors:  
 Using a total area of 35.55(Ac.) (Ref: fig. E-4)

5-minute factor = 0.998	Adjusted rainfall = 0.177(In)
30-minute factor = 0.998	Adjusted rainfall = 0.363(In)
1-hour factor = 0.998	Adjusted rainfall = 0.479(In)
3-hour factor = 1.000	Adjusted rainfall = 0.842(In)
6-hour factor = 1.000	Adjusted rainfall = 1.200(In)
24-hour factor = 1.000	Adjusted rainfall = 2.000(In)

Unit Hydrograph

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Interval Number	'S' Graph Mean values	Unit Hydrograph ((CFS))
(K = 429.93 (CFS))		
1	3.895	16.748
2	29.905	111.824
3	60.095	129.795
4	73.423	57.303
5	81.171	33.310
6	86.350	22.266
7	89.979	15.602
8	92.596	11.252
9	94.600	8.614
10	96.121	6.542
11	97.251	4.856
12	97.988	3.171
13	98.524	2.302
14	99.117	2.550
15	99.607	2.107
16	100.000	1.691

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Total soil rain loss = 0.79(In)  
 Total effective rainfall = 1.21(In)  
 Peak flow rate in flood hydrograph = 27.14(CFS)

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BASINB-2 2 YEAR UH  
24 - H O U R S T O R M  
R u n o f f H y d r o g r a p h

-----  
Hydrograph in 5 Minute intervals ((CFS))  
-----

Time(h+m)	Volume	Ac.Ft	Q(CFS)	0	7.5	15.0	22.5	30.0
0+ 5	0.0002		0.02	Q				
0+10	0.0015		0.19	Q				
0+15	0.0041		0.38	Q				
0+20	0.0073		0.47	Q				
0+25	0.0109		0.52	Q				
0+30	0.0147		0.55	Q				
0+35	0.0187		0.58	Q				
0+40	0.0228		0.60	Q				
0+45	0.0270		0.61	Q				
0+50	0.0313		0.62	Q				
0+55	0.0356		0.63	Q				
1+ 0	0.0400		0.64	Q				
1+ 5	0.0445		0.64	Q				
1+10	0.0489		0.65	Q				
1+15	0.0535		0.66	Q				
1+20	0.0580		0.66	Q				
1+25	0.0626		0.66	Q				
1+30	0.0671		0.67	Q				
1+35	0.0717		0.67	Q				
1+40	0.0764		0.67	Q				
1+45	0.0810		0.67	Q				
1+50	0.0856		0.67	Q				
1+55	0.0903		0.68	QV				
2+ 0	0.0950		0.68	QV				
2+ 5	0.0997		0.68	QV				
2+10	0.1044		0.68	QV				
2+15	0.1091		0.69	QV				
2+20	0.1139		0.69	QV				
2+25	0.1186		0.69	QV				
2+30	0.1234		0.70	QV				
2+35	0.1282		0.70	QV				
2+40	0.1331		0.70	QV				
2+45	0.1379		0.70	QV				
2+50	0.1428		0.71	QV				
2+55	0.1476		0.71	QV				
3+ 0	0.1525		0.71	QV				
3+ 5	0.1575		0.71	QV				
3+10	0.1624		0.72	QV				
3+15	0.1674		0.72	QV				
3+20	0.1723		0.72	QV				
3+25	0.1773		0.73	QV				
3+30	0.1824		0.73	Q V				
3+35	0.1874		0.73	Q V				
3+40	0.1925		0.73	Q V				
3+45	0.1975		0.74	Q V				
3+50	0.2026		0.74	Q V				
3+55	0.2078		0.74	Q V				
4+ 0	0.2129		0.75	Q V				
4+ 5	0.2181		0.75	QV				
4+10	0.2233		0.75	QV				
4+15	0.2285		0.76	QV				
4+20	0.2337		0.76	QV				
4+25	0.2390		0.76	QV				
4+30	0.2443		0.77	QV				
4+35	0.2496		0.77	QV				
4+40	0.2549		0.77	QV				
4+45	0.2603		0.78	QV				
4+50	0.2657		0.78	QV				

BASINB-2 2 YEAR UH

4+55	0.2711	0.78	Q	V			
5+ 0	0.2765	0.79	Q	V			
5+ 5	0.2819	0.79	Q	V			
5+10	0.2874	0.80	Q	V			
5+15	0.2929	0.80	Q	V			
5+20	0.2985	0.80	Q	V			
5+25	0.3040	0.81	Q	V			
5+30	0.3096	0.81	Q	V			
5+35	0.3152	0.82	Q	V			
5+40	0.3209	0.82	Q	V			
5+45	0.3265	0.82	Q	V			
5+50	0.3322	0.83	Q	V			
5+55	0.3380	0.83	Q	V			
6+ 0	0.3437	0.84	Q	V			
6+ 5	0.3495	0.84	Q	V			
6+10	0.3553	0.84	Q	V			
6+15	0.3612	0.85	Q	V			
6+20	0.3671	0.85	Q	V			
6+25	0.3730	0.86	Q	V			
6+30	0.3789	0.86	Q	V			
6+35	0.3849	0.87	Q	V			
6+40	0.3909	0.87	Q	V			
6+45	0.3969	0.88	Q	V			
6+50	0.4030	0.88	Q	V			
6+55	0.4091	0.89	Q	V			
7+ 0	0.4152	0.89	Q	V			
7+ 5	0.4214	0.90	Q	V			
7+10	0.4276	0.90	Q	V			
7+15	0.4339	0.91	Q	V			
7+20	0.4402	0.91	Q	V			
7+25	0.4465	0.92	Q	V			
7+30	0.4528	0.92	Q	V			
7+35	0.4592	0.93	Q	V			
7+40	0.4657	0.93	Q	V			
7+45	0.4721	0.94	Q	V			
7+50	0.4787	0.95	Q	V			
7+55	0.4852	0.95	Q	V			
8+ 0	0.4918	0.96	Q	V			
8+ 5	0.4984	0.96	Q	V			
8+10	0.5051	0.97	Q	V			
8+15	0.5118	0.98	Q	V			
8+20	0.5186	0.98	Q	V			
8+25	0.5254	0.99	Q	V			
8+30	0.5323	1.00	Q	V			
8+35	0.5392	1.00	Q	V			
8+40	0.5462	1.01	Q	V			
8+45	0.5532	1.02	Q	V			
8+50	0.5602	1.02	Q	V			
8+55	0.5673	1.03	Q	V			
9+ 0	0.5745	1.04	Q	V			
9+ 5	0.5817	1.05	Q	V			
9+10	0.5889	1.05	Q	V			
9+15	0.5962	1.06	Q	V			
9+20	0.6036	1.07	Q	V			
9+25	0.6110	1.08	Q	V			
9+30	0.6185	1.09	Q	V			
9+35	0.6261	1.09	Q	V			
9+40	0.6337	1.10	Q	V			
9+45	0.6413	1.11	Q	V			
9+50	0.6490	1.12	Q	V			
9+55	0.6568	1.13	Q	V			
10+ 0	0.6647	1.14	Q	V			
10+ 5	0.6726	1.15	Q	V			
10+10	0.6806	1.16	Q	V			
10+15	0.6886	1.17	Q	V			
10+20	0.6967	1.18	Q	V			

BASINB-2 2 YEAR UH

10+25	0.7049	1.19	Q	V					
10+30	0.7132	1.20	Q	V					
10+35	0.7216	1.21	Q	V					
10+40	0.7300	1.22	Q	V					
10+45	0.7385	1.23	Q	V					
10+50	0.7471	1.25	Q	V					
10+55	0.7557	1.26	Q	V					
11+ 0	0.7645	1.27	Q	V					
11+ 5	0.7733	1.28	Q	V					
11+10	0.7823	1.30	Q	V					
11+15	0.7913	1.31	Q	V					
11+20	0.8004	1.32	Q	V					
11+25	0.8096	1.34	Q	V					
11+30	0.8189	1.35	Q	V					
11+35	0.8284	1.37	Q	V					
11+40	0.8379	1.38	Q	V					
11+45	0.8475	1.40	Q	V					
11+50	0.8573	1.42	Q	V					
11+55	0.8672	1.43	Q	V					
12+ 0	0.8771	1.45	Q	V					
12+ 5	0.8874	1.49	Q	V					
12+10	0.8989	1.66	Q	V					
12+15	0.9117	1.86	Q	V					
12+20	0.9252	1.96	Q	V					
12+25	0.9392	2.03	Q	V					
12+30	0.9535	2.08	Q	V					
12+35	0.9681	2.13	Q	V					
12+40	0.9831	2.17	Q	V					
12+45	0.9982	2.20	Q	V					
12+50	1.0136	2.24	Q	V					
12+55	1.0293	2.27	Q	V					
13+ 0	1.0451	2.30	Q	V					
13+ 5	1.0612	2.34	Q	V					
13+10	1.0775	2.37	Q	V					
13+15	1.0941	2.40	Q	V					
13+20	1.1109	2.44	Q	V					
13+25	1.1279	2.47	Q	V					
13+30	1.1452	2.51	Q	V					
13+35	1.1627	2.55	Q	V					
13+40	1.1806	2.59	Q	V					
13+45	1.1986	2.63	Q	V					
13+50	1.2170	2.67	Q	V					
13+55	1.2357	2.71	Q	V					
14+ 0	1.2547	2.76	Q	V					
14+ 5	1.2741	2.81	Q	V					
14+10	1.2939	2.87	Q	V					
14+15	1.3140	2.93	Q	V					
14+20	1.3346	2.99	Q	V					
14+25	1.3556	3.05	Q	V					
14+30	1.3771	3.12	Q	V					
14+35	1.3991	3.20	Q	V					
14+40	1.4217	3.27	Q	V					
14+45	1.4448	3.36	Q	V					
14+50	1.4686	3.45	Q	V					
14+55	1.4931	3.55	Q	V					
15+ 0	1.5183	3.67	Q	V					
15+ 5	1.5444	3.79	Q	V					
15+10	1.5715	3.93	Q	V					
15+15	1.5995	4.08	Q	V					
15+20	1.6288	4.25	Q	V					
15+25	1.6591	4.40	Q	V					
15+30	1.6891	4.35	Q	V					
15+35	1.7185	4.28	Q	V					
15+40	1.7493	4.46	Q	V					
15+45	1.7822	4.78	Q	V					
15+50	1.8186	5.29	Q	V					

BASINB-2 2 YEAR UH

15+55	1.8602	6.04	Q	V		
16+ 0	1.9122	7.54	Q	V		
16+ 5	1.9975	12.39	Q	V		
16+10	2.1768	26.04	Q	V		
16+15	2.3637	27.14		V		Q
16+20	2.4732	15.89		V		Q
16+25	2.5511	11.32		V		
16+30	2.6137	9.09	Q	V		
16+35	2.6670	7.74	Q	V		
16+40	2.7130	6.68	Q	V		
16+45	2.7538	5.91	Q	V		
16+50	2.7901	5.27	Q	V		
16+55	2.8227	4.73	Q	V		
17+ 0	2.8519	4.24	Q	V		
17+ 5	2.8788	3.92	Q	V		
17+10	2.9047	3.76	Q	V		
17+15	2.9289	3.52	Q	V		
17+20	2.9515	3.28	Q	V		
17+25	2.9715	2.90	Q	V		
17+30	2.9907	2.78	Q	V		
17+35	3.0092	2.69	Q	V		
17+40	3.0271	2.60	Q	V		
17+45	3.0444	2.52	Q	V		
17+50	3.0612	2.44	Q	V		
17+55	3.0776	2.38	Q	V		
18+ 0	3.0936	2.32	Q	V		
18+ 5	3.1090	2.24	Q	V		
18+10	3.1230	2.03	Q	V		
18+15	3.1354	1.81	Q	V		
18+20	3.1470	1.68	Q	V		
18+25	3.1580	1.60	Q	V		
18+30	3.1686	1.53	Q	V		
18+35	3.1787	1.47	Q	V		
18+40	3.1886	1.43	Q	V		
18+45	3.1981	1.38	Q	V		
18+50	3.2074	1.35	Q	V		
18+55	3.2164	1.31	Q	V		
19+ 0	3.2252	1.28	Q	V		
19+ 5	3.2338	1.25	Q	V		
19+10	3.2423	1.23	Q	V		
19+15	3.2506	1.20	Q	V		
19+20	3.2587	1.18	Q	V		
19+25	3.2666	1.16	Q	V		
19+30	3.2745	1.14	Q	V		
19+35	3.2822	1.12	Q	V		
19+40	3.2897	1.10	Q	V		
19+45	3.2972	1.08	Q	V		
19+50	3.3046	1.07	Q	V		
19+55	3.3118	1.05	Q	V		
20+ 0	3.3189	1.04	Q	V		
20+ 5	3.3260	1.02	Q	V		
20+10	3.3329	1.01	Q	V		
20+15	3.3398	0.99	Q	V		
20+20	3.3465	0.98	Q	V		
20+25	3.3532	0.97	Q	V		
20+30	3.3598	0.96	Q	V		
20+35	3.3663	0.94	Q	V		
20+40	3.3727	0.93	Q	V		
20+45	3.3790	0.92	Q	V		
20+50	3.3853	0.91	Q	V		
20+55	3.3915	0.90	Q	V		
21+ 0	3.3976	0.89	Q	V		
21+ 5	3.4037	0.88	Q	V		
21+10	3.4097	0.87	Q	V		
21+15	3.4156	0.86	Q	V		
21+20	3.4215	0.85	Q	V		

BASINB-2 2 YEAR UH

21+25	3.4273	0.84	Q			V
21+30	3.4330	0.83	Q			V
21+35	3.4387	0.83	Q			V
21+40	3.4444	0.82	Q			V
21+45	3.4499	0.81	Q			V
21+50	3.4555	0.80	Q			V
21+55	3.4609	0.79	Q			V
22+ 0	3.4664	0.79	Q			V
22+ 5	3.4717	0.78	Q			V
22+10	3.4771	0.77	Q			V
22+15	3.4823	0.77	Q			V
22+20	3.4876	0.76	Q			V
22+25	3.4927	0.75	Q			V
22+30	3.4979	0.75	Q			V
22+35	3.5030	0.74	Q			V
22+40	3.5080	0.73	Q			V
22+45	3.5130	0.73	Q			V
22+50	3.5180	0.72	Q			V
22+55	3.5229	0.72	Q			V
23+ 0	3.5278	0.71	Q			V
23+ 5	3.5327	0.70	Q			V
23+10	3.5375	0.70	Q			V
23+15	3.5423	0.69	Q			V
23+20	3.5470	0.69	Q			V
23+25	3.5517	0.68	Q			V
23+30	3.5564	0.68	Q			V
23+35	3.5610	0.67	Q			V
23+40	3.5657	0.67	Q			V
23+45	3.5702	0.66	Q			V
23+50	3.5748	0.66	Q			V
23+55	3.5793	0.65	Q			V
24+ 0	3.5838	0.65	Q			V

U n i t   H y d r o g r a p h   A n a l y s i s

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Study date 03/12/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

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BASIN B1  
100 YEAR 24 HR UNIT HYDROGRAPH  
RANCHO MIRAMONTE  
-----

Storm Event Year = 100

Antecedent Moisture Condition = 3

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity isohyetal data:

Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 100		
29.20	1	1.22
7.35	1	1.22
5.21	1	1.22

-----  
Rainfall data for year 100  
41.76                      6                      3.00  
-----

-----  
Rainfall data for year 100  
41.76                      24                      6.50  
-----

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\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

SCS curve No.(AMCII)	SCS curve NO.(AMC 3)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm (In/Hr)
69.0	86.2	29.20	0.699	0.262	0.190	0.050
69.0	86.2	7.35	0.176	0.262	0.200	0.052
69.0	86.2	5.21	0.125	0.262	0.100	0.026

Area-averaged adjusted loss rate Fm (In/Hr) = 0.047

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area (Ac.)	Area Fract	SCS CN (AMC2)	SCS CN (AMC3)	S	Pervious Yield Fr
5.55	0.133	69.0	86.2	1.60	0.755
23.65	0.566	98.0	98.0	0.20	0.963
1.47	0.035	69.0	86.2	1.60	0.755
5.88	0.141	98.0	98.0	0.20	0.963
0.52	0.012	69.0	86.2	1.60	0.755
4.69	0.112	98.0	98.0	0.20	0.963

Area-averaged catchment yield fraction, Y = 0.926

Area-averaged low loss fraction, Yb = 0.074

User entry of time of concentration = 0.540 (hours)

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Watershed area = 41.76(Ac.)

Catchment Lag time = 0.432 hours

Unit interval = 5.000 minutes

Unit interval percentage of lag time = 19.2901

Hydrograph baseflow = 0.00(CFS)

Average maximum watershed loss rate(Fm) = 0.047(In/Hr)

Average low loss rate fraction (Yb) = 0.074 (decimal)

DESERT S-Graph Selected

Computed peak 5-minute rainfall = 0.452(In)

Computed peak 30-minute rainfall = 0.925(In)

Specified peak 1-hour rainfall = 1.220(In)

Computed peak 3-hour rainfall = 2.118(In)

Specified peak 6-hour rainfall = 3.000(In)

Specified peak 24-hour rainfall = 6.500(In)

Rainfall depth area reduction factors:

Using a total area of 41.76(Ac.) (Ref: fig. E-4)

5-minute factor = 0.998 Adjusted rainfall = 0.451(In)

30-minute factor = 0.998 Adjusted rainfall = 0.923(In)

1-hour factor = 0.998 Adjusted rainfall = 1.218(In)

3-hour factor = 1.000 Adjusted rainfall = 2.118(In)

6-hour factor = 1.000                      Adjusted rainfall = 3.000(In)  
 24-hour factor = 1.000                     Adjusted rainfall = 6.500(In)

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 U n i t   H y d r o g r a p h

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Interval                      'S' Graph                      Unit Hydrograph  
 Number                        Mean values                      ((CFS))

-----  
 (K =                      505.03 (CFS))

1	0.944	4.769
2	4.246	16.675
3	10.392	31.037
4	23.829	67.862
5	40.957	86.503
6	53.094	61.294
7	61.260	41.245
8	67.075	29.367
9	71.724	23.477
10	75.446	18.797
11	78.589	15.875
12	81.156	12.965
13	83.341	11.036
14	85.300	9.891
15	87.066	8.921
16	88.581	7.649
17	89.796	6.138
18	90.904	5.596
19	91.928	5.170
20	92.843	4.623
21	93.693	4.290
22	94.401	3.576
23	95.057	3.312
24	95.678	3.136
25	96.188	2.575
26	96.689	2.530
27	97.101	2.083
28	97.449	1.754
29	97.777	1.661
30	97.991	1.076
31	98.183	0.974
32	98.392	1.052
33	98.623	1.168
34	98.855	1.169
35	99.086	1.169
36	99.318	1.169
37	99.521	1.027
38	99.646	0.632
39	99.767	0.609
40	99.887	0.609
41	100.000	0.569

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 -----  
 Total soil rain loss = 0.44(In)  
 Total effective rainfall = 6.06(In)  
 Peak flow rate in flood hydrograph = 68.62(CFS)  
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 24 - H O U R S T O R M  
 R u n o f f H y d r o g r a p h  
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Hydrograph in 5 Minute intervals ((CFS))

Time(h+m)	Volume Ac.Ft	Q(CFS)	0	17.5	35.0	52.5	70.0
0+ 5	0.0004	0.06	Q				
0+10	0.0021	0.25	Q				
0+15	0.0063	0.61	Q				
0+20	0.0160	1.41	Q				
0+25	0.0326	2.42	VQ				
0+30	0.0543	3.14	VQ				
0+35	0.0792	3.63	V Q				
0+40	0.1066	3.98	V Q				
0+45	0.1359	4.26	V Q				
0+50	0.1668	4.49	V Q				
0+55	0.1991	4.68	V Q				
1+ 0	0.2325	4.85	V Q				
1+ 5	0.2668	4.99	V Q				
1+10	0.3020	5.11	V Q				
1+15	0.3381	5.23	V Q				
1+20	0.3748	5.33	V Q				
1+25	0.4121	5.42	V Q				
1+30	0.4499	5.49	V Q				
1+35	0.4883	5.57	V Q				
1+40	0.5271	5.64	V Q				
1+45	0.5663	5.70	V Q				
1+50	0.6060	5.76	V Q				
1+55	0.6460	5.81	V Q				
2+ 0	0.6864	5.86	V Q				
2+ 5	0.7270	5.91	V Q				
2+10	0.7680	5.95	V Q				
2+15	0.8093	5.99	V Q				
2+20	0.8507	6.03	V Q				
2+25	0.8925	6.06	V Q				
2+30	0.9344	6.09	V Q				
2+35	0.9765	6.12	V Q				
2+40	1.0188	6.14	V Q				
2+45	1.0614	6.17	VQ				
2+50	1.1041	6.20	VQ				
2+55	1.1470	6.23	VQ				
3+ 0	1.1902	6.26	VQ				

3+ 5	1.2335	6.29	VQ
3+10	1.2770	6.32	VQ
3+15	1.3207	6.34	VQ
3+20	1.3646	6.37	VQ
3+25	1.4086	6.39	VQ
3+30	1.4527	6.41	VQ
3+35	1.4970	6.43	VQ
3+40	1.5414	6.45	VQ
3+45	1.5859	6.46	Q
3+50	1.6305	6.48	Q
3+55	1.6753	6.50	Q
4+ 0	1.7202	6.52	Q
4+ 5	1.7652	6.54	Q
4+10	1.8104	6.56	Q
4+15	1.8557	6.58	Q
4+20	1.9011	6.60	Q
4+25	1.9467	6.62	Q
4+30	1.9924	6.64	Q
4+35	2.0383	6.66	Q
4+40	2.0843	6.68	Q
4+45	2.1304	6.70	QV
4+50	2.1767	6.72	QV
4+55	2.2231	6.74	QV
5+ 0	2.2696	6.76	QV
5+ 5	2.3164	6.78	QV
5+10	2.3632	6.80	QV
5+15	2.4102	6.83	QV
5+20	2.4574	6.85	QV
5+25	2.5047	6.87	QV
5+30	2.5522	6.89	QV
5+35	2.5998	6.92	QV
5+40	2.6476	6.94	Q V
5+45	2.6956	6.96	Q V
5+50	2.7437	6.99	Q V
5+55	2.7919	7.01	QV
6+ 0	2.8404	7.03	QV
6+ 5	2.8890	7.06	QV
6+10	2.9378	7.08	QV
6+15	2.9867	7.11	QV
6+20	3.0359	7.13	QV
6+25	3.0852	7.16	QV
6+30	3.1347	7.19	QV
6+35	3.1843	7.21	Q V
6+40	3.2342	7.24	Q V
6+45	3.2842	7.26	Q V
6+50	3.3344	7.29	Q V
6+55	3.3848	7.32	Q V
7+ 0	3.4354	7.35	Q V
7+ 5	3.4862	7.38	Q V
7+10	3.5372	7.40	Q V
7+15	3.5884	7.43	Q V
7+20	3.6398	7.46	Q V

7+25	3.6914	7.49	Q	V		
7+30	3.7432	7.52	Q	V		
7+35	3.7952	7.55	Q	V		
7+40	3.8474	7.58	Q	V		
7+45	3.8999	7.61	Q	V		
7+50	3.9525	7.65	Q	V		
7+55	4.0054	7.68	Q	V		
8+ 0	4.0585	7.71	Q	V		
8+ 5	4.1118	7.74	Q	V		
8+10	4.1654	7.78	Q	V		
8+15	4.2192	7.81	Q	V		
8+20	4.2732	7.85	Q	V		
8+25	4.3275	7.88	Q	V		
8+30	4.3820	7.92	Q	V		
8+35	4.4368	7.95	Q	V		
8+40	4.4918	7.99	Q	V		
8+45	4.5471	8.03	Q	V		
8+50	4.6026	8.06	Q	V		
8+55	4.6584	8.10	Q	V		
9+ 0	4.7145	8.14	Q	V		
9+ 5	4.7708	8.18	Q	V		
9+10	4.8274	8.22	Q	V		
9+15	4.8843	8.26	Q	V		
9+20	4.9415	8.30	Q	V		
9+25	4.9989	8.34	Q	V		
9+30	5.0567	8.39	Q	V		
9+35	5.1148	8.43	Q	V		
9+40	5.1731	8.47	Q	V		
9+45	5.2318	8.52	Q	V		
9+50	5.2908	8.57	Q	V		
9+55	5.3501	8.61	Q	V		
10+ 0	5.4098	8.66	Q	V		
10+ 5	5.4697	8.71	Q	V		
10+10	5.5300	8.76	Q	V		
10+15	5.5907	8.81	Q	V		
10+20	5.6517	8.86	Q	V		
10+25	5.7131	8.91	Q	V		
10+30	5.7748	8.96	Q	V		
10+35	5.8370	9.02	Q	V		
10+40	5.8994	9.07	Q	V		
10+45	5.9623	9.13	Q	V		
10+50	6.0256	9.19	Q	V		
10+55	6.0893	9.25	Q	V		
11+ 0	6.1534	9.31	Q	V		
11+ 5	6.2179	9.37	Q	V		
11+10	6.2829	9.43	Q	V		
11+15	6.3483	9.50	Q	V		
11+20	6.4141	9.56	Q	V		
11+25	6.4804	9.63	Q	V		
11+30	6.5472	9.70	Q	V		
11+35	6.6145	9.77	Q	V		
11+40	6.6823	9.84	Q	V		

11+45	6.7505	9.91	Q	V
11+50	6.8193	9.99	Q	V
11+55	6.8887	10.07	Q	V
12+ 0	6.9585	10.15	Q	V
12+ 5	7.0289	10.22	Q	V
12+10	7.0996	10.27	Q	V
12+15	7.1705	10.29	Q	V
12+20	7.2409	10.23	Q	V
12+25	7.3107	10.14	Q	V
12+30	7.3803	10.10	Q	V
12+35	7.4499	10.11	Q	V
12+40	7.5198	10.15	Q	V
12+45	7.5901	10.20	Q	V
12+50	7.6608	10.27	Q	V
12+55	7.7320	10.35	Q	V
13+ 0	7.8039	10.43	Q	V
13+ 5	7.8764	10.53	Q	V
13+10	7.9496	10.63	Q	V
13+15	8.0236	10.74	Q	V
13+20	8.0984	10.86	Q	V
13+25	8.1740	10.98	Q	V
13+30	8.2506	11.12	Q	V
13+35	8.3281	11.26	Q	V
13+40	8.4066	11.40	Q	V
13+45	8.4862	11.56	Q	V
13+50	8.5670	11.72	Q	V
13+55	8.6489	11.89	Q	V
14+ 0	8.7320	12.07	Q	V
14+ 5	8.8165	12.27	Q	V
14+10	8.9024	12.47	Q	V
14+15	8.9897	12.69	Q	V
14+20	9.0787	12.92	Q	V
14+25	9.1694	13.17	Q	V
14+30	9.2619	13.43	Q	V
14+35	9.3563	13.71	Q	V
14+40	9.4527	14.00	Q	V
14+45	9.5514	14.32	Q	V
14+50	9.6523	14.66	Q	V
14+55	9.7558	15.02	Q	V
15+ 0	9.8619	15.42	Q	V
15+ 5	9.9711	15.85	Q	V
15+10	10.0835	16.32	Q	V
15+15	10.1994	16.84	Q	V
15+20	10.3194	17.42	Q	V
15+25	10.4434	18.01	Q	V
15+30	10.5714	18.58	Q	V
15+35	10.7031	19.11	Q	V
15+40	10.8368	19.42	Q	V
15+45	10.9728	19.74	Q	V
15+50	11.1145	20.57	Q	V
15+55	11.2664	22.07	Q	V
16+ 0	11.4356	24.56	Q	V

16+ 5	11.6404	29.73			Q		V		
16+10	11.9039	38.26					QV		
16+15	12.2360	48.23					V		
16+20	12.6731	63.46					V		Q
16+25	13.1456	68.62					V		
16+30	13.5369	56.81					V		
16+35	13.8562	46.36					Q		Q
16+40	14.1289	39.60							
16+45	14.3744	35.65					Q		
16+50	14.5971	32.34							
16+55	14.8020	29.75							
17+ 0	14.9903	27.35							
17+ 5	15.1657	25.46							
17+10	15.3307	23.96							
17+15	15.4864	22.61							
17+20	15.6324	21.21							
17+25	15.7691	19.84							
17+30	15.8993	18.91							
17+35	16.0239	18.09							
17+40	16.1428	17.27							
17+45	16.2569	16.56							
17+50	16.3655	15.77							
17+55	16.4700	15.18							
18+ 0	16.5709	14.64							
18+ 5	16.6674	14.02							
18+10	16.7611	13.61							
18+15	16.8515	13.12							
18+20	16.9394	12.77							
18+25	17.0259	12.56							
18+30	17.1098	12.18							
18+35	17.1922	11.97							
18+40	17.2736	11.82							
18+45	17.3540	11.68							
18+50	17.4332	11.50							
18+55	17.5112	11.31							
19+ 0	17.5877	11.11							
19+ 5	17.6624	10.85							
19+10	17.7348	10.52							
19+15	17.8059	10.33							
19+20	17.8757	10.14							
19+25	17.9441	9.93							
19+30	18.0099	9.56							
19+35	18.0747	9.40							
19+40	18.1385	9.27							
19+45	18.2015	9.15							
19+50	18.2637	9.03							
19+55	18.3251	8.92							
20+ 0	18.3858	8.81							
20+ 5	18.4457	8.71							
20+10	18.5050	8.61							
20+15	18.5637	8.52							
20+20	18.6217	8.42							

20+25	18.6791	8.34	Q	V
20+30	18.7359	8.25	Q	V
20+35	18.7922	8.17	Q	V
20+40	18.8479	8.09	Q	V
20+45	18.9031	8.02	Q	V
20+50	18.9578	7.94	Q	V
20+55	19.0121	7.87	Q	V
21+ 0	19.0658	7.80	Q	V
21+ 5	19.1191	7.74	Q	V
21+10	19.1719	7.67	Q	V
21+15	19.2243	7.61	Q	V
21+20	19.2763	7.55	Q	V
21+25	19.3278	7.49	Q	V
21+30	19.3790	7.43	Q	V
21+35	19.4297	7.37	Q	V
21+40	19.4801	7.31	Q	V
21+45	19.5300	7.26	Q	V
21+50	19.5796	7.20	Q	V
21+55	19.6289	7.15	Q	V
22+ 0	19.6778	7.10	Q	V
22+ 5	19.7263	7.05	Q	V
22+10	19.7745	7.00	Q	V
22+15	19.8224	6.95	Q	V
22+20	19.8699	6.90	Q	V
22+25	19.9172	6.86	Q	V
22+30	19.9641	6.81	Q	V
22+35	20.0107	6.77	Q	V
22+40	20.0571	6.73	Q	V
22+45	20.1031	6.69	Q	V
22+50	20.1489	6.65	Q	V
22+55	20.1944	6.60	Q	V
23+ 0	20.2396	6.57	Q	V
23+ 5	20.2845	6.53	Q	V
23+10	20.3292	6.49	Q	V
23+15	20.3737	6.45	Q	V
23+20	20.4178	6.42	Q	V
23+25	20.4618	6.38	Q	V
23+30	20.5055	6.34	Q	V
23+35	20.5489	6.31	Q	V
23+40	20.5921	6.28	Q	V
23+45	20.6351	6.24	Q	V
23+50	20.6779	6.21	Q	V
23+55	20.7204	6.18	Q	V
24+ 0	20.7628	6.15	Q	V

BASINB-2 100 YEAR UH

Unit Hydrograph Analysis

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Study date 03/12/15

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San Bernardino County Synthetic Unit Hydrology Method  
Manual date - August 1986

Program License Serial Number 6288

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BASIN B-2 UNIT HYDROGRAPH  
100 YEAR 24 HOUR  
RANCHO MIRAMONTE  
-----

Storm Event Year = 100

Antecedent Moisture Condition = 3

English (in-lb) Input Units Used

English Rainfall Data (Inches) Input Values Used

English Units used in output format

Area averaged rainfall intensity isohyetal data:

Sub-Area (Ac.)	Duration (hours)	Isohyetal (In)
Rainfall data for year 100 35.55	1	1.22

Rainfall data for year 100 35.55	6	3.00
-------------------------------------	---	------

Rainfall data for year 100 35.55	24	6.50
-------------------------------------	----	------

\*\*\*\*\* Area-averaged max loss rate, Fm \*\*\*\*\*

(In/Hr)	SCS curve No. (AMCII)	SCS curve NO. (AMC 3)	Area (Ac.)	Area Fraction	Fp(Fig C6) (In/Hr)	Ap (dec.)	Fm
	70.0	87.0	35.55	1.000	0.247	0.406	0.100

Area-averaged adjusted loss rate Fm (In/Hr) = 0.100

\*\*\*\*\* Area-Averaged low loss rate fraction, Yb \*\*\*\*\*

Area	Area	SCS CN	SCS CN	S	Pervious
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		BASINB-2 100 YEAR UH			
(Ac.)	Fract	(AMC2)	(AMC3)		Yield Fr
14.43	0.406	70.0	87.0	1.49	0.769
21.12	0.594	98.0	98.0	0.20	0.963

Area-averaged catchment yield fraction, Y = 0.884  
Area-averaged low loss fraction, Yb = 0.116  
User entry of time of concentration = 0.210 (hours)  
++++  
Watershed area = 35.55(Ac.)  
Catchment Lag time = 0.168 hours  
Unit interval = 5.000 minutes  
Unit interval percentage of lag time = 49.4854  
Hydrograph baseflow = 0.00(CFS)  
Average maximum watershed loss rate(Fm) = 0.100(In/Hr)  
Average low loss rate fraction (Yb) = 0.116 (decimal)  
DESERT S-Graph Selected  
Computed peak 5-minute rainfall = 0.452(In)  
Computed peak 30-minute rainfall = 0.925(In)  
Specified peak 1-hour rainfall = 1.220(In)  
Computed peak 3-hour rainfall = 2.118(In)  
Specified peak 6-hour rainfall = 3.000(In)  
Specified peak 24-hour rainfall = 6.500(In)

Rainfall depth area reduction factors:  
Using a total area of 35.55(Ac.) (Ref: fig. E-4)

5-minute factor = 0.998	Adjusted rainfall = 0.451(In)
30-minute factor = 0.998	Adjusted rainfall = 0.923(In)
1-hour factor = 0.998	Adjusted rainfall = 1.218(In)
3-hour factor = 1.000	Adjusted rainfall = 2.118(In)
6-hour factor = 1.000	Adjusted rainfall = 3.000(In)
24-hour factor = 1.000	Adjusted rainfall = 6.500(In)

U n i t H y d r o g r a p h

++++

Interval Number	'S' Graph Mean values	Unit Hydrograph (CFS)
	(K = 429.93 (CFS))	
1	3.895	16.748
2	29.905	111.824
3	60.095	129.795
4	73.423	57.303
5	81.171	33.310
6	86.350	22.266
7	89.979	15.602
8	92.596	11.252
9	94.600	8.614
10	96.121	6.542
11	97.251	4.856
12	97.988	3.171
13	98.524	2.302
14	99.117	2.550
15	99.607	2.107
16	100.000	1.691

-----  
Total soil rain loss = 0.69(In)  
Total effective rainfall = 5.81(In)  
Peak flow rate in flood hydrograph = 82.04(CFS)  
-----

BASINB-2 100 YEAR UH  
24 - H O U R     S T O R M  
R u n o f f     H y d r o g r a p h

-----  
Hydrograph in 5 Minute intervals ((CFS))  
-----

Time(h+m)	Volume	Ac.Ft	Q(CFS)	0	22.5	45.0	67.5	90.0
0+ 5	0.0013		0.19	Q				
0+10	0.0112		1.43	Q				
0+15	0.0310		2.88	VQ				
0+20	0.0553		3.53	VQ				
0+25	0.0822		3.91	VQ				
0+30	0.1108		4.16	VQ				
0+35	0.1408		4.35	VQ				
0+40	0.1716		4.48	VQ				
0+45	0.2032		4.59	V Q				
0+50	0.2354		4.67	V Q				
0+55	0.2680		4.74	V Q				
1+ 0	0.3010		4.78	V Q				
1+ 5	0.3342		4.82	V Q				
1+10	0.3676		4.86	V Q				
1+15	0.4014		4.90	V Q				
1+20	0.4353		4.93	VQ				
1+25	0.4693		4.94	VQ				
1+30	0.5034		4.95	VQ				
1+35	0.5376		4.96	VQ				
1+40	0.5719		4.98	VQ				
1+45	0.6062		4.99	VQ				
1+50	0.6407		5.00	VQ				
1+55	0.6752		5.01	VQ				
2+ 0	0.7098		5.03	VQ				
2+ 5	0.7445		5.04	VQ				
2+10	0.7793		5.05	VQ				
2+15	0.8142		5.07	VQ				
2+20	0.8492		5.08	VQ				
2+25	0.8843		5.09	Q				
2+30	0.9195		5.11	Q				
2+35	0.9547		5.12	Q				
2+40	0.9901		5.13	Q				
2+45	1.0256		5.15	Q				
2+50	1.0611		5.16	Q				
2+55	1.0968		5.18	Q				
3+ 0	1.1325		5.19	Q				
3+ 5	1.1684		5.21	Q				
3+10	1.2043		5.22	Q				
3+15	1.2404		5.23	Q				
3+20	1.2765		5.25	Q				
3+25	1.3128		5.26	QV				
3+30	1.3491		5.28	QV				
3+35	1.3856		5.29	QV				
3+40	1.4222		5.31	QV				
3+45	1.4588		5.33	QV				
3+50	1.4956		5.34	QV				
3+55	1.5325		5.36	QV				
4+ 0	1.5695		5.37	QV				
4+ 5	1.6066		5.39	QV				
4+10	1.6439		5.41	QV				
4+15	1.6812		5.42	QV				
4+20	1.7187		5.44	QV				
4+25	1.7562		5.46	Q V				
4+30	1.7939		5.47	Q V				
4+35	1.8317		5.49	Q V				
4+40	1.8697		5.51	Q V				
4+45	1.9077		5.52	Q V				
4+50	1.9459		5.54	Q V				

BASINB-2 100 YEAR UH

4+55	1.9842	5.56	Q	V			
5+ 0	2.0226	5.58	Q	V			
5+ 5	2.0611	5.60	Q	V			
5+10	2.0998	5.62	Q	V			
5+15	2.1386	5.63	Q	V			
5+20	2.1775	5.65	Q	V			
5+25	2.2166	5.67	Q	V			
5+30	2.2558	5.69	Q	V			
5+35	2.2951	5.71	Q	V			
5+40	2.3346	5.73	Q	V			
5+45	2.3742	5.75	Q	V			
5+50	2.4139	5.77	Q	V			
5+55	2.4538	5.79	Q	V			
6+ 0	2.4938	5.81	Q	V			
6+ 5	2.5340	5.83	Q	V			
6+10	2.5743	5.85	Q	V			
6+15	2.6148	5.87	Q	V			
6+20	2.6554	5.90	Q	V			
6+25	2.6961	5.92	Q	V			
6+30	2.7371	5.94	Q	V			
6+35	2.7781	5.96	Q	V			
6+40	2.8193	5.99	Q	V			
6+45	2.8607	6.01	Q	V			
6+50	2.9023	6.03	Q	V			
6+55	2.9440	6.06	Q	V			
7+ 0	2.9858	6.08	Q	V			
7+ 5	3.0279	6.10	Q	V			
7+10	3.0701	6.13	Q	V			
7+15	3.1125	6.15	Q	V			
7+20	3.1550	6.18	Q	V			
7+25	3.1977	6.20	Q	V			
7+30	3.2406	6.23	Q	V			
7+35	3.2837	6.26	Q	V			
7+40	3.3270	6.28	Q	V			
7+45	3.3704	6.31	Q	V			
7+50	3.4141	6.34	Q	V			
7+55	3.4579	6.36	Q	V			
8+ 0	3.5019	6.39	Q	V			
8+ 5	3.5462	6.42	Q	V			
8+10	3.5906	6.45	Q	V			
8+15	3.6352	6.48	Q	V			
8+20	3.6801	6.51	Q	V			
8+25	3.7251	6.54	Q	V			
8+30	3.7703	6.57	Q	V			
8+35	3.8158	6.60	Q	V			
8+40	3.8615	6.63	Q	V			
8+45	3.9074	6.67	Q	V			
8+50	3.9535	6.70	Q	V			
8+55	3.9999	6.73	Q	V			
9+ 0	4.0465	6.77	Q	V			
9+ 5	4.0933	6.80	Q	V			
9+10	4.1404	6.84	Q	V			
9+15	4.1877	6.87	Q	V			
9+20	4.2353	6.91	Q	V			
9+25	4.2831	6.94	Q	V			
9+30	4.3312	6.98	Q	V			
9+35	4.3795	7.02	Q	V			
9+40	4.4281	7.06	Q	V			
9+45	4.4770	7.10	Q	V			
9+50	4.5262	7.14	Q	V			
9+55	4.5756	7.18	Q	V			
10+ 0	4.6253	7.22	Q	V			
10+ 5	4.6754	7.26	Q	V			
10+10	4.7257	7.31	Q	V			
10+15	4.7763	7.35	Q	V			
10+20	4.8272	7.40	Q	V			

			BASINB-2 100 YEAR UH				
10+25	4.8785	7.44	Q	V			
10+30	4.9301	7.49	Q	V			
10+35	4.9820	7.54	Q	V			
10+40	5.0342	7.59	Q	V			
10+45	5.0868	7.64	Q	V			
10+50	5.1397	7.69	Q	V			
10+55	5.1930	7.74	Q	V			
11+ 0	5.2467	7.79	Q	V			
11+ 5	5.3008	7.85	Q	V			
11+10	5.3552	7.90	Q	V			
11+15	5.4100	7.96	Q	V			
11+20	5.4653	8.02	Q	V			
11+25	5.5209	8.08	Q	V			
11+30	5.5770	8.14	Q	V			
11+35	5.6335	8.20	Q	V			
11+40	5.6905	8.27	Q	V			
11+45	5.7479	8.34	Q	V			
11+50	5.8057	8.40	Q	V			
11+55	5.8641	8.47	Q	V			
12+ 0	5.9230	8.55	Q	V			
12+ 5	5.9821	8.59	Q	V			
12+10	6.0402	8.44	Q	V			
12+15	6.0970	8.25	Q	V			
12+20	6.1536	8.21	Q	V			
12+25	6.2103	8.23	Q	V			
12+30	6.2672	8.27	Q	V			
12+35	6.3246	8.33	Q	V			
12+40	6.3825	8.40	Q	V			
12+45	6.4409	8.48	Q	V			
12+50	6.4999	8.57	Q	V			
12+55	6.5595	8.66	Q	V			
13+ 0	6.6199	8.76	Q	V			
13+ 5	6.6809	8.87	Q	V			
13+10	6.7428	8.98	Q	V			
13+15	6.8054	9.10	Q	V			
13+20	6.8689	9.22	Q	V			
13+25	6.9333	9.35	Q	V			
13+30	6.9987	9.49	Q	V			
13+35	7.0650	9.63	Q	V			
13+40	7.1324	9.78	Q	V			
13+45	7.2009	9.94	Q	V			
13+50	7.2705	10.11	Q	V			
13+55	7.3413	10.28	Q	V			
14+ 0	7.4134	10.47	Q	V			
14+ 5	7.4869	10.66	Q	V			
14+10	7.5618	10.88	Q	V			
14+15	7.6383	11.10	Q	V			
14+20	7.7164	11.34	Q	V			
14+25	7.7962	11.59	Q	V			
14+30	7.8778	11.86	Q	V			
14+35	7.9614	12.14	Q	V			
14+40	8.0472	12.45	Q	V			
14+45	8.1352	12.78	Q	V			
14+50	8.2257	13.14	Q	V			
14+55	8.3189	13.53	Q	V			
15+ 0	8.4151	13.97	Q	V			
15+ 5	8.5145	14.44	Q	V			
15+10	8.6177	14.97	Q	V			
15+15	8.7249	15.56	Q	V			
15+20	8.8367	16.24	Q	V			
15+25	8.9528	16.85	Q	V			
15+30	9.0679	16.72	Q	V			
15+35	9.1819	16.55	Q	V			
15+40	9.3012	17.32	Q	V			
15+45	9.4293	18.60	Q	V			
15+50	9.5712	20.60	Q	V			

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15+55	9.7339	23.63		Q		V	
16+ 0	9.9377	29.59			Q	V	
16+ 5	10.2417	44.13				V	
16+10	10.7913	79.80				V	
16+15	11.3563	82.04				V	Q
16+20	11.7111	51.51			Q	V	Q
16+25	11.9727	37.99				V	
16+30	12.1878	31.22			Q	V	
16+35	12.3743	27.09		Q		V	
16+40	12.5377	23.72		Q		V	
16+45	12.6836	21.18		Q		V	
16+50	12.8148	19.06		Q		V	
16+55	12.9337	17.25		Q		V	
17+ 0	13.0415	15.66		Q		V	
17+ 5	13.1416	14.53		Q		V	
17+10	13.2371	13.87		Q		V	
17+15	13.3266	13.00		Q		V	
17+20	13.4104	12.17		Q		V	
17+25	13.4864	11.02		Q		V	
17+30	13.5591	10.56		Q		V	
17+35	13.6292	10.18		Q		V	
17+40	13.6969	9.84		Q		V	
17+45	13.7625	9.53		Q		V	
17+50	13.8262	9.24		Q		V	
17+55	13.8881	8.99		Q		V	
18+ 0	13.9484	8.75		Q		V	
18+ 5	14.0074	8.57		Q		V	
18+10	14.0666	8.60		Q		V	
18+15	14.1264	8.68		Q		V	
18+20	14.1857	8.62		Q		V	
18+25	14.2445	8.53		Q		V	
18+30	14.3024	8.42		Q		V	
18+35	14.3597	8.31		Q		V	
18+40	14.4161	8.19		Q		V	
18+45	14.4718	8.08		Q		V	
18+50	14.5267	7.97		Q		V	
18+55	14.5809	7.87		Q		V	
19+ 0	14.6343	7.76		Q		V	
19+ 5	14.6871	7.66		Q		V	
19+10	14.7392	7.57		Q		V	
19+15	14.7907	7.47		Q		V	
19+20	14.8415	7.38		Q		V	
19+25	14.8918	7.29		Q		V	
19+30	14.9414	7.21		Q		V	
19+35	14.9905	7.13		Q		V	
19+40	15.0390	7.05		Q		V	
19+45	15.0870	6.97		Q		V	
19+50	15.1345	6.90		Q		V	
19+55	15.1815	6.82		Q		V	
20+ 0	15.2280	6.75		Q		V	
20+ 5	15.2741	6.69	Q			V	
20+10	15.3197	6.62	Q			V	
20+15	15.3649	6.56	Q			V	
20+20	15.4096	6.50	Q			V	
20+25	15.4540	6.44	Q			V	
20+30	15.4980	6.38	Q			V	
20+35	15.5415	6.33	Q			V	
20+40	15.5847	6.27	Q			V	
20+45	15.6276	6.22	Q			V	
20+50	15.6701	6.17	Q			V	
20+55	15.7122	6.12	Q			V	
21+ 0	15.7540	6.07	Q			V	
21+ 5	15.7955	6.02	Q			V	
21+10	15.8367	5.98	Q			V	
21+15	15.8775	5.93	Q			V	
21+20	15.9181	5.89	Q			V	

BASINB-2 100 YEAR UH				
21+25	15.9583	5.85	Q	V
21+30	15.9983	5.80	Q	V
21+35	16.0380	5.76	Q	V
21+40	16.0774	5.72	Q	V
21+45	16.1165	5.68	Q	V
21+50	16.1554	5.65	Q	V
21+55	16.1940	5.61	Q	V
22+ 0	16.2324	5.57	Q	V
22+ 5	16.2705	5.54	Q	V
22+10	16.3084	5.50	Q	V
22+15	16.3460	5.47	Q	V
22+20	16.3835	5.43	Q	V
22+25	16.4206	5.40	Q	V
22+30	16.4576	5.37	Q	V
22+35	16.4943	5.33	Q	V
22+40	16.5309	5.30	Q	V
22+45	16.5672	5.27	Q	V
22+50	16.6033	5.24	Q	V
22+55	16.6392	5.21	Q	V
23+ 0	16.6749	5.18	Q	V
23+ 5	16.7104	5.16	Q	V
23+10	16.7457	5.13	Q	V
23+15	16.7809	5.10	Q	V
23+20	16.8158	5.07	Q	V
23+25	16.8506	5.05	Q	V
23+30	16.8852	5.02	Q	V
23+35	16.9196	5.00	Q	V
23+40	16.9538	4.97	Q	V
23+45	16.9879	4.95	Q	V
23+50	17.0218	4.92	Q	V
23+55	17.0555	4.90	Q	V
24+ 0	17.0891	4.87	Q	V

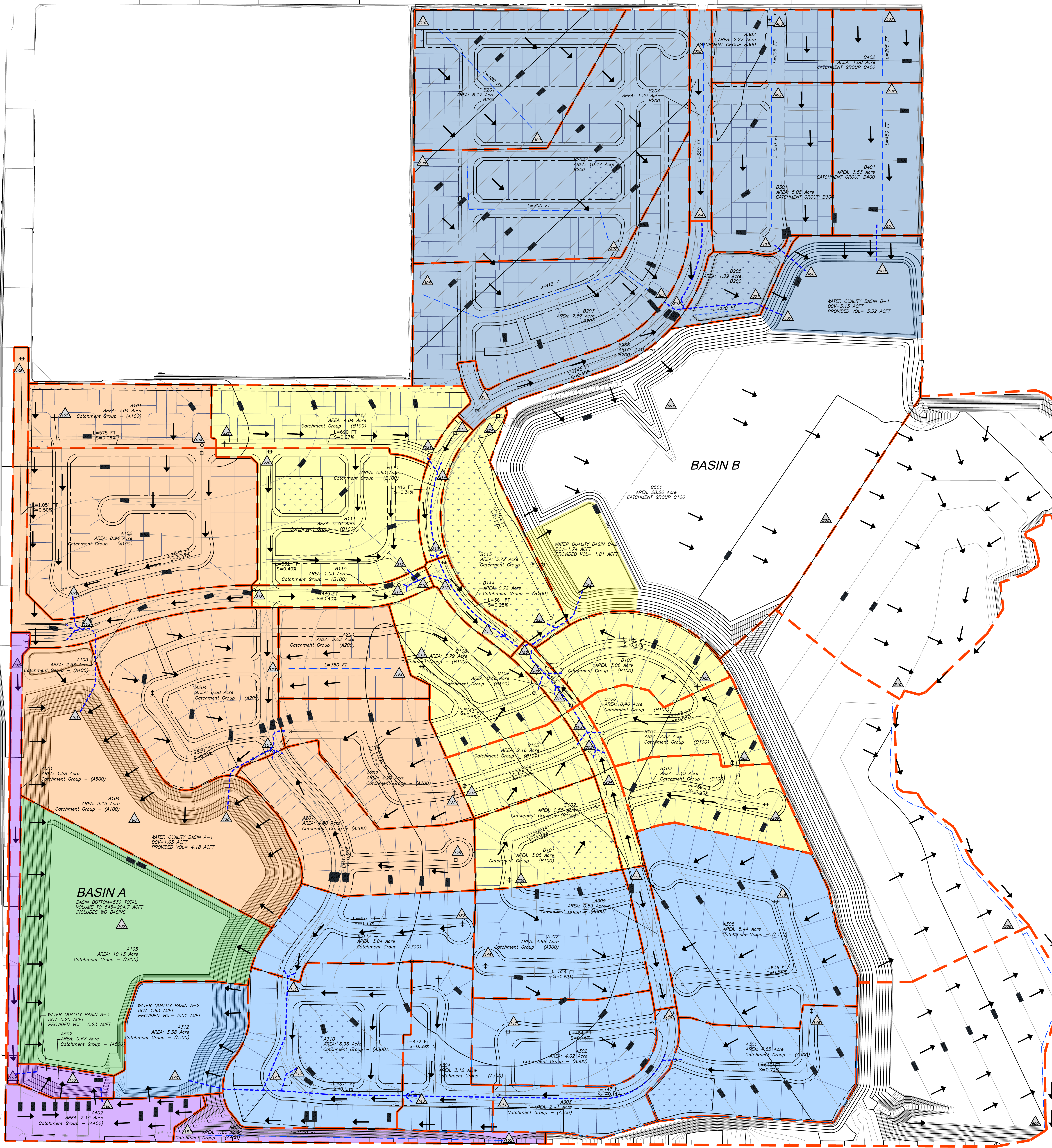
Prepared for the City of Chino, CA

***APPENDIX I: ACOE REPLACEMENT VOLUME SUMMARY***

Contour Elevation	Basin Depth	Existing Condition			Proposed Condition			May-14	
		Area (Acres)	Incremental volume (CY)	Volume (CY) Subtotal	Area (Acres)	Incremental volume (CY)	Volume (CY) Subtotal	1' incement proposed vs existing	Overall proposed vs existing
509	1	0.4	354	354	0.4	369	369	104.24%	104.24%
510	2	0.5	569	923	0.7	589	958	103.51%	103.79%
511	3	0.9	1,139	2,062	1.2	1,160	2,118	101.84%	102.72%
512	4	1.4	1,825	3,887	1.7	1,837	3,955	100.66%	101.75%
513	5	2.2	2,728	6,615	2.2	2,748	6,703	100.73%	101.33%
514	6	2.5	3,443	10,058	2.6	3,469	10,172	100.76%	101.13%
515	7	3.0	4,182	14,240	3.5	4,215	14,387	100.79%	101.03%
516	8	4.3	5,470	19,710	4.6	5,490	19,877	100.37%	100.85%
517	9	5.3	7,081	26,791	5.7	7,167	27,044	101.21%	100.94%
518	10	6.7	9,061	35,852	6.9	9,122	36,166	100.67%	100.88%
519	11	7.9	11,171	47,023	9.6	12,652	48,818	113.26%	103.82%
520	12	8.6	12,442	59,465	13.2	17,110	65,928	137.52%	110.87%
521	13	9.4	13,665	73,130	17.4	23,309	89,237	170.57%	122.03%
522	14	10.3	14,992	88,122	23.1	31,101	120,338	207.45%	136.56%
523	15	10.9	16,247	104,369	27.3	38,368	158,706	236.15%	152.06%
524	16	11.6	17,241	121,610	30.5	44,509	203,215	258.16%	167.10%
525	17	12.5	18,671	140,281	33.9	50,258	253,473	269.18%	180.69%
526	18	14.7	21,345	161,626	65.9	56,195	309,668	263.27%	191.60%
527	19	17.5	25,630	187,256	70.1	63,341	373,009	247.14%	199.20%
528	20	20.1	30,358	217,614	72.5	69,367	442,376	228.50%	203.28%
529	21	22.0	33,085	250,699	75.0	72,903	515,279	220.35%	205.54%
530	22	23.3	35,699	286,398	76.8	75,867	591,146	212.52%	206.41%
531	23	25.3	38,493	324,891	80.6	100,353	691,499	260.70%	212.84%
532	24	28.5	42,753	367,644	82.7	103,241	794,740	241.48%	216.17%
533	25	33.8	50,870	418,514	84.8	105,813	900,553	208.01%	215.18%
534	26	36.0	55,856	474,370	86.3	107,846	1,008,399	193.08%	212.58%
535	27	37.7	58,570	532,940	88.2	109,832	1,118,231	187.52%	209.82%
536	28	39.9	61,694	594,634	89.9	112,196	1,230,427	181.86%	206.92%
537	29	42.5	65,593	660,227	91.4	114,013	1,344,440	173.82%	203.63%
538	30	44.0	69,039	729,266	92.5	115,632	1,460,072	167.49%	200.21%
539	31	45.4	71,387	800,653	93.7	116,454	1,576,526	163.13%	196.91%
540	32	46.6	73,412	874,065	94.8	117,205	1,693,731	159.65%	193.78%
541	33	48.5	75,951	950,016	95.8	123,278	1,817,009	162.31%	191.26%
542	34	50.4	79,055	1,029,071	96.8	124,265	1,941,274	157.19%	188.64%
543	35	52.2	81,845	1,110,916	97.7	125,152	2,066,426	152.91%	186.01%
544	36	53.9	84,531	1,195,447	98.7	125,895	2,192,321	148.93%	183.39%
545	37	56.1	87,574	1,283,021	99.7	126,754	2,319,075	144.74%	180.75%
546	38	60.9	94,058	1,377,079	101.2	161,656	2,480,731	171.87%	180.14%
547	39	66.1	100,577	1,477,656	102.1	163,327	2,644,058	162.39%	178.94%
548	40	71.8	108,979	1,586,635	103.5	164,924	2,808,982	151.34%	177.04%
549	41	82.7	123,894	1,710,529	104.6	166,417	2,975,399	134.32%	173.95%
550	42	90.4	137,710	1,848,239	105.6	167,912	3,143,311	121.93%	170.07%
551	43	97.8	150,047	1,998,286	108.6	171,913	3,315,224	114.57%	165.90%
552	44	105.1	161,901	2,160,187	109.7	173,353	3,488,577	107.07%	161.49%
553	45	113.4	174,749	2,334,936	110.9	174,796	3,663,373	100.03%	156.89%
554	46	120.7	187,076	2,522,012	112.0	176,239	3,839,612	94.21%	152.24%
555	47	127.5	198,386	2,720,398	113.2	177,631	4,017,243	89.54%	147.67%
556	48	135.1	209,891	2,930,289	114.3	178,769	4,196,012	85.17%	143.19%
557	49	143.0	222,477	3,152,766	115.3	179,548	4,375,560	80.70%	138.78%
558	50	157.7	245,137	3,397,903	116.2	180,604	4,556,164	73.67%	134.09%
559	51	167.8	260,963	3,658,866	117.2	181,951	4,738,115	69.72%	129.50%
560	52	177.7	277,342	3,936,208	118.3	183,301	4,921,416	66.09%	125.03%
561	53	187.0	293,376	4,229,584	119.7	185,231	5,106,647	63.14%	120.74%
562	54	195.8	307,233	4,536,817	121.6	187,480	5,294,127	61.02%	116.69%
563	55	205.1	321,861	4,858,678	123.4	189,736	5,483,863	58.95%	112.87%
564	56	217.5	339,189	5,197,867	124.8	191,984	5,675,847	56.60%	109.20%
565	57	229.8	361,467	5,559,334	126.5	194,072	5,869,919	53.69%	105.59%
566	58	239.1	377,021	5,936,355	129.0	195,885	6,065,804	51.96%	102.18%

43560

Kyle: 2-16-15 Worksheet					1' incement		Overall	
Area (sf)	Area (ac)	incremental volume (cy)	Volume (CY) Subtotal	proposed vs existing	proposed vs existing			
13,689	0.31	354	354	100.00%	100.00%			
4,430	0.10	569	923	100.00%	100.00%			
17,349	0.40	1,139	2,062	100.00%	100.00%			
21,642	0.50	1,825	3,887	100.00%	100.00%			
28,170	0.65	2,728	6,615	100.00%	100.00%			
19,733	0.45	3,443	10,058	100.00%	100.00%			
16,801	0.39	4,182	14,240	100.00%	100.00%			
27,529	0.63	5,470	19,710	100.00%	100.00%			
68,923	1.58	7,081	26,791	100.00%	100.00%			
53,614	1.23	9,061	35,852	100.00%	100.00%			
119,372	2.74	12,529	48,381	112.16%	102.89%			
139,530	3.20	16,989	71,595	136.55%	120.40%			
193,734	4.45	23,214	101,816	169.88%	139.23%			
250,778	5.76	30,221	140,505	201.58%	159.44%			
187,698	4.31	38,689	189,621	238.13%	181.68%			
157,437	3.61	49,116	248,551	284.88%	204.38%			
143,761	3.30	58,930	324,947	315.62%	231.64%			
179,843	4.13	76,396	419,805	357.91%	259.74%			
206,845	4.75	94,858	519,820	370.11%	277.60%			
92,417	2.12	100,015	622,044	329.45%	285.85%			
97,110	2.23	102,224	726,794	308.97%	289.91%			
69,687	1.60	104,750	833,663	293.43%	291.09%			
1,036,594	23.80	106,869	942,643	277.63%	290.14%			
66,045	1.52	108,980	1,053,626	254.91%	286.59%			
89,194	2.05	110,983	1,166,564	218.17%	278.74%			
51,958	1.19	112,938	1,281,371	202.19%	270.12%			
68,224	1.57	114,807	1,398,415	196.02%	262.40%			
299,144	6.87	117,044	1,517,174	189.72%	255.14%			
78,132	1.79	118,759	1,637,428	181.05%	248.01%			
42,940	0.99	120,254	1,758,382	174.18%	241.12%			
31,396	0.72	120,954	1,879,956	169.43%	234.80%			
30,046	0.69	121,574	2,002,179	165.61%	229.07%			
178,099	4.09	122,223	2,125,179	160.92%	223.70%			
39,994	0.92	123,000	2,255,102	155.59%	219.14%			
33,520	0.77	129,923	2,385,611	158.74%	214.74%			
3,161	0.07	130,509	2,516,824	154.39%	210.53%			
38,786	0.89	131,213	2,676,709	149.83%	208.63%			
54,331	1.25	159,885	2,838,247	169.99%	206.11%			
44,428	1.02	161,538	3,001,230	160.61%	203.11%			
41,052	0.94	162,983	3,165,722	149.55%	199.52%			
40,971	0.94	164,492	3,331,314	132.77%	194.75%			
38,539	0.88	165,592	3,501,174	120.25%	189.43%			
123,749	2.84	169,860	3,672,602	113.20%	183.79%			
33,790	0.78	171,428	3,845,477	105.88%	178.02%			
39,834	0.91	172,875	4,019,615	98.93%	172.15%			
34,587	0.79	174,138	4,195,062	93.08%	166.34%			
40,153	0.92	175,447	4,371,905	88.44%	160.71%			
29,585	0.68	176,843	4,549,779	84.25%	155.27%			
27,778	0.64	177,874	4,728,670	79.95%	149.98%			
27,729	0.64	178,891	4,908,597	72.98%	144.46%			
27,943	0.64	179,927	5,089,541	68.95%	139.10%			
30,598	0.70	180,944	5,271,515	65.24%	133.92%			
29,130	0.67	181,974	5,454,479	62.03%	128.96%			
63,658	1.46	182,964	5,638,458	59.55%	124.28%			
67,443	1.55	183,979	5,823,437	57.16%	119.86%			
48,542	1.11	184,979	6,009,476	54.54%	115.61%			
46,726	1.07	186,039	6,196,695	51.47%	111.46%			
2,133	0.05	187,219	6,383,914	49.66%	107.54%			

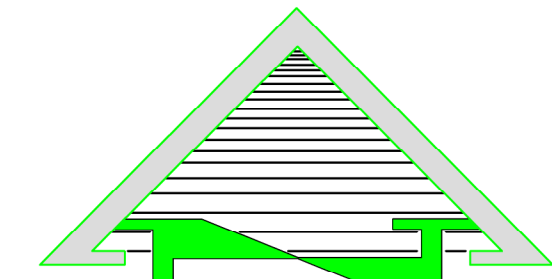


**LEGEND**

- LINK NODE
- FLOW DIRECTION
- DRAINAGE AREA BOUNDARY
- PROPOSED STORM DRAIN PIPE
- DRAINAGE PATH LENGTH  
SUB AREA DRAINAGE PATH

**Rational Method Routed Peak Flows**

Basin A Design Flows					Basin B Design Flows					Existing Condition Flows	
Design Point (Ft)	Elevation (Ft)	Tc100 (Min)	100-year Flow (CFS)	2-year Flow (CFS)	Design Point (Ft)	Elevation (Ft)	Tc100 (Min)	100-year Flow (CFS)	2-year Flow (CFS)	Design Point (Ft)	Flow (CFS)
101	535	19.7	30.4	10.0	198	541	12.31	93.44	32.6	301	110
120	535	120	52.8	17.8	400	540	10.17	23.16	8.3	300	51
140	555	153	95.3	31.0	500	540	12.18	14.83	5.3	510	12.6
160-190	535	23.3	7.0	2.2						505	198.7
191-193	535	11.5	7.3	2.6						600	211.9



GRAPHIC SCALE  
( IN FEET )  
1 inch = 100 ft.

DATE: 03-12-15

**DEVELOPED  
HYDROLOGY MAP**

# **Preliminary Water Quality Management Plan (PWQMP)**

**For:**

**Mill Creek Farming Associates, LLC**

**TENTATIVE TRACT 18846**

**Prepared for:**

**Millcreek Farming Associates, LLC**

**1730 Evergreen Street**

**Duarte, CA 91010**

**(310) 300-5440**

**Prepared by:**

**RMB**

**8175 Limonite Avenue, Suite E**

**Jurupa Valley, CA 92509**

**(951) 317-2041**

**RMbeers777@hotmail.com**

**Approval Date: \_\_\_\_\_**

### Project Owner's Certification

This Water Quality Management Plan (WQMP) has been prepared for Millcreek Farming Associates, LLC by rmb Engineers. The WQMP is intended to comply with the requirements of the City of Chino and the NPDES Areawide Stormwater Program requiring the preparation of a WQMP. The undersigned, while it owns the subject property, is responsible for the implementation of the provisions of this plan and will ensure that this plan is amended as appropriate to reflect up-to-date conditions on the site consistent with San Bernardino County's Municipal Storm Water Management Program and the intent of the NPDES Permit for San Bernardino County and the incorporated cities of San Bernardino County within the Santa Ana Region. Once the undersigned transfers its interest in the property, its successors in interest and the city/county shall be notified of the transfer. The new owner will be informed of its responsibility under this WQMP. A copy of the approved WQMP shall be available on the subject site in perpetuity.

"I certify under a penalty of law that the provisions (implementation, operation, maintenance, and funding) of the WQMP have been accepted and that the plan will be transferred to future successors."

Project Data			
Permit/Application Number(s):	TBD	Grading Permit Number(s):	TBD
Tract/Parcel Map Number(s):	Tentative Tract 18846	Building Permit Number(s):	TBD
CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract):			057-191-05-0000
Owner's Signature			
<b>Owner Name:</b> Joe Blum			
Title	Director of Development		
Company	Millcreek Farming Associates, LLC		
Address	1730 Evergreen Street, Duarte, CA 91010		
Email	jblum@butier.com		
Telephone #	(714) 448-0216		
Signature			Date

### Preparer's Certification

Project Data			
Permit/Application Number(s):	TBD	Grading Permit Number(s):	TBD
Tract/Parcel Map Number(s):	Tentative Tract 18846	Building Permit Number(s):	TBD
CUP, SUP, and/or APN (Specify Lot Numbers if Portions of Tract):			057-191-05-0000

“The selection, sizing and design of stormwater treatment and other stormwater quality and quantity control measures in this plan were prepared under my oversight and meet the requirements of Regional Water Quality Control Board Order No. R8-2010-0036.”

<b>Engineer:</b> Bob Beers		PE Stamp Below
Title	Professional Engineer	
Company	RMB	
Address	8175 Limonite Ave, Suite E, Jurupa Valley, CA 92509	
Email	Rmbeers777@hotmail.com	
Telephone #	(951) 317-2041	
Signature		
Date		

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## Section 1 Discretionary Permit(s)

<b>Form 1-1 Project Information</b>					
Project Name		Mill Creek Master Plan			
Project Owner Contact Name:		Joe Blum			
Mailing Address:	1730 Evergreen Street, Duarte, CA 91010	E-mail Address:	jblum@butier.com	Telephone:	(714) 448-0216
Permit/Application Number(s):		TBD	Tract/Parcel Map Number(s):		Tentative Tract 18846
Additional Information/Comments:		Subject Property is a portion of the Preserve Specific Plan			
Description of Project:		<p>The property is located to the north of Prado Reservoir, with Mill Creek traversing the easterly edge of the property. The proposed project will have both attached and detached single-family residential development, with a zoning overlay allowing a small amount of commercial uses, a church, and an assisted living facility. The property is nominally 272-acres in size, with nominally 143-acres being built out as housing and streets, and nominally 128-acres remaining as active and passive open space. A master homeowners association will be formed for the project.</p>			
Provide summary of Conceptual WQMP conditions (if previously submitted and approved). Attach complete copy.		<p>Large areas of the property have been used for agricultural and dairy purposes. In order to mitigate soil issues associated with these uses, some material may have to be removed due to high concentrations of organic material. That material will be removed, hauled offsite, and disposed of in accordance with local regulations. Mass grading of the site will address any issues associated with undocumented fills, low levels of organic material, and alluvium soils requiring over excavation and recompaction. Per the 2006 CWA Section 303(d) list of Water Quality Limited Segments for the Santa Ana Regional Board found in Attachment 6-1, the Mill Creek (Prado Area) is impaired due to nutrients, pathogens, and total suspended solids.</p> <p>Watershed Impact of project hydrological studies for the 2-year and 100-year frequency storm events have been prepared for the project. The proposed project has been designed to reduce runoff for both cases. The difference between flows for pre-development conditions versus post development conditions for the 2-year and 100-year frequency storms is greater than the flows calculated for Qbmp.</p> <p>The project includes two drainage areas shown as A and B. Drainage area A is being constructed with a large detention basin which is designed to detain 100% of the 24 hour 100-year storm event and draw down in less than 96 hours. Biofiltration water quality basins are being constructed in both A and B drainage areas which will treat the design capture volume (DCV) produced by the water quality storm event. Drainage Area B was designed to have positive drainage into Mill Creek and allow for additional storage for Prado Reservoir per direction of the Army Core of Engineers (ACOE). The eastern portion of Drainage Area B is developed similar to the existing conditions in that it drains directly into Mill Creek. Increases in runoff are not anticipated from the project. Basin A as well as the multiple water quality basins throughout the project will mitigate the runoff generated by the developed site to levels acceptable by the City and State Agencies.</p> <p>For this Preliminary Water Quality Management Plan (PWQMP) we have analyzed the 2-year, 24-hour storm using the methods contained in the San Bernardino County Hydrology Manual (1986) for the areas tributary to Basin A and Drainage Area B.</p>			

For the Basin A tributary area, in the existing condition, approximately 72.75-acres drains towards the southwest property line near Cucamonga Avenue. In the developed condition, approximately 101.2-acres of the site will drain into Basin A. There are five storm drain systems that outlet into Basin A. These storm drain lines will outlet into a series of pretreatment devices such as Vortech Units upstream of infiltration basins prior to discharging into Basin A. Basin A has a total storage volume of 204.7 acre-feet. The volume of a 100-year, 24-hour unit hydrograph for the areas tributary to Basin A is 47.4 acre- feet, which is only 23.2% of the total storage volume. We have not included an analysis of the existing condition unit hydrograph for the two-year storm event since there will be a 100% retention of the 2-year storm runoff volume within Basin A. We have included a 2-year, 24 hour unit hydrograph for the post-developed condition for reference. This volume is approximately 8.55-acre feet, which accounts for 4.2% of the total storage volume of Basin A.

In the existing condition Drainage Area B consists of approximately 167.3 acres draining directly into Mill Creek. These areas are shown as E3 and E4 in the Existing Condition Hydrology Map. In order to provide water quality treatment to the developed areas in the proposed condition these areas have been broken down into separate Drainage Areas B and C. In the proposed condition the total drainage area from Drainage Area B and C is approximately 144.2 acres. The net effect of the proposed grading design is to reduce the footprint of the area that drains directly into Mill Creek or Prado Basin from 167.3-acres to 144.26-acres.

In the proposed condition there are four storm drain systems that outlet into Basin B. These storm drains are designed to discharge into two separate water quality biofiltration basins as shown on the WQMP BMP location map in Attachment 6-1. The eastern portion of Drainage Area B consists of developed slopes with natural vegetation to mimic the existing conditions.

## Section 2 Project Description

### 2.1 Project Information

This section of the WQMP should provide the information listed below. The information provided for Conceptual/ Preliminary WQMP should give sufficient detail to identify the major proposed site design and LID BMPs and other anticipated water quality features that impact site planning. Final Project WQMP must specifically identify all BMP incorporated into the final site design and provide other detailed information as described herein.

The purpose of this information is to help determine the applicable development category, pollutants of concern, watershed description, and long term maintenance responsibilities for the project, and any applicable water quality credits. This information will be used in conjunction with the information in Section 3, Site Description, to establish the performance criteria and to select the LID BMP or other BMP for the project or other alternative programs that the project will participate in, which are described in Section 4.

<b>Form 2.1-1 Description of Proposed Project</b>					
<b>1</b> Development Category (Select all that apply):					
<input type="checkbox"/> Significant re-development involving the addition or replacement of 5,000 ft <sup>2</sup> or more of impervious surface on an already developed site	<input checked="" type="checkbox"/> New development involving the creation of 10,000 ft <sup>2</sup> or more of impervious surface collectively over entire site	<input type="checkbox"/> Automotive repair shops with standard industrial classification (SIC) codes 5013, 5014, 5541, 7532- 7534, 7536-7539	<input type="checkbox"/> Restaurants (with SIC code 5812) where the land area of development is 5,000 ft <sup>2</sup> or more		
<input type="checkbox"/> Hillside developments of 5,000 ft <sup>2</sup> or more which are located on areas with known erosive soil conditions or where the natural slope is 25 percent or more	<input checked="" type="checkbox"/> Developments of 2,500 ft <sup>2</sup> of impervious surface or more adjacent to (within 200 ft) or discharging directly into environmentally sensitive areas or waterbodies listed on the CWA Section 303(d) list of impaired waters.	<input type="checkbox"/> Parking lots of 5,000 ft <sup>2</sup> or more exposed to storm water	<input type="checkbox"/> Retail gasoline outlets that are either 5,000 ft <sup>2</sup> or more, or have a projected average daily traffic of 100 or more vehicles per day		
<input type="checkbox"/> Non-Priority / Non-Category Project <i>May require source control LID BMPs and other LIP requirements. Please consult with local jurisdiction on specific requirements.</i>					
<b>2</b> Project Area (ft <sup>2</sup> ):	8,660,163 sf	<b>3</b> Number of Dwelling Units:	up to 1,074	<b>4</b> SIC Code:	1521 (SFH)
<b>5</b> Is Project going to be phased? Yes <input checked="" type="checkbox"/> No <input type="checkbox"/> <i>If yes, ensure that the WQMP evaluates each phase as a distinct DA, requiring LID BMPs to address runoff at time of completion.</i>					
<b>6</b> Does Project include roads? Yes <input checked="" type="checkbox"/> No <input type="checkbox"/> <i>If yes, ensure that applicable requirements for transportation projects are addressed (see Appendix A of TGD for WQMP)</i>					

## 2.2 Property Ownership/Management

Describe the ownership/management of all portions of the project and site. State whether any infrastructure will transfer to public agencies (City, County, Caltrans, etc.) after project completion. State if a homeowners or property owners association will be formed and be responsible for the long-term maintenance of project stormwater facilities. Describe any lot-level stormwater features that will be the responsibility of individual property owners.

<b>Form 2.2-1 Property Ownership/Management</b>
Describe property ownership/management responsible for long-term maintenance of WQMP stormwater facilities:
A Homeowner's Association will be created and be responsible for the long-term maintenance of the project storm drain facilities.

## 2.3 Potential Stormwater Pollutants

Determine and describe expected stormwater pollutants of concern based on land uses and site activities (refer to Table 3-3 in the TGD for WQMP).

<b>Form 2.3-1 Pollutants of Concern</b>			
Pollutant	Please check: E=Expected, N=Not Expected		Additional Information and Comments
Pathogens (Bacterial / Virus)	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Nutrients - Phosphorous	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Nutrients - Nitrogen	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Noxious Aquatic Plants	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Sediment	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Metals	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Oil and Grease	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Trash/Debris	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Pesticides / Herbicides	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Organic Compounds	E <input checked="" type="checkbox"/>	N <input type="checkbox"/>	
Other:	E <input type="checkbox"/>	N <input type="checkbox"/>	
Other:	E <input type="checkbox"/>	N <input type="checkbox"/>	
Other:	E <input type="checkbox"/>	N <input type="checkbox"/>	

## 2.4 Water Quality Credits

A water quality credit program is applicable for certain types of development projects if it is not feasible to meet the requirements for on-site LID. Proponents for eligible projects, as described below, can apply for water quality credits that would reduce project obligations for selecting and sizing other treatment BMP or participating in other alternative compliance programs. Refer to Section 6.2 in the TGD for WQMP to determine if water quality credits are applicable for the project.

<b>Form 2.4-1 Water Quality Credits</b>			
<b>1</b> Project Types that Qualify for Water Quality Credits: <i>Select all that apply</i>			
<input type="checkbox"/> Redevelopment projects that reduce the overall impervious footprint of the project site. [Credit = % impervious reduced]	Higher density development projects <input type="checkbox"/> Vertical density [20%] <input type="checkbox"/> 7 units/ acre [5%]	<input type="checkbox"/> Mixed use development, (combination of residential, commercial, industrial, office, institutional, or other land uses which incorporate design principles that demonstrate environmental benefits not realized through single use projects) [20%]	<input type="checkbox"/> Brownfield redevelopment (redevelop real property complicated by presence or potential of hazardous contaminants) [25%]
<input type="checkbox"/> Redevelopment projects in established historic district, historic preservation area, or similar significant core city center areas [10%]	<input type="checkbox"/> Transit-oriented developments (mixed use residential or commercial area designed to maximize access to public transportation) [20%]	<input type="checkbox"/> In-fill projects (conversion of empty lots & other underused spaces < 5 acres, substantially surrounded by urban land uses, into more beneficially used spaces, such as residential or commercial areas) [10%]	<input type="checkbox"/> Live-Work developments (variety of developments designed to support residential and vocational needs) [20%]
<b>2</b> Total Credit % <i>(Total all credit percentages up to a maximum allowable credit of 50 percent)</i>			
Description of Water Quality Credit Eligibility (if applicable)	N/A		

## Section 3 Site and Watershed Description

Describe the project site conditions that will facilitate the selection of BMP through an analysis of the physical conditions and limitations of the site and its receiving waters. Identify distinct drainage areas (DA) that collect flow from a portion of the site and describe how runoff from each DA (and sub-watershed DMAs) is conveyed to the site outlet(s). Refer to Section 3.2 in the TGD for WQMP. The form below is provided as an example. Then complete Forms 3.2 and 3.3 for each DA on the project site. ***If the project has more than one drainage area for stormwater management, then complete additional versions of these forms for each DA / outlet.***

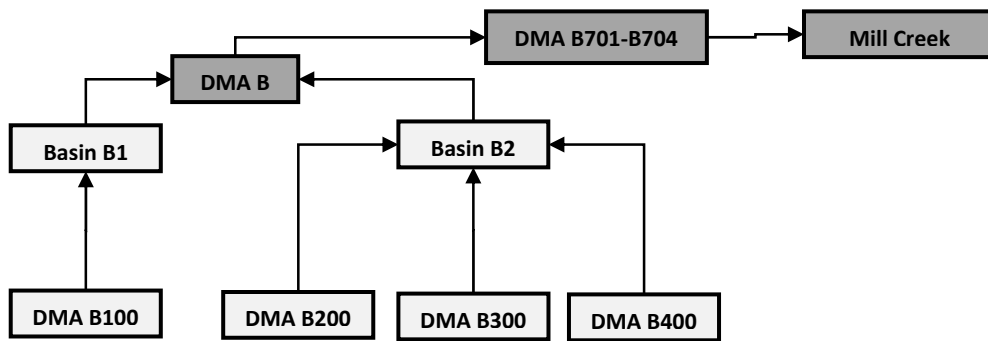
Form 3-1.1 Site Location and Hydrologic Features			
Site coordinates take GPS measurement at approximate center of site	Latitude 33.94	Longitude -117.625	Thomas Bros Map page 712 F4
<p><sup>1</sup> San Bernardino County climatic region: <input checked="" type="checkbox"/> Valley <input type="checkbox"/> Mountain</p>			
<p><sup>2</sup> Does the site have more than one drainage area (DA): Yes <input checked="" type="checkbox"/> No <input type="checkbox"/> If no, proceed to Form 3-2. If yes, then use this form to show a conceptual schematic describing DMAs and hydrologic feature connecting DMAs to the site outlet(s). An example is provided below that can be modified for proposed project or a drawing clearly showing DMA and flow routing may be attached</p>			
<pre> graph TD     A100[DMA A100] --&gt; A1[Basin A1]     A200[DMA A200] --&gt; A1     A1 --&gt; A[Basin A]     A300[DMA A300] --&gt; A2[Basin A2]     A2 --&gt; A     A400[DMA A400] --&gt; A3[Basin A3]     A500[DMA A500] --&gt; A3     A3 --&gt; A     </pre>			
Conveyance	Briefly describe on-site drainage features to convey runoff that is not retained within a DMA		
DMA A100 and DMA A200 flows to Basin A1	DMA A100 (14.6 acres) and DMA A200 (18.7 acres) flow to water quality Basin A1 which flows to detention Basin A. Basin A detains 100% of the 100-year, 24-hour storm event.		
DMA A300 flows to Basin A2	DMA A300 (39.5 acres) flows to water quality Basin A2 which flows to detention Basin A. Basin A detains 100% of the 100-year, 24-hour storm event.		
DMA A400 and DMA A500 flow to Basin A3	DMA A400 (3.8 acres) and DMA A500 (1.3 acres) flow to water quality Basin A3 which flows to Basin A. Basin A detains 100% of the 100-year, 24-hour storm event.		
DMA A600	DMA A600 consists of 10.3 acres and makes up the footprint of Basin A.		

## Form 3-1.2 Site Location and Hydrologic Features

Site coordinates <i>take GPS measurement at approximate center of site</i>	Latitude 33.94	Longitude -117.625	Thomas Bros Map page 712 F4
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<sup>1</sup> San Bernardino County climatic region:  Valley  Mountain

<sup>2</sup> Does the site have more than one drainage area (DA): Yes  No  *If no, proceed to Form 3-2. If yes, then use this form to show a conceptual schematic describing DMAs and hydrologic feature connecting DMAs to the site outlet(s). An example is provided below that can be modified for proposed project or a drawing clearly showing DMA and flow routing may be attached*



Conveyance	Briefly describe on-site drainage features to convey runoff that is not retained within a DMA
DMA B100 Basin B1	DMA B100 (35.55 acres) flows north west within the developed area to the water quality basin B1. Basin B1 drains east through DMA B which drains into Mill Creek.
DMA B200, B300 and DMA B400 flow to Basin B2	DMA B200 (29.2 acres), B300 (7.35 acres), and B400 (5.21 acres) all flow southeast within the developed area to the water quality basin B2. Basin B2 drains southeast through DMA B which outlets into Mill Creek.
DMA B500	DMA B500 consists of 28.2 acres and makes up the footprint of the outfall to DMA B.
DMA B701, B702, B703, and B704	DMA B701 to B704 consists of 60.78 acres of developed open space similar to the existing condition. This flows directly into Mill Creek.

<b>Form 3-2 Existing Hydrologic Characteristics for Project Drainage Area</b>						
For Drainage Area A's sub-watershed DMA, provide the following characteristics	DMA E1	DMA E2	DMA E3	DMA E4	DMA E5	DMA E6
<b>1</b> DMA drainage area (ft <sup>2</sup> )	1,623,046	1,545,944	811,523	4,261,340	1,560,864	654,316
<b>2</b> Existing site impervious area (ft <sup>2</sup> )	162,305	154,594	81,152	426,134	156,086	65,432
<b>3</b> Antecedent moisture condition <i>For desert areas, use <a href="http://www.sbcounty.gov/dpw/floodcontrol/pdf/20100412_map.pdf">http://www.sbcounty.gov/dpw/floodcontrol/pdf/20100412_map.pdf</a></i>	3	3	3	3	3	3
<b>4</b> Hydrologic soil group <i>Refer to Watershed Mapping Tool – <a href="http://sbcounty.permitrack.com/WAP">http://sbcounty.permitrack.com/WAP</a></i>	C	C	C	C	C	C
<b>5</b> Longest flow path length (ft)	1,631	1,158	914	4,500	1,200	1,150
<b>6</b> Longest flow path slope (ft/ft)	0.019	0.016	0.025	0.013	0.014	0.005
<b>7</b> Current land cover type(s) <i>Select from Fig C-3 of Hydrology Manual</i>	79	79	79	79	79	79
<b>8</b> Pre-developed pervious area condition: <i>Based on the extent of wet season vegetated cover good &gt;75%; Fair 50-75%; Poor &lt;50% Attach photos of site to support rating</i>	75	75	75	75	75	75

<b>Form 3-3 Watershed Description for Project Drainage Area</b>	
Receiving waters <i>Refer to Watershed Mapping Tool - <a href="http://sbcounty.permitrack.com/WAP">http://sbcounty.permitrack.com/WAP</a></i> See "Drainage Facilities" link at this website	Cucamonga Channel/Mill Creek (Prado Area) 801.25, Chino Creek Reach 1A, Santa Ana River Watershed Reach 5, 3, 2, and 1
Applicable TMDLs <i>Refer to Local Implementation Plan</i>	Pathogens
303(d) listed impairments <i>Refer to Local Implementation Plan and Watershed Mapping Tool - <a href="http://sbcounty.permitrack.com/WAP">http://sbcounty.permitrack.com/WAP</a> and State Water Resources Control Board website - <a href="http://www.waterboards.ca.gov/santaana/water_issues/programs/tmdl/index.shtml">http://www.waterboards.ca.gov/santaana/water_issues/programs/tmdl/index.shtml</a></i>	Mill Creek (Prado Area) – Nutrients, Total Suspended Solids, Pathogens Santa Ana River Watershed Reach 5 – Nutrients, Pathogens Santa Ana River Watershed Reach 3 - Copper, Lead, Pathogens Santa Ana River Watershed Reach 2 – Indicator Bacteria
Environmentally Sensitive Areas (ESA) <i>Refer to Watershed Mapping Tool - <a href="http://sbcounty.permitrack.com/WAP">http://sbcounty.permitrack.com/WAP</a></i>	Riparian/Wetland
Unlined Downstream Water Bodies <i>Refer to Watershed Mapping Tool - <a href="http://sbcounty.permitrack.com/WAP">http://sbcounty.permitrack.com/WAP</a></i>	Mill Creek
Hydrologic Conditions of Concern	<input checked="" type="checkbox"/> Yes Complete Hydrologic Conditions of Concern (HCOC) Assessment. Include Forms 4.2-2 through Form 4.2-5 and Hydromodification BMP Form 4.3-10 in submittal <input type="checkbox"/> No
Watershed-based BMP included in a RWQCB approved WAP	<input type="checkbox"/> Yes Attach verification of regional BMP evaluation criteria in WAP <ul style="list-style-type: none"> <li>• More Effective than On-site LID</li> <li>• Remaining Capacity for Project DCV</li> <li>• Upstream of any Water of the US</li> <li>• Operational at Project Completion</li> <li>• Long-Term Maintenance Plan</li> </ul> <input checked="" type="checkbox"/> No

## Section 4 Best Management Practices (BMP)

### 4.1 Source Control BMP

#### 4.1.1 Pollution Prevention

Non-structural and structural source control BMP are required to be incorporated into all new development and significant redevelopment projects. Form 4.1-1 and 4.1-2 are used to describe specific source control BMPs used in the WQMP or to explain why a certain BMP is not applicable. Table 7-3 of the TGD for WQMP provides a list of applicable source control BMP for projects with specific types of potential pollutant sources or activities. The source control BMP in this table must be implemented for projects with these specific types of potential pollutant sources or activities.

The preparers of this WQMP have reviewed the source control BMP requirements for new development and significant redevelopment projects. The preparers have also reviewed the specific BMP required for project as specified in Forms 4.1-1 and 4.1-2. All applicable non-structural and structural source control BMP shall be implemented in the project.

<b>Form 4.1-1 Non-Structural Source Control BMPs</b>				
Identifier	Name	Check One		Describe BMP Implementation OR, if not applicable, state reason
		Included	Not Applicable	
N1	Education of Property Owners, Tenants and Occupants on Stormwater BMPs	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Education of Property Owners" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2
N2	Activity Restrictions	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Activity Restrictions" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2
N3	Landscape Management BMPs	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Landscape Management" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2
N4	BMP Maintenance	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "BMP Maintenance" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N5	Title 22 CCR Compliance (How development will comply)	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Title 22 CCR Compliance" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N6	Local Water Quality Ordinances	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Local Water Quality Ordinances" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N7	Spill Contingency Plan	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Spill Contingency" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N8	Underground Storage Tank Compliance	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There will be no underground storage tanks on property.
N9	Hazardous Materials Disclosure Compliance	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There will be no hazardous material on property.

<b>Form 4.1-1 Non-Structural Source Control BMPs</b>				
Identifier	Name	Check One		Describe BMP Implementation OR, if not applicable, state reason
		Included	Not Applicable	
N10	Uniform Fire Code Implementation	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "uniform Fire Code Implementation" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N11	Litter/Debris Control Program	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Litter/Debris Control Program" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N12	Employee Training	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Employee Training" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N13	Housekeeping of Loading Docks	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no housekeeping of loading docks within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of loading docks in order to protect water quality.
N14	Catch Basin Inspection Program	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Catch Basin Inspection Program" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N15	Vacuum Sweeping of Private Streets and Parking Lots	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Street Sweeping Private Streets and Parking Lots" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
N16	Other Non-structural Measures for Public Agency Projects	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no other non-structural measures for Public Measures for Agency.
N17	Comply with all other applicable NPDES permits	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure compliance with all other applicable NPDES permits" for our specific project site. Detailed information on the implementation of this planned source permits control BMP will be determined and discussed in the Final WQMP, Section 3.2.

<b>Form 4.1-2 Structural Source Control BMPs</b>				
Identifier	Name	Check One		Describe BMP Implementation OR, If not applicable, state reason
		Included	Not Applicable	
S1	Provide storm drain system stencilling and signage (CASQA New Development BMP Handbook SD-13)	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Storm Drain Signage" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
S2	Design and construct outdoor material storage areas to reduce pollution introduction (CASQA New Development BMP Handbook SD-34)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no Outdoor Material Storage areas within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of outdoor material storage areas in order to protect water quality.
S3	Design and construct trash and waste storage areas to reduce pollution introduction (CASQA New Development BMP Handbook SD-32)	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Trash Storage Areas" BMP will be implemented for our specific project site. Detailed information on the implementation of this will be determined and discussed in the Final WQMP, Section 3.2.
S4	Use efficient irrigation systems & landscape design, water conservation, smart controllers, and source control (Statewide Model Landscape Ordinance; CASQA New Development BMP Handbook SD-12)	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Efficient Irrigation" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
S5	Finish grade of landscaped areas at a minimum of 1-2 inches below top of curb, sidewalk, or pavement	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Finish grade of landscaped areas be at a minimum Finish grade of landscaped areas at a minimum of 1-2 inches below top of curb, sidewalk, or pavement" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
S6	Protect slopes and channels and provide energy dissipation (CASQA New Development BMP Handbook SD-10)	<input checked="" type="checkbox"/>	<input type="checkbox"/>	The Developer will ensure the "Protect Slopes and Drainage" BMP will be implemented for our specific project site. Detailed information on the implementation of this planned source control BMP will be determined and discussed in the Final WQMP, Section 3.2.
S7	Covered dock areas (CASQA New Development BMP Handbook SD-31)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There will be no covered dock areas within the project.
S8	Covered maintenance bays with spill containment plans (CASQA New Development BMP Handbook SD-31)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no maintenance bays or docks within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of maintenance bays or docks in order to protect water quality.

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S9	Vehicle wash areas with spill containment plans (CASQA New Development BMP Handbook SD-33)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	The owner shall prohibit vehicle-washing areas within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of vehicle-washing areas in order to protect water quality.
S10	Covered outdoor processing areas (CASQA New Development BMP Handbook SD-36)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no outdoor processing areas within the project site. Not a proposed use. The residents terms and tenant/occupant's lease terms will prohibit the use of outdoor processing areas in order to protect water quality.
S11	Equipment wash areas with spill containment plans (CASQA New Development BMP Handbook SD-33)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no equipment wash areas within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of equipment wash areas in order to project water quality.
S12	Fueling areas (CASQA New Development BMP Handbook SD-30)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no fueling areas within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of fueling areas in order to project water quality.
S13	Hillside landscaping (CASQA New Development BMP Handbook SD-10)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no existing hillsides within the project.
S14	Wash water control for food preparation areas	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no food preparation areas within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of food preparation areas in order to protect water quality.
S15	Community car wash racks (CASQA New Development BMP Handbook SD-33)	<input type="checkbox"/>	<input checked="" type="checkbox"/>	There are no community car wash racks within the project site. Not a proposed land use. The residents terms and tenant/occupant's lease terms will prohibit the use of car wash areas in order to project water quality.

### 4.1.2 Preventative LID Site Design Practices

Site design practices associated with new LID requirements in the MS4 Permit should be considered in the earliest phases of a project. Preventative site design practices can result in smaller DCV for LID BMP and hydromodification control BMP by reducing runoff generation. Describe site design and drainage plan including:

- A narrative of site design practices utilized or rationale for not using practices
- A narrative of how site plan incorporates preventive site design practices
- Include an attached Site Plan layout which shows how preventative site design practices are included in WQMP

Refer to Section 5.2 of the TGD for WQMP for more details.

<b>Form 4.1-3 Preventative LID Site Design Practices Checklist</b>
<p>Site Design Practices <i>If yes, explain how preventative site design practice is addressed in project site plan. If no, other LID BMPs must be selected to meet targets</i></p>
<p>Minimize impervious areas: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: THE PROJECT WILL MINIMIZE THE USE OF IMPERVIOUS SURFACES AND WILL MINIMIZE THE USE DECORATIVE CONCRETE IN THE LANDSCAPE DESIGN.</p>
<p>Maximize natural infiltration capacity: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: ON-SITE SOILS ARE POORLY DRAINING AND HAVE RELATIVELY HIGH GROUNDWATER. IN THE PROPOSED CONDITION, BASINS A AND B ARE DESIGNED TO PROVIDE TREATMENT AND INCIDENTAL INFILTRATION TO THE EXTENT POSSIBLE. DUE TO EXISTING SOIL CONDITIONS INFILTRATION IS NOT FEASIBLE.</p>
<p>Preserve existing drainage patterns and time of concentration: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: IN PROPOSED CONDITION, BASIN A WILL DETAIN 100% OF THE 100-YEAR, 24-HOUR STORM.</p>
<p>Disconnect impervious areas: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: DRAINING INTO VEGETATED SWALES AND ULTIMATELY, BASINS A &amp; B.</p>
<p>Protect existing vegetation and sensitive areas: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: APPROXIMATELY 47% OF THE SITE WILL REMAIN AS ACTIVE AND PASSIVE OPEN SPACE. CONSERVATION EASEMENTS OVER PORTIONS OF THE OPEN SPACE ALONG MILL CREEK WILL CONSERVE NATURAL OPEN SPACE AREAS.</p>
<p>Re-vegetate disturbed areas: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: NEW LANDSCAPEING WILL BE PROPOSED IN ALL AREAS TO RE-ESTABLISH DISTURBED VEGETATION.</p>
<p>Minimize unnecessary compaction in stormwater retention/infiltration basin/trench areas: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: BASIN BOTIOMS WILL HAVE AMENDED SOILS TO INCREASE INFILTRATION AND MINIMIZE COMPACTION.</p>
<p>Utilize vegetated drainage swales in place of underground piping or imperviously lined swales: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: THE DEVELOPMENT WILL USE VEGETATED DRAINAGE SWALES IN LIEU OF IMPERVIOUSLY LINED SWALES WHERE DETERMINED FEASIBLE DURING FINAL ENGINEERING DESIGN.</p>
<p>Stake off areas that will be used for landscaping to minimize compaction during construction : Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>                      Explanation: STAKED OFF AREAS WILL BE DELINEATED IN LANDSCAPE DRAWINGS.</p>

## 4.2 Project Performance Criteria

The purpose of this section of the Project WQMP is to establish targets for post-development hydrology based on performance criteria specified in the MS4 Permit. These targets include runoff volume for water quality control (referred to as LID design capture volume), and runoff volume, time of concentration, and peak runoff for protection of any downstream waterbody segments with a HCOC. ***If the project has more than one outlet for stormwater runoff, then complete additional versions of these forms for each DA / outlet.***

Methods applied in the following forms include:

- For LID BMP Design Capture Volume (DCV), the San Bernardino County Stormwater Program requires use of the P<sub>6</sub> method (MS4 Permit Section XI.D.6a.ii) – Form 4.2-1
- For HCOC pre- and post-development hydrologic calculation, the San Bernardino County Stormwater Program requires the use of the Rational Method (San Bernardino County Hydrology Manual Section D). Forms 4.2-2 through Form 4.2-5 calculate hydrologic variables including runoff volume, time of concentration, and peak runoff from the project site pre- and post-development using the Hydrology Manual Rational Method approach. For projects greater than 640 acres (1.0 mi<sup>2</sup>), the Rational Method and these forms should not be used. For such projects, the Unit Hydrograph Method (San Bernardino County Hydrology Manual Section E) shall be applied for hydrologic calculations for HCOC performance criteria.

Refer to Section 4 in the TGD for WQMP for detailed guidance and instructions.

<b>Form 4.2-1 LID BMP Performance Criteria for Design Capture Volume (DMA A100 and A200)</b>		
<b>1</b> Project area DA 1 (ft <sup>2</sup> ): 1,450,984	<b>2</b> Imperviousness after applying preventative site design practices (Imp%): 0.60	<b>3</b> Runoff Coefficient (Rc): _0.419 <i>R<sub>c</sub> = 0.858(Imp%)<sup>0.3</sup> - 0.78(Imp%)<sup>0.2</sup> + 0.774(Imp%) + 0.04</i>
<b>4</b> Determine 1-hour rainfall depth for a 2-year return period P <sub>2yr-1hr</sub> (in): 0.5 <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>		
<b>5</b> Compute P <sub>6</sub> , Mean 6-hr Precipitation (inches): 0.740 <i>P<sub>6</sub> = Item 4 * C<sub>1</sub>, where C<sub>1</sub> is a function of site climatic region specified in Form 3-1 Item 1 (Valley = 1.4807; Mountain = 1.909; Desert = 1.2371)</i>		
<b>6</b> Drawdown Rate <i>Use 48 hours as the default condition. Selection and use of the 24 hour drawdown time condition is subject to approval by the local jurisdiction. The necessary BMP footprint is a function of drawdown time. While shorter drawdown times reduce the performance criteria for LID BMP design capture volume, the depth of water that can be stored is also reduced.</i>		24-hrs <input type="checkbox"/> 48-hrs <input checked="" type="checkbox"/>
<b>7</b> Compute design capture volume, DCV (ft <sup>3</sup> ): 71,890 <i>DCV = 1/12 * [Item 1 * Item 3 * Item 5 * C<sub>2</sub>], where C<sub>2</sub> is a function of drawdown rate (24-hr = 1.582; 48-hr = 1.963)</i> <i>Compute separate DCV for each outlet from the project site per schematic drawn in Form 3-1 Item 2</i>		

<b>Form 4.2-1 LID BMP Performance Criteria for Design Capture Volume (DMA A300)</b>		
<b>1</b> Project area DA 1 (ft <sup>2</sup> ): 1,718,878	<b>2</b> Imperviousness after applying preventative site design practices (Imp%): 0.59	<b>3</b> Runoff Coefficient (Rc): <u>0.41</u> $R_c = 0.858(\text{Imp}\%)^{0.3} - 0.78(\text{Imp}\%)^{0.2} + 0.774(\text{Imp}\%) + 0.04$
<b>4</b> Determine 1-hour rainfall depth for a 2-year return period $P_{2\text{yr-1hr}}$ (in): 0.5 <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>		
<b>5</b> Compute $P_6$ , Mean 6-hr Precipitation (inches): 0.740 $P_6 = \text{Item 4} * C_1$ , where $C_1$ is a function of site climatic region specified in Form 3-1 Item 1 (Valley = 1.4807; Mountain = 1.909; Desert = 1.2371)		
<b>6</b> Drawdown Rate <i>Use 48 hours as the default condition. Selection and use of the 24 hour drawdown time condition is subject to approval by the local jurisdiction. The necessary BMP footprint is a function of drawdown time. While shorter drawdown times reduce the performance criteria for LID BMP design capture volume, the depth of water that can be stored is also reduced.</i>		24-hrs <input type="checkbox"/> 48-hrs <input checked="" type="checkbox"/>
<b>7</b> Compute design capture volume, DCV (ft <sup>3</sup> ): 84,213 $DCV = 1/12 * [\text{Item 1} * \text{Item 3} * \text{Item 5} * C_2]$ , where $C_2$ is a function of drawdown rate (24-hr = 1.582; 48-hr = 1.963) <i>Compute separate DCV for each outlet from the project site per schematic drawn in Form 3-1 Item 2</i>		

<b>Form 4.2-1 LID BMP Performance Criteria for Design Capture Volume (DMA A400 and A500)</b>		
<b>1</b> Project area DA 1 (ft <sup>2</sup> ): 222,156	<b>2</b> Imperviousness after applying preventative site design practices (Imp%): 0.49	<b>3</b> Runoff Coefficient (Rc): <u>0.33</u> $R_c = 0.858(\text{Imp}\%)^{0.3} - 0.78(\text{Imp}\%)^{0.2} + 0.774(\text{Imp}\%) + 0.04$
<b>4</b> Determine 1-hour rainfall depth for a 2-year return period $P_{2\text{yr-1hr}}$ (in): 0.5 <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>		
<b>5</b> Compute $P_6$ , Mean 6-hr Precipitation (inches): 0.740 $P_6 = \text{Item 4} * C_1$ , where $C_1$ is a function of site climatic region specified in Form 3-1 Item 1 (Valley = 1.4807; Mountain = 1.909; Desert = 1.2371)		
<b>6</b> Drawdown Rate <i>Use 48 hours as the default condition. Selection and use of the 24 hour drawdown time condition is subject to approval by the local jurisdiction. The necessary BMP footprint is a function of drawdown time. While shorter drawdown times reduce the performance criteria for LID BMP design capture volume, the depth of water that can be stored is also reduced.</i>		24-hrs <input type="checkbox"/> 48-hrs <input checked="" type="checkbox"/>
<b>7</b> Compute design capture volume, DCV (ft <sup>3</sup> ): 9,134 $DCV = 1/12 * [\text{Item 1} * \text{Item 3} * \text{Item 5} * C_2]$ , where $C_2$ is a function of drawdown rate (24-hr = 1.582; 48-hr = 1.963) <i>Compute separate DCV for each outlet from the project site per schematic drawn in Form 3-1 Item 2</i>		

<b>Form 4.2-1 LID BMP Performance Criteria for Design Capture Volume (DMA B100)</b>		
<b>1</b> Project area DA 1 (ft <sup>2</sup> ): 1,548,558	<b>2</b> Imperviousness after applying preventative site design practices (Imp%): 0.59	<b>3</b> Runoff Coefficient (Rc): <u>0.41</u> $R_c = 0.858(\text{Imp}\%)^{0.3} - 0.78(\text{Imp}\%)^{0.2} + 0.774(\text{Imp}\%) + 0.04$
<b>4</b> Determine 1-hour rainfall depth for a 2-year return period $P_{2\text{yr-1hr}}$ (in): 0.5 <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>		
<b>5</b> Compute $P_6$ , Mean 6-hr Precipitation (inches): 0.740 $P_6 = \text{Item 4} * C_1$ , where $C_1$ is a function of site climatic region specified in Form 3-1 Item 1 (Valley = 1.4807; Mountain = 1.909; Desert = 1.2371)		
<b>6</b> Drawdown Rate <i>Use 48 hours as the default condition. Selection and use of the 24 hour drawdown time condition is subject to approval by the local jurisdiction. The necessary BMP footprint is a function of drawdown time. While shorter drawdown times reduce the performance criteria for LID BMP design capture volume, the depth of water that can be stored is also reduced.</i>		24-hrs <input type="checkbox"/> 48-hrs <input checked="" type="checkbox"/>
<b>7</b> Compute design capture volume, DCV (ft <sup>3</sup> ): 75,868 $DCV = 1/12 * [\text{Item 1} * \text{Item 3} * \text{Item 5} * C_2]$ , where $C_2$ is a function of drawdown rate (24-hr = 1.582; 48-hr = 1.963) <i>Compute separate DCV for each outlet from the project site per schematic drawn in Form 3-1 Item 2</i>		

<b>Form 4.2-1 LID BMP Performance Criteria for Design Capture Volume (DMA B200, B300, and B400)</b>		
<b>1</b> Project area DA 1 (ft <sup>2</sup> ): 1,819,066	<b>2</b> Imperviousness after applying preventative site design practices (Imp%): 0.82	<b>3</b> Runoff Coefficient (Rc): <u>0.62</u> $R_c = 0.858(\text{Imp}\%)^{0.3} - 0.78(\text{Imp}\%)^{0.2} + 0.774(\text{Imp}\%) + 0.04$
<b>4</b> Determine 1-hour rainfall depth for a 2-year return period $P_{2\text{yr-1hr}}$ (in): 0.5 <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>		
<b>5</b> Compute $P_6$ , Mean 6-hr Precipitation (inches): 0.740 $P_6 = \text{Item 4} * C_1$ , where $C_1$ is a function of site climatic region specified in Form 3-1 Item 1 (Valley = 1.4807; Mountain = 1.909; Desert = 1.2371)		
<b>6</b> Drawdown Rate <i>Use 48 hours as the default condition. Selection and use of the 24 hour drawdown time condition is subject to approval by the local jurisdiction. The necessary BMP footprint is a function of drawdown time. While shorter drawdown times reduce the performance criteria for LID BMP design capture volume, the depth of water that can be stored is also reduced.</i>		24-hrs <input type="checkbox"/> 48-hrs <input checked="" type="checkbox"/>
<b>7</b> Compute design capture volume, DCV (ft <sup>3</sup> ): 137,266 $DCV = 1/12 * [\text{Item 1} * \text{Item 3} * \text{Item 5} * C_2]$ , where $C_2$ is a function of drawdown rate (24-hr = 1.582; 48-hr = 1.963) <i>Compute separate DCV for each outlet from the project site per schematic drawn in Form 3-1 Item 2</i>		

### Form 4.2-2 Summary of HCOC Assessment (DA A)

Does project have the potential to cause or contribute to an HCOC in a downstream channel: Yes  No

Go to: <http://sbcounty.permitrack.com/WAP>

If "Yes", then complete HCOC assessment of site hydrology for 2yr storm event using Forms 4.2-3 through 4.2-5 and insert results below  
(Forms 4.2-3 through 4.2-5 may be replaced by computer software analysis based on the San Bernardino County Hydrology Manual)

If "No," then proceed to Section 4.3 Project Conformance Analysis

Condition	Runoff Volume (ft <sup>3</sup> )	Time of Concentration (min)	Peak Runoff (cfs)
Pre-developed	<sup>1</sup> 173,312 <i>Form 4.2-3 Item 12</i>	<sup>2</sup> 26.48 <i>Form 4.2-4 Item 6</i>	<sup>3</sup> 54.8 <i>Form 4.2-5 Item 5</i>
Post-developed	<sup>4</sup> 208,980 <i>Form 4.2-3 Item 13</i>	<sup>5</sup> 24.29 <i>Form 4.2-4 Item 7</i>	<sup>6</sup> 7.5 <i>Form 4.2-5 Item 6</i>
Difference	<sup>7</sup> 35,668 <i>Item 4 – Item 1</i>	<sup>8</sup> 2.19 <i>Item 2 – Item 5</i>	<sup>9</sup> 47.3 <i>Item 6 – Item 3</i>
Difference (as % of pre-developed)	<sup>10</sup> 21 % <i>Item 7 / Item 1</i>	<sup>11</sup> 8.2 % <i>Item 8 / Item 2</i>	<sup>12</sup> 86 % <i>Item 9 / Item 3</i>

### Form 4.2-2 Summary of HCOC Assessment (DA B)

Does project have the potential to cause or contribute to an HCOC in a downstream channel: Yes  No

Go to: <http://sbcounty.permitrack.com/WAP>

If "Yes", then complete HCOC assessment of site hydrology for 2yr storm event using Forms 4.2-3 through 4.2-5 and insert results below  
(Forms 4.2-3 through 4.2-5 may be replaced by computer software analysis based on the San Bernardino County Hydrology Manual)

If "No," then proceed to Section 4.3 Project Conformance Analysis

Condition	Runoff Volume (ft <sup>3</sup> )	Time of Concentration (min)	Peak Runoff (cfs)
Pre-developed	<sup>1</sup> 281,989 <i>Form 4.2-3 Item 12</i>	<sup>2</sup> 122.5 <i>Form 4.2-4 Item 6</i>	<sup>3</sup> 19.35 <i>Form 4.2-5 Item 5</i>
Post-developed	<sup>4</sup> 288,475 <i>Form 4.2-3 Item 13</i>	<sup>5</sup> 18.31 <i>Form 4.2-4 Item 7</i>	<sup>6</sup> 57.9 ( <i>Form 4.2-5 Item 6</i> ) <sup>20.5</sup> ( <i>Basin Routed UH Method</i> )
Difference	<sup>7</sup> 6,487 <i>Item 4 – Item 1</i>	<sup>8</sup> 104.2 <i>Item 2 – Item 5</i>	<sup>9</sup> 1.15 <i>Item 6 – Item 3</i>
Difference (as % of pre-developed)	<sup>10</sup> 2.3 % <i>Item 7 / Item 1</i>	<sup>11</sup> 85 % <i>Item 8 / Item 2</i>	<sup>12</sup> 5.9% <i>Item 9 / Item 3</i>

Basins in area B have a total volume of 213,134 cf and will detain the difference between the post and pre-development. The peak discharge is also reduced to 20.5 cfs when routed through the basins.

Form 4.2-3 HCOC Assessment for Runoff Volume (DA A)							
<b>Weighted Curve Number Determination for: Pre-developed DA</b>	DMA E1		DMA E2		DMA E3		
<b>1a</b> Land Cover type	Agricultural Dryland with Fair Cover						
<b>2a</b> Hydrologic Soil Group (HSG)	C		C		C		
<b>3a</b> DMA Area, ft <sup>2</sup> sum of areas of DMA should equal area of DA	1,623,045.6		1,545,944.4		811,522.8		
<b>4a</b> Curve Number (CN) use Items 1 and 2 to select the appropriate CN from Appendix C-2 of the TGD for WQMP	79		79		79		
<b>Weighted Curve Number Determination for: Post-developed DA</b>	DMA A100	DMA A200	DMA A300	DMA A400	DMA A500	DMA A600	
<b>1b</b> Land Cover type	Residential / Commercial Good Cover						
<b>2b</b> Hydrologic Soil Group (HSG)	C	C	C	C	C	C	
<b>3b</b> DMA Area, ft <sup>2</sup> sum of areas of DMA should equal area of DA	1,034,550	814,572	1,834,747	163,350	55,757	441,263	
<b>4b</b> Curve Number (CN) use Items 5 and 6 to select the appropriate CN from Appendix C-2 of the TGD for WQMP	82.3	82.3	82.3	82.3	82.3	77	
<b>5</b> Pre-Developed area-weighted CN: 79	<b>7</b> Pre-developed soil storage capacity, S (in): 2.7 $S = (1000 / \text{Item } 5) - 10$				<b>9</b> Initial abstraction, I <sub>a</sub> (in): 0.5 $I_a = 0.2 * \text{Item } 7$		
<b>6</b> Post-Developed area-weighted CN: 82	<b>8</b> Post-developed soil storage capacity, S (in): 2.2 $S = (1000 / \text{Item } 6) - 10$				<b>10</b> Initial abstraction, I <sub>a</sub> (in): 0.43 $I_a = 0.2 * \text{Item } 8$		
<b>11</b> Precipitation for 2 yr, 24 hr storm (in): 2.0 Go to: <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/so/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/so/sca_pfds.html</a>							
<b>12</b> Pre-developed Volume (ft <sup>3</sup> ): 173,312 $V_{pre} = (1 / 12) * (\text{Item sum of Item } 3) * [(\text{Item } 11 - \text{Item } 9)^2 / ((\text{Item } 11 - \text{Item } 9 + \text{Item } 7))$							
<b>13</b> Post-developed Volume (ft <sup>3</sup> ): 208,980 $V_{pre} = (1 / 12) * (\text{Item sum of Item } 3) * [(\text{Item } 11 - \text{Item } 10)^2 / ((\text{Item } 11 - \text{Item } 10 + \text{Item } 8))$							
<b>14</b> Volume Reduction needed to meet HCOC Requirement, (ft <sup>3</sup> ): 35,668 $V_{HCOC} = (\text{Item } 13 * 0.95) - \text{Item } 12$							

**Form 4.2-3 HCOC Assessment for Runoff Volume (DA B)**

<b>Weighted Curve Number Determination for: Pre-developed DA</b>	DMA E4		DMA E5		DMA E6			
<b>1a</b> Land Cover type	Agricultural Dryland with Fair Cover							
<b>2a</b> Hydrologic Soil Group (HSG)	C		C		C			
<b>3a</b> DMA Area, ft <sup>2</sup> sum of areas of DMA should equal area of DA	4,261,340		1,560,864		654,316			
<b>4a</b> Curve Number (CN) use Items 1 and 2 to select the appropriate CN from Appendix C-2 of the TGD for WQMP	79		79		79			
<b>Weighted Curve Number Determination for: Post-developed DA</b>	DMA B100	DMA B200	DMA B300	DMA B400	DMA B701	DMA B702	DMA B703	DMA B704
<b>1b</b> Land Cover type	Residential / Commercial Good Cover				Open Brush Fair Cover			
<b>2b</b> Hydrologic Soil Group (HSG)	C	C	C	C	C	C	C	C
<b>3b</b> DMA Area, ft <sup>2</sup> sum of areas of DMA should equal area of DA	1,548,558	1,217,952	320,166	226,948	344,124	1,274,66	256,568	772,319
<b>4b</b> Curve Number (CN) use Items 5 and 6 to select the appropriate CN from Appendix C-2 of the TGD for WQMP	80	80	80	80	77	77	77	77
<b>5</b> Pre-Developed area-weighted CN: 79	<b>7</b> Pre-developed soil storage capacity, S (in): 2.7 $S = (1000 / \text{Item } 5) - 10$				<b>9</b> Initial abstraction, I <sub>a</sub> (in): 0.5 $I_a = 0.2 * \text{Item } 7$			
<b>6</b> Post-Developed area-weighted CN: 80	<b>8</b> Post-developed soil storage capacity, S (in): 2.5 $S = (1000 / \text{Item } 6) - 10$				<b>10</b> Initial abstraction, I <sub>a</sub> (in): 0.5 $I_a = 0.2 * \text{Item } 8$			
<b>11</b> Precipitation for 2 yr, 24 hr storm (in): 2 Go to: <a href="http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html">http://hdsc.nws.noaa.gov/hdsc/pfds/sa/sca_pfds.html</a>								
<b>12</b> Pre-developed Volume (ft <sup>3</sup> ): 281,989 $V_{pre} = (1 / 12) * (\text{Item sum of Item } 3) * [(\text{Item } 11 - \text{Item } 9)^2 / ((\text{Item } 11 - \text{Item } 9 + \text{Item } 7))$								
<b>13</b> Post-developed Volume (ft <sup>3</sup> ): 288,475 $V_{pre} = (1 / 12) * (\text{Item sum of Item } 3) * [(\text{Item } 11 - \text{Item } 10)^2 / ((\text{Item } 11 - \text{Item } 10 + \text{Item } 8))$								
<b>14</b> Volume Reduction needed to meet HCOC Requirement, (ft <sup>3</sup> ): 7,937 $V_{HCOC} = (\text{Item } 13 * 0.95) - \text{Item } 12$								

**Preliminary Water Quality Management Plan (PWQMP)**

Time of concentration and peak runoff results were calculated using the computer program CivilD, a San Bernardino County approved hydrology program, and the San Bernardino County Hydrology Manual. Forms 4.2-4 and 4.2-5 have been modified from their original layout to summarize the results from the 2 year storm event analysis. Post-developed results in both Forms 4.2-4 and 4.2-5 were developed using the Rational method and don't include the proposed basin routing. Brief summaries are included after each form to summarize the results of the post-developed condition with the proposed basin routing. The results from the routed analysis are included to show the impacts of the proposed basins and show compliance with the HCOC requirements.

<b>Form 4.2-4 HCOC Assessment for Time of Concentration (DA A)</b>								
Compute time of concentration for pre and post developed conditions for each DA ( <i>For projects using the Hydrology Manual complete the form below</i> )								
Variables	Pre-developed DA A <i>Use additional forms if there are more than 4 DMA</i>				Post-developed DA A <i>Use additional forms if there are more than 4 DMA</i>			
	DMA E1	DMA E2	DMA E3		DMA A100	DMA A200	DMA A300	DMA A400
<b>1</b> Land cover	Agricultural Dryland with Fair Cover				Single Family Residential and Commercial			
<b>2</b> Total time of concentration (min) $T_c$	13.26	26.48	19.21		21.84	12.94	22.26	24.29
<b>3</b> Pre-developed time of concentration (min): 26.48								
<b>4</b> Post-developed time of concentration (min): See next table								
<b>5</b> Additional time of concentration needed to meet HCOC requirement (min): See next table $T_{C-HCOC} = (Item\ 6 * 0.95) - Item\ 7$								

Form 4.2-4 HCOC Assessment for Time of Concentration (DA A)								
Compute time of concentration for pre and post developed conditions for each DA (For projects using the Hydrology Manual complete the form below)								
Variables	Pre-developed DA A <i>Use additional forms if there are more than 4 DMA</i>				Post-developed DA A <i>Use additional forms if there are more than 4 DMA</i>			
					DMA A500	DMA A600		
1 Land cover					Single Family Residential and Commercial			
2 Total time of concentration (min) $T_c$					13.66	34.34		
3 Pre-developed time of concentration (min): 26.48								
4 Post-developed time of concentration (min): 24.29								
8 Additional time of concentration needed to meet HCOC requirement (min): 0.87 $T_{C-HCOC} = (Item\ 6 * 0.95) - Item\ 7$								

Form 4.2-4 HCOC Assessment for Time of Concentration (DA B)								
Compute time of concentration for pre and post developed conditions for each DA (For projects using the Hydrology Manual complete the form below)								
Variables	Pre-developed DA B <i>Use additional forms if there are more than 4 DMA</i>				Post-developed DA B <i>Use additional forms if there are more than 4 DMA</i>			
	DMA E4	DMA E5	DMA E6		DMA B100	DMA B200	DMA B300	DMA B400
4 Land cover	Agricultural Dryland with Fair Cover				Single Family Residential and Commercial			
5 Total time of concentration (min) $T_c$	122.5	60.17	71.06		15.19	20.47	11.09	16.77
6 Pre-developed time of concentration (min): 122.5								
7 Post-developed time of concentration (min): See next table								
8 Additional time of concentration needed to meet HCOC requirement (min): See next table $T_{C-HCOC} = (Item\ 6 * 0.95) - Item\ 7$								

### Form 4.2-4 HCOC Assessment for Time of Concentration (DA B)

Compute time of concentration for pre and post developed conditions for each DA (*For projects using the Hydrology Manual complete the form below*)

Variables	Pre-developed DA B <i>Use additional forms if there are more than 4 DMA</i>				Post-developed DA B <i>Use additional forms if there are more than 4 DMA</i>			
					DMA B701	DMA B702	DMA B703	
<b>1</b> Land cover					Single Family Residential and Commercial			
<b>2</b> Total time of concentration (min) $\tau_c$					33.72	61.12	27.86	
<b>3</b> Pre-developed time of concentration (min): 122.5								
<b>4</b> Post-developed time of concentration (min): 18.31								
<b>5</b> Additional time of concentration needed to meet HCOC requirement (min): 98.06 $T_{C-HCOC} = (Item\ 3 * 0.95) - Item\ 4$								

### Form 4.2-5 HCOC Assessment for Peak Runoff (DA A)

Compute peak runoff for pre- and post-developed conditions

Variables	Pre-developed DA to Project Outlet			Post-developed DA to Project Outlet			
	DMA E1	DMA E2	DMA E3	DMA A100	DMA A200	DMA A300	DMA A400
<b>1</b> Rainfall Intensity for storm duration equal to time of concentration (in/hr)	1.19	0.78	0.951	0.88	1.21	0.87	0.83
<b>2</b> Drainage Area of each DMA (Acres)	43.21	35.49	18.63	23.75	18.70	42.12	3.75
<b>3</b> Ratio of pervious area to total area	1.0	1.0	1.0	0.574	0.4	0.442	0.484
<b>4</b> Peak Flow from DMA (cfs)	31.94	13.42	9.46	13.424	16.87	30.2	2.13
<b>5</b> Peak runoff from pre-developed condition confluence analysis (cfs): 54.82							
<b>6</b> Peak runoff from post-developed condition confluence analysis (cfs): See next table							
<b>7</b> Peak runoff reduction needed to meet HCOC Requirement (cfs): See next table							

### Form 4.2-5 HCOC Assessment for Peak Runoff (DA A)

Compute peak runoff for pre- and post-developed conditions

Variables	Pre-developed DA to Project Outlet			Post-developed DA to Project Outlet			
				DMA A500	DMA A600		
<b>1</b> Rainfall Intensity for storm duration equal to time of concentration				1.17	0.67		
<b>2</b> Drainage Area of each DMA (Acres)				1.28	10.13		
<b>3</b> Ratio of pervious area to total area				0.1	1.0		
<b>4</b> Peak Flow from DMA (cfs)				1.28	0.8		
<b>5</b> Peak runoff from pre-developed condition confluence analysis (cfs): 54.82							
<b>6</b> Peak runoff from post-developed condition confluence analysis (cfs): 7.5							
<b>7</b> Peak runoff reduction needed to meet HCOC Requirement (cfs): 0 $Q_{p-HCOC} = (Item\ 6 * 0.95) - Item\ 5$							

### Form 4.2-5 HCOC Assessment for Peak Runoff (DA B)

Compute peak runoff for pre- and post-developed conditions

Variables	Pre-developed DA to Project Outlet			Post-developed DA to Project Outlet			
	DMA E4	DMA E5	DMA E6	DMA B100	DMA B200	DMA B300	DMA B400
<b>1</b> Rainfall Intensity for storm duration equal to time of concentration	0.313	0.479	0.434	1.094	0.915	1.235	1.03
<b>2</b> Drainage Area of each DMA (Acres)	97.89	35.83	15.02	35.78	29.20	7.35	5.21
<b>3</b> Ratio of pervious area to total area	1.0	1.0	1.0	0.45	0.1	.2	0.1
<b>4</b> Peak Flow from DMA (cfs)	19.35	4.15	1.85	30.89	20.47	8.07	4.90
<b>5</b> Peak runoff from pre-developed condition confluence analysis (cfs): 19.35							
<b>6</b> Peak runoff from post-developed condition confluence analysis (cfs): See next table							
<b>7</b> Peak runoff reduction needed to meet HCOC Requirement (cfs): See next table							

### Form 4.2-5 HCOC Assessment for Peak Runoff (DA B)

Compute peak runoff for pre- and post-developed conditions

Variables	Pre-developed DA to Project Outlet			Post-developed DA to Project Outlet			
				DMA B701	DMA B702	DMA B703	
<b>1</b> Rainfall Intensity for storm duration equal to time of concentration				0.68	0.475	0.761	
<b>2</b> Drainage Area of each DMA (Acres)				7.90	29.26	23.62	
<b>3</b> Ratio of pervious area to total area				1.0	1.0	1.0	
<b>4</b> Peak Flow from DMA (cfs)				2.69	5.82		
<b>5</b> Peak runoff from pre-developed condition confluence analysis (cfs): 19.35							
<b>6</b> Peak runoff from post-developed condition confluence analysis (cfs): 57.95							
<b>7</b> Peak runoff reduction needed to meet HCOC Requirement (cfs): 38.6 $Q_{p-HCOC} = (Item\ 6 * 0.95) - Item\ 5$							

All hydrology calculations are provided in the stand alone Hydrology Report for this project.

## 4.3 Project Conformance Analysis

Complete the following forms for each project site DA to document that the proposed LID BMPs conform to the project DCV developed to meet performance criteria specified in the MS<sub>4</sub> Permit (WQMP Template Section 4.2). For the LID DCV, the forms are ordered according to hierarchy of BMP selection as required by the MS<sub>4</sub> Permit (see Section 5.3.1 in the TGD for WQMP). The forms compute the following for on-site LID BMP:

- Site Design and Hydrologic Source Controls (Form 4.3-2)
- Retention and Infiltration (Form 4.3-3)
- Harvested and Use (Form 4.3-4) or
- Biotreatment (Form 4.3-5).

At the end of each form, additional fields facilitate the determination of the extent of mitigation provided by the specific BMP category, allowing for use of the next category of BMP in the hierarchy, if necessary.

The first step in the analysis, using Section 5.3.2.1 of the TGD for WQMP, is to complete Forms 4.3-1 and 4.3-3) to determine if retention and infiltration BMPs are infeasible for the project. For each feasibility criterion in Form 4.3-1, if the answer is “Yes,” provide all study findings that includes relevant calculations, maps, data sources, etc. used to make the determination of infeasibility.

Next, complete Forms 4.3-2 and 4.3-4 to determine the feasibility of applicable HSC and harvest and use BMPs, and, if their implementation is feasible, the extent of mitigation of the DCV.

If no site constraints exist that would limit the type of BMP to be implemented in a DA, evaluate the use of combinations of LID BMPs, including all applicable HSC BMPs to maximize on-site retention of the DCV. If no combination of BMP can mitigate the entire DCV, implement the single BMP type, or combination of BMP types, that maximizes on-site retention of the DCV within the minimum effective area.

If the combination of LID HSC, retention and infiltration, and harvest and use BMPs are unable to mitigate the entire DCV, then biotreatment BMPs may be implemented by the project proponent. If biotreatment BMPs are used, then they must be sized to provide sufficient capacity for effective treatment of the remainder of the volume-based performance criteria that cannot be achieved with LID BMPs (TGD for WQMP Section 5.4.4.2).

**Under no circumstances shall any portion of the DCV be released from the site without effective mitigation and/or treatment.**

## Form 4.3-1 Infiltration BMP Feasibility (DA 1)

Feasibility Criterion – Complete evaluation for each DA on the Project Site

**1** Would infiltration BMP pose significant risk for groundwater related concerns? Yes  No

*Refer to Section 5.3.2.1 of the TGD for WQMP*

If Yes, Provide basis: (attach)

**2** Would installation of infiltration BMP significantly increase the risk of geotechnical hazards? Yes  No

(Yes, if the answer to any of the following questions is yes, as established by a geotechnical expert):

- The location is less than 50 feet away from slopes steeper than 15 percent
- The location is less than eight feet from building foundations or an alternative setback.
- A study certified by a geotechnical professional or an available watershed study determines that stormwater infiltration would result in significantly increased risks of geotechnical hazards.

If Yes, Provide basis: (attach)

**3** Would infiltration of runoff on a Project site violate downstream water rights? Yes  No

If Yes, Provide basis: (attach)

**4** Is proposed infiltration facility located on hydrologic soil group (HSG) D soils or does the site geotechnical investigation indicate presence of soil characteristics, which support categorization as D soils? Yes  No

If Yes, Provide basis: (attach)

**5** Is the design infiltration rate, after accounting for safety factor of 2.0, below proposed facility less than 0.3 in/hr (accounting for soil amendments)? Yes  No

If Yes, Provide basis: (attach)

**6** Would on-site infiltration or reduction of runoff over pre-developed conditions be partially or fully inconsistent with watershed management strategies as defined in the WAP, or impair beneficial uses? Yes  No

*See Section 3.5 of the TGD for WQMP and WAP*

If Yes, Provide basis: (attach)

**7** Any answer from Item 1 through Item 3 is “Yes”: Yes  No

*If yes, infiltration of any volume is not feasible onsite. Proceed to Form 4.3-4, Harvest and Use BMP. If no, then proceed to Item 8 below.*

**8** Any answer from Item 4 through Item 6 is “Yes”: Yes  No

*If yes, infiltration is permissible but is not required to be considered. Proceed to Form 4.3-2, Hydrologic Source Control BMP. If no, then proceed to Item 9, below.*

**9** All answers to Item 1 through Item 6 are “No”:

*Infiltration of the full DCV is potentially feasible, LID infiltration BMP must be designed to infiltrate the full DCV to the MEP. Proceed to Form 4.3-2, Hydrologic Source Control BMP.*

### 4.3.1 Site Design Hydrologic Source Control BMP

Section XI.E. of the Permit emphasizes the use of LID preventative measures; and the use of LID HSC BMPs reduces the portion of the DCV that must be addressed in downstream BMPs. Therefore, all applicable HSC shall be provided except where they are mutually exclusive with each other, or with other BMPs. Mutual exclusivity may result from overlapping BMP footprints such that either would be potentially feasible by itself, but both could not be implemented. Please note that while there are no numeric standards regarding the use of HSC, if a project cannot feasibly meet BMP sizing requirements or cannot fully address HCOCs, feasibility of all applicable HSC must be part of demonstrating that the BMP system has been designed to retain the maximum feasible portion of the DCV. Complete Form 4.3-2 to identify and calculate estimated retention volume from implementing site design HSC BMP. Refer to Section 5.4.1 in the TGD for more detailed guidance.

<b>Form 4.3-2 Site Design Hydrologic Source Control BMPs (DA 1)</b>			
<b>1</b> Implementation of Impervious Area Dispersion BMP (i.e. routing runoff from impervious to pervious areas), excluding impervious areas planned for routing to on-lot infiltration BMP: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, complete Items 2-5; If no, proceed to Item 6</i>	DA DMA BMP Type	DA DMA BMP Type	DA DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>2</b> Total impervious area draining to pervious area (ft <sup>2</sup> )			
<b>3</b> Ratio of pervious area receiving runoff to impervious area			
<b>4</b> Retention volume achieved from impervious area dispersion (ft <sup>3</sup> ) $V = \text{Item 2} * \text{Item 3} * (0.5/12)$ , assuming retention of 0.5 inches of runoff			
<b>5</b> Sum of retention volume achieved from impervious area dispersion (ft <sup>3</sup> ):		$V_{\text{retention}} = \text{Sum of Item 4 for all BMPs}$	
<b>6</b> Implementation of Localized On-lot Infiltration BMPs (e.g. on-lot rain gardens): Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, complete Items 7-13 for aggregate of all on-lot infiltration BMP in each DA; If no, proceed to Item 14</i>	DA DMA BMP Type	DA DMA BMP Type	DA DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>7</b> Ponding surface area (ft <sup>2</sup> )			
<b>8</b> Ponding depth (ft)			
<b>9</b> Surface area of amended soil/gravel (ft <sup>2</sup> )			
<b>10</b> Average depth of amended soil/gravel (ft)			
<b>11</b> Average porosity of amended soil/gravel			
<b>12</b> Retention volume achieved from on-lot infiltration (ft <sup>3</sup> ) $V_{\text{retention}} = (\text{Item 7} * \text{Item 8}) + (\text{Item 9} * \text{Item 10} * \text{Item 11})$			
<b>13</b> Runoff volume retention from on-lot infiltration (ft <sup>3</sup> ):		$V_{\text{retention}} = \text{Sum of Item 12 for all BMPs}$	

### Form 4.3-2 cont. Site Design Hydrologic Source Control BMPs (DA 1)

<b>14</b> Implementation of evapotranspiration BMP (green, brown, or blue roofs): Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, complete Items 15-20. If no, proceed to Item 21</i>	DA    DMA BMP Type	DA    DMA BMP Type	DA    DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>15</b> Rooftop area planned for ET BMP (ft <sup>2</sup> )			
<b>16</b> Average wet season ET demand (in/day) <i>Use local values, typical ~ 0.1</i>			
<b>17</b> Daily ET demand (ft <sup>3</sup> /day) <i>Item 15 * (Item 16 / 12)</i>			
<b>18</b> Drawdown time (hrs) <i>Copy Item 6 in Form 4.2-1</i>			
<b>19</b> Retention Volume (ft <sup>3</sup> ) <i>V<sub>retention</sub> = Item 17 * (Item 18 / 24)</i>			
<b>20</b> Runoff volume retention from evapotranspiration BMPs (ft <sup>3</sup> ): <span style="float: right;"><i>V<sub>retention</sub> = Sum of Item 19 for all BMPs</i></span>			
<b>21</b> Implementation of Street Trees: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, complete Items 22-25. If no, proceed to Item 26</i>	DA    DMA BMP Type	DA    DMA BMP Type	DA    DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>22</b> Number of Street Trees			
<b>23</b> Average canopy cover over impervious area (ft <sup>2</sup> )			
<b>24</b> Runoff volume retention from street trees (ft <sup>3</sup> ) <i>V<sub>retention</sub> = Item 22 * Item 23 * (0.05/12) assume runoff retention of 0.05 inches</i>			
<b>25</b> Runoff volume retention from street tree BMPs (ft <sup>3</sup> ): <span style="float: right;"><i>V<sub>retention</sub> = Sum of Item 24 for all BMPs</i></span>			
<b>26</b> Implementation of residential rain barrel/cisterns: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, complete Items 27-29; If no, proceed to Item 30</i>	DA    DMA BMP Type	DA    DMA BMP Type	DA    DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>27</b> Number of rain barrels/cisterns			
<b>28</b> Runoff volume retention from rain barrels/cisterns (ft <sup>3</sup> ) <i>V<sub>retention</sub> = Item 27 * 3</i>			
<b>29</b> Runoff volume retention from residential rain barrels/Cisterns (ft <sup>3</sup> ): <span style="float: right;"><i>V<sub>retention</sub> = Sum of Item 28 for all BMPs</i></span>			
<b>30</b> Total Retention Volume from Site Design Hydrologic Source Control BMPs:		<i>Sum of Items 5, 13, 20, 25 and 29</i>	

### 4.3.2 Infiltration BMPs

Use Form 4.3-3 to compute on-site retention of runoff from proposed retention and infiltration BMPs. Volume retention estimates are sensitive to the percolation rate used, which determines the amount of runoff that can be infiltrated within the specified drawdown time. The infiltration safety factor reduces field measured percolation to account for potential inaccuracy associated with field measurements, declining BMP performance over time, and compaction during construction. Appendix D of the TGD for WQMP provides guidance on estimating an appropriate safety factor to use in Form 4.3-3.

If site constraints limit the use of BMPs to a single type and implementation of retention and infiltration BMPs mitigate no more than 40% of the DCV, then they are considered infeasible and the Project Proponent may evaluate the effectiveness of BMPs lower in the LID hierarchy of use (Section 5.5.1 of the TGD for WQMP)

If implementation of infiltrations BMPs is feasible as determined using Form 4.3-1, then LID infiltration BMPs shall be implemented to the MEP (section 4.1 of the TGD for WQMP).

### Form 4.3-3 Infiltration LID BMP - including underground BMPs (DA A)

<b>1</b> Remaining LID DCV not met by site design HSC BMP (ft <sup>3</sup> ): 164,852 $V_{unmet} = \text{Form 4.2-1 Item 7} - \text{Form 4.3-2 Item 30}$			
BMP Type Use columns to the right to compute runoff volume retention from proposed infiltration BMP (select BMP from Table 5-4 in TGD for WQMP) - Use additional forms for more BMPs			
<b>2</b> Infiltration rate of underlying soils (in/hr) See Section 5.4.2 and Appendix D of the TGD for WQMP for minimum requirements for assessment methods			
<b>3</b> Infiltration safety factor See TGD Section 5.4.2 and Appendix D			
<b>4</b> Design percolation rate (in/hr) $P_{design} = \text{Item 2} / \text{Item 3}$			
<b>5</b> Pondered water drawdown time (hr) Copy Item 6 in Form 4.2-1			
<b>6</b> Maximum ponding depth (ft) BMP specific, see Table 5-4 of the TGD for WQMP for BMP design details			
<b>7</b> Ponding Depth (ft) $d_{BMP} = \text{Minimum of } (1/12 * \text{Item 4} * \text{Item 5}) \text{ or Item 6}$			
<b>8</b> Infiltrating surface area, $SA_{BMP}$ (ft <sup>2</sup> ) the lesser of the area needed for infiltration of full DCV or minimum space requirements from Table 5.7 of the TGD for WQMP			
<b>9</b> Amended soil depth, $d_{media}$ (ft) Only included in certain BMP types, see Table 5-4 in the TGD for WQMP for reference to BMP design details			
<b>10</b> Amended soil porosity			
<b>11</b> Gravel depth, $d_{media}$ (ft) Only included in certain BMP types, see Table 5-4 of the TGD for WQMP for BMP design details			
<b>12</b> Gravel porosity			
<b>13</b> Duration of storm as basin is filling (hrs) Typical ~ 3hrs	3	3	3
<b>14</b> Above Ground Retention Volume (ft <sup>3</sup> ) $V_{retention} = \text{Item 8} * [\text{Item 7} + (\text{Item 9} * \text{Item 10}) + (\text{Item 11} * \text{Item 12}) + (\text{Item 13} * (\text{Item 4} / 12))]$			
<b>15</b> Underground Retention Volume (ft <sup>3</sup> ) Volume determined using manufacturer's specifications and calculations			
<b>16</b> Total Retention Volume from LID Infiltration BMPs: (Sum of Items 14 and 15 for all infiltration BMP included in plan)			
<b>17</b> Fraction of DCV achieved with infiltration BMP: $\text{Retention}\% = \text{Item 16} / \text{Form 4.2-1 Item 7}$			
<b>18</b> Is full LID DCV retained on-site with combination of hydrologic source control and LID retention and infiltration BMPs? Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> If yes, demonstrate conformance using Form 4.3-10; If no, then reduce Item 3, Factor of Safety to 2.0 and increase Item 8, Infiltrating Surface Area, such that the portion of the site area used for retention and infiltration BMPs equals or exceeds the minimum effective area thresholds (Table 5-7 of the TGD for WQMP) for the applicable category of development and repeat all above calculations.			

### Form 4.3-3 Infiltration LID BMP - including underground BMPs (DA B)

<b>1</b> Remaining LID DCV not met by site design HSC BMP (ft <sup>3</sup> ): 213,134 $V_{unmet} = \text{Form 4.2-1 Item 7} - \text{Form 4.3-2 Item 30}$			
<b>BMP Type</b> Use columns to the right to compute runoff volume retention from proposed infiltration BMP (select BMP from Table 5-4 in TGD for WQMP) - Use additional forms for more BMPs			
<b>2</b> Infiltration rate of underlying soils (in/hr) See Section 5.4.2 and Appendix D of the TGD for WQMP for minimum requirements for assessment methods			
<b>3</b> Infiltration safety factor See TGD Section 5.4.2 and Appendix D			
<b>4</b> Design percolation rate (in/hr) $P_{design} = \text{Item 2} / \text{Item 3}$			
<b>5</b> Ponded water drawdown time (hr) Copy Item 6 in Form 4.2-1			
<b>6</b> Maximum ponding depth (ft) BMP specific, see Table 5-4 of the TGD for WQMP for BMP design details			
<b>7</b> Ponding Depth (ft) $d_{BMP} = \text{Minimum of } (1/12 * \text{Item 4} * \text{Item 5}) \text{ or Item 6}$			
<b>8</b> Infiltrating surface area, $SA_{BMP}$ (ft <sup>2</sup> ) the lesser of the area needed for infiltration of full DCV or minimum space requirements from Table 5.7 of the TGD for WQMP			
<b>9</b> Amended soil depth, $d_{media}$ (ft) Only included in certain BMP types, see Table 5-4 in the TGD for WQMP for reference to BMP design details			
<b>10</b> Amended soil porosity			
<b>11</b> Gravel depth, $d_{media}$ (ft) Only included in certain BMP types, see Table 5-4 of the TGD for WQMP for BMP design details			
<b>12</b> Gravel porosity			
<b>13</b> Duration of storm as basin is filling (hrs) Typical ~ 3hrs			
<b>14</b> Above Ground Retention Volume (ft <sup>3</sup> ) $V_{retention} = \text{Item 8} * [\text{Item 7} + (\text{Item 9} * \text{Item 10}) + (\text{Item 11} * \text{Item 12}) + (\text{Item 13} * (\text{Item 4} / 12))]$			
<b>15</b> Underground Retention Volume (ft <sup>3</sup> ) Volume determined using manufacturer's specifications and calculations			
<b>16</b> Total Retention Volume from LID Infiltration BMPs: (Sum of Items 14 and 15 for all infiltration BMP included in plan)			
<b>17</b> Fraction of DCV achieved with infiltration BMP: $\text{Retention\%} = \text{Item 16} / \text{Form 4.2-1 Item 7}$			
<b>18</b> Is full LID DCV retained on-site with combination of hydrologic source control and LID retention and infiltration BMPs? Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> If yes, demonstrate conformance using Form 4.3-10; If no, then reduce Item 3, Factor of Safety to 2.0 and increase Item 8, Infiltrating Surface Area, such that the portion of the site area used for retention and infiltration BMPs equals or exceeds the minimum effective area thresholds (Table 5-7 of the TGD for WQMP) for the applicable category of development and repeat all above calculations.			

### 4.3.3 Harvest and Use BMP

Harvest and use BMP may be considered if the full LID DCV cannot be met by maximizing infiltration BMPs. Use Form 4.3-4 to compute on-site retention of runoff from proposed harvest and use BMPs.

Volume retention estimates for harvest and use BMPs are sensitive to the on-site demand for captured stormwater. Since irrigation water demand is low in the wet season, when most rainfall events occur in San Bernardino County, the volume of water that can be used within a specified drawdown period is relatively low. The bottom portion of Form 4.3-4 facilitates the necessary computations to show infeasibility if a minimum incremental benefit of 40 percent of the LID DCV would not be achievable with MEP implementation of on-site harvest and use of stormwater (Section 5.5.4 of the TGD for WQMP).

<b>Form 4.3-4 Harvest and Use BMPs (DA 1)</b>			
<b>1</b> Remaining LID DCV not met by site design HSC or infiltration BMP (ft <sup>3</sup> ): <i>V<sub>unmet</sub> = Form 4.2-1 Item 7 - Form 4.3-2 Item 30 - Form 4.3-3 Item 16</i>			
BMP Type(s) <i>Compute runoff volume retention from proposed harvest and use BMP (Select BMPs from Table 5-4 of the TGD for WQMP) - Use additional forms for more BMPs</i>	DA BMP Type	DMA BMP Type	DA DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>2</b> Describe cistern or runoff detention facility			
<b>3</b> Storage volume for proposed detention type (ft <sup>3</sup> ) <i>Volume of cistern</i>			
<b>4</b> Landscaped area planned for use of harvested stormwater (ft <sup>2</sup> )			
<b>5</b> Average wet season daily irrigation demand (in/day) <i>Use local values, typical ~ 0.1 in/day</i>			
<b>6</b> Daily water demand (ft <sup>3</sup> /day) <i>Item 4 * (Item 5 / 12)</i>			
<b>7</b> Drawdown time (hrs) <i>Copy Item 6 from Form 4.2-1</i>			
<b>8</b> Retention Volume (ft <sup>3</sup> ) <i>V<sub>retention</sub> = Minimum of (Item 3) or (Item 6 * (Item 7 / 24))</i>			
<b>9</b> Total Retention Volume (ft <sup>3</sup> ) from Harvest and Use BMP <span style="float: right;"><i>Sum of Item 8 for all harvest and use BMP included in plan</i></span>			
<b>10</b> Is the full DCV retained with a combination of LID HSC, retention and infiltration, and harvest and use BMPs? Yes <input type="checkbox"/> No <input type="checkbox"/> <i>If yes, demonstrate conformance using Form 4.3-10. If no, then re-evaluate combinations of all LID BMP and optimize their implementation such that the maximum portion of the DCV is retained on-site (using a single BMP type or combination of BMP types). If the full DCV cannot be mitigated after this optimization process, proceed to Section 4.3.4.</i>			

### 4.3.4 Biotreatment BMP

Biotreatment BMPs may be considered if the full LID DCV cannot be met by maximizing retention and infiltration, and harvest and use BMPs. A key consideration when using biotreatment BMP is the effectiveness of the proposed BMP in addressing the pollutants of concern for the project (see Table 5-5 of the TGD for WQMP).

Use Form 4.3-5 to summarize the potential for volume based and/or flow based biotreatment options to biotreat the remaining unmet LID DCV w. Biotreatment computations are included as follows:

- Use Form 4.3-6 to compute biotreatment in small volume based biotreatment BMP (e.g. bioretention w/underdrains);
- Use Form 4.3-7 to compute biotreatment in large volume based biotreatment BMP (e.g. constructed wetlands);
- Use Form 4.3-8 to compute sizing criteria for flow-based biotreatment BMP (e.g. bioswales)

<b>Form 4.3-5 Selection and Evaluation of Biotreatment BMP (DA A)</b>		
<p><b>1</b> Remaining LID DCV not met by site design HSC, infiltration, or harvest and use BMP for potential biotreatment (ft<sup>3</sup>): 165,236 <i>Form 4.2-1 Item 7 - Form 4.3-2 Item 30 – Form 4.3-3 Item 16- Form 4.3-4 Item 9</i></p>	<p>List pollutants of concern <i>Copy from Form 2.3-1.</i> Pathogens, Nutrients, Noxious Aquatic Plants, Sediment, Metals, Oil and Grease, Trash and Debris, Pesticides and Herbicides, and Organic Compounds</p>	
<p><b>2</b> Biotreatment BMP Selected <i>(Select biotreatment BMP(s) necessary to ensure all pollutants of concern are addressed through Unit Operations and Processes, described in Table 5-5 of the TGD for WQMP)</i></p>	<p style="text-align: center;">Volume-based biotreatment <i>Use Forms 4.3-6 and 4.3-7 to compute treated volume</i></p> <p><input checked="" type="checkbox"/> Bioretention with underdrain <input type="checkbox"/> Planter box with underdrain <input type="checkbox"/> Constructed wetlands <input type="checkbox"/> Wet extended detention <input type="checkbox"/> Dry extended detention</p>	<p style="text-align: center;">Flow-based biotreatment <i>Use Form 4.3-8 to compute treated volume</i></p> <p><input type="checkbox"/> Vegetated swale <input type="checkbox"/> Vegetated filter strip <input type="checkbox"/> Proprietary biotreatment</p>
<p><b>3</b> Volume biotreated in volume based biotreatment BMP (ft<sup>3</sup>): 271,263 <i>Form 4.3-6 Item 15 + Form 4.3-7 Item 13</i></p>	<p><b>4</b> Compute remaining LID DCV with implementation of volume based biotreatment BMP (ft<sup>3</sup>): -106,027 <i>Item 1 – Item 3</i></p>	<p><b>5</b> Remaining fraction of LID DCV for sizing flow based biotreatment BMP: 0% <i>Item 4 / Item 1</i></p>
<p><b>6</b> Flow-based biotreatment BMP capacity provided (cfs): NA <i>Use Figure 5-2 of the TGD for WQMP to determine flow capacity required to provide biotreatment of remaining percentage of unmet LID DCV (Item 5), for the project’s precipitation zone (Form 3-1 Item 1)</i></p>		
<p><b>7</b> Metrics for MEP determination:</p> <ul style="list-style-type: none"> <li>• Provided a WQMP with the portion of site area used for suite of LID BMP equal to minimum thresholds in Table 5-7 of the TGD for WQMP for the proposed category of development: <input type="checkbox"/> <i>If maximized on-site retention BMPs is feasible for partial capture, then LID BMP implementation must be optimized to retain and infiltrate the maximum portion of the DCV possible within the prescribed minimum effective area. The remaining portion of the DCV shall then be mitigated using biotreatment BMP.</i></li> </ul>		

<b>Form 4.3-5 Selection and Evaluation of Biotreatment BMP (DA B)</b>		
<p><b>1</b> Remaining LID DCV not met by site design HSC, infiltration, or harvest and use BMP for potential biotreatment (ft<sup>3</sup>): 213,134 <i>Form 4.2-1 Item 7 - Form 4.3-2 Item 30 – Form 4.3-3 Item 16- Form 4.3-4 Item 9</i></p>	<p>List pollutants of concern <i>Copy from Form 2.3-1.</i>                      Pathogens, Nutrients, Noxious Aquatic Plants, Sediment, Metals, Oil and Grease, Trash and Debris, Pesticides and Herbicides, and Organic Compounds</p>	
<p><b>2</b> Biotreatment BMP Selected                       (Select biotreatment BMP(s) necessary to ensure all pollutants of concern are addressed through Unit Operations and Processes, described in Table 5-5 of the TGD for WQMP)</p>	<p style="text-align: center;">Volume-based biotreatment <i>Use Forms 4.3-6 and 4.3-7 to compute treated volume</i></p> <p><input checked="" type="checkbox"/> Bioretention with underdrain  <input type="checkbox"/> Planter box with underdrain  <input type="checkbox"/> Constructed wetlands  <input type="checkbox"/> Wet extended detention  <input type="checkbox"/> Dry extended detention</p>	<p style="text-align: center;">Flow-based biotreatment <i>Use Form 4.3-8 to compute treated volume</i></p> <p><input type="checkbox"/> Vegetated swale  <input type="checkbox"/> Vegetated filter strip  <input type="checkbox"/> Proprietary biotreatment</p>
<p><b>3</b> Volume biotreated in volume based biotreatment BMP (ft<sup>3</sup>): 349,895 <i>Form 4.3-6 Item 15 + Form 4.3-7 Item 13</i></p>	<p><b>4</b> Compute remaining LID DCV with implementation of volume based biotreatment BMP (ft<sup>3</sup>): -136,761 <i>Item 1 – Item 3</i></p>	<p><b>5</b> Remaining fraction of LID DCV for sizing flow based biotreatment BMP: 0% <i>Item 4 / Item 1</i></p>
<p><b>6</b> Flow-based biotreatment BMP capacity provided (cfs): NA <i>Use Figure 5-2 of the TGD for WQMP to determine flow capacity required to provide biotreatment of remaining percentage of unmet LID DCV (Item 5), for the project’s precipitation zone (Form 3-1 Item 1)</i></p>		
<p><b>7</b> Metrics for MEP determination:</p> <ul style="list-style-type: none"> <li>• Provided a WQMP with the portion of site area used for suite of LID BMP equal to minimum thresholds in Table 5-7 of the TGD for WQMP for the proposed category of development: <input type="checkbox"/> <i>If maximized on-site retention BMPs is feasible for partial capture, then LID BMP implementation must be optimized to retain and infiltrate the maximum portion of the DCV possible within the prescribed minimum effective area. The remaining portion of the DCV shall then be mitigated using biotreatment BMP.</i></li> </ul>		

<b>Form 4.3-6 Volume Based Biotreatment (DA A) – Bioretention and Planter Boxes with Underdrains</b>			
Biotreatment BMP Type <i>(All basins are bioretention w/underdrain and include a conservative amended soil infiltration rate for the Preliminary WQMP. It is anticipated that basins will have an internal water storage zone for additional nutrient removal.)</i>	DA A DMA A100 and A200 Basin A-1	DA A DMA A300 Basin A-2	DA A DMA A400 and A500 Basin A-3
<b>1</b> Pollutants addressed with BMP <i>List all pollutant of concern that will be effectively reduced through specific Unit Operations and Processes described in Table 5-5 of the TGD for WQMP</i>	Pathogens, Nutrients, Sediment, Metals, Oil and Grease, Trash and Debris, Pesticides and Herbicides, and Organic Compounds		
<b>2</b> Amended soil infiltration rate <i>Typical ~ 5.0</i>	2.5	2.5	2.5
<b>3</b> Amended soil infiltration safety factor <i>Typical ~ 2.0</i>	2	2	2
<b>4</b> Amended soil design percolation rate (in/hr) $P_{design} = \text{Item 2} / \text{Item 3}$	1.25	1.25	1.25
<b>5</b> Ponded water drawdown time (hr) <i>Copy Item 6 from Form 4.2-1</i>	48	48	48
<b>6</b> Maximum ponding depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	5	5	5
<b>7</b> Ponding Depth (ft) $d_{BMP} = \text{Minimum of } (1/12 * \text{Item 4} * \text{Item 5}) \text{ or Item 6}$	1.5	1.5	1.5
<b>8</b> Amended soil surface area (ft <sup>2</sup> )	47,927	56,142	6,089
<b>9</b> Amended soil depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	2	2	2
<b>10</b> Amended soil porosity, <i>n</i>	0.35	0.35	0.35
<b>11</b> Gravel depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	2	2	2
<b>12</b> Gravel porosity, <i>n</i>	0.35	0.35	0.35
<b>13</b> Duration of storm as basin is filling (hrs) <i>Typical ~ 3hrs</i>	3	3	3
<b>14</b> Biotreated Volume (ft <sup>3</sup> ) $V_{biotreated} = \text{Item 8} * [( \text{Item 7} / 2 ) + ( \text{Item 9} * \text{Item 10} ) + ( \text{Item 11} * \text{Item 12} ) + ( \text{Item 13} * ( \text{Item 4} / 12 ) )]$	118,020	138,249	14,994
<b>15</b> Total biotreated volume from bioretention and/or planter box with underdrains BMP: 271,263 cf <i>Sum of Item 14 for all volume-based BMPs included in this form</i>			

<b>Form 4.3-6 Volume Based Biotreatment (DA B) – Bioretention and Planter Boxes with Underdrains</b>			
Biotreatment BMP Type <i>(All basins are bioretention w/underdrain and include a conservative amended soil infiltration rate for the Preliminary WQMP. It is anticipated that basins will have an internal water storage zone for additional nutrient removal.)</i>	DA B DMA B100 Basin B-1	DA B DMA B200, B300, and B400 Basin B-2	
<b>1</b> Pollutants addressed with BMP <i>List all pollutant of concern that will be effectively reduced through specific Unit Operations and Processes described in Table 5-5 of the TGD for WQMP</i>	Pathogens, Nutrients, Sediment, Metals, Oil and Grease, Trash and Debris, Pesticides and Herbicides, and Organic Compounds		
<b>2</b> Amended soil infiltration rate <i>Typical ~ 5.0</i>	2.5	2.5	
<b>3</b> Amended soil infiltration safety factor <i>Typical ~ 2.0</i>	2	2	
<b>4</b> Amended soil design percolation rate (in/hr) $P_{design} = \text{Item 2} / \text{Item 3}$	1.25	1.25	
<b>5</b> Ponded water drawdown time (hr) <i>Copy Item 6 from Form 4.2-1</i>	48	48	
<b>6</b> Maximum ponding depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	5	5	
<b>7</b> Ponding Depth (ft) $d_{BMP} = \text{Minimum of } (1/12 * \text{Item 4} * \text{Item 5}) \text{ or Item 6}$	1.5	1.5	
<b>8</b> Amended soil surface area (ft <sup>2</sup> )	50,579	91,510	
<b>9</b> Amended soil depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	2	2	
<b>10</b> Amended soil porosity, <i>n</i>	0.35	0.35	
<b>11</b> Gravel depth (ft) <i>see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>	2	2	
<b>12</b> Gravel porosity, <i>n</i>	0.35	0.35	
<b>13</b> Duration of storm as basin is filling (hrs) <i>Typical ~ 3hrs</i>	3	3	
<b>14</b> Biotreated Volume (ft <sup>3</sup> ) $V_{biotreated} = \text{Item 8} * [(\text{Item 7}/2) + (\text{Item 9} * \text{Item 10}) + (\text{Item 11} * \text{Item 12}) + (\text{Item 13} * (\text{Item 4} / 12))]$	124,550	225,344	
<b>15</b> Total biotreated volume from bioretention and/or planter box with underdrains BMP: 349,895 cf <i>Sum of Item 14 for all volume-based BMPs included in this form</i>			

## Form 4.3-7 Volume Based Biotreatment (DA 1) – Constructed Wetlands and Extended Detention

Biotreatment BMP Type <i>Constructed wetlands, extended wet detention, extended dry detention, or other comparable proprietary BMP. If BMP includes multiple modules (e.g. forebay and main basin), provide separate estimates for storage and pollutants treated in each module.</i>	DA    DMA BMP Type		DA    DMA BMP Type <i>(Use additional forms for more BMPs)</i>	
	Forebay	Basin	Forebay	Basin
<b>1</b> Pollutants addressed with BMP forebay and basin <i>List all pollutant of concern that will be effectively reduced through specific Unit Operations and Processes described in Table 5-5 of the TGD for WQMP</i>				
<b>2</b> Bottom width (ft)				
<b>3</b> Bottom length (ft)				
<b>4</b> Bottom area (ft <sup>2</sup> ) <i>A<sub>bottom</sub> = Item 2 * Item 3</i>				
<b>5</b> Side slope (ft/ft)				
<b>6</b> Depth of storage (ft)				
<b>7</b> Water surface area (ft <sup>2</sup> ) <i>A<sub>surface</sub> = (Item 2 + (2 * Item 5 * Item 6)) * (Item 3 + (2 * Item 5 * Item 6))</i>				
<b>8</b> Storage volume (ft <sup>3</sup> ) <i>For BMP with a forebay, ensure fraction of total storage is within ranges specified in BMP specific fact sheets, see Table 5-6 of the TGD for WQMP for reference to BMP design details</i> <i>V = Item 6 / 3 * [Item 4 + Item 7 + (Item 4 * Item 7)<sup>0.5</sup>]</i>				
<b>9</b> Drawdown Time (hrs) <i>Copy Item 6 from Form 2.1</i>				
<b>10</b> Outflow rate (cfs) <i>Q<sub>BMP</sub> = (Item 8<sub>forebay</sub> + Item 8<sub>basin</sub>) / (Item 9 * 3600)</i>				
<b>11</b> Duration of design storm event (hrs)				
<b>12</b> Biotreated Volume (ft <sup>3</sup> ) <i>V<sub>biotreated</sub> = (Item 8<sub>forebay</sub> + Item 8<sub>basin</sub>) + (Item 10 * Item 11 * 3600)</i>				
<b>13</b> Total biotreated volume from constructed wetlands, extended dry detention, or extended wet detention : <i>(Sum of Item 12 for all BMP included in plan)</i>				

<b>Form 4.3-8 Flow Based Biotreatment (DA 1)</b>			
Biotreatment BMP Type <i>Vegetated swale, vegetated filter strip, or other comparable proprietary BMP</i>	DA    DMA BMP Type	DA    DMA BMP Type	DA    DMA BMP Type <i>(Use additional forms for more BMPs)</i>
<b>1</b> Pollutants addressed with BMP <i>List all pollutant of concern that will be effectively reduced through specific Unit Operations and Processes described in TGD Table 5-5</i>			
<b>2</b> Flow depth for water quality treatment (ft) <i>BMP specific, see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>			
<b>3</b> Bed slope (ft/ft) <i>BMP specific, see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>			
<b>4</b> Manning's roughness coefficient			
<b>5</b> Bottom width (ft) <i><math>b_w = (\text{Form 4.3-5 Item 6} * \text{Item 4}) / (1.49 * \text{Item 2}^{1.67} * \text{Item 3}^{0.5})</math></i>			
<b>6</b> Side Slope (ft/ft) <i>BMP specific, see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>			
<b>7</b> Cross sectional area (ft <sup>2</sup> ) <i><math>A = (\text{Item 5} * \text{Item 2}) + (\text{Item 6} * \text{Item 2}^2)</math></i>			
<b>8</b> Water quality flow velocity (ft/sec) <i><math>V = \text{Form 4.3-5 Item 6} / \text{Item 7}</math></i>			
<b>9</b> Hydraulic residence time (min) <i>Pollutant specific, see Table 5-6 of the TGD for WQMP for reference to BMP design details</i>			
<b>10</b> Length of flow based BMP (ft) <i><math>L = \text{Item 8} * \text{Item 9} * 60</math></i>			
<b>11</b> Water surface area at water quality flow depth (ft <sup>2</sup> ) <i><math>SA_{top} = (\text{Item 5} + (2 * \text{Item 2} * \text{Item 6})) * \text{Item 10}</math></i>			

### 4.3.5 Conformance Summary

Complete Form 4.3-9 to demonstrate how on-site LID DCV is met with proposed site design hydrologic source control, infiltration, harvest and use, and/or biotreatment BMP. The bottom line of the form is used to describe the basis for infeasibility determination for on-site LID BMP to achieve full LID DCV, and provides methods for computing remaining volume to be addressed in an alternative compliance plan. If the project has more than one outlet, then complete additional versions of this form for each outlet.

Form 4.3-9 Conformance Summary and Alternative Compliance Volume Estimate	
<b>1</b>	Total LID DCV for the Project (ft <sup>3</sup> ): 378,370 <i>Copy Item 7 in Form 4.2-1</i>
<b>2</b>	On-site retention with site design hydrologic source control LID BMP (ft <sup>3</sup> ): NA <i>Copy Item 30 in Form 4.3-2</i>
<b>3</b>	On-site retention with LID infiltration BMP (ft <sup>3</sup> ): NA <i>Copy Item 16 in Form 4.3-3</i>
<b>4</b>	On-site retention with LID harvest and use BMP (ft <sup>3</sup> ): NA <i>Copy Item 9 in Form 4.3-4</i>
<b>5</b>	On-site biotreatment with volume based biotreatment BMP (ft <sup>3</sup> ): 621,158 <i>Copy Item 3 in Form 4.3-5</i>
<b>6</b>	Flow capacity provided by flow based biotreatment BMP (cfs): NA <i>Copy Item 6 in Form 4.3-5</i>
<b>7</b>	<p>LID BMP performance criteria are achieved if answer to any of the following is "Yes":</p> <ul style="list-style-type: none"> <li>• Full retention of LID DCV with site design HSC, infiltration, or harvest and use BMP: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, sum of Items 2, 3, and 4 is greater than Item 1</i></li> <li>• Combination of on-site retention BMPs for a portion of the LID DCV and volume-based biotreatment BMP that address all pollutants of concern for the remaining LID DCV: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/> <i>If yes, a) sum of Items 2, 3, 4, and 5 is greater than Item 1, and Items 2, 3 and 4 are maximized; or b) Item 6 is greater than Form 4.3-5 Item 6 and Items 2, 3 and 4 are maximized</i></li> <li>▪ On-site retention and infiltration is determined to be infeasible and biotreatment BMP provide biotreatment for all pollutants of concern for full LID DCV: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/> <i>If yes, Form 4.3-1 Items 7 and 8 were both checked yes</i></li> </ul>
<b>8</b>	<p>If the LID DCV is not achieved by any of these means, then the project may be allowed to develop an alternative compliance plan. Check box that describes the scenario which caused the need for alternative compliance:</p> <ul style="list-style-type: none"> <li>• Combination of HSC, retention and infiltration, harvest and use, and biotreatment BMPs provide less than full LID DCV capture: <input type="checkbox"/> <i>Checked yes for Form 4.3-5 Item 7, Item 6 is zero, and sum of Items 2, 3, 4, and 5 is less than Item 1. If so, apply water quality credits and calculate volume for alternative compliance, <math>V_{alt} = (Item\ 1 - Item\ 2 - Item\ 3 - Item\ 4 - Item\ 5) * (100 - Form\ 2.4-1\ Item\ 2)\%</math></i></li> <li>• An approved Watershed Action Plan (WAP) demonstrates that water quality and hydrologic impacts of urbanization are more effective when managed in at an off-site facility: <input type="checkbox"/> <i>Attach appropriate WAP section, including technical documentation, showing effectiveness comparisons for the project site and regional watershed</i></li> </ul>

### 4.3.6 Hydromodification Control BMP

Use Form 4.3-10 to compute the remaining runoff volume retention, after LID BMP are implemented, needed to address HCOC, and the increase in time of concentration and decrease in peak runoff necessary to meet targets for protection of waterbodies with a potential HCOC. Describe hydromodification control BMP that address HCOC, which may include off-site BMP and/or in-stream controls. Section 5.6 of the TGD for WQMP provides additional details on selection and evaluation of hydromodification control BMP.

<b>Form 4.3-10 Hydromodification Control BMPs (DA A)</b>	
<p><b>1</b> Volume reduction needed for HCOC performance criteria (ft<sup>3</sup>): 35,668 (Form 4.2-2 Item 4 * 0.95) – Form 4.2-2 Item 1</p>	<p><b>2</b> On-site retention with site design hydrologic source control, infiltration, and harvest and use LID BMP (ft<sup>3</sup>): 0 <i>Sum of Form 4.3-9 Items 2, 3, and 4 Evaluate option to increase implementation of on-site retention in Forms 4.3-2, 4.3-3, and 4.3-4 in excess of LID DCV toward achieving HCOC volume reduction</i></p>
<p><b>3</b> Remaining volume for HCOC volume capture (ft<sup>3</sup>): 35,668 <i>Item 1 – Item 2</i></p>	<p><b>4</b> Volume capture provided by incorporating additional on-site or off-site retention BMPs (ft<sup>3</sup>): 0 <i>Existing downstream BMP may be used to demonstrate additional volume capture (if so, attach to this WQMP a hydrologic analysis showing how the additional volume would be retained during a 2-yr storm event for the regional watershed)</i></p>
<p><b>5</b> If Item 4 is less than Item 3, incorporate in-stream controls on downstream waterbody segment to prevent impacts due to hydromodification <input checked="" type="checkbox"/> SEE DISCUSSION BELOW <i>Attach in-stream control BMP selection and evaluation to this WQMP</i></p>	
<p><b>6</b> Is Form 4.2-2 Item 11 less than or equal to 5%: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>  <i>If yes, HCOC performance criteria is achieved. If no, select one or more mitigation options below:</i></p> <ul style="list-style-type: none"> <li>• Demonstrate increase in time of concentration achieved by proposed LID site design, LID BMP, and additional on-site or off-site retention BMP <input type="checkbox"/>  <i>BMP upstream of a waterbody segment with a potential HCOC may be used to demonstrate increased time of concentration through hydrograph attenuation (if so, show that the hydraulic residence time provided in BMP for a 2-year storm event is equal or greater than the addition time of concentration requirement in Form 4.2-4 Item 15)</i></li> <li>• Increase time of concentration by preserving pre-developed flow path and/or increase travel time by reducing slope and increasing cross-sectional area and roughness for proposed on-site conveyance facilities <input type="checkbox"/></li> <li>• Incorporate appropriate in-stream controls for downstream waterbody segment to prevent impacts due to hydromodification, in a plan approved and signed by a licensed engineer in the State of California <input type="checkbox"/></li> </ul>	
<p><b>7</b> Form 4.2-2 Item 12 less than or equal to 5%: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>  <i>If yes, HCOC performance criteria is achieved. If no, select one or more mitigation options below:</i></p> <ul style="list-style-type: none"> <li>• Demonstrate reduction in peak runoff achieved by proposed LID site design, LID BMPs, and additional on-site or off-site retention BMPs <input type="checkbox"/>  <i>BMPs upstream of a waterbody segment with a potential HCOC may be used to demonstrate additional peak runoff reduction through hydrograph attenuation (if so, attach to this WQMP, a hydrograph analysis showing how the peak runoff would be reduced during a 2-yr storm event)</i></li> <li>• Incorporate appropriate in-stream controls for downstream waterbody segment to prevent impacts due to hydromodification, in a plan approved and signed by a licensed engineer in the State of California <input type="checkbox"/></li> </ul>	

Drainage Area A ultimately drains to Basin A which detains 8,917,723 cf. This volume is slowly released through an outlet riser to drain within 96 hours. Due to soil constraints the HCOC volume condition is not able to be met however the flow rate and time of concentration HCOC conditions are being met and mitigating for HCOC concerns for the post-developed conditions. Basin A has a total volume of 8,917,723 cf and will detain the entire

## Preliminary Water Quality Management Plan (PWQMP)

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100 year storm volume and release it downstream through a low flow orifice riser within 96 hours. The following table has been inserted to show the average flow rate from the orifice at different water surface elevations. The total 100 year volume draining to Basin A is 2,141,250 cf and is 5' deep. In order to drain in 96 hours an average flow rate of 6.2 cfs is needed. As shown in the table below the flow rates up to a depth of 1.5' exceed 6.2 cfs and will not have any issue draining in 96 hours. During final design a stage storage curve will be included to show the exact time of draw down for the basin.

$$\text{Orifice flow} = CA(2gh)^{1/2}$$

$$\text{Orifice } C = 0.66$$

$$\text{Pipe Dia (in)} = 36''$$

$$\text{Orifice } \text{1 (inch)} = 16'' \text{ at bottom of basin}$$

Stage	Discharge
Head (Feet)	Outlet 1 (cfs)
0	0.00
1	3.70
1.5	6.40
2	8.27
3	11.09
3.5	12.26
4	13.33
4.5	14.32
5	15.25

<b>Form 4.3-10 Hydromodification Control BMPs (DA B)</b>	
<p><b>1</b> Volume reduction needed for HCOC performance criteria (ft<sup>3</sup>): 7,937 (Form 4.2-2 Item 4 * 0.95) – Form 4.2-2 Item 1</p>	<p><b>2</b> On-site retention with site design hydrologic source control, infiltration, and harvest and use LID BMP (ft<sup>3</sup>): 0 <i>Sum of Form 4.3-9 Items 2, 3, and 4 Evaluate option to increase implementation of on-site retention in Forms 4.3-2, 4.3-3, and 4.3-4 in excess of LID DCV toward achieving HCOC volume reduction</i></p>
<p><b>3</b> Remaining volume for HCOC volume capture (ft<sup>3</sup>): 0</p>	<p><b>4</b> Volume capture provided by incorporating additional on-site or off-site retention BMPs (ft<sup>3</sup>): 0 <i>Existing downstream BMP may be used to demonstrate additional volume capture (if so, attach to this WQMP a hydrologic analysis showing how the additional volume would be retained during a 2-yr storm event for the regional watershed)</i></p>
<p><b>5</b> If Item 4 is less than Item 3, incorporate in-stream controls on downstream waterbody segment to prevent impacts due to hydromodification <input checked="" type="checkbox"/> SEE DISCUSSION BELOW <i>Attach in-stream control BMP selection and evaluation to this WQMP</i></p>	
<p><b>6</b> Is Form 4.2-2 Item 11 less than or equal to 5%: Yes <input checked="" type="checkbox"/> No <input type="checkbox"/>  <i>If yes, HCOC performance criteria is achieved. If no, select one or more mitigation options below:</i></p> <ul style="list-style-type: none"> <li>• Demonstrate increase in time of concentration achieved by proposed LID site design, LID BMP, and additional on-site or off-site retention BMP <input type="checkbox"/>  <i>BMP upstream of a waterbody segment with a potential HCOC may be used to demonstrate increased time of concentration through hydrograph attenuation (if so, show that the hydraulic residence time provided in BMP for a 2-year storm event is equal or greater than the addition time of concentration requirement in Form 4.2-4 Item 15)</i></li> <li>• Increase time of concentration by preserving pre-developed flow path and/or increase travel time by reducing slope and increasing cross-sectional area and roughness for proposed on-site conveyance facilities <input type="checkbox"/></li> <li>• Incorporate appropriate in-stream controls for downstream waterbody segment to prevent impacts due to hydromodification, in a plan approved and signed by a licensed engineer in the State of California <input type="checkbox"/></li> </ul>	
<p><b>7</b> Form 4.2-2 Item 12 less than or equal to 5%: Yes <input type="checkbox"/> No <input checked="" type="checkbox"/>  <i>If yes, HCOC performance criteria is achieved. If no, select one or more mitigation options below:</i></p> <ul style="list-style-type: none"> <li>• Demonstrate reduction in peak runoff achieved by proposed LID site design, LID BMPs, and additional on-site or off-site retention BMPs <input checked="" type="checkbox"/> SEE DISCUSSION BELOW  <i>BMPs upstream of a waterbody segment with a potential HCOC may be used to demonstrate additional peak runoff reduction through hydrograph attenuation (if so, attach to this WQMP, a hydrograph analysis showing how the peak runoff would be reduced during a 2-yr storm event)</i></li> <li>• Incorporate appropriate in-stream controls for downstream waterbody segment to prevent impacts due to hydromodification, in a plan approved and signed by a licensed engineer in the State of California <input type="checkbox"/></li> </ul>	

Drainage area B has been designed to treat the DCV, mitigate hydromodification impacts, and attenuate peak runoff through the incorporation of Basin B-1 and B-2. These basins reduce the 2-year peak runoff to be within 5.9% of the existing condition runoff. This exceeds the allowable increase by 0.9%. This will be addressed by incorporating LID BMPs throughout drainage area B. These BMPs will be addressed in detail in the Final Project WQMP. All hydrology calculations are provided in the stand alone Hydrology Report for this project.

## 4.4 Alternative Compliance Plan (if applicable)

Describe an alternative compliance plan (if applicable) for projects not fully able to infiltrate, harvest and use, or biotreat the DCV via on-site LID practices. A project proponent must develop an alternative compliance plan to address the remainder of the LID DCV. Depending on project type some projects may qualify for water quality credits that can be applied to reduce the DCV that must be treated prior to development of an alternative compliance plan (see Form 2.4-1, Water Quality Credits). Form 4.3-9 Item 8 includes instructions on how to apply water quality credits when computing the DCV that must be met through alternative compliance. Alternative compliance plans may include one or more of the following elements:

- On-site structural treatment control BMP - All treatment control BMP should be located as close to possible to the pollutant sources and should not be located within receiving waters;
- Off-site structural treatment control BMP - Pollutant removal should occur prior to discharge of runoff to receiving waters;
- Urban runoff fund or In-lieu program, if available

Depending upon the proposed alternative compliance plan, approval by the executive officer may or may not be required (see Section 6 of the TGD for WQMP).

## Section 5 Inspection and Maintenance Responsibility for Post Construction BMP

All BMP included as part of the project WQMP are required to be maintained through regular scheduled inspection and maintenance (refer to Section 8, Post Construction BMP Requirements, in the TGD for WQMP). Fully complete Form 5-1 summarizing all BMP included in the WQMP. Attach additional forms as needed. The WQMP shall also include a detailed Operation and Maintenance Plan for all BMP and may require a Maintenance Agreement (consult the jurisdiction's LIP). If a Maintenance Agreement is required, it must also be attached to the WQMP.

<b>Form 5-1 BMP Inspection and Maintenance (use additional forms as necessary)</b>			
BMP	Reponsible Party(s)	Inspection/ Maintenance Activities Required	Minimum Frequency of Activities
SD-10	HOA	per BMP recommendation	per BMP recommendation
SD-12	HOA	per BMP recommendation	per BMP recommendation
SD-13	HOA	per BMP recommendation	per BMP recommendation
SD-32	HOA	per BMP recommendation	per BMP recommendation
TC-11	HOA	per BMP recommendation	per BMP recommendation
TC-22	HOA	per BMP recommendation	per BMP recommendation
TC-30	HOA	per BMP recommendation	per BMP recommendation
TC-31	HOA	per BMP recommendation	per BMP recommendation

<b>Form 5-1 BMP Inspection and Maintenance (Continued)</b>			
BMP	Reponsible Party(s)	Inspection/ Maintenance Activities Required	Minimum Frequency of Activities
TC-50	HOA	per BMP recommendation	per BMP recommendation
TC-60	HOA	per BMP recommendation	per BMP recommendation
MP-51	HOA	per BMP recommendation	per BMP recommendation
MP-52	HOA	per BMP recommendation	per BMP recommendation

# Section 6WQMP Attachments

## 6.1.Site Plan and Drainage Plan

Include a site plan and drainage plan sheet set containing the following minimum information:

- Project location
- Site boundary
- Land uses and land covers, as applicable
- Suitability/feasibility constraints
- Structural Source Control BMP locations
- Site Design Hydrologic Source Control BMP locations
- LID BMP details
- Drainage delineations and flow information
- Drainage connections

## 6.2Electronic Data Submittal

Minimum requirements include submittal of PDF exhibits in addition to hard copies. Format must not require specialized software to open. If the local jurisdiction requires specialized electronic document formats (as described in their local Local Implementation Plan), this section will describe the contents (e.g., layering, nomenclature, geo-referencing, etc.) of these documents so that they may be interpreted efficiently and accurately.

## 6.3Post Construction

Attach all O&M Plans and Maintenance Agreements for BMP to the WQMP.

## 6.4Other Supporting Documentation

- BMP Educational Materials
- Activity Restriction – C, C&R's & Lease Agreements

# **ATTACHMENT 6-1**

## **Site Plan and Drainage Plan**

- Location Map
- Site Map
- Land Use Plan
- WQMP EXHIBIT
- EXISTING HYDROLOGY MAP
- BASIS FOR SEASONAL HIGH GROUNDWATER



Project Site



robert m. beers pe

8175 Limonte Avenue, Suite E  
Jurupa Valley, CA 92509  
Ph. (951) 360-2070 Fax (951) 360-2080  
Cell: (951) 317-2041  
rmbears777@hotmail.com

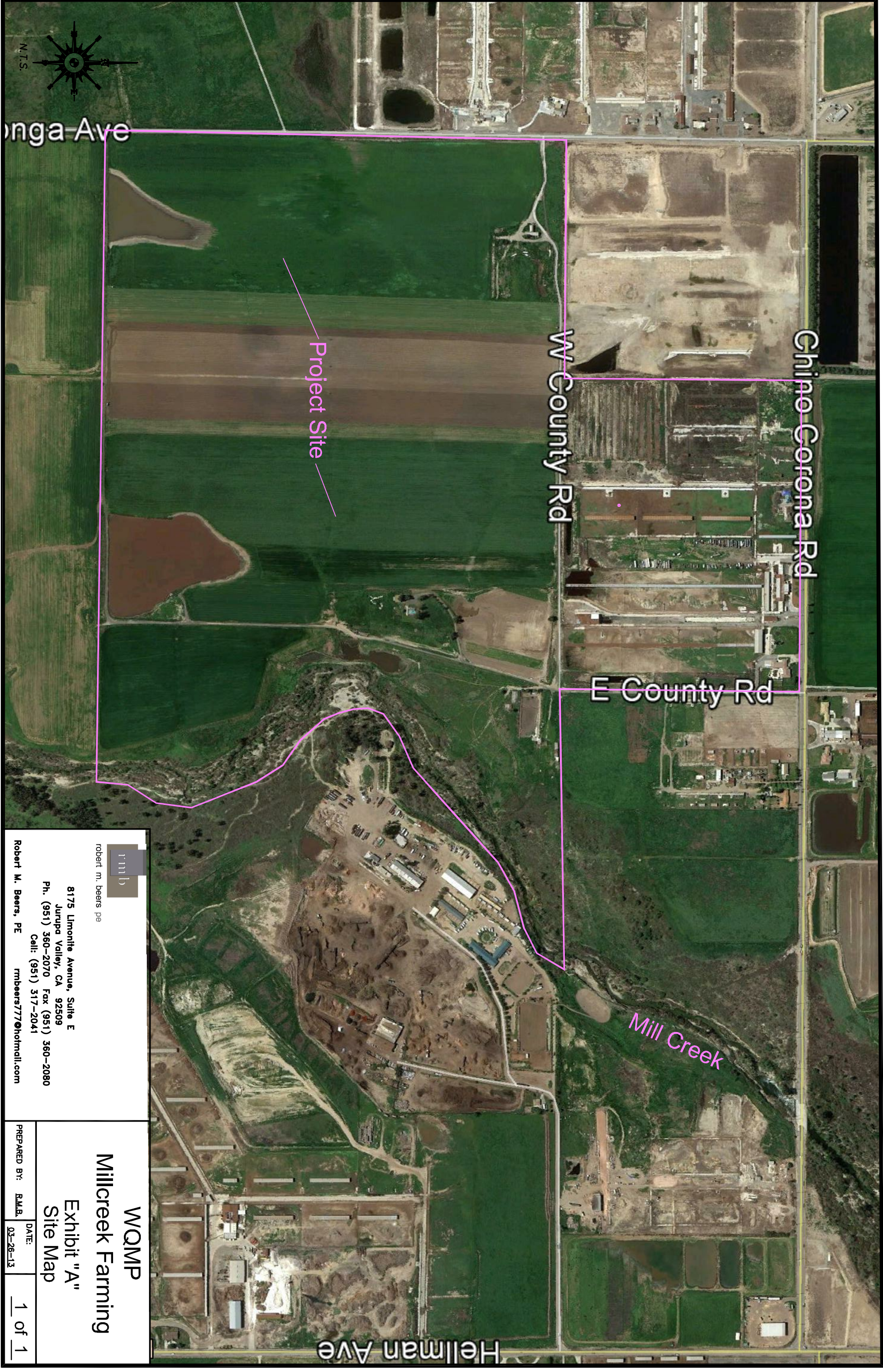
# WQMP

## Millcreek Farming

### Exhibit "A" Location Map

PREPARED BY: R.M.B. DATE: 03-26-13 1 of 1





Chino Corona Rd

W County Rd

E County Rd

Project Site

Mill Creek

Hellman Ave

ongga Ave



N.T.S.

rmb

robert m. beers pe

8175 Limonite Avenue, Suite E  
Jurupa Valley, CA 92509  
Ph. (951) 360-2070 Fax (951) 360-2080  
Cell: (951) 317-2041  
rmbbeers777@hotmail.com

WQMP

Millcreek Farming

Exhibit "A"  
Site Map

PREPARED BY: R.M.B. DATE: 03-26-13 1 of 1

CALIFORNIA DEPARTMENT OF  
**TRANSPORTATION**

[Caltrans](#) > [DEA](#) > [Stormwater](#) > **Water Quality Planning Tool**

**Mill Creek (Prado Area)**



**TMDLS & 303(D) LIST (2010) FOR MILL CREEK (PRADO AREA)**

**Key:** Pollutant on 303(d) list    Pollutant with a TMDL

Pollutant	Source	Size	Status	Comments
Nutrients	Agriculture	1.58 Miles	TMDL required	
Nutrients	Dairies	1.58 Miles	TMDL required	
Pathogens	Dairies	1.58 Miles	Being addressed with USEPA approved TMDL	
Total Suspended Solids (TSS)	Dairies	1.58 Miles	TMDL required	

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## BASIS FOR SEASONAL HIGH GROUNDWATER ELEVATION

Plates from the 2007 GMU soils report out of 2009 EIR for Edgewater SPA provide maps for the borings and include water surface elevations. The seasonal high groundwater elevation will be re-evaluated in final design.

March 16, 2007

Project 04-15-03 (Edgewater, Chino)

 GMU Page 9

mapping done by Mendenhall around 1900 (Fife, 1976) documented relatively shallow groundwater levels throughout the basin, as well as artesian conditions in a large area northwest of the site. An increase in population and agricultural activity triggered a decline in groundwater elevations until the 1970s. During the first half of the 20<sup>th</sup> century, significant areas of subsidence were documented across the Basin. In the 1970s efforts were started to restore the groundwater regime (Fife, 1976, and Wildermuth Environmental, 1999). Groundwater management by the Chino Basin Watermaster as well as imported water have led to more stabilized groundwater elevations within the Chino Basin.

Groundwater within the Chino Basin consists of several stacked aquifers within the basin alluvial deposits, which are up to several hundred feet thick. The shallowest of these varies across the basin in elevation and thickness. Historical regional data on the Chino Basin Watermaster website indicates groundwater elevations of approximately 525 feet msl underlying the site in the 1930s, and elevations of approximately 500 feet msl in the late 1990s. Current regional groundwater elevations have not yet been posted by the Watermaster, but are anticipated to be higher than reported in the 1990s.

**Local Groundwater Conditions.** Groundwater data collected during our previous investigation indicated groundwater depths of approximately 13 to 23 feet below the site (elevations of approximately 523 to 547 feet msl). Our current investigation encountered perched zones of seepage in scattered locations across the site. These locations are likely local zones of minor seepage. Groundwater was encountered between approximately 6 and 40 feet below the existing

ground surface across the site (elevations of approximately 519 to 539 feet msl) during our current investigation. Groundwater elevations observed during our current investigation were compiled with data from our previous investigation to complete the site subsurface groundwater model. The following table lists the groundwater data collected:

<b>Exploration Number</b>	<b>Date of Reading/Test</b>	<b>Depth to Groundwater (feet bgs)</b>	<b>Elevation of Groundwater (feet msl)</b>
CPT-1	2-13-04	16	546
CPT-3	2-13-04	13	547
CPT-5	2-13-04	23	523
DH-3	2-7-07	26.2	535.8
DH-5	2-7-07	6.8	529.2
DH-6	2-7-07	38.6	519.4
DH-7	2-7-07	20.0	525.0
DH-8	2-7-07	30.2	519.8

In general, the groundwater surface beneath the site appears to slope gently to the south and east. The shallowest groundwater elevations occur along the southern boundary of the site (approximately 520 feet msl) and the eastern boundary, adjacent to Mill Creek (approximately 520 to 525 feet msl). The groundwater surface ramps upward toward the northern portion of the site, with elevations up to approximately 545 feet msl.

In comparison to historical data, the current groundwater table appears to be up to approximately 20 feet higher in the northern portion of the site. As a result, current groundwater elevations were utilized in both liquefaction and slope stability analyses as the historic high elevations.

**ATTACHMENT 6-2**  
**Electronic Data Submittal**

## **ATTACHMENT 6-3**

### **Post Construction**

- **O&M Plans and Maintenance Agreements**

# Attachment A-1

## Maintenance Mechanisms

**A-1.1** The Agency shall not accept storm water structural BMPs as meeting the WQMP requirements standard, unless an O&M Plan is prepared (see WQMP Section 4.1) and a mechanism is in place that will ensure ongoing long-term maintenance of all structural and non-structural BMPs. This mechanism can be provided by the Agency or by the project proponent. As part of project review, if a project proponent is required to include interim or permanent structural and non-structural BMPs in project plans, and if the Agency does not provide a mechanism for BMP maintenance, the Agency shall require that the applicant provide verification of maintenance requirements through such means as may be appropriate, at the discretion of the Agency, including, but not limited to covenants, legal agreements, maintenance agreements, conditional use permits and/or funding arrangements.

### **A-1.2 Maintenance Mechanisms**

1. **Public entity maintenance:** The Agency may approve a public or acceptable quasi-public entity (e.g., the County Flood Control District, or annex to an existing assessment district, an existing utility district, a state or federal resource agency, or a conservation conservancy) to assume responsibility for operation, maintenance, repair and replacement of the BMP. Unless otherwise acceptable to individual Agencies, public entity maintenance agreements shall ensure estimated costs are front-funded or reliably guaranteed, (e.g., through a trust fund, assessment district fees, bond, letter of credit or similar means). In addition, the Permittees may seek protection from liability by appropriate releases and indemnities.

The Agency shall have the authority to approve storm water BMPs proposed for transfer to any other public entity within its jurisdiction before installation. The Permittee shall be involved in the negotiation of maintenance requirements with any other public entities accepting maintenance responsibilities within their respective jurisdictions; and in negotiations with the resource agencies responsible for issuing permits for the construction and/or maintenance of the facilities. The Agency must be identified as a third party beneficiary empowered to enforce any such maintenance agreement within their respective jurisdictions.

2. **Project proponent agreement to maintain storm water BMPs:** The Agency may enter into a contract with the project proponent obliging the project proponent to maintain, repair and replace the storm water BMP as necessary into perpetuity. Security or a funding mechanism with a "no sunset" clause may be required.
3. **Assessment districts:** The Agency may approve an Assessment District or other funding mechanism created by the project proponent to provide funds for storm water BMP maintenance, repair and replacement on an ongoing basis. Any agreement with such a District shall be subject to the Public Entity Maintenance Provisions above.

4. **Lease provisions:** In those cases where the Agency holds title to the land in question, and the land is being leased to another party for private or public use, the Agency may assure storm water BMP maintenance, repair and replacement through conditions in the lease.
5. **Conditional use permits:** For discretionary projects only, the Agency may assure maintenance of storm water BMPs through the inclusion of maintenance conditions in the conditional use permit. Security may be required.
6. **Alternative mechanisms:** The Agency may accept alternative maintenance mechanisms if such mechanisms are as protective as those listed above.

# Attachment A-2



## INSTRUCTIONS FOR COMPLETING THE WATER QUALITY MANAGEMENT PLAN (WQMP) TRANSFER, ACCESS, AND MAINTENANCE AGREEMENT

In order for your project to receive approval, you will need to prepare a WQMP and Storm Water Best Management Practices Transfer, Access and Maintenance Agreement (Maintenance Agreement). ***Please contact the Department of Public Works, Environmental Management Division after the final WQMP is approved, and at least ten (10) business days prior to your project's construction being completed, to receive the current Maintenance Agreement template. You can contact the Environmental Management Division by phone, by calling (909) 387 – 8109, or by email, at [evarga@dpw.sbcounty.gov](mailto:evarga@dpw.sbcounty.gov).*** The Maintenance Agreement shall be completed and approved according to the procedure outlined below, and must include the following information and attachments:

1. Provide all written information requested in the Maintenance Agreement template, including project and property owner's signature(s).
2. Attach a legal description of the project location, as well as a legal description of the locations of where all proposed storm water Best Management Practices will be constructed / installed, including a listing of the applicable Assessor's Parcel Numbers (APN). This page will be labeled as "EXHIBIT A, LEGAL DESCRIPTION".
3. Attach an 8.5" x 11" or 8.5" x 14" sized project plan sheet (labeled as "EXHIBIT B, BMP SITE PLAN"), illustrating the proposed storm water Best Management Practices and maintenance / access points. Cross sections and other pertinent details of such measures shall also be included. (Plan should be legible)
4. Attach a completed Notary page, with notary's wet seal affixed, **and having the following statement inserted on the Notary page:**

"FOR: Maintenance Agreement, dated \_\_\_\_\_, for the project known as \_\_\_\_\_ (APN \_\_\_\_\_), as described in the WQMP dated \_\_\_\_\_."

5. Attach a copy of the cover sheet for the most current WQMP that was approved. The cover sheet must display the information that is requested in the most current WQMP Guidance and Template document. Please visit the Environmental Management Division's website for the current WQMP Guidance and Template document ([http://www.sbcounty.gov/dpw/land/environmental\\_mgmt.asp](http://www.sbcounty.gov/dpw/land/environmental_mgmt.asp)).

6. The completed Maintenance Agreement (along with attachments) shall be submitted for review to:

Department of Public Works  
Environmental Management Division  
825 E. Third Street, Room 201  
San Bernardino, CA 92415-0835

7. Staff at the Environmental Management Division will review the Maintenance Agreement, and request any necessary changes.
8. When the Maintenance Agreement has been approved, the project and property owner(s) must sign the Maintenance Agreement and have his / her signature notarized.
9. The Maintenance Agreement will then be returned to the Environmental Management Division, where appropriate staff will obtain the notarized signature of the Director of Public Works (this may take up to ten (10) business days, depending on the Director's schedule and availability of the notary). Please provide a telephone number where you can be reached when the Maintenance Agreement is ready to be picked up for you to record.

**Please Note:** The Director of Public Works will not sign the Maintenance Agreement until the proposed Best Management Practices, as documented in the project WQMP, are constructed / installed, and all outstanding charges and invoices are paid.

10. The fully executed Maintenance Agreement must now be recorded at the San Bernardino County Recorder's Office at:

222 W. Hospitality Lane (behind the Souplantation restaurant)  
San Bernardino, CA 92415-0018

11. A photocopy of the final recorded Maintenance Agreement must be returned to the Environmental Management Division. You must also show the recorded Maintenance Agreement to the Land Development Division of the Department of Public Works, in order to receive their final approval on your project.

If during project construction, there are any field changes to the storm water Best Management Practices and maintenance / access points proposed in the WQMP, then the WQMP must be revised and re-submitted for approval by the County, and a new Maintenance Agreement must also be completed and re-submitted for approval by the County, according to the procedure outlined above.

If you have any further questions about this process, please call the Environmental Management Division, County Storm Water Program, at (909) 387-8109.

**RECORDING REQUESTED BY:**

County of San Bernardino  
Department of Public Works

**AND WHEN RECORDED MAIL TO:**

County of San Bernardino  
Department of Public Works  
825 E. Third Street, Room 201  
San Bernardino, CA 92415-0835

---

SPACE ABOVE THIS LINE FOR RECORDER'S USE

---

**AGREEMENT**

**THIS PAGE ADDED TO PROVIDE ADEQUATE SPACE FOR RECORDING  
INFORMATION (Additional Recording Fees Apply)**

**Water Quality Management Plan and Storm Water Best Management Practices  
Transfer, Access and Maintenance Agreement**

**OWNER NAME :** \_\_\_\_\_

**PROPERTY ADDRESS:** \_\_\_\_\_

**APN:** \_\_\_\_\_

**THIS AGREEMENT** is made and entered into in

\_\_\_\_\_, California, this \_\_\_\_\_ day of

\_\_\_\_\_, by and between

\_\_\_\_\_, hereinafter

referred to as Owner, and the COUNTY OF SAN BERNARDINO, a municipal corporation, located in the County of San Bernardino, State of California, hereinafter referred to as County;

**WHEREAS**, the Owner owns real property ("Property") in the County of San Bernardino, State of California, more specifically described in Exhibit "A" and depicted in Exhibit "B", each of which exhibits is attached hereto and incorporated herein by this reference;

**WHEREAS**, at the time of initial approval of development project known as

\_\_\_\_\_ within the Property described herein, the County required the project to employ Best Management Practices, hereinafter referred to as "BMPs," to minimize pollutants in urban runoff;

**WHEREAS**, the Owner has chosen to install and/or implement BMPs as described in the Water Quality Management Plan, on file with the County, hereinafter referred to as "WQMP", to minimize pollutants in urban runoff and to minimize other adverse impacts of urban runoff;

**WHEREAS**, said WQMP has been certified by the Owner and reviewed and approved by the County;

**WHEREAS**, the Owner is aware that periodic and continuous maintenance, including, but not necessarily limited to, filter material replacement and sediment removal, is required to assure peak performance of all BMPs in the WQMP and that, furthermore, such maintenance activity will require compliance with all Local, State, or Federal laws and regulations, including those pertaining to confined space and waste disposal methods, in effect at the time such maintenance occurs;

**NOW THEREFORE**, it is mutually stipulated and agreed as follows:

1. All maintenance or replacement of BMPs proposed as part of the WQMP are the sole responsibility of the Owner in accordance with the terms of this Agreement.
2. Owner hereby provides the County of San Bernardino's designee complete access, of any duration, to the BMPs and their immediate vicinity at any time, upon reasonable notice, or in the event of emergency, as determined by the County Director of Public Works, no advance notice, for the purpose of inspection, sampling, testing of the Device, and in case of emergency, to undertake all necessary repairs or other preventative measures at owner's expense as provided in paragraph 3 below. The County shall make every effort at all times to minimize or avoid interference with Owner's use of the Property. Denial of access to any premises or facility that contains WQMP features is a violation of the County Storm Water Ordinance, County Code 3587. If there is reasonable cause to believe that an illicit discharge or breach of the WQMP operation and maintenance commitments is occurring on the premises then the authorized enforcement agency may seek issuance of a search warrant from any court of competent jurisdiction in addition to other enforcement actions.
3. Owner shall use its best efforts diligently to maintain all BMPs in a manner assuring peak performance at all times. All reasonable precautions shall be exercised by Owner and Owner's representative or contractor in the removal and extraction of any material(s) from the BMPs and the ultimate disposal of the material(s) in a manner consistent with all relevant laws and regulations in effect at the time. As may be requested from time to time by the County, the Owner shall provide the County with documentation identifying the material(s) removed, the quantity, and disposal destination.
4. In the event Owner, or its successors or assigns, fails to accomplish the necessary maintenance contemplated by this Agreement, within five (5) days of being given written notice by the County, the County is hereby authorized to cause any maintenance necessary to be done and charge the entire cost and expense against the property and/or to the Owner or Owner's successors or assigns, including administrative costs, attorneys fees and interest thereon at the maximum rate authorized by the County Code from the date of the notice of expense until paid in full.
5. The County may require the owner to post security in form and for a time period satisfactory to the County to guarantee the performance of the obligations stated herein. Should the Owner fail to perform the obligations under the Agreement, the County may, in the case of a cash bond, act for the Owner using the proceeds from it, or in the case of a surety bond, require the sureties to perform the obligations of the Agreement. As an additional remedy, the Director of Public Works may withdraw any previous storm water-related approval with respect to the property on which BMPs have been installed and/or implemented until such time as Owner repays to County its reasonable costs incurred in accordance with paragraph 3 above.

6. This agreement shall be recorded in the Office of the Recorder of San Bernardino County, California, at the expense of the Owner and shall constitute notice to all successors and assigns of the title to said Property of the obligation herein set forth, and also a lien in such amount as will fully reimburse the County, including interest as herein above set forth, subject to foreclosure in event of default in payment.
7. In event of legal action occasioned by any default or action of the Owner, or its successors or assigns, then the Owner and its successors or assigns agree(s) to hold the County harmless and pay all costs incurred by the County in enforcing the terms of this Agreement, including reasonable attorney's fees and costs, and that the same shall become a part of the lien against said Property.
8. It is the intent of the parties hereto that burdens and benefits herein undertaken shall constitute covenants that run with said Property and constitute a lien there against.
9. The obligations herein undertaken shall be binding upon the heirs, successors, executors, administrators and assigns of the parties hereto. The term "Owner" shall include not only the present Owner, but also its heirs, successors, executors, administrators, and assigns. Owner shall notify any successor to title of all or part of the Property about the existence of this Agreement. Owner shall provide such notice prior to such successor obtaining an interest in all or part of the Property. Owner shall provide a copy of such notice to the County at the same time such notice is provided to the successor.
10. Time is of the essence in the performance of this Agreement.
11. Any notice to a party required or called for in this Agreement shall be served in person, or by deposit in the U.S. Mail, first class postage prepaid, to the address set forth below. Notice(s) shall be deemed effective upon receipt, or seventy-two (72) hours after deposit in the U.S. Mail, whichever is earlier. A party may change a notice address only by providing written notice thereof to the other party.
12. The Owner its successors and assigns, hereby agrees to save and hold harmless the County, any of its departments, agencies, officers or employees, all of whom while working within their respective authority, from all cost, injury and damage incurred by any of the above, and from any other injury or damage to any person or property whatsoever, any of which is caused by an activity, condition or event arising out of the performance, preparation for performance or nonperformance of any provision of this agreement by the Owner, its agents, or any of its independent contractors.

[REMAINDER OF THIS PAGE INTENTIONALLY LEFT BLANK]

**IF TO COUNTY :**

**IF TO OWNER:**

Director of Public Works \_\_\_\_\_

\_\_\_\_\_

825 E. Third Street, Room 201 \_\_\_\_\_

\_\_\_\_\_

San Bernardino, CA 92415-0835 \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

**IN WITNESS THEREOF**, the parties hereto have affixed their signatures as of the date first written above.

**OWNER:**

Signature: \_\_\_\_\_

Name: \_\_\_\_\_

\_\_\_\_\_

Title

**OWNER:**

Signature: \_\_\_\_\_

Name: \_\_\_\_\_

\_\_\_\_\_

Title

**NOTARIES ON FOLLOWING PAGE**

A notary acknowledgement is required for recordation (attach appropriate acknowledgement).

ACCEPTED BY:

\_\_\_\_\_  
GERRY NEWCOMBE, Director of Public Works

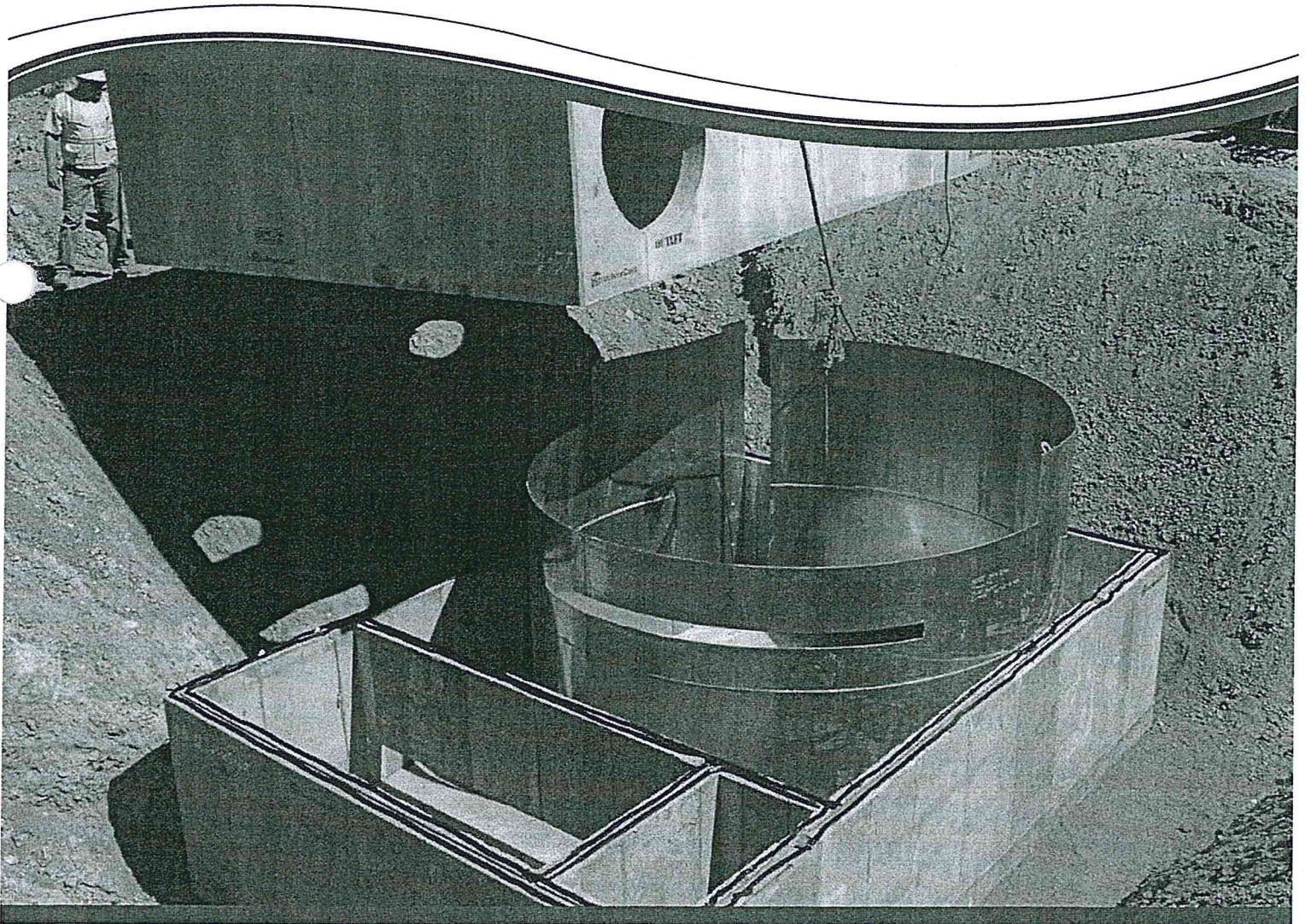
Date: \_\_\_\_\_

Attachment: Standard Notary Acknowledgement

**EXHIBIT A**  
**(Legal Description)**

**EXHIBIT B**  
**(Map/illustration)**

## Vortechs<sup>®</sup> Guide Operation, Design, Performance and Maintenance



## Vortechs®

The Vortechs system is a high-performance hydrodynamic separator that effectively removes finer sediment (e.g. 50-microns ( $\mu\text{m}$ ), oil, and floating debris). The swirl concentration operation and flow controls work together to minimize turbulence and provide stable storage of captured pollutants. Precast models can treat peak design flows up to 30-cfs (850-l/s); cast-in-place models handle even greater flows. A typical system is sized to provide a specific removal efficiency of a predefined particle size distribution (PSD).

### Operation Overview

Stormwater enters the swirl chamber inducing a gentle swirling flow pattern and enhancing gravitational separation. Sinking pollutants stay in the swirl chamber while floatables are stopped at the baffle wall. Vortechs systems are usually sized to efficiently treat the frequently occurring runoff events and are primarily controlled by the low flow control orifice. This orifice effectively reduces inflow velocity and turbulence by inducing a slight backwater that is appropriate to the site.

During larger storms, the water level rises above the low flow control orifice and begins to flow through the high flow control. Any layer of floating pollutants is elevated above the invert of the Floatables Baffle Wall, preventing release. Swirling action increases in relation to the storm intensity, while sediment pile remains stable. When the storm drain is flowing at peak capacity, the water surface in the system approaches the top of the high flow control. The Vortechs system will be sized large enough so that previously captured pollutants are retained in the system, even during these infrequent events.

As a storm subsides, treated runoff decants out of the Vortechs system at a controlled rate, restoring the water level to a dry-weather level equal to the invert of the inlet pipe. The low water level facilitates easier inspection and cleaning, and significantly reduces maintenance costs by reducing pump-out volume.

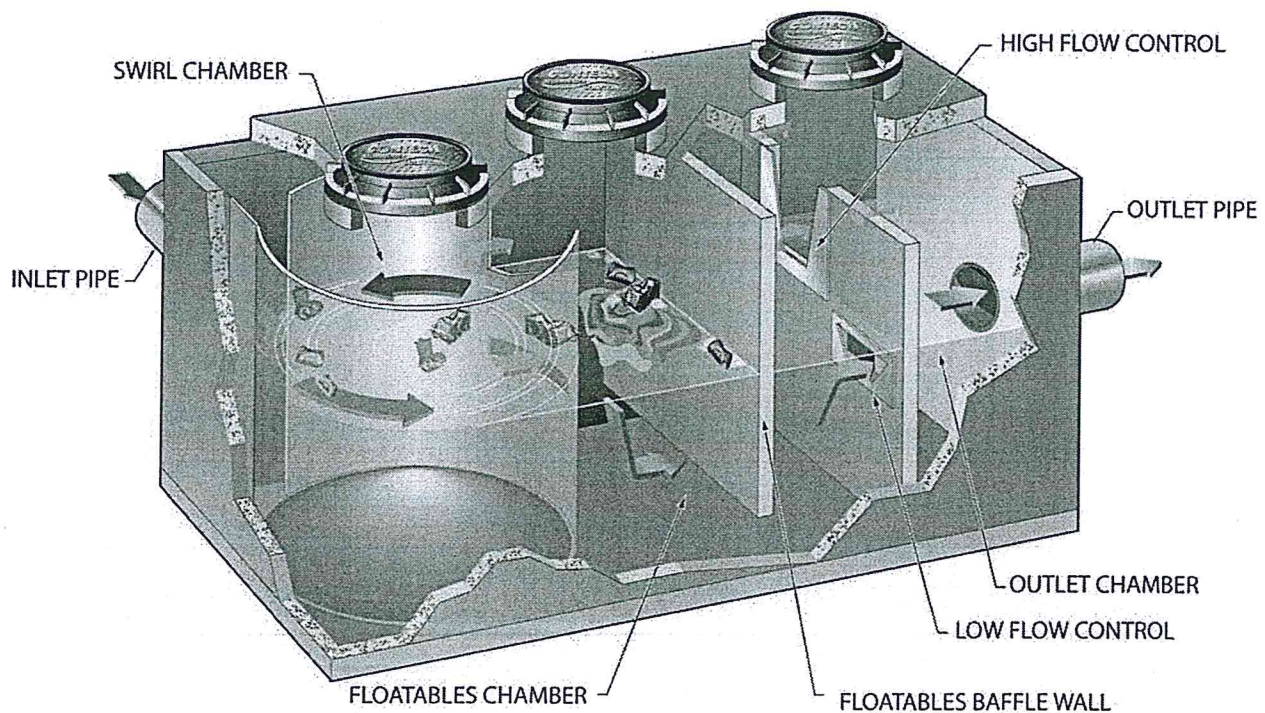
### Design Basics

Each Vortechs system is custom designed based on site size, site runoff coefficient, regional precipitation intensity distribution, and anticipated pollutant characteristics. There are two primary methods of sizing a Vortechs system. The first is to determine which model size provides the desired removal efficiency at a given flow for a defined particle size or PSD. The second and more in depth method is the summation of Rational Rainfall Method™ which uses a summation process described below in detail and is used when a specific removal efficiency of the net annual sediment load is required.

Typically Vortechs systems are designed to achieve an 80% annual solids load reduction based on lab generated performance curves for either 50- $\mu\text{m}$  particles, or a particle gradation found in typical urban runoff (see performance section of this manual for more information).

#### The Rational Rainfall Method™

Differences in local climate, topography and scale make every site hydraulically unique. It is important to take these factors into consideration when estimating the long-term performance of any stormwater treatment system. The Rational Rainfall Method combines site-specific information with laboratory generated performance data, and local historical precipitation records to estimate removal efficiencies as accurately as possible.



Short duration rain gauge records from across the United States and Canada were analyzed to determine the percent of the total annual rainfall that fell at a range of intensities. US stations' depths were totaled every 15 minutes or hourly and recorded in 0.01-inch increments. Depths were recorded hourly with 1-mm resolution at Canadian stations. One trend was consistent at all sites; the vast majority of precipitation fell at low intensities and high intensity storms contributed relatively little to the total annual depth.

These intensities, along with the total drainage area and runoff coefficient for each specific site, are translated into flow rates using the Rational Rainfall Method. Since most sites are relatively small and highly impervious, the Rational Rainfall Method is appropriate. Based on the runoff flow rates calculated for each intensity, operating rates within a proposed Vortechs system are determined. Performance efficiency curve determined from full scale laboratory tests on defined sediment PSDs is applied to calculate solids removal efficiency. The relative removal efficiency at each operating rate is added to produce a net annual pollutant removal efficiency estimate.

Once a system size is established, the internal elements of the system are designed based on information provided by the site engineer. Flow control sizes and shapes, sump depth, oil spill storage capacity, sediment storage volume and inlet and outlet orientation are determined for each system. In addition, bypass weir calculations are made for off-line systems.

## Flow Control Calculations

### Low Flow Control

The low flow control, or orifice, is typically sized to submerge the inlet pipe when the Vortechs system is operating at 20% of its treatment capacity. The orifice is typically a Cippoletti shaped aperture defined by its flat crest and sides which incline outwardly at a slope of 1 horizontal to 4 vertical.

$$Q_{\text{orifice}} = C_d \cdot A \cdot \sqrt{2gh}$$

Where:

$Q_{\text{orifice}}$  = flow through orifice, cfs (L/s)

$C_d$  = orifice coefficient of discharge = 0.56 (based on lab tests)

$A$  = orifice flow area, ft<sup>2</sup> (m<sup>2</sup>) (calculated by orifice geometry)

$h$  = design head, ft (m) (equal to the inlet pipe diameter)

$g$  = acceleration due to gravity (32.2-ft/s<sup>2</sup> (9.81-m/s<sup>2</sup>))

The minimum orifice crest length is 3-in (76-mm) and the minimum orifice height is 4-in (102-mm). If flow must be restricted beyond what can be provided by this size aperture, a Fluidic-Amp™ HydroBrake flow control will be used. The HydroBrake allows the minimum flow constriction to remain at 3-in (76-mm) or greater while further reducing flow due to its unique throttling action.

### High Flow Control

The high flow control, or weir, is sized to pass the peak system capacity minus the peak orifice flow when the water surface elevation is at the top of the weir. This flow control is also a Cippoletti type weir.

The weir flow control is sized by solving for the crest length and head in the following equation:

$$Q_{\text{weir}} = C_d \cdot L \cdot (h)^{3/2}$$

Where:

$Q_{\text{weir}}$  = flow through weir, cfs (L/s)

$C_d$  = Cippoletti weir coefficient = 3.37 (based on lab testing)

$h$  = available head, ft (m) (height of weir)

$L$  = design weir crest length, ft (m)

### Bypass Calculations

In most all cases, pollutant removal goals can be met without treating peak flow rates and it is most feasible to use a smaller Vortechs system configured with an external bypass. In such cases, a bypass design is recommended by CONTECH Stormwater Solutions for each off-line system. To calculate the bypass capacity, first subtract the system's treatment capacity from the peak conveyance capacity of the collection system (minimum of 10-year recurrence interval). The result is the flow rate that must be bypassed to avoid surcharging the Vortechs system. Then use the following arrangement of the Francis formula to calculate the depth of flow over the bypass weir.

$$H = (Q_{\text{bypass}} / (C_d \cdot L))^{2/3}$$

Where:

$H$  = depth of flow over bypass weir crest, ft (m)

$Q_{\text{bypass}}$  = required bypass flow, cfs (L/s)

$C_d$  = discharge coefficient = 3.3 for rectangular weir

$L$  = length of bypass weir crest, ft

The bypass weir crest elevation is then calculated to be the elevation at the top of the Cippoletti weir minus the depth of flow.

### Hydraulic Capacity

In the event that the peak design flow from the site is exceeded, it is important that the Vortechs system is not a constriction to runoff leaving the site. Therefore, each system is designed with enough hydraulic capacity to pass the 100-year flow rate. It is important to note that at operating rates above 100-gpm/ft<sup>2</sup> (68-Lps/m<sup>2</sup>) of the swirl chamber area (peak treatment capacity), captured pollutants may be lost.

When the system is operating at peak hydraulic capacity, water will be flowing through the gap over the top of the flow control wall as well as the orifice and the weir.

## Performance

### Full Scale Laboratory Test Results

Laboratory testing was conducted on a full scale Vortechs model 2000. The 150- $\mu\text{m}$  curve demonstrates the results of tests using particles that passed through a 60-mesh sieve and were retained on a 100-mesh sieve. The 50- $\mu\text{m}$  curve is based on tests of particles passing through a 200-mesh sieve and retained on a 400-mesh sieve (38- $\mu\text{m}$ ). A gradation with an average particle size (d50) of 80- $\mu\text{m}$ , containing particles ranging from 38-500- $\mu\text{m}$  in diameter was used to represent typical stormwater solids. (Table 1)

Particle Size Distribution ( $\mu\text{m}$ )	Percentage of Sample Make-Up
<63	42%
63 - 75	4%
75 - 100	9%
100 - 150	7%
150 - 250	11%
>250	27%

Table 1: Particle gradation of typical urban runoff used for efficiency curve

As shown, the Vortechs system maintains positive total suspended solids (TSS), defined by the tested gradations, removal efficiencies over the full range of operating rates. This allows the system to effectively treat all runoff from large, infrequent design storms, as well as runoff from more frequent low-intensity storms.

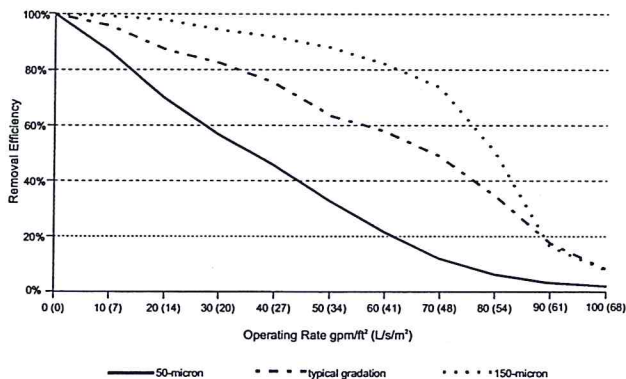


Figure 1: Vortechs model 2000 Removal Efficiencies

Typical Vortechs systems are designed to treat peak flows from 1.6-cfs (45-L/s) up to 30-cfs (850-L/s) online without the need for bypass. However, external bypasses can be configured to convey peak flows around the system if treatment capacity is exceeded. The system can also be configured to direct low flows from the last chamber of the system to polishing treatment when more stringent water quality standards are imposed. In all configurations, high removal efficiencies are achieved during the lower intensity storms, which constitute the majority of annual rainfall volume.

Full report available at [www.contechstormwater.com](http://www.contechstormwater.com).

### Laboratory Testing

Full reports available at [www.contechstormwater.com](http://www.contechstormwater.com)

Technical Bulletin 1: Removal Efficiencies for Selected Particle Gradations

Technical Bulletin 2: Particle Distribution of Sediments and the Effect on Heavy Metal Removal

Technical Bulletin 3: Sizing for Net Annual Sediment Removal

Technical Bulletin 3a: Determining Bypass Weir Elevation for Off-Line Systems

Technical Bulletin 4: Modeling Long Term Load Reduction: The Rational Rainfall Method

Technical Bulletin 5: Oil Removal Efficiency

### Field Monitoring

Following are brief summaries of the field tests completed to date.

Full reports available at [www.contechstormwater.com](http://www.contechstormwater.com)

### DeLorme Mapping Company

Yarmouth, ME

### CONTECH Stormwater Solutions

Prior to this premier field test of the Vortechs system, CONTECH developed an extensive body of laboratory data to document total suspended solids (TSS) removal efficiency. CONTECH performed this field study in order to compare the performance predicted using laboratory data to the performance of a correctly sized system in the field.

The study site was the headquarters of DeLorme Mapping in Yarmouth, Maine. The building, driveway, parking lot and ancillary facilities were constructed in 1996. A Vortechs model 11000 was installed to treat runoff from the 300-space, 4-acre (1.62-ha) parking lot.

Testing Period	May 1999 to Dec 1999
# of Storms Sampled	20
Mean Influent Concentration	328-mg/L
Mean Effluent Concentration	60-mg/L
Removal Efficiency	82%

The main purpose of the DeLorme study was to verify that the sizing methodology developed from our full-scale laboratory testing was valid and an accurate means of predicting field performance. The results of the study confirmed our sizing methodology.

### Village Marine Drainage

Lake George, NY

### New York State Department of Environmental Conservation, Division of Water

The New York State DEC used funds obtained in a Section 319 grant to initiate a study of the effectiveness of the Vortechs system to remove sediment and other pollutants transported

by stormwater to Lake George, Lake George Village, New York. "Since the 1970s, when there was a rapid increase in the rate and concentration of development along the southwestern shores of Lake George, we have been concerned about the impact of stormwater discharges into the lake," said Tracy West, co-author of the study.

Testing Period	Feb 2000 to Dec 2000
# of Storms Sampled	13
Mean Influent Concentration	801-mg/L
Mean Effluent Concentration	105-mg/L
Removal Efficiency	88%

The study concluded that the Village and Town of Lake George should consider installing additional Vortechs systems in areas where sedimentation and erosion have been identified as non-point source pollution problems.

**Harding Township Rest Area  
Harding Township, NJ  
RTP Environmental Associates**

This third party evaluation was performed under a U.S. Environmental Protection Agency grant, administered by the New Jersey Department of Environmental Protection. A. Roger Greenway, principal of RTP Environmental Associates, Inc., conducted the study in conjunction with Thonet Associates, which assisted with data analysis and helped develop best management practices (BMP) recommendations.

The Vortechs model 4000 was sized to handle a 100-year storm from the 3 acre (1.21 ha) paved parking area at the Harding Rest Stop, located off the northbound lane of I-287 in Harding Township, New Jersey.

Testing Period	May 1999 to Nov 2000
# of Storms Sampled	5
Mean Influent Concentration (TSS)	493-mg/L
Mean Effluent Concentration (TSS)	35-mg/L
Removal Efficiency (TSS)	93%
Mean Influent Concentration (TPH)	16-mg/L
Mean Effluent Concentration (TPH)	5-mg/L
Removal Efficiency (TPH)	67%

The study concluded that truck rest stops and similar parking areas would benefit from installing stormwater treatment systems to mitigate the water quality impacts associated with stormwater runoff from these sites.

**Timothy Edwards Middle School  
South Windsor, CT**

**UCONN Department of Civil & Environmental Engineering**

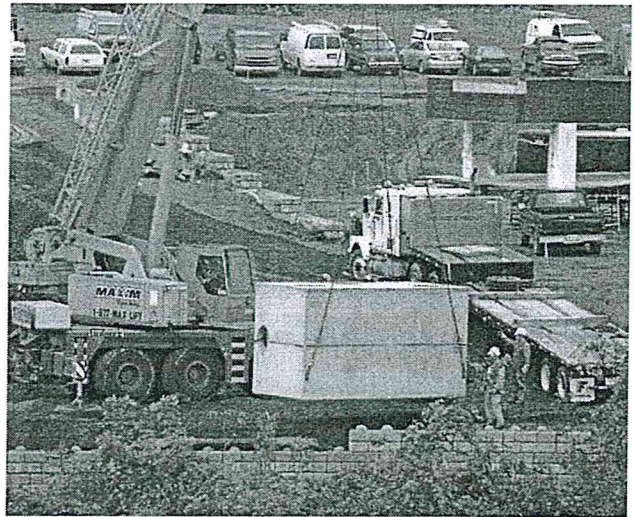
This study of the Vortechs system was published as a thesis by Susan Mary Board, as part of the requirements for a Master of Science degree from the University of Connecticut. Her objective was to determine how well the Vortechs system retained pollutants from parking lot runoff, including total suspended solids (TSS), nutrients, metals, and petroleum hydrocarbons.

A Vortechs model 5000 was installed in 1998 to treat runoff from the 82-space parking lot of Timothy Edwards Middle School. The entire watershed was approximately 2 acres (0.81 ha), and was 80% impervious.

Testing Period	Jul 2000 to Apr 2001
# of Storms Sampled	weekly composite samples taken
Mean Influent Concentration	324-mg/L
Mean Effluent Concentration	73-mg/L
Removal Efficiency	77%

Additionally, the Vortechs system was particularly effective in removing zinc (85%), lead (46%), copper (56%), phosphorus (67%) and nitrate (54%).

The study concluded that the Vortechs system significantly reduced effluent concentrations of many pollutants in stormwater runoff.



## Maintenance

The Vortechs system should be inspected at regular intervals and maintained 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., unstable soils or heavy winter sanding will cause the swirl chamber to fill more quickly but regular sweeping will slow accumulation.

### Inspection

Inspection is the key to effective maintenance and is easily performed. Pollutant deposition and transport may vary from year to year and regular inspections will help ensure that the system is cleaned out at the appropriate time. Inspections should be performed twice per year (i.e. spring and fall) however more frequent inspections may be necessary in equipment washdown areas and in climates where winter sanding operations may lead to rapid accumulations. It is useful and often required as part of a permit to keep a record of each inspection. A simple inspection and maintenance log form for doing so is provided on the following page, and is also available on [contechstormwater.com](http://contechstormwater.com).

The Vortechs system should be cleaned when inspection reveals that the sediment depth has accumulated to within 12 to 18 inches (300 to 450 mm) of the dry-weather water surface elevation. This determination can be made by taking two measurements with a stadia rod or similar measuring device; one measurement from the manhole opening to the top of the sediment pile and the other from the manhole opening to the water surface. **Note:** To avoid underestimating the volume of sediment in the chamber, the measuring device must be carefully lowered to the top of the sediment pile. 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.

### Cleaning

Cleaning of the Vortechs system should be done during dry weather conditions when no flow is entering the system. Clean-out of the Vortechs system with a vacuum truck is generally the most effective and convenient method of excavating pollutants from the system. If such a truck is not available, a "clamshell" grab may be used, but it is difficult to remove all accumulated pollutants using a "clamshell".

In installations where the risk of petroleum spills is small, liquid contaminants may not accumulate as quickly as sediment. However, an oil or gasoline spill should be cleaned out immediately. Motor oil and other hydrocarbons that accumulate on a more routine basis should be removed when an appreciable layer has been captured. To remove these pollutants, it may be preferable to use adsorbent pads to solidify the oil since these pads are usually much easier to remove from the unit individually and less expensive to dispose of than the oil/water emulsion that may be created by vacuuming the oily layer. Floating trash can be netted out if you wish to separate it from the other pollutants.

Cleaning of a Vortechs system is typically done by inserting a vacuum hose into the swirl chamber and evacuating this chamber of water and pollutants. As water is evacuated, the water level outside of the swirl chamber will drop to a level roughly equal to the crest of the lower aperture of the swirl chamber. The water outside the swirl chamber should remain

near this level throughout pumping as the bottom and sides of the swirl chamber are sealed to the tank floor and walls. This "water lock" feature prevents water from migrating into the swirl chamber, exposing the bottom of the baffle wall and creating excess pump-out volume. Floating pollutants will decant into the swirl chamber as the water level is drawn down. This allows most floating material to be withdrawn from the same access point above the swirl chamber. Floating material that does not decant into the swirl chamber during draw down should be skimmed from the baffle chamber. If maintenance is not performed as recommended, sediment may accumulate outside the swirl chamber. If this is the case, it may be necessary to pump out other chambers. It is advisable to check for sediment accumulation in all chambers during inspection and maintenance.

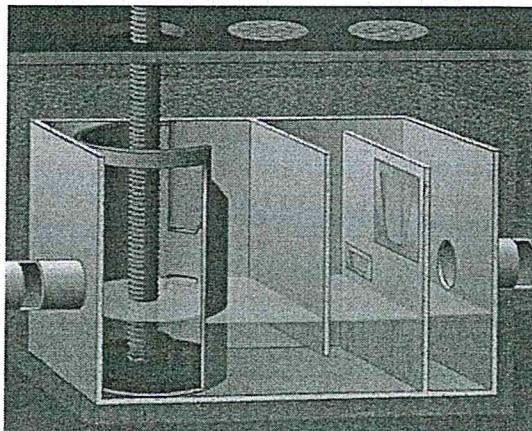
These maintenance recommendations apply to all Vortechs systems with the following exceptions:

1. It is strongly recommended that when cleaning systems larger than the Model 16000 the baffle chamber be drawn down to depth of three feet prior to beginning clean-out of the swirl chamber. Drawing down this chamber prior to the swirl chamber reduces adverse structural forces pushing upstream on the swirl chamber once that chamber is empty.
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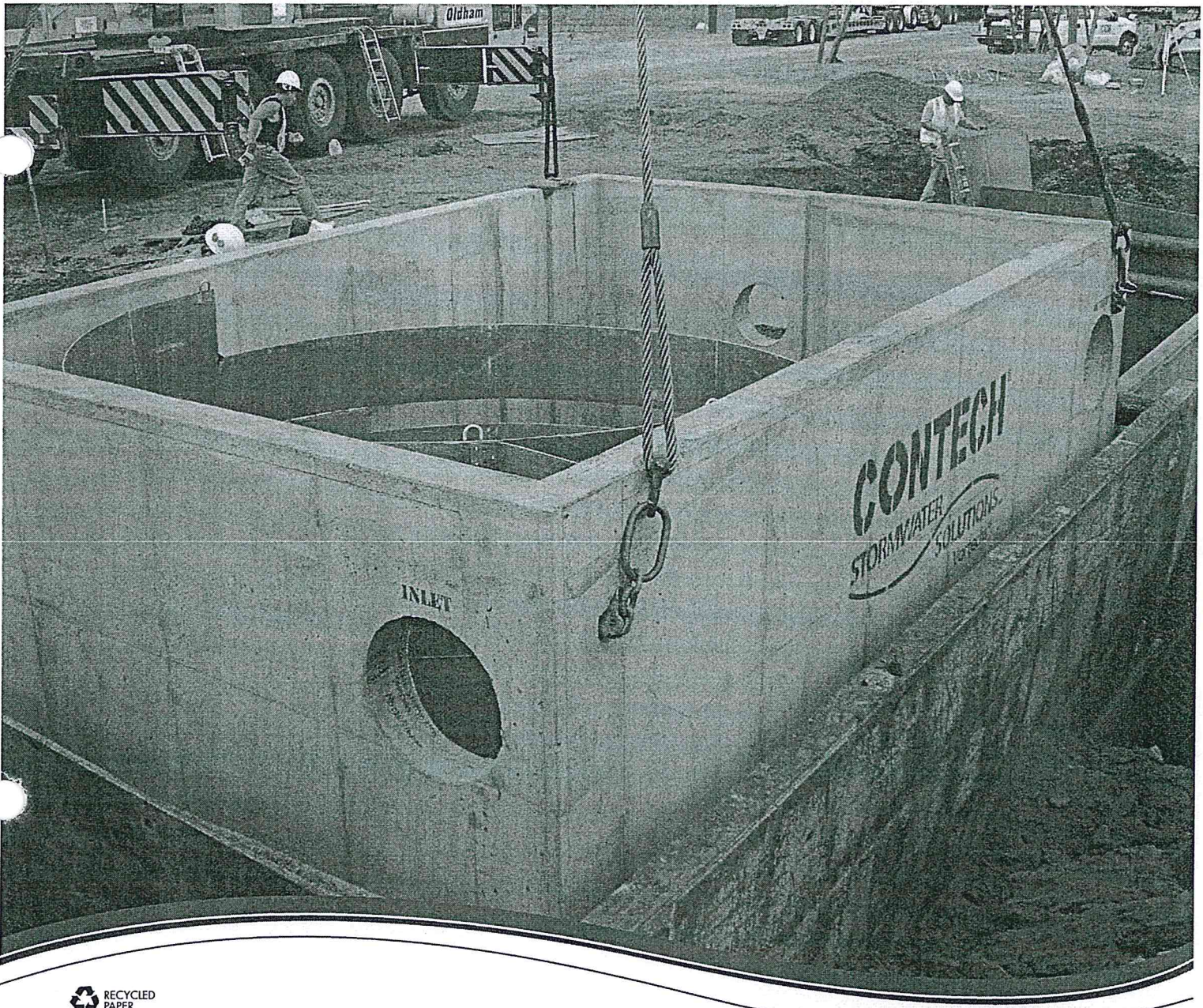
Manhole covers should be securely seated following cleaning activities to prevent leakage of runoff into the system from above and also to ensure proper safety precautions. If anyone physically enters the unit, Confined Space Entry procedures need to be followed.

Disposal of all material removed from the Vortechs system should be done in accordance with local regulations. In many locations, disposal of evacuated sediments may be handled in the same manner as disposal of sediments removed from catch basins or deep sump manholes. Check your local regulations for specific requirements on disposal.

For assistance with maintaining your Vortechs system, contact us regarding the CONTECH Maintenance Compliance Certification Program.







800.925.5240  
contechstormwater.com

## Support

- Drawings and specifications are available at [www.contechstormwater.com](http://www.contechstormwater.com).
- Site-specific design support is available from our engineers.

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## Vortechs® Maintenance

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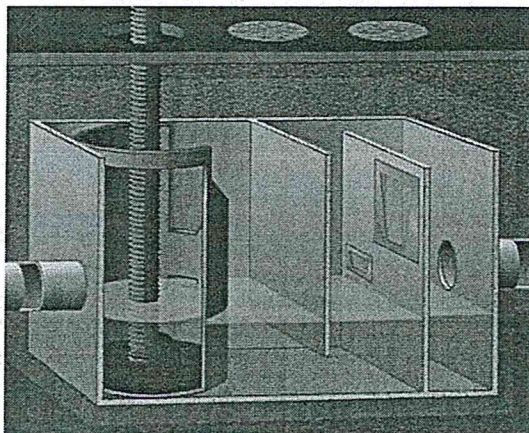
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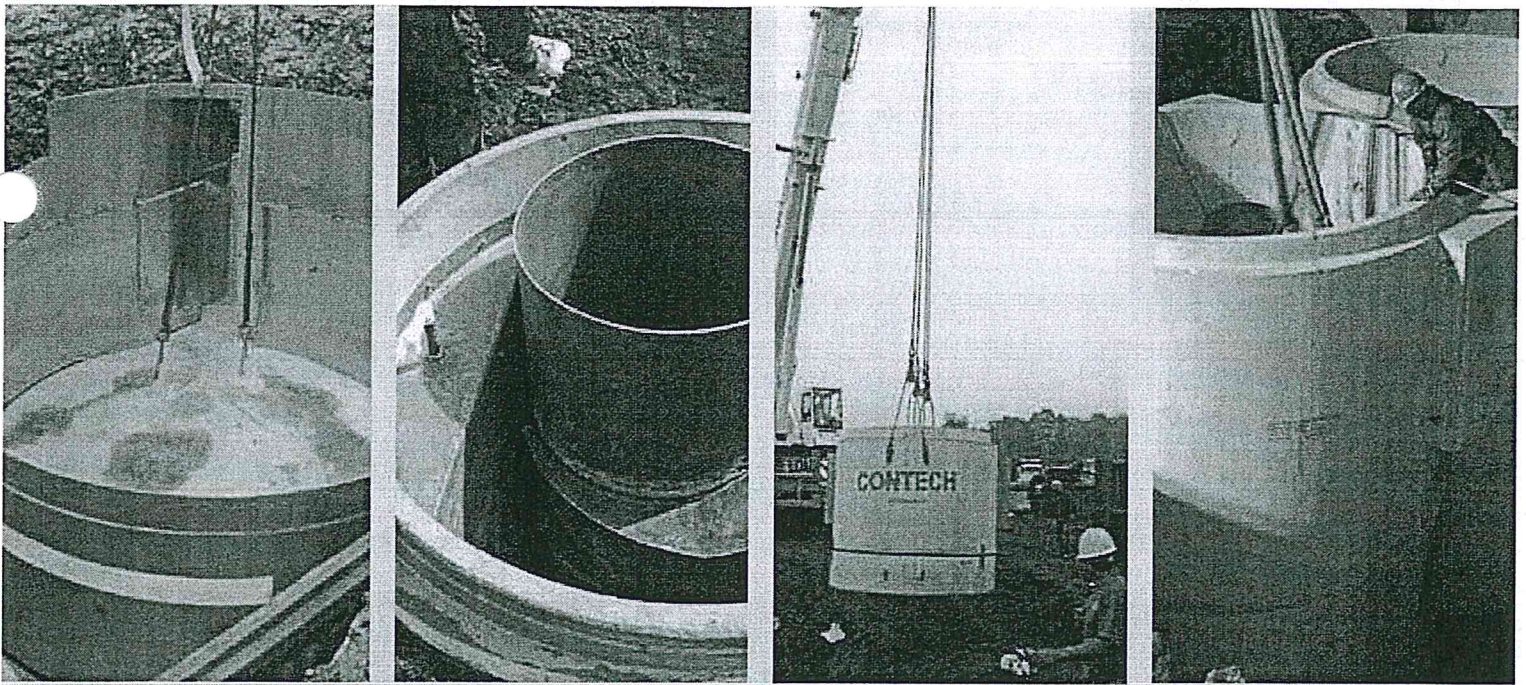
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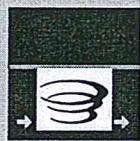






**CONTECH**<sup>®</sup>  
ENGINEERED SOLUTIONS

# Hydrodynamic Separation



Solutions  
Guide

Scan Me!



URBANGREEN<sup>™</sup>

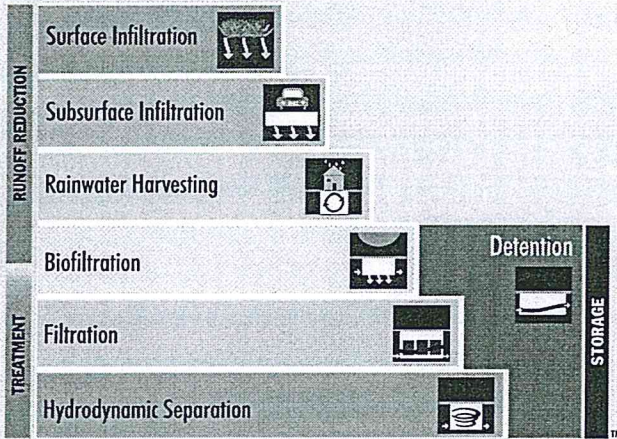


# Hydrodynamic Separation



Selecting the right stormwater solution just got easier...

It's simple to choose the right low impact development (LID) solution to achieve your runoff reduction goals with the Contech UrbanGreen Staircase. First, select the runoff reduction practices that are most appropriate for your site, paying particular attention to pretreatment needs. If the entire design storm cannot be retained, select a treatment best management practice (BMP) for the balance. Finally, select a detention system to address any outstanding downstream erosion.



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## Removing Pollutants with Hydrodynamic Separation

Hydrodynamic separators are some of the first technologies to be developed for treating stormwater. Our hydrodynamic separation (HDS) products have been providing reliable stormwater treatment solutions for more than 20 years. With performance proven in the lab and in the field at sites across the country, these systems are widely accepted for effective solids removal. They are an optimal choice for pretreatment systems, especially efficient on gross solids, trash and debris, while also removing total suspended solids (TSS).

### Fundamentals of HDS

- Create a low velocity vortex action to:
  - Increase efficiency by increasing length of flow path and eliminating short circuiting
  - Concentrate solids in stable, low velocity flow field
- Incorporate flow controls to:
  - Minimize turbulence and velocity
  - Prevent flow surges and resuspension
  - Retain floating pollutants
- Provide easy access to captured pollutants to make maintenance easy

Learn more about hydrodynamic separation at [www.ContechES/stormwater](http://www.ContechES/stormwater)

Our hydrodynamic separation products have been providing reliable stormwater treatment solutions for more than 20 years ❖ ❖ ❖

# Applications

HDS products work well as standalone or end-of-pipe treatment systems and can easily be implemented in a retrofit scenario. They are particularly effective at removal of solids, trash and debris – and can help you meet TMDL requirements for these pollutants. HDS systems are also optimal pretreatment systems – and an important building block in a low impact development (LID) design. By removing solids, trash and debris prior to detention, infiltration or re-use systems, you can significantly increase their service life.

## Water Quality

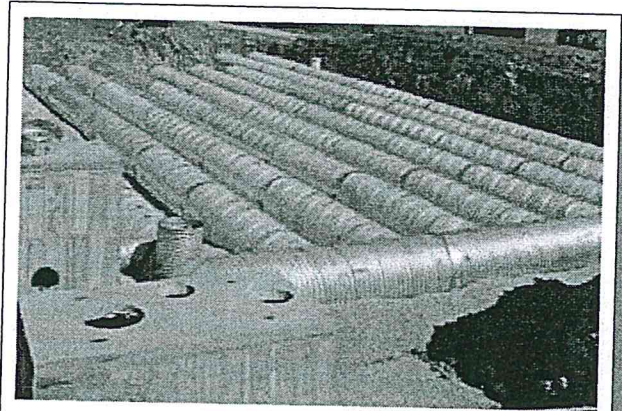
HDS products provide high-performance stormwater pollutant removal. These systems are effective in removing solids to meet water quality goals and can be designed to achieve site treatment goals for TSS or oil.

## Pretreatment for Low Impact Development (LID) Designs

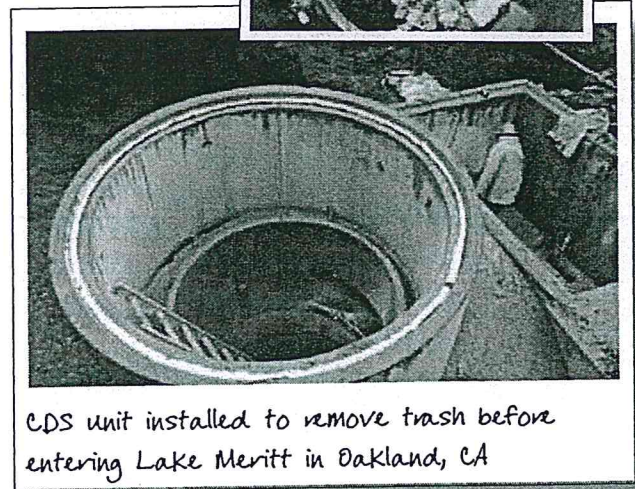
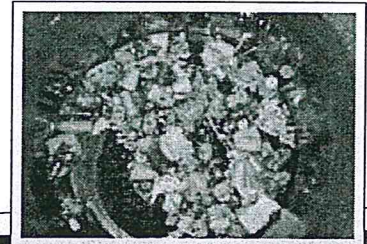
Hydrodynamic separation systems installed as pretreatment reduce downstream loading to reduce maintenance

## Inlet and Outlet Pollution Control

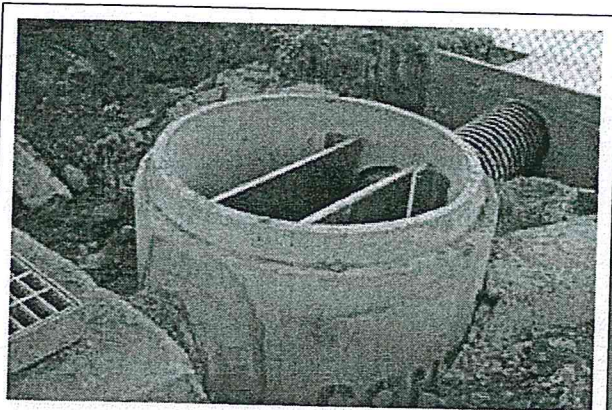
Our HDS products are especially effective for solids and trash and debris. They can be installed at either the inlet or outlet of a drainage system to prevent pollutants from being discharged into lakes, streams or the ocean.



*A Vortechs protects detention system from sediment build-up and reduces maintenance*



*CDS unit installed to remove trash before entering Lake Meritt in Oakland, CA*



*VortSentry HS is an effective option when space is limited*

The CDS is a swirl concentrator hybrid technology that provides continuous deflective separation – a combination of swirl concentration and patented indirect screening – into a uniquely capable product. It effectively screens, separates and traps debris, sediment and oil from stormwater runoff and is an ideal system to meet trash Total Maximum Daily Load (TMDL) requirements.

## Features & Benefits

### One-of-a-Kind Screening Technology

- Captures and retains 100% of floatables and neutrally buoyant debris 2.4mm or larger
- Effectively removes solids down to 100µm
- Self-cleaning screen – the only non-blocking screening technology available
- Water velocities within the swirl chamber continually shear debris off the screen to keep it clean
- Various screening apertures available

### Proven Performance

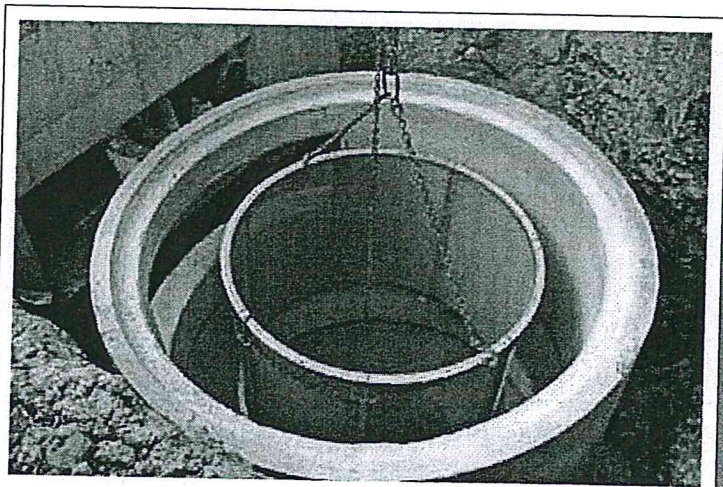
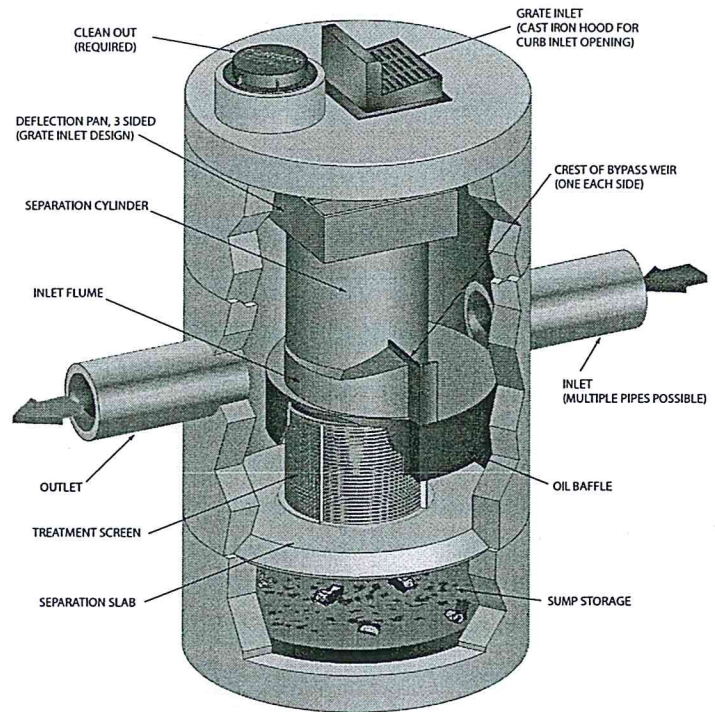
- Performance verified by NJ CAT and WA Ecology

### Excellent Pollutant Retention

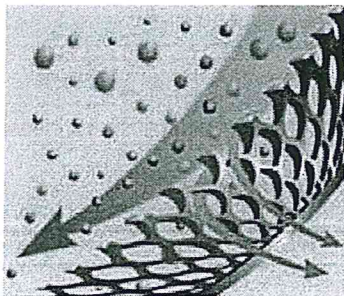
- Isolated Storage Sump eliminates scour potential
- Oil Baffle improves hydrocarbon removal

### Multiple Options to Meet Site-Specific Needs

- Inline, offline, grate inlet and drop inlet configuration
- Accepts multiple pipe inlets and 90-180° angles – eliminate the need for junction manholes
- Internal and external peak bypass options available



*CDS removes fine sediments and trash debris*



Continuous deflective separation — water velocities within the swirl chamber continually shear debris off the screen to keep it clean

# Vortechs®

The Vortechs system's swirl concentrator and flow controls work together to create a low energy environment, ideal for capturing and storing fine particles and other pollutants of concern. With comprehensive lab and field testing, the system delivers proven results and site-specific solutions.

## Features & Benefits

### Shallow Profile

- Easy and cost-effective installation, especially on sites with high groundwater or bedrock
- Typical invert only 3 feet below pipe

### Effective Fine Solids Removal

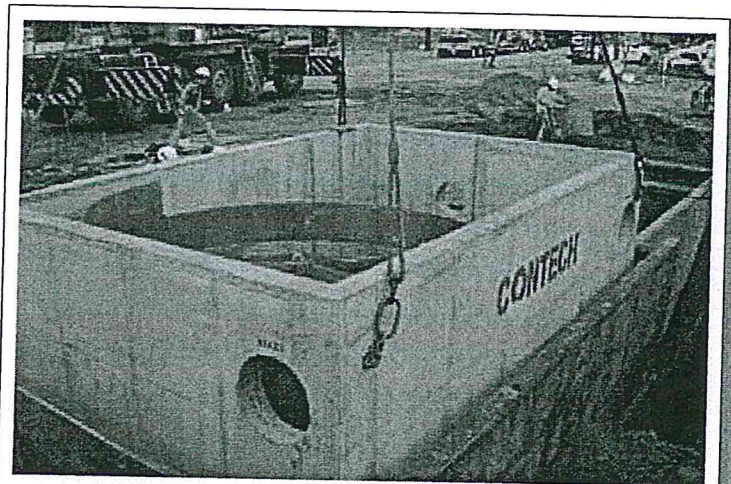
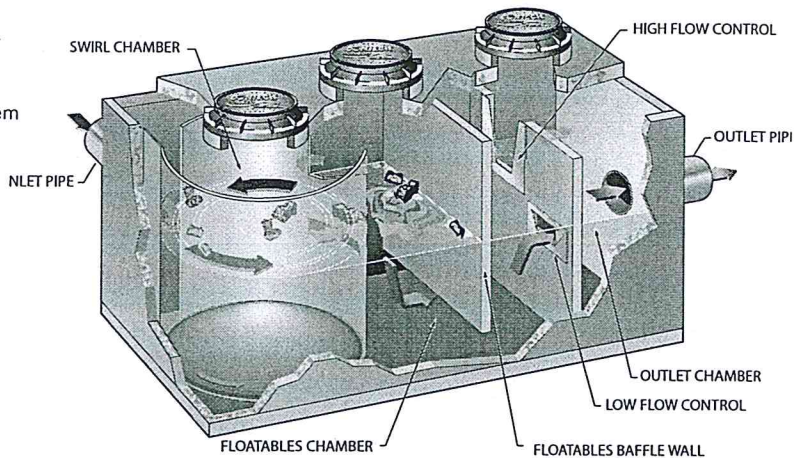
- Large swirl chamber – Enhances very fine particle removal (down to 50 microns)
- Flow controls reduce inflow velocity and increase residence time
- Largest treatment zone surface area of any swirl concentrator system available

### Easy Maintenance

- Unobstructed access to stored pollutants
- Sealed swirl chamber decreases clean-out volume

### Proven Performance

- Performance verified by NJ CAT and WA Ecology



*Large diameter swirl chamber for enhancement of sediment removal in a low profile unit*

Our systems are widely accepted for effective solids removal ❖ ❖ ❖

# VortSentry® HS

The VortSentry HS hydrodynamic separator has a small footprint making it an effective pretreatment or treatment option for projects where space is at a premium.

## Helical Flow Pattern

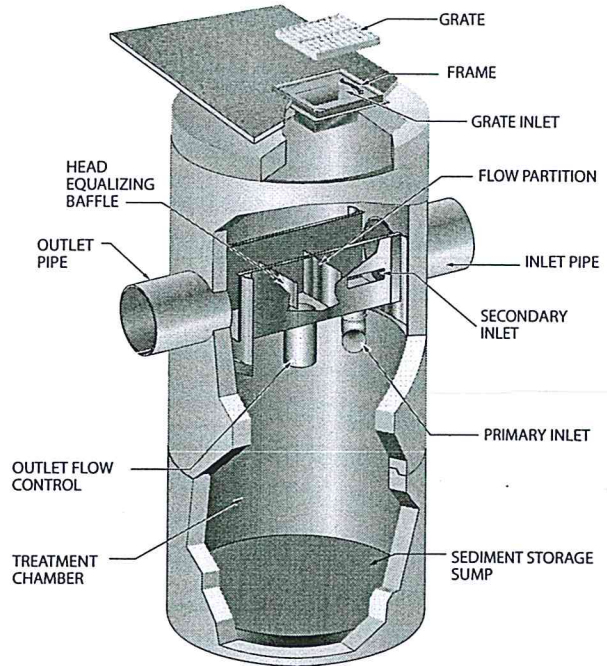
- Enhances trapping and containment of pollutants
- Provides effective removal of settleable solids and floating contaminants

## Unique Internal Bypass

- Accepts a wide range of pipe sizes to treat and convey a wide range of flows
- Higher flows can be diverted without the use of external bypass structures
- Secondary inlet enhances floatable debris capture

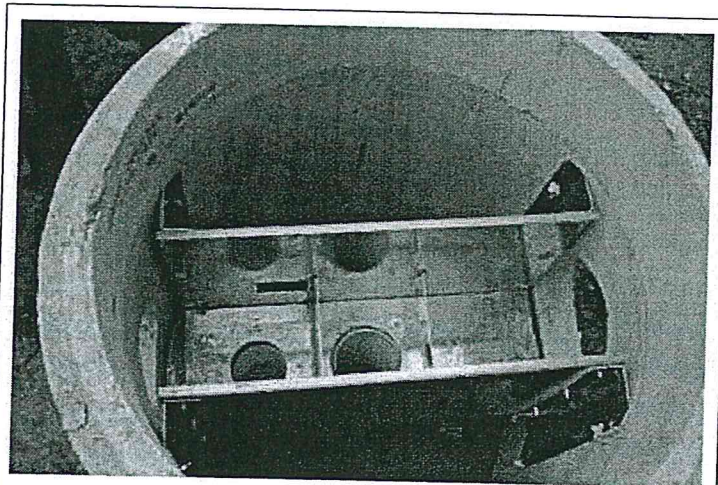
## Flexible, Compact Design

- Small manhole footprint
- Inlet and grated inlet configuration available



## VortSentry®

Similar to the VortSentry HS, the VortSentry is a compact manhole hydrodynamic separator; however it does not have the same treatment flow routing components/controls as the VortSentry HS does. This limits the flow rate and pipe sizes the system can accept. The VortSentry has received approval and is accepted by many municipalities, and is currently available in only those areas. Please see [www.ContechES.com/vortsentry](http://www.ContechES.com/vortsentry) for more information.



*VSHS unique internal bypass design treats high flows and bypasses peak flow, eliminating washout*

# Maintenance

All stormwater treatment systems – whether natural or manufactured – should be maintained regularly. Despite the widespread implementation of BMPs, water quality goals will not be met if the treatment structures are not properly cleaned and maintained.

Systems vary in their maintenance needs, and the selection of a cost-effective and easy-to-access treatment system can mean a huge difference in maintenance expenses for years to come.

We design our products to minimize maintenance and make it as easy and inexpensive as possible to keep our systems working properly.

## Inspection

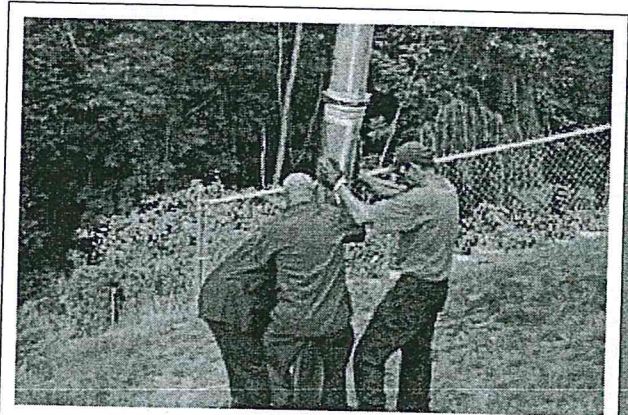
Inspection is the key to effective maintenance. Pollutant deposition and transport may vary from year to year and site to site. Semi-annual inspections will help ensure that the system is cleaned out at the appropriate time. Inspections should be performed more frequently where site conditions may cause rapid accumulation of pollutants.

## Vortechs, VortSentry and VortSentry HS

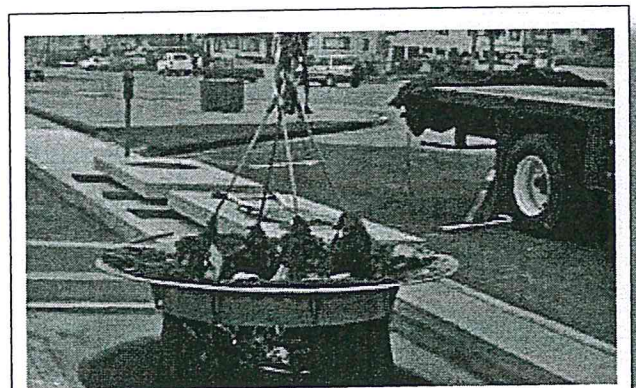
These systems should be cleaned out when sediment has accumulated to a specific depth (refer to the respective maintenance guidelines for details). Maintaining these systems is easiest when there is no flow entering the system. A vacuum truck is generally the most effective and convenient method of excavating pollutants from the systems.

## CDS

The recommended cleanout of solids within the CDS unit's sump should occur at 75% of the sump capacity. Access to the CDS unit is typically achieved through two manhole access covers – one allows inspection and cleanout of the separation chamber and sump, and another allows inspection and cleanout of sediment captured and retained behind the screen. A vacuum truck is recommended for cleanout of the CDS unit and can be easily accomplished in less than 30 minutes for most installations.

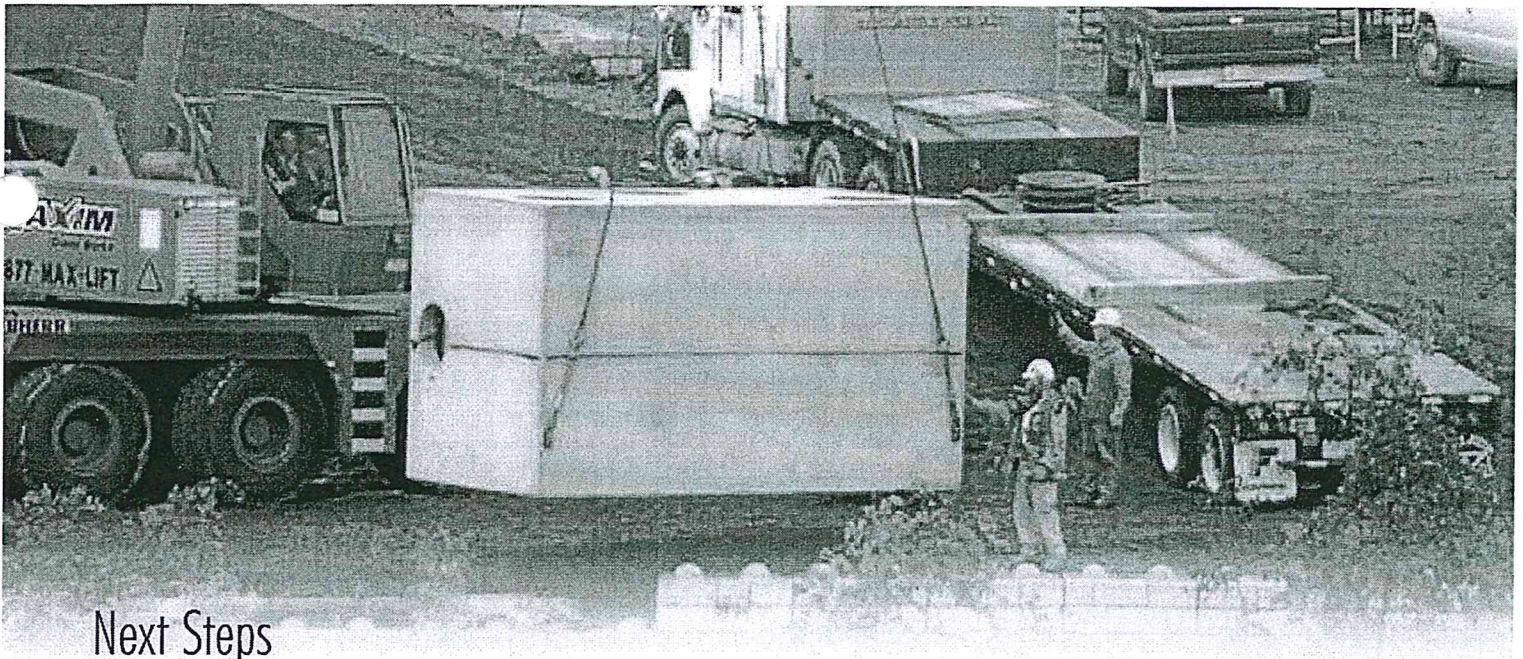


*A vacuum truck excavates pollutants from the systems*



*A CDS unit can be easily cleaned out in less than 30 minutes*

All stormwater systems should be maintained regularly ❖ ❖ ❖



## Next Steps

### Learn more

See our HDS systems in action. Flash animations available at [www.ContechES.com/videos](http://www.ContechES.com/videos)

### Connect with us

We're always available to make your job easier. Contact your local project consultant for design assistance. Search online at [www.ContechES.com](http://www.ContechES.com). While you're there, be sure to check out our upcoming seminar schedule or request an in-house technical presentation.

### Start a Project

If you are ready to begin a project, visit us at [www.ContechES.com/designtoolbox](http://www.ContechES.com/designtoolbox)

Contech Engineered Solutions LLC provides site solutions for the civil engineering industry. Contech's portfolio includes bridges, drainage, retaining walls, sanitary sewer, stormwater, erosion control and soil stabilization products.

For more information, visit our web site: [www.ContechES.com](http://www.ContechES.com) or call 800.338.1122

The product(s) described may be protected by one or more of the following US patents: 5,322,629; 5,624,576; 5,707,527; 5,759,415; 5,788,848; 5,985,157; 6,027,639; 6,350,374; 6,406,218; 6,641,720; 6,511,595; 6,649,048; 6,991,114; 6,998,038; 7,186,058; 7,296,692; 7,297,266 related foreign patents or other patents pending.

The Stormwater Management StormFilter, MFS and CDS are trademarks, registered trademarks, or licensed trademarks of Contech Engineered Solutions LLC.

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FSC

## **ATTACHMENT 6-4**

### **Other Supporting Documentation**

- **BMP Educational Materials**
- **Activity Restrictions- C, C&R's and Lease Agreements**

# Site Design & Landscape Planning SD-10



## Design Objectives

- Maximize Infiltration
- Provide Retention
- Slow Runoff
- Minimize Impervious Land Coverage
- Prohibit Dumping of Improper Materials
- Contain Pollutants
- Collect and Convey

## Description

Each project site possesses unique topographic, hydrologic, and vegetative features, some of which are more suitable for development than others. Integrating and incorporating appropriate landscape planning methodologies into the project design is the most effective action that can be done to minimize surface and groundwater contamination from stormwater.

## Approach

Landscape planning should couple consideration of land suitability for urban uses with consideration of community goals and projected growth. Project plan designs should conserve natural areas to the extent possible, maximize natural water storage and infiltration opportunities, and protect slopes and channels.

## Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment.

## Design Considerations

Design requirements for site design and landscapes planning should conform to applicable standards and specifications of agencies with jurisdiction and be consistent with applicable General Plan and Local Area Plan policies.



# **SD-10 Site Design & Landscape Planning**

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## ***Designing New Installations***

Begin the development of a plan for the landscape unit with attention to the following general principles:

- Formulate the plan on the basis of clearly articulated community goals. Carefully identify conflicts and choices between retaining and protecting desired resources and community growth.
- Map and assess land suitability for urban uses. Include the following landscape features in the assessment: wooded land, open unwooded land, steep slopes, erosion-prone soils, foundation suitability, soil suitability for waste disposal, aquifers, aquifer recharge areas, wetlands, floodplains, surface waters, agricultural lands, and various categories of urban land use. When appropriate, the assessment can highlight outstanding local or regional resources that the community determines should be protected (e.g., a scenic area, recreational area, threatened species habitat, farmland, fish run). Mapping and assessment should recognize not only these resources but also additional areas needed for their sustenance.

Project plan designs should conserve natural areas to the extent possible, maximize natural water storage and infiltration opportunities, and protect slopes and channels.

## ***Conserve Natural Areas during Landscape Planning***

If applicable, the following items are required and must be implemented in the site layout during the subdivision design and approval process, consistent with applicable General Plan and Local Area Plan policies:

- Cluster development on least-sensitive portions of a site while leaving the remaining land in a natural undisturbed condition.
- Limit clearing and grading of native vegetation at a site to the minimum amount needed to build lots, allow access, and provide fire protection.
- Maximize trees and other vegetation at each site by planting additional vegetation, clustering tree areas, and promoting the use of native and/or drought tolerant plants.
- Promote natural vegetation by using parking lot islands and other landscaped areas.
- Preserve riparian areas and wetlands.

## ***Maximize Natural Water Storage and Infiltration Opportunities Within the Landscape Unit***

- Promote the conservation of forest cover. Building on land that is already deforested affects basin hydrology to a lesser extent than converting forested land. Loss of forest cover reduces interception storage, detention in the organic forest floor layer, and water losses by evapotranspiration, resulting in large peak runoff increases and either their negative effects or the expense of countering them with structural solutions.
- Maintain natural storage reservoirs and drainage corridors, including depressions, areas of permeable soils, swales, and intermittent streams. Develop and implement policies and

# Site Design & Landscape Planning SD-10

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regulations to discourage the clearing, filling, and channelization of these features. Utilize them in drainage networks in preference to pipes, culverts, and engineered ditches.

- Evaluating infiltration opportunities by referring to the stormwater management manual for the jurisdiction and pay particular attention to the selection criteria for avoiding groundwater contamination, poor soils, and hydrogeological conditions that cause these facilities to fail. If necessary, locate developments with large amounts of impervious surfaces or a potential to produce relatively contaminated runoff away from groundwater recharge areas.

## *Protection of Slopes and Channels during Landscape Design*

- Convey runoff safely from the tops of slopes.
- Avoid disturbing steep or unstable slopes.
- Avoid disturbing natural channels.
- Stabilize disturbed slopes as quickly as possible.
- Vegetate slopes with native or drought tolerant vegetation.
- Control and treat flows in landscaping and/or other controls prior to reaching existing natural drainage systems.
- Stabilize temporary and permanent channel crossings as quickly as possible, and ensure that increases in run-off velocity and frequency caused by the project do not erode the channel.
- 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.
- Line on-site conveyance channels where appropriate, to reduce erosion caused by increased flow velocity due to increases in tributary impervious area. The first choice for linings should be grass or some other vegetative surface, since these materials not only reduce runoff velocities, but also provide water quality benefits from filtration and infiltration. If velocities in the channel are high enough to erode grass or other vegetative linings, riprap, concrete, soil cement, or geo-grid stabilization are other alternatives.
- Consider other design principles that are comparable and equally effective.

## *Redeveloping Existing Installations*

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define “redevelopment” in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of “redevelopment” must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under “designing new installations” above should be followed.

# **SD-10 Site Design & Landscape Planning**

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Redevelopment may present significant opportunity to add features which had not previously been implemented. Examples include incorporation of depressions, areas of permeable soils, and swales in newly redeveloped areas. While some site constraints may exist due to the status of already existing infrastructure, opportunities should not be missed to maximize infiltration, slow runoff, reduce impervious areas, disconnect directly connected impervious areas.

## **Other Resources**

A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, May 2002.

Stormwater Management Manual for Western Washington, Washington State Department of Ecology, August 2001.

Model Standard Urban Storm Water Mitigation Plan (SUSMP) for San Diego County, Port of San Diego, and Cities in San Diego County, February 14, 2002.

Model Water Quality Management Plan (WQMP) for County of Orange, Orange County Flood Control District, and the Incorporated Cities of Orange County, Draft February 2003.

Ventura Countywide Technical Guidance Manual for Stormwater Quality Control Measures, July 2002.



## Design Objectives

- Maximize Infiltration
- Provide Retention
- Slow Runoff
- Minimize Impervious Land Coverage
- Prohibit Dumping of Improper Materials
- Contain Pollutants
- Collect and Convey

## Description

Irrigation water provided to landscaped areas may result in excess irrigation water being conveyed into stormwater drainage systems.

## Approach

Project plan designs for development and redevelopment should include application methods of irrigation water that minimize runoff of excess irrigation water into the stormwater conveyance system.

## Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment. (Detached residential single-family homes are typically excluded from this requirement.)

## Design Considerations

### *Designing New Installations*

The following methods to reduce excessive irrigation runoff should be considered, and incorporated and implemented where determined applicable and feasible by the Permittee:

- Employ rain-triggered shutoff devices to prevent irrigation after precipitation.
- Design irrigation systems to each landscape area's specific water requirements.
- Include design featuring flow reducers or shutoff valves triggered by a pressure drop to control water loss in the event of broken sprinkler heads or lines.
- Implement landscape plans consistent with County or City water conservation resolutions, which may include provision of water sensors, programmable irrigation times (for short cycles), etc.



- Design timing and application methods of irrigation water to minimize the runoff of excess irrigation water into the storm water drainage system.
- Group plants with similar water requirements in order to reduce excess irrigation runoff and promote surface filtration. Choose plants with low irrigation requirements (for example, native or drought tolerant species). Consider design features such as:
  - Using mulches (such as wood chips or bar) in planter areas without ground cover to minimize sediment in runoff
  - Installing appropriate plant materials for the location, in accordance with amount of sunlight and climate, and use native plant materials where possible and/or as recommended by the landscape architect
  - Leaving a vegetative barrier along the property boundary and interior watercourses, to act as a pollutant filter, where appropriate and feasible
  - Choosing plants that minimize or eliminate the use of fertilizer or pesticides to sustain growth
- Employ other comparable, equally effective methods to reduce irrigation water runoff.

***Redeveloping Existing Installations***

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define “redevelopment” in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of “redevelopment” must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under “designing new installations” above should be followed.

**Other Resources**

A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, May 2002.

Model Standard Urban Storm Water Mitigation Plan (SUSMP) for San Diego County, Port of San Diego, and Cities in San Diego County, February 14, 2002.

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## Design Objectives

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- Prohibit Dumping of Improper Materials
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- Collect and Convey

## Description

Waste materials dumped into storm drain inlets can have severe impacts on receiving and ground waters. Posting notices regarding discharge prohibitions at storm drain inlets can prevent waste dumping. Storm drain signs and stencils are highly visible source controls that are typically placed directly adjacent to storm drain inlets.

## Approach

The stencil or affixed sign contains a brief statement that prohibits dumping of improper materials into the urban runoff conveyance system. Storm drain messages have become a popular method of alerting the public about the effects of and the prohibitions against waste disposal.

## Suitable Applications

Stencils and signs alert the public to the destination of pollutants discharged to the storm drain. Signs are appropriate in residential, commercial, and industrial areas, as well as any other area where contributions or dumping to storm drains is likely.

## Design Considerations

Storm drain message markers or placards are recommended at all storm drain inlets within the boundary of a development project. The marker should be placed in clear sight facing toward anyone approaching the inlet from either side. All storm drain inlet locations should be identified on the development site map.

## Designing New Installations

The following methods should be considered for inclusion in the project design and show on project plans:

- Provide stenciling or labeling of all storm drain inlets and catch basins, constructed or modified, within the project area with prohibitive language. Examples include "NO DUMPING



– DRAINS TO OCEAN” and/or other graphical icons to discourage illegal dumping.

- Post signs with prohibitive language and/or graphical icons, which prohibit illegal dumping at public access points along channels and creeks within the project area.

Note - Some local agencies have approved specific signage and/or storm drain message placards for use. Consult local agency stormwater staff to determine specific requirements for placard types and methods of application.

### ***Redeveloping Existing Installations***

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define “redevelopment” in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. If the project meets the definition of “redevelopment”, then the requirements stated under “designing new installations” above should be included in all project design plans.

### **Additional Information**

#### ***Maintenance Considerations***

- Legibility of markers and signs should be maintained. If required by the agency with jurisdiction over the project, the owner/operator or homeowner’s association should enter into a maintenance agreement with the agency or record a deed restriction upon the property title to maintain the legibility of placards or signs.

#### ***Placement***

- Signage on top of curbs tends to weather and fade.
- Signage on face of curbs tends to be worn by contact with vehicle tires and sweeper brooms.

### **Supplemental Information**

#### ***Examples***

- Most MS4 programs have storm drain signage programs. Some MS4 programs will provide stencils, or arrange for volunteers to stencil storm drains as part of their outreach program.

### **Other Resources**

A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, May 2002.

Model Standard Urban Storm Water Mitigation Plan (SUSMP) for San Diego County, Port of San Diego, and Cities in San Diego County, February 14, 2002.

Model Water Quality Management Plan (WQMP) for County of Orange, Orange County Flood Control District, and the Incorporated Cities of Orange County, Draft February 2003.

Ventura Countywide Technical Guidance Manual for Stormwater Quality Control Measures, July 2002.

## Description

Trash storage areas are areas where a trash receptacle (s) are located for use as a repository for solid wastes. Stormwater runoff from areas where trash is stored or disposed of can be polluted. In addition, loose trash and debris can be easily transported by water or wind into nearby storm drain inlets, channels, and/or creeks. Waste handling operations that may be sources of stormwater pollution include dumpsters, litter control, and waste piles.

## Approach

This fact sheet contains details on the specific measures required to prevent or reduce pollutants in stormwater runoff associated with trash storage and handling. Preventative measures including enclosures, containment structures, and impervious pavements to mitigate spills, should be used to reduce the likelihood of contamination.

## Suitable Applications

Appropriate applications include residential, commercial and industrial areas planned for development or redevelopment. (Detached residential single-family homes are typically excluded from this requirement.)

## Design Considerations

Design requirements for waste handling areas are governed by Building and Fire Codes, and by current local agency ordinances and zoning requirements. The design criteria described in this fact sheet are meant to enhance and be consistent with these code and ordinance requirements. Hazardous waste should be handled in accordance with legal requirements established in Title 22, California Code of Regulation.

Wastes from commercial and industrial sites are typically hauled by either public or commercial carriers that may have design or access requirements for waste storage areas. The design criteria in this fact sheet are recommendations and are not intended to be in conflict with requirements established by the waste hauler. The waste hauler should be contacted prior to the design of your site trash collection areas. Conflicts or issues should be discussed with the local agency.

## Designing New Installations

Trash storage areas should be designed to consider the following structural or treatment control BMPs:

- Design trash container areas so that drainage from adjoining roofs and pavement is diverted around the area(s) to avoid run-on. This might include berming or grading the waste handling area to prevent run-on of stormwater.
- Make sure trash container areas are screened or walled to prevent off-site transport of trash.

## Design Objectives

Maximize Infiltration

Provide Retention

Slow Runoff

Minimize Impervious Land Coverage

Prohibit Dumping of Improper Materials

Contain Pollutants

Collect and Convey



- Use lined bins or dumpsters to reduce leaking of liquid waste.
- Provide roofs, awnings, or attached lids on all trash containers to minimize direct precipitation and prevent rainfall from entering containers.
- Pave trash storage areas with an impervious surface to mitigate spills.
- Do not locate storm drains in immediate vicinity of the trash storage area.
- Post signs on all dumpsters informing users that hazardous materials are not to be disposed of therein.

***Redeveloping Existing Installations***

Various jurisdictional stormwater management and mitigation plans (SUSMP, WQMP, etc.) define “redevelopment” in terms of amounts of additional impervious area, increases in gross floor area and/or exterior construction, and land disturbing activities with structural or impervious surfaces. The definition of “redevelopment” must be consulted to determine whether or not the requirements for new development apply to areas intended for redevelopment. If the definition applies, the steps outlined under “designing new installations” above should be followed.

**Additional Information*****Maintenance Considerations***

The integrity of structural elements that are subject to damage (i.e., screens, covers, and signs) must be maintained by the owner/operator. Maintenance agreements between the local agency and the owner/operator may be required. Some agencies will require maintenance deed restrictions to be recorded of the property title. If required by the local agency, maintenance agreements or deed restrictions must be executed by the owner/operator before improvement plans are approved.

**Other Resources**

A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP), Los Angeles County Department of Public Works, May 2002.

Model Standard Urban Storm Water Mitigation Plan (SUSMP) for San Diego County, Port of San Diego, and Cities in San Diego County, February 14, 2002.

Model Water Quality Management Plan (WQMP) for County of Orange, Orange County Flood Control District, and the Incorporated Cities of Orange County, Draft February 2003.

Ventura Countywide Technical Guidance Manual for Stormwater Quality Control Measures, July 2002.



## Design Considerations

- Soil for Infiltration
- Tributary Area
- Slope
- Aesthetics
- Environmental Side-effects

## Description

The bioretention best management practice (BMP) functions as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. These facilities normally consist of a grass buffer strip, sand bed, ponding area, organic layer or mulch layer, planting soil, and plants. The runoff's velocity is reduced by passing over or through buffer strip and subsequently distributed evenly along a ponding area. Exfiltration of the stored water in the bioretention area planting soil into the underlying soils occurs over a period of days.

## California Experience

None documented. Bioretention has been used as a stormwater BMP since 1992. In addition to Prince George's County, MD and Alexandria, VA, bioretention has been used successfully at urban and suburban areas in Montgomery County, MD; Baltimore County, MD; Chesterfield County, VA; Prince William County, VA; Smith Mountain Lake State Park, VA; and Cary, NC.

## Advantages

- Bioretention provides stormwater treatment that enhances the quality of downstream water bodies by temporarily storing runoff in the BMP and releasing it over a period of four days to the receiving water (EPA, 1999).
- The vegetation provides shade and wind breaks, absorbs noise, and improves an area's landscape.

## Limitations

- The bioretention BMP is not recommended for areas with slopes greater than 20% or where mature tree removal would

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	■
<input checked="" type="checkbox"/>	Nutrients	▲
<input checked="" type="checkbox"/>	Trash	■
<input checked="" type="checkbox"/>	Metals	■
<input checked="" type="checkbox"/>	Bacteria	■
<input checked="" type="checkbox"/>	Oil and Grease	■
<input checked="" type="checkbox"/>	Organics	■

## Legend (Removal Effectiveness)

- Low
- High
- ▲ Medium



be required since clogging may result, particularly if the BMP receives runoff with high sediment loads (EPA, 1999).

- Bioretention is not a suitable BMP at locations where the water table is within 6 feet of the ground surface and where the surrounding soil stratum is unstable.
- By design, bioretention BMPs have the potential to create very attractive habitats for mosquitoes and other vectors because of highly organic, often heavily vegetated areas mixed with shallow water.
- In cold climates the soil may freeze, preventing runoff from infiltrating into the planting soil.

### **Design and Sizing Guidelines**

- The bioretention area should be sized to capture the design storm runoff.
- In areas where the native soil permeability is less than 0.5 in/hr an underdrain should be provided.
- Recommended minimum dimensions are 15 feet by 40 feet, although the preferred width is 25 feet. Excavated depth should be 4 feet.
- Area should drain completely within 72 hours.
- Approximately 1 tree or shrub per 50 ft<sup>2</sup> of bioretention area should be included.
- Cover area with about 3 inches of mulch.

### **Construction/Inspection Considerations**

Bioretention area should not be established until contributing watershed is stabilized.

### **Performance**

Bioretention removes stormwater pollutants through physical and biological processes, including adsorption, filtration, plant uptake, microbial activity, decomposition, sedimentation and volatilization (EPA, 1999). Adsorption is the process whereby particulate pollutants attach to soil (e.g., clay) or vegetation surfaces. Adequate contact time between the surface and pollutant must be provided for in the design of the system for this removal process to occur. Thus, the infiltration rate of the soils must not exceed those specified in the design criteria or pollutant removal may decrease. Pollutants removed by adsorption include metals, phosphorus, and hydrocarbons. Filtration occurs as runoff passes through the bioretention area media, such as the sand bed, ground cover, and planting soil.

Common particulates removed from stormwater include particulate organic matter, phosphorus, and suspended solids. Biological processes that occur in wetlands result in pollutant uptake by plants and microorganisms in the soil. Plant growth is sustained by the uptake of nutrients from the soils, with woody plants locking up these nutrients through the seasons. Microbial activity within the soil also contributes to the removal of nitrogen and organic matter. Nitrogen is removed by nitrifying and denitrifying bacteria, while aerobic bacteria are responsible for the decomposition of the organic matter. Microbial processes require oxygen and can result in depleted oxygen levels if the bioretention area is not adequately

aerated. Sedimentation occurs in the swale or ponding area as the velocity slows and solids fall out of suspension.

The removal effectiveness of bioretention has been studied during field and laboratory studies conducted by the University of Maryland (Davis et al, 1998). During these experiments, synthetic stormwater runoff was pumped through several laboratory and field bioretention areas to simulate typical storm events in Prince George's County, MD. Removal rates for heavy metals and nutrients are shown in Table 1.

<b>Pollutant</b>	<b>Removal Rate</b>
Total Phosphorus	70-83%
Metals (Cu, Zn, Pb)	93-98%
TKN	68-80%
Total Suspended Solids	90%
Organics	90%
Bacteria	90%

Results for both the laboratory and field experiments were similar for each of the pollutants analyzed. Doubling or halving the influent pollutant levels had little effect on the effluent pollutants concentrations (Davis et al, 1998).

The microbial activity and plant uptake occurring in the bioretention area will likely result in higher removal rates than those determined for infiltration BMPs.

### **Siting Criteria**

Bioretention BMPs are generally used to treat stormwater from impervious surfaces at commercial, residential, and industrial areas (EPA, 1999). Implementation of bioretention for stormwater management is ideal for median strips, parking lot islands, and swales. Moreover, the runoff in these areas can be designed to either divert directly into the bioretention area or convey into the bioretention area by a curb and gutter collection system.

The best location for bioretention areas is upland from inlets that receive sheet flow from graded areas and at areas that will be excavated (EPA, 1999). In order to maximize treatment effectiveness, the site must be graded in such a way that minimizes erosive conditions as sheet flow is conveyed to the treatment area. Locations where a bioretention area can be readily incorporated into the site plan without further environmental damage are preferred. Furthermore, to effectively minimize sediment loading in the treatment area, bioretention only should be used in stabilized drainage areas.

### Additional Design Guidelines

The layout of the bioretention area is determined after site constraints such as location of utilities, underlying soils, existing vegetation, and drainage are considered (EPA, 1999). Sites with loamy sand soils are especially appropriate for bioretention because the excavated soil can be backfilled and used as the planting soil, thus eliminating the cost of importing planting soil.

The use of bioretention may not be feasible given an unstable surrounding soil stratum, soils with clay content greater than 25 percent, a site with slopes greater than 20 percent, and/or a site with mature trees that would be removed during construction of the BMP.

Bioretention can be designed to be off-line or on-line of the existing drainage system (EPA, 1999). The drainage area for a bioretention area should be between 0.1 and 0.4 hectares (0.25 and 1.0 acres). Larger drainage areas may require multiple bioretention areas. Furthermore, the maximum drainage area for a bioretention area is determined by the expected rainfall intensity and runoff rate. Stabilized areas may erode when velocities are greater than 5 feet per second (1.5 meter per second). The designer should determine the potential for erosive conditions at the site.

The size of the bioretention area, which is a function of the drainage area and the runoff generated from the area is sized to capture the water quality volume.

The recommended minimum dimensions of the bioretention area are 15 feet (4.6 meters) wide by 40 feet (12.2 meters) long, where the minimum width allows enough space for a dense, randomly-distributed area of trees and shrubs to become established. Thus replicating a natural forest and creating a microclimate, thereby enabling the bioretention area to tolerate the effects of heat stress, acid rain, runoff pollutants, and insect and disease infestations which landscaped areas in urban settings typically are unable to tolerate. The preferred width is 25 feet (7.6 meters), with a length of twice the width. Essentially, any facilities wider than 20 feet (6.1 meters) should be twice as long as they are wide, which promotes the distribution of flow and decreases the chances of concentrated flow.

In order to provide adequate storage and prevent water from standing for excessive periods of time the ponding depth of the bioretention area should not exceed 6 inches (15 centimeters). Water should not be left to stand for more than 72 hours. A restriction on the type of plants that can be used may be necessary due to some plants' water intolerance. Furthermore, if water is left standing for longer than 72 hours mosquitoes and other insects may start to breed.

The appropriate planting soil should be backfilled into the excavated bioretention area. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content ranging from 10 to 25 percent.

Generally the soil should have infiltration rates greater than 0.5 inches (1.25 centimeters) per hour, which is typical of sandy loams, loamy sands, or loams. The pH of the soil should range between 5.5 and 6.5, where pollutants such as organic nitrogen and phosphorus can be adsorbed by the soil and microbial activity can flourish. Additional requirements for the planting soil include a 1.5 to 3 percent organic content and a maximum 500 ppm concentration of soluble salts.

Soil tests should be performed for every 500 cubic yards (382 cubic meters) of planting soil, with the exception of pH and organic content tests, which are required only once per bioretention area (EPA, 1999). Planting soil should be 4 inches (10.1 centimeters) deeper than the bottom of the largest root ball and 4 feet (1.2 meters) altogether. This depth will provide adequate soil for the plants' root systems to become established, prevent plant damage due to severe wind, and provide adequate moisture capacity. Most sites will require excavation in order to obtain the recommended depth.

Planting soil depths of greater than 4 feet (1.2 meters) may require additional construction practices such as shoring measures (EPA, 1999). Planting soil should be placed in 18 inches or greater lifts and lightly compacted until the desired depth is reached. Since high canopy trees may be destroyed during maintenance the bioretention area should be vegetated to resemble a terrestrial forest community ecosystem that is dominated by understory trees. Three species each of both trees and shrubs are recommended to be planted at a rate of 2500 trees and shrubs per hectare (1000 per acre). For instance, a 15 foot (4.6 meter) by 40 foot (12.2 meter) bioretention area (600 square feet or 55.75 square meters) would require 14 trees and shrubs. The shrub-to-tree ratio should be 2:1 to 3:1.

Trees and shrubs should be planted when conditions are favorable. Vegetation should be watered at the end of each day for fourteen days following its planting. Plant species tolerant of pollutant loads and varying wet and dry conditions should be used in the bioretention area.

The designer should assess aesthetics, site layout, and maintenance requirements when selecting plant species. Adjacent non-native invasive species should be identified and the designer should take measures, such as providing a soil breach to eliminate the threat of these species invading the bioretention area. Regional landscaping manuals should be consulted to ensure that the planting of the bioretention area meets the landscaping requirements established by the local authorities. The designers should evaluate the best placement of vegetation within the bioretention area. Plants should be placed at irregular intervals to replicate a natural forest. Trees should be placed on the perimeter of the area to provide shade and shelter from the wind. Trees and shrubs can be sheltered from damaging flows if they are placed away from the path of the incoming runoff. In cold climates, species that are more tolerant to cold winds, such as evergreens, should be placed in windier areas of the site.

Following placement of the trees and shrubs, the ground cover and/or mulch should be established. Ground cover such as grasses or legumes can be planted at the beginning of the growing season. Mulch should be placed immediately after trees and shrubs are planted. Two to 3 inches (5 to 7.6 cm) of commercially-available fine shredded hardwood mulch or shredded hardwood chips should be applied to the bioretention area to protect from erosion.

## Maintenance

The primary maintenance requirement for bioretention areas is that of inspection and repair or replacement of the treatment area's components. Generally, this involves nothing more than the routine periodic maintenance that is required of any landscaped area. Plants that are appropriate for the site, climatic, and watering conditions should be selected for use in the bioretention cell. Appropriately selected plants will aide in reducing fertilizer, pesticide, water, and overall maintenance requirements. Bioretention system components should blend over time through plant and root growth, organic decomposition, and the development of a natural

soil horizon. These biologic and physical processes over time will lengthen the facility's life span and reduce the need for extensive maintenance.

Routine maintenance should include a biannual health evaluation of the trees and shrubs and subsequent removal of any dead or diseased vegetation (EPA, 1999). Diseased vegetation should be treated as needed using preventative and low-toxic measures to the extent possible. BMPs have the potential to create very attractive habitats for mosquitoes and other vectors because of highly organic, often heavily vegetated areas mixed with shallow water. Routine inspections for areas of standing water within the BMP and corrective measures to restore proper infiltration rates are necessary to prevent creating mosquito and other vector habitat. In addition, bioretention BMPs are susceptible to invasion by aggressive plant species such as cattails, which increase the chances of water standing and subsequent vector production if not routinely maintained.

In order to maintain the treatment area's appearance it may be necessary to prune and weed. Furthermore, mulch replacement is suggested when erosion is evident or when the site begins to look unattractive. Specifically, the entire area may require mulch replacement every two to three years, although spot mulching may be sufficient when there are random void areas. Mulch replacement should be done prior to the start of the wet season.

New Jersey's Department of Environmental Protection states in their bioretention systems standards that accumulated sediment and debris removal (especially at the inflow point) will normally be the primary maintenance function. Other potential tasks include replacement of dead vegetation, soil pH regulation, erosion repair at inflow points, mulch replenishment, unclogging the underdrain, and repairing overflow structures. There is also the possibility that the cation exchange capacity of the soils in the cell will be significantly reduced over time. Depending on pollutant loads, soils may need to be replaced within 5-10 years of construction (LID, 2000).

## **Cost**

### ***Construction Cost***

Construction cost estimates for a bioretention area are slightly greater than those for the required landscaping for a new development (EPA, 1999). A general rule of thumb (Coffman, 1999) is that residential bioretention areas average about \$3 to \$4 per square foot, depending on soil conditions and the density and types of plants used. Commercial, industrial and institutional site costs can range between \$10 to \$40 per square foot, based on the need for control structures, curbing, storm drains and underdrains.

Retrofitting a site typically costs more, averaging \$6,500 per bioretention area. The higher costs are attributed to the demolition of existing concrete, asphalt, and existing structures and the replacement of fill material with planting soil. The costs of retrofitting a commercial site in Maryland, Kettering Development, with 15 bioretention areas were estimated at \$111,600.

In any bioretention area design, the cost of plants varies substantially and can account for a significant portion of the expenditures. While these cost estimates are slightly greater than those of typical landscaping treatment (due to the increased number of plantings, additional soil excavation, backfill material, use of underdrains etc.), those landscaping expenses that would be required regardless of the bioretention installation should be subtracted when determining the net cost.

Perhaps of most importance, however, the cost savings compared to the use of traditional structural stormwater conveyance systems makes bioretention areas quite attractive financially. For example, the use of bioretention can decrease the cost required for constructing stormwater conveyance systems at a site. A medical office building in Maryland was able to reduce the amount of storm drain pipe that was needed from 800 to 230 feet - a cost savings of \$24,000 (PGDER, 1993). And a new residential development spent a total of approximately \$100,000 using bioretention cells on each lot instead of nearly \$400,000 for the traditional stormwater ponds that were originally planned (Rappahanock, ). Also, in residential areas, stormwater management controls become a part of each property owner's landscape, reducing the public burden to maintain large centralized facilities.

### ***Maintenance Cost***

The operation and maintenance costs for a bioretention facility will be comparable to those of typical landscaping required for a site. Costs beyond the normal landscaping fees will include the cost for testing the soils and may include costs for a sand bed and planting soil.

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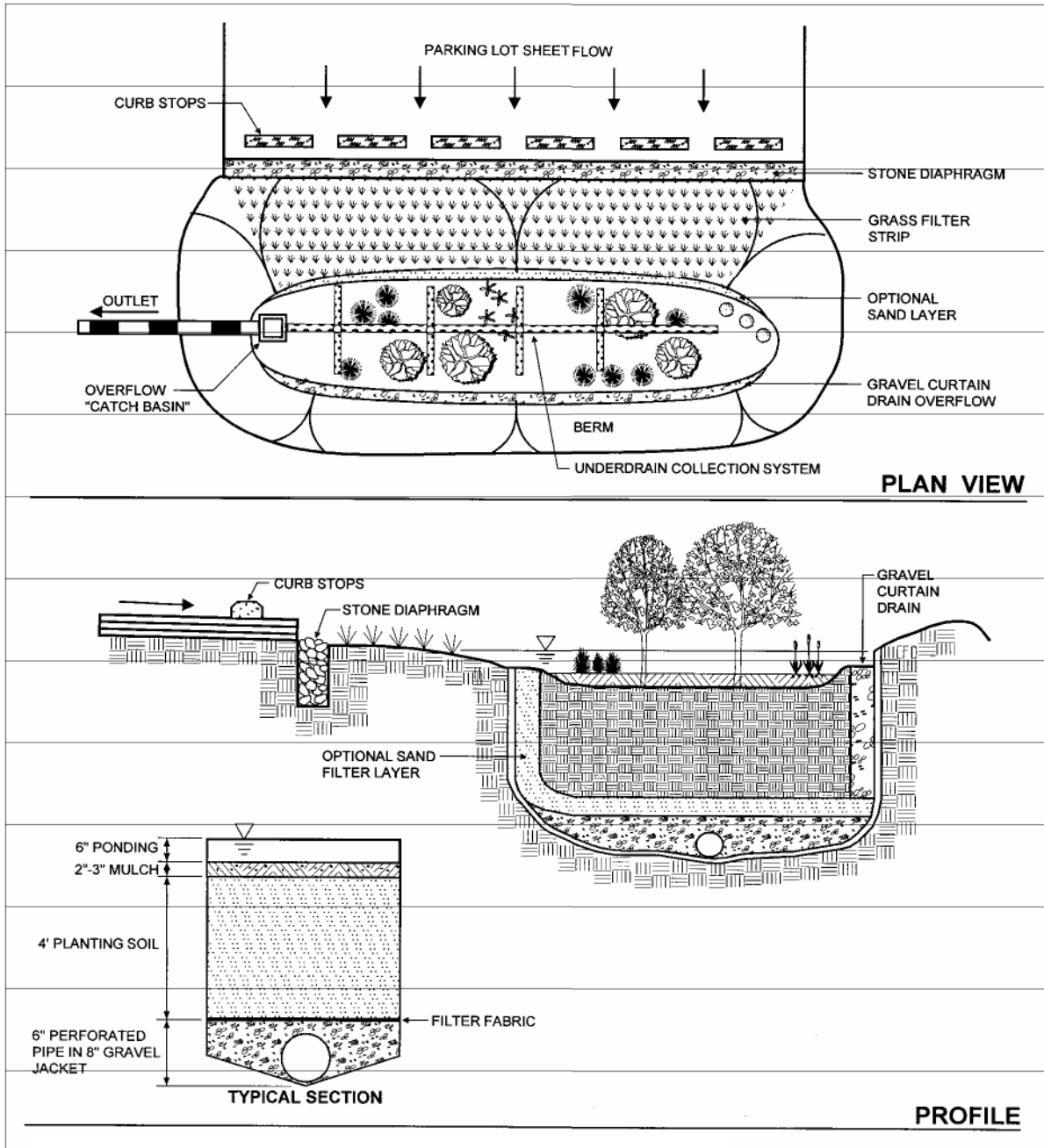
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Schematic of a Bioretention Facility (MDE, 2000)



## Description

An infiltration basin is a shallow impoundment that is designed to infiltrate stormwater. Infiltration basins use the natural filtering ability of the soil to remove pollutants in stormwater runoff. Infiltration facilities store runoff until it gradually exfiltrates through the soil and eventually into the water table. This practice has high pollutant removal efficiency and can also help recharge groundwater, thus helping to maintain low flows in stream systems. Infiltration basins can be challenging to apply on many sites, however, because of soils requirements. In addition, some studies have shown relatively high failure rates compared with other management practices.

## California Experience

Infiltration basins have a long history of use in California, especially in the Central Valley. Basins located in Fresno were among those initially evaluated in the National Urban Runoff Program and were found to be effective at reducing the volume of runoff, while posing little long-term threat to groundwater quality (EPA, 1983; Schroeder, 1995). Proper siting of these devices is crucial as underscored by the experience of Caltrans in siting two basins in Southern California. The basin with marginal separation from groundwater and soil permeability failed immediately and could never be rehabilitated.

## Advantages

- Provides 100% reduction in the load discharged to surface waters.
- The principal benefit of infiltration basins is the approximation of pre-development hydrology during which a

## Design Considerations

- Soil for Infiltration
- Slope
- Aesthetics

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	■
<input checked="" type="checkbox"/>	Nutrients	■
<input checked="" type="checkbox"/>	Trash	■
<input checked="" type="checkbox"/>	Metals	■
<input checked="" type="checkbox"/>	Bacteria	■
<input checked="" type="checkbox"/>	Oil and Grease	■
<input checked="" type="checkbox"/>	Organics	■

### Legend (Removal Effectiveness)

- Low                      ■ High  
▲ Medium



significant portion of the average annual rainfall runoff is infiltrated and evaporated rather than flushed directly to creeks.

- If the water quality volume is adequately sized, infiltration basins can be useful for providing control of channel forming (erosion) and high frequency (generally less than the 2-year) flood events.

**Limitations**

- May not be appropriate for industrial sites or locations where spills may occur.
- Infiltration basins require a minimum soil infiltration rate of 0.5 inches/hour, not appropriate at sites with Hydrologic Soil Types C and D.
- If infiltration rates exceed 2.4 inches/hour, then the runoff should be fully treated prior to infiltration to protect groundwater quality.
- Not suitable on fill sites or steep slopes.
- Risk of groundwater contamination in very coarse soils.
- Upstream drainage area must be completely stabilized before construction.
- Difficult to restore functioning of infiltration basins once clogged.

**Design and Sizing Guidelines**

- Water quality volume determined by local requirements or sized so that 85% of the annual runoff volume is captured.
- Basin sized so that the entire water quality volume is infiltrated within 48 hours.
- Vegetation establishment on the basin floor may help reduce the clogging rate.

**Construction/Inspection Considerations**

- Before construction begins, stabilize the entire area draining to the facility. If impossible, place a diversion berm around the perimeter of the infiltration site to prevent sediment entrance during construction or remove the top 2 inches of soil after the site is stabilized. Stabilize the entire contributing drainage area, including the side slopes, before allowing any runoff to enter once construction is complete.
- Place excavated material such that it can not be washed back into the basin if a storm occurs during construction of the facility.
- Build the basin without driving heavy equipment over the infiltration surface. Any equipment driven on the surface should have extra-wide ("low pressure") tires. Prior to any construction, rope off the infiltration area to stop entrance by unwanted equipment.
- After final grading, till the infiltration surface deeply.
- Use appropriate erosion control seed mix for the specific project and location.

## Performance

As water migrates through porous soil and rock, pollutant attenuation mechanisms include precipitation, sorption, physical filtration, and bacterial degradation. If functioning properly, this approach is presumed to have high removal efficiencies for particulate pollutants and moderate removal of soluble pollutants. Actual pollutant removal in the subsurface would be expected to vary depending upon site-specific soil types. This technology eliminates discharge to surface waters except for the very largest storms; consequently, complete removal of all stormwater constituents can be assumed.

There remain some concerns about the potential for groundwater contamination despite the findings of the NURP and Nightingale (1975; 1987a,b,c; 1989). For instance, a report by Pitt et al. (1994) highlighted the potential for groundwater contamination from intentional and unintentional stormwater infiltration. That report recommends that infiltration facilities not be sited in areas where high concentrations are present or where there is a potential for spills of toxic material. Conversely, Schroeder (1995) reported that there was no evidence of groundwater impacts from an infiltration basin serving a large industrial catchment in Fresno, CA.

## Siting Criteria

The key element in siting infiltration basins is identifying sites with appropriate soil and hydrogeologic properties, which is critical for long term performance. In one study conducted in Prince George's County, Maryland (Galli, 1992), all of the infiltration basins investigated clogged within 2 years. It is believed that these failures were for the most part due to allowing infiltration at sites with rates of less than 0.5 in/hr, basing siting on soil type rather than field infiltration tests, and poor construction practices that resulted in soil compaction of the basin invert.

A study of 23 infiltration basins in the Pacific Northwest showed better long-term performance in an area with highly permeable soils (Hilding, 1996). In this study, few of the infiltration basins had failed after 10 years. Consequently, the following guidelines for identifying appropriate soil and subsurface conditions should be rigorously adhered to.

- Determine soil type (consider RCS soil type 'A, B or C' only) from mapping and consult USDA soil survey tables to review other parameters such as the amount of silt and clay, presence of a restrictive layer or seasonal high water table, and estimated permeability. The soil should not have more than 30% clay or more than 40% of clay and silt combined. Eliminate sites that are clearly unsuitable for infiltration.
- Groundwater separation should be at least 3 m from the basin invert to the measured ground water elevation. There is concern at the state and regional levels of the impact on groundwater quality from infiltrated runoff, especially when the separation between groundwater and the surface is small.
- Location away from buildings, slopes and highway pavement (greater than 6 m) and wells and bridge structures (greater than 30 m). Sites constructed of fill, having a base flow or with a slope greater than 15% should not be considered.
- Ensure that adequate head is available to operate flow splitter structures (to allow the basin to be offline) without ponding in the splitter structure or creating backwater upstream of the splitter.

- Base flow should not be present in the tributary watershed.

### **Secondary Screening Based on Site Geotechnical Investigation**

- At least three in-hole conductivity tests shall be performed using USBR 7300-89 or Bouwer-Rice procedures (the latter if groundwater is encountered within the boring), two tests at different locations within the proposed basin and the third down gradient by no more than approximately 10 m. The tests shall measure permeability in the side slopes and the bed within a depth of 3 m of the invert.
- The minimum acceptable hydraulic conductivity as measured in any of the three required test holes is 13 mm/hr. If any test hole shows less than the minimum value, the site should be disqualified from further consideration.
- Exclude from consideration sites constructed in fill or partially in fill unless no silts or clays are present in the soil boring. Fill tends to be compacted, with clays in a dispersed rather than flocculated state, greatly reducing permeability.
- The geotechnical investigation should be such that a good understanding is gained as to how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water.

### **Additional Design Guidelines**

- (1) Basin Sizing - The required water quality volume is determined by local regulations or sufficient to capture 85% of the annual runoff.
- (2) Provide pretreatment if sediment loading is a maintenance concern for the basin.
- (3) Include energy dissipation in the inlet design for the basins. Avoid designs that include a permanent pool to reduce opportunity for standing water and associated vector problems.
- (4) Basin invert area should be determined by the equation:

$$A = \frac{WQV}{kt}$$

where A = Basin invert area (m<sup>2</sup>)

WQV = water quality volume (m<sup>3</sup>)

k = 0.5 times the lowest field-measured hydraulic conductivity (m/hr)

t = drawdown time ( 48 hr)

- (5) The use of vertical piping, either for distribution or infiltration enhancement shall not be allowed to avoid device classification as a Class V injection well per 40 CFR146.5(e)(4).

## Maintenance

Regular maintenance is critical to the successful operation of infiltration basins. Recommended operation and maintenance guidelines include:

- Inspections and maintenance to ensure that water infiltrates into the subsurface completely (recommended infiltration rate of 72 hours or less) and that vegetation is carefully managed to prevent creating mosquito and other vector habitats.
- Observe drain time for the design storm after completion or modification of the facility to confirm that the desired drain time has been obtained.
- Schedule semiannual inspections for beginning and end of the wet season to identify potential problems such as erosion of the basin side slopes and invert, standing water, trash and debris, and sediment accumulation.
- Remove accumulated trash and debris in the basin at the start and end of the wet season.
- Inspect for standing water at the end of the wet season.
- Trim vegetation at the beginning and end of the wet season to prevent establishment of woody vegetation and for aesthetic and vector reasons.
- Remove accumulated sediment and regrade when the accumulated sediment volume exceeds 10% of the basin.
- If erosion is occurring within the basin, revegetate immediately and stabilize with an erosion control mulch or mat until vegetation cover is established.
- To avoid reversing soil development, scarification or other disturbance should only be performed when there are actual signs of clogging, rather than on a routine basis. Always remove deposited sediments before scarification, and use a hand-guided rotary tiller, if possible, or a disc harrow pulled by a very light tractor.

## Cost

Infiltration basins are relatively cost-effective practices because little infrastructure is needed when constructing them. One study estimated the total construction cost at about \$2 per ft (adjusted for inflation) of storage for a 0.25-acre basin (SWRPC, 1991). As with other BMPs, these published cost estimates may deviate greatly from what might be incurred at a specific site. For instance, Caltrans spent about \$18/ft<sup>3</sup> for the two infiltration basins constructed in southern California, each of which had a water quality volume of about 0.34 ac.-ft. Much of the higher cost can be attributed to changes in the storm drain system necessary to route the runoff to the basin locations.

Infiltration basins typically consume about 2 to 3% of the site draining to them, which is relatively small. Additional space may be required for buffer, landscaping, access road, and fencing. Maintenance costs are estimated at 5 to 10% of construction costs.

One cost concern associated with infiltration practices is the maintenance burden and longevity. If improperly maintained, infiltration basins have a high failure rate. Thus, it may be necessary to replace the basin with a different technology after a relatively short period of time.

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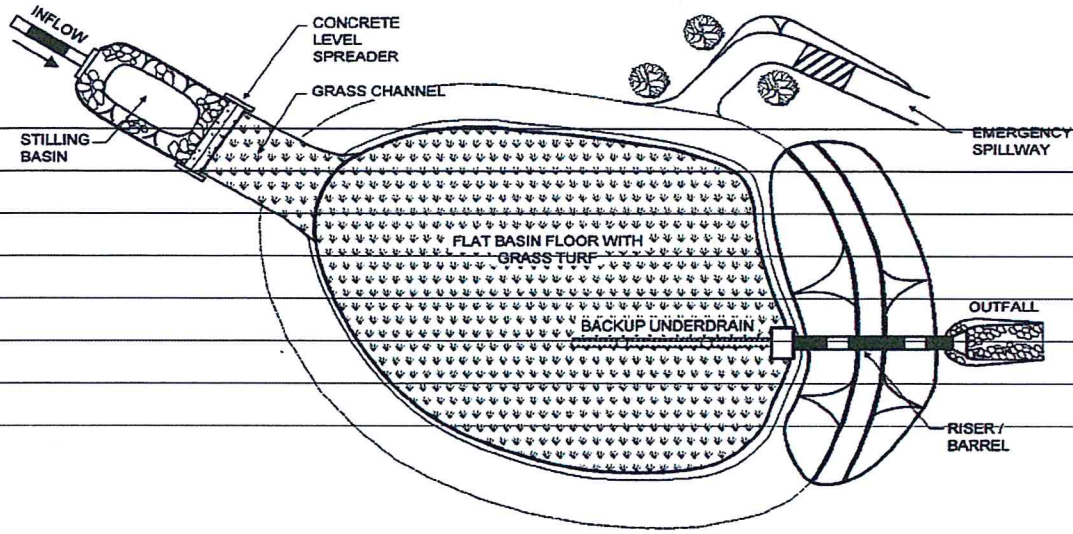
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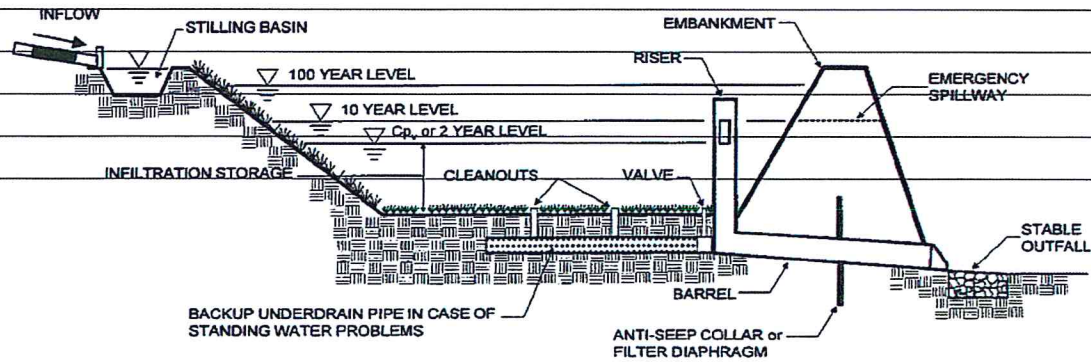
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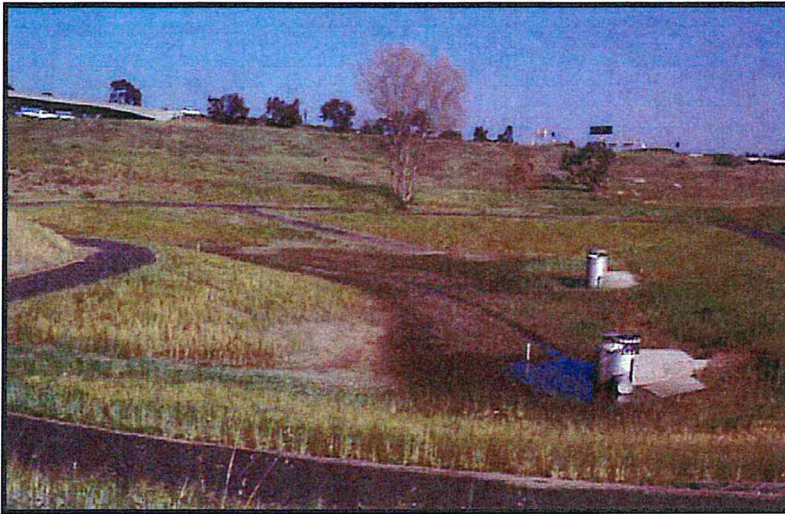
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PLAN VIEW



PROFILE



## Description

Dry extended detention ponds (a.k.a. dry ponds, extended detention basins, detention ponds, extended detention ponds) are basins whose outlets have been designed to detain the stormwater runoff from a water quality design storm for some minimum time (e.g., 48 hours) to allow particles and associated pollutants to settle. Unlike wet ponds, these facilities do not have a large permanent pool. They can also be used to provide flood control by including additional flood detention storage.

## California Experience

Caltrans constructed and monitored 5 extended detention basins in southern California with design drain times of 72 hours. Four of the basins were earthen, less costly and had substantially better load reduction because of infiltration that occurred, than the concrete basin. The Caltrans study reaffirmed the flexibility and performance of this conventional technology. The small headloss and few siting constraints suggest that these devices are one of the most applicable technologies for stormwater treatment.

## Advantages

- Due to the simplicity of design, extended detention basins are relatively easy and inexpensive to construct and operate.
- Extended detention basins can provide substantial capture of sediment and the toxics fraction associated with particulates.
- Widespread application with sufficient capture volume can provide significant control of channel erosion and enlargement caused by changes to flow frequency

## Design Considerations

- Tributary Area
- Area Required
- Hydraulic Head

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	▲
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	■
<input checked="" type="checkbox"/>	Metals	▲
<input checked="" type="checkbox"/>	Bacteria	▲
<input checked="" type="checkbox"/>	Oil and Grease	▲
<input checked="" type="checkbox"/>	Organics	▲

## Legend (Removal Effectiveness)

- |   |        |   |      |
|---|--------|---|------|
| ● | Low    | ■ | High |
| ▲ | Medium |   |      |



relationships resulting from the increase of impervious cover in a watershed.

### Limitations

- Limitation of the diameter of the orifice may not allow use of extended detention in watersheds of less than 5 acres (would require an orifice with a diameter of less than 0.5 inches that would be prone to clogging).
- Dry extended detention ponds have only moderate pollutant removal when compared to some other structural stormwater practices, and they are relatively ineffective at removing soluble pollutants.
- Although wet ponds can increase property values, dry ponds can actually detract from the value of a home due to the adverse aesthetics of dry, bare areas and inlet and outlet structures.

### Design and Sizing Guidelines

- Capture volume determined by local requirements or sized to treat 85% of the annual runoff volume.
- Outlet designed to discharge the capture volume over a period of hours.
- Length to width ratio of at least 1.5:1 where feasible.
- Basin depths optimally range from 2 to 5 feet.
- Include energy dissipation in the inlet design to reduce resuspension of accumulated sediment.
- A maintenance ramp and perimeter access should be included in the design to facilitate access to the basin for maintenance activities and for vector surveillance and control.
- Use a draw down time of 48 hours in most areas of California. Draw down times in excess of 48 hours may result in vector breeding, and should be used only after coordination with local vector control authorities. Draw down times of less than 48 hours should be limited to BMP drainage areas with coarse soils that readily settle and to watersheds where warming may be determined to downstream fisheries.

### Construction/Inspection Considerations

- Inspect facility after first large to storm to determine whether the desired residence time has been achieved.
- When constructed with small tributary area, orifice sizing is critical and inspection should verify that flow through additional openings such as bolt holes does not occur.

### Performance

One objective of stormwater management practices can be to reduce the flood hazard associated with large storm events by reducing the peak flow associated with these storms. Dry extended detention basins can easily be designed for flood control, and this is actually the primary purpose of most detention ponds.

Dry extended detention basins provide moderate pollutant removal, provided that the recommended design features are incorporated. Although they can be effective at removing some pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a permanent pool. Several studies are available on the effectiveness of dry extended detention ponds including one recently concluded by Caltrans (2002).

The load reduction is greater than the concentration reduction because of the substantial infiltration that occurs. Although the infiltration of stormwater is clearly beneficial to surface receiving waters, there is the potential for groundwater contamination. Previous research on the effects of incidental infiltration on groundwater quality indicated that the risk of contamination is minimal.

There were substantial differences in the amount of infiltration that were observed in the earthen basins during the Caltrans study. On average, approximately 40 percent of the runoff entering the unlined basins infiltrated and was not discharged. The percentage ranged from a high of about 60 percent to a low of only about 8 percent for the different facilities. Climatic conditions and local water table elevation are likely the principal causes of this difference. The least infiltration occurred at a site located on the coast where humidity is higher and the basin invert is within a few meters of sea level. Conversely, the most infiltration occurred at a facility located well inland in Los Angeles County where the climate is much warmer and the humidity is less, resulting in lower soil moisture content in the basin floor at the beginning of storms.

Vegetated detention basins appear to have greater pollutant removal than concrete basins. In the Caltrans study, the concrete basin exported sediment and associated pollutants during a number of storms. Export was not as common in the earthen basins, where the vegetation appeared to help stabilize the retained sediment.

## **Siting Criteria**

Dry extended detention ponds are among the most widely applicable stormwater management practices and are especially useful in retrofit situations where their low hydraulic head requirements allow them to be sited within the constraints of the existing storm drain system. In addition, many communities have detention basins designed for flood control. It is possible to modify these facilities to incorporate features that provide water quality treatment and/or channel protection. Although dry extended detention ponds can be applied rather broadly, designers need to ensure that they are feasible at the site in question. This section provides basic guidelines for siting dry extended detention ponds.

In general, dry extended detention ponds should be used on sites with a minimum area of 5 acres. With this size catchment area, the orifice size can be on the order of 0.5 inches. On smaller sites, it can be challenging to provide channel or water quality control because the orifice diameter at the outlet needed to control relatively small storms becomes very small and thus prone to clogging. In addition, it is generally more cost-effective to control larger drainage areas due to the economies of scale.

Extended detention basins can be used with almost all soils and geology, with minor design adjustments for regions of rapidly percolating soils such as sand. In these areas, extended detention ponds may need an impermeable liner to prevent ground water contamination.

The base of the extended detention facility should not intersect the water table. A permanently wet bottom may become a mosquito breeding ground. Research in Southwest Florida (Santana et al., 1994) demonstrated that intermittently flooded systems, such as dry extended detention ponds, produce more mosquitoes than other pond systems, particularly when the facilities remained wet for more than 3 days following heavy rainfall.

A study in Prince George's County, Maryland, found that stormwater management practices can increase stream temperatures (Galli, 1990). Overall, dry extended detention ponds increased temperature by about 5°F. In cold water streams, dry ponds should be designed to detain stormwater for a relatively short time (i.e., 24 hours) to minimize the amount of warming that occurs in the basin.

### Additional Design Guidelines

In order to enhance the effectiveness of extended detention basins, the dimensions of the basin must be sized appropriately. Merely providing the required storage volume will not ensure maximum constituent removal. By effectively configuring the basin, the designer will create a long flow path, promote the establishment of low velocities, and avoid having stagnant areas of the basin. To promote settling and to attain an appealing environment, the design of the basin should consider the length to width ratio, cross-sectional areas, basin slopes and pond configuration, and aesthetics (Young et al., 1996).

Energy dissipation structures should be included for the basin inlet to prevent resuspension of accumulated sediment. The use of stilling basins for this purpose should be avoided because the standing water provides a breeding area for mosquitoes.

Extended detention facilities should be sized to completely capture the water quality volume. A micropool is often recommended for inclusion in the design and one is shown in the schematic diagram. These small permanent pools greatly increase the potential for mosquito breeding and complicate maintenance activities; consequently, they are not recommended for use in California.

A large aspect ratio may improve the performance of detention basins; consequently, the outlets should be placed to maximize the flowpath through the facility. The ratio of flowpath length to width from the inlet to the outlet should be at least 1.5:1 (L:W) where feasible. Basin depths optimally range from 2 to 5 feet.

The facility's drawdown time should be regulated by an orifice or weir. In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes. The outlet design implemented by Caltrans in the facilities constructed in San Diego County used an outlet riser with orifices



**Figure 1**  
**Example of Extended Detention Outlet Structure**

sized to discharge the water quality volume, and the riser overflow height was set to the design storm elevation. A stainless steel screen was placed around the outlet riser to ensure that the orifices would not become clogged with debris. Sites either used a separate riser or broad crested weir for overflow of runoff for the 25 and greater year storms. A picture of a typical outlet is presented in Figure 1.

The outflow structure should be sized to allow for complete drawdown of the water quality volume in 72 hours. No more than 50% of the water quality volume should drain from the facility within the first 24 hours. The outflow structure can be fitted with a valve so that discharge from the basin can be halted in case of an accidental spill in the watershed.

### ***Summary of Design Recommendations***

- (1) **Facility Sizing** - The required water quality volume is determined by local regulations or the basin should be sized to capture and treat 85% of the annual runoff volume. See Section 5.5.1 of the handbook for a discussion of volume-based design.

**Basin Configuration** – A high aspect ratio may improve the performance of detention basins; consequently, the outlets should be placed to maximize the flowpath through the facility. The ratio of flowpath length to width from the inlet to the outlet should be at least 1.5:1 (L:W). The flowpath length is defined as the distance from the inlet to the outlet as measured at the surface. The width is defined as the mean width of the basin. Basin depths optimally range from 2 to 5 feet. The basin may include a sediment forebay to provide the opportunity for larger particles to settle out.

A micropool should not be incorporated in the design because of vector concerns. For online facilities, the principal and emergency spillways must be sized to provide 1.0 foot of freeboard during the 25-year event and to safely pass the flow from 100-year storm.

- (2) **Pond Side Slopes** - Side slopes of the pond should be 3:1 (H:V) or flatter for grass stabilized slopes. Slopes steeper than 3:1 (H:V) must be stabilized with an appropriate slope stabilization practice.
- (3) **Basin Lining** – Basins must be constructed to prevent possible contamination of groundwater below the facility.
- (4) **Basin Inlet** – Energy dissipation is required at the basin inlet to reduce resuspension of accumulated sediment and to reduce the tendency for short-circuiting.
- (5) **Outflow Structure** - The facility's drawdown time should be regulated by a gate valve or orifice plate. In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes.

The outflow structure should be sized to allow for complete drawdown of the water quality volume in 72 hours. No more than 50% of the water quality volume should drain from the facility within the first 24 hours. The outflow structure should be fitted with a valve so that discharge from the basin can be halted in case of an accidental spill in the watershed. This same valve also can be used to regulate the rate of discharge from the basin.

The discharge through a control orifice is calculated from:

$$Q = CA(2g(H-H_o))^{0.5}$$

where: Q = discharge (ft<sup>3</sup>/s)  
 C = orifice coefficient  
 A = area of the orifice (ft<sup>2</sup>)  
 g = gravitational constant (32.2)  
 H = water surface elevation (ft)  
 H<sub>o</sub> = orifice elevation (ft)

Recommended values for C are 0.66 for thin materials and 0.80 when the material is thicker than the orifice diameter. This equation can be implemented in spreadsheet form with the pond stage/volume relationship to calculate drain time. To do this, use the initial height of the water above the orifice for the water quality volume. Calculate the discharge and assume that it remains constant for approximately 10 minutes. Based on that discharge, estimate the total discharge during that interval and the new elevation based on the stage volume relationship. Continue to iterate until H is approximately equal to H<sub>o</sub>. When using multiple orifices the discharge from each is summed.

- (6) Splitter Box - When the pond is designed as an offline facility, a splitter structure is used to isolate the water quality volume. The splitter box, or other flow diverting approach, should be designed to convey the 25-year storm event while providing at least 1.0 foot of freeboard along pond side slopes.
- (7) Erosion Protection at the Outfall - For online facilities, special consideration should be given to the facility's outfall location. Flared pipe end sections that discharge at or near the stream invert are preferred. The channel immediately below the pond outfall should be modified to conform to natural dimensions, and lined with large stone riprap placed over filter cloth. Energy dissipation may be required to reduce flow velocities from the primary spillway to non-erosive velocities.
- (8) Safety Considerations - Safety is provided either by fencing of the facility or by managing the contours of the pond to eliminate dropoffs and other hazards. Earthen side slopes should not exceed 3:1 (H:V) and should terminate on a flat safety bench area. Landscaping can be used to impede access to the facility. The primary spillway opening must not permit access by small children. Outfall pipes above 48 inches in diameter should be fenced.

### Maintenance

Routine maintenance activity is often thought to consist mostly of sediment and trash and debris removal; however, these activities often constitute only a small fraction of the maintenance hours. During a recent study by Caltrans, 72 hours of maintenance was performed annually, but only a little over 7 hours was spent on sediment and trash removal. The largest recurring activity was vegetation management, routine mowing. The largest absolute number of hours was associated with vector control because of mosquito breeding that occurred in the stilling basins (example of standing water to be avoided) installed as energy dissipaters. In most cases, basic housekeeping practices such as removal of debris accumulations and vegetation

management to ensure that the basin dewatered completely in 48-72 hours is sufficient to prevent creating mosquito and other vector habitats.

Consequently, maintenance costs should be estimated based primarily on the mowing frequency and the time required. Mowing should be done at least annually to avoid establishment of woody vegetation, but may need to be performed much more frequently if aesthetics are an important consideration.

Typical activities and frequencies include:

- 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.
- Remove accumulated trash and debris in the basin and around the riser pipe during the semiannual inspections. The frequency of this activity may be altered to meet specific site conditions.
- 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.
- Remove accumulated sediment and re-grade about every 10 years or when the accumulated sediment volume exceeds 10 percent of the basin volume. Inspect the basin each year for accumulated sediment volume.

## Cost

### **Construction Cost**

The construction costs associated with extended detention basins vary considerably. One recent study evaluated the cost of all pond systems (Brown and Schueler, 1997). Adjusting for inflation, the cost of dry extended detention ponds can be estimated with the equation:

$$C = 12.4V^{0.760}$$

where: C = Construction, design, and permitting cost, and  
V = Volume (ft<sup>3</sup>).

Using this equation, typical construction costs are:

\$ 41,600 for a 1 acre-foot pond

\$ 239,000 for a 10 acre-foot pond

\$ 1,380,000 for a 100 acre-foot pond

Interestingly, these costs are generally slightly higher than the predicted cost of wet ponds (according to Brown and Schueler, 1997) on a cost per total volume basis, which highlights the difficulty of developing reasonably accurate construction estimates. In addition, a typical facility constructed by Caltrans cost about \$160,000 with a capture volume of only 0.3 ac-ft.

An economic concern associated with dry ponds is that they might detract slightly from the value of adjacent properties. One study found that dry ponds can actually detract from the

perceived value of homes adjacent to a dry pond by between 3 and 10 percent (Emmerling-Dinovo, 1995).

### **Maintenance Cost**

For ponds, the annual cost of routine maintenance is typically estimated at about 3 to 5 percent of the construction cost (EPA website). Alternatively, a community can estimate the cost of the maintenance activities outlined in the maintenance section. Table 1 presents the maintenance costs estimated by Caltrans based on their experience with five basins located in southern California. Again, it should be emphasized that the vast majority of hours are related to vegetation management (mowing).

<b>Activity</b>	<b>Labor Hours</b>	<b>Equipment &amp; Material (\$)</b>	<b>Cost</b>
Inspections	4	7	183
Maintenance	49	126	2282
Vector Control	0	0	0
Administration	3	0	132
Materials	-	535	535
<b>Total</b>	<b>56</b>	<b>\$668</b>	<b>\$3,132</b>

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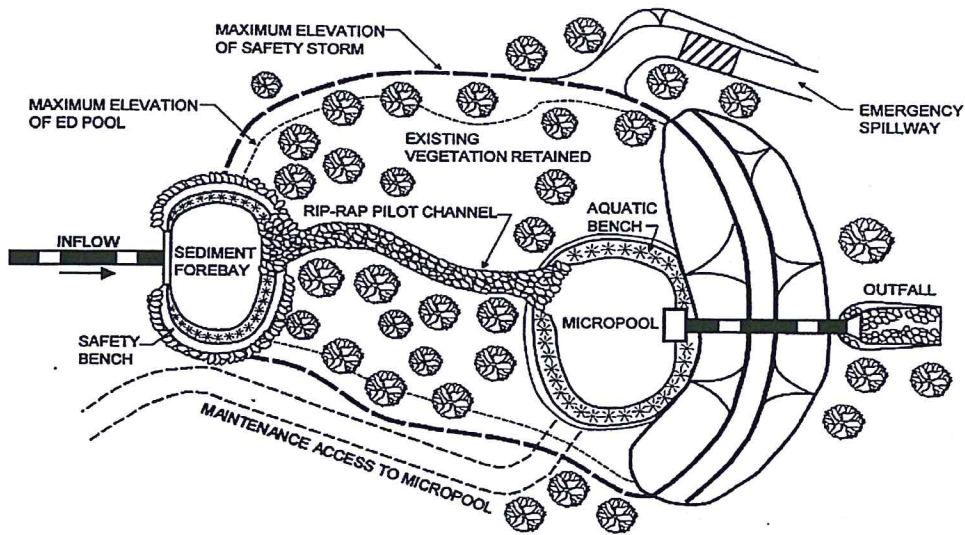
Young, G.K., et al., 1996, *Evaluation and Management of Highway Runoff Water Quality*, Publication No. FHWA-PD-96-032, U.S. Department of Transportation, Federal Highway Administration, Office of Environment and Planning.

### **Information Resources**

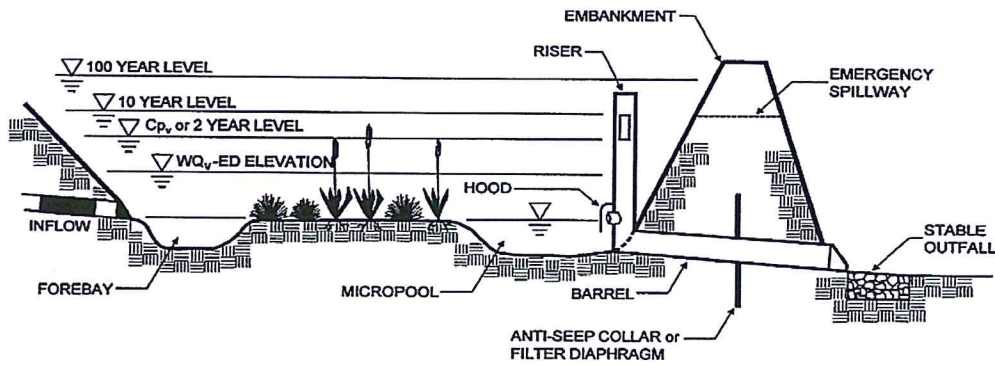
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PLAN VIEW



PROFILE

Schematic of an Extended Detention Basin (MDE, 2000)



## Design Considerations

- Tributary Area
- Area Required
- Slope
- Water Availability

## Description

Vegetated swales are open, shallow channels with vegetation covering the side slopes and bottom that collect and slowly convey runoff flow to downstream discharge points. They are designed to treat runoff through filtering by the vegetation in the channel, filtering through a subsoil matrix, and/or infiltration into the underlying soils. Swales can be natural or manmade. They trap particulate pollutants (suspended solids and trace metals), promote infiltration, and reduce the flow velocity of stormwater runoff. Vegetated swales can serve as part of a stormwater drainage system and can replace curbs, gutters and storm sewer systems.

## California Experience

Caltrans constructed and monitored six vegetated swales in southern California. These swales were generally effective in reducing the volume and mass of pollutants in runoff. Even in the areas where the annual rainfall was only about 10 inches/yr, the vegetation did not require additional irrigation. One factor that strongly affected performance was the presence of large numbers of gophers at most of the sites. The gophers created earthen mounds, destroyed vegetation, and generally reduced the effectiveness of the controls for TSS reduction.

## Advantages

- If properly designed, vegetated, and operated, swales can serve as an aesthetic, potentially inexpensive urban development or roadway drainage conveyance measure with significant collateral water quality benefits.

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	▲
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	●
<input checked="" type="checkbox"/>	Metals	▲
<input checked="" type="checkbox"/>	Bacteria	●
<input checked="" type="checkbox"/>	Oil and Grease	▲
<input checked="" type="checkbox"/>	Organics	▲

## Legend (Removal Effectiveness)

- Low
- High
- ▲ Medium



- Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible.

### Limitations

- Can be difficult to avoid channelization.
- May not be appropriate for industrial sites or locations where spills may occur
- Grassed swales cannot treat a very large drainage area. Large areas may be divided and treated using multiple swales.
- A thick vegetative cover is needed for these practices to function properly.
- They are impractical in areas with steep topography.
- They are not effective and may even erode when flow velocities are high, if the grass cover is not properly maintained.
- In some places, their use is restricted by law: many local municipalities require curb and gutter systems in residential areas.
- Swales are more susceptible to failure if not properly maintained than other treatment BMPs.

### Design and Sizing Guidelines

- Flow rate based design determined by local requirements or sized so that 85% of the annual runoff volume is discharged at less than the design rainfall intensity.
- Swale should be designed so that the water level does not exceed 2/3rds the height of the grass or 4 inches, which ever is less, at the design treatment rate.
- Longitudinal slopes should not exceed 2.5%
- Trapezoidal channels are normally recommended but other configurations, such as parabolic, can also provide substantial water quality improvement and may be easier to mow than designs with sharp breaks in slope.
- Swales constructed in cut are preferred, or in fill areas that are far enough from an adjacent slope to minimize the potential for gopher damage. Do not use side slopes constructed of fill, which are prone to structural damage by gophers and other burrowing animals.
- A diverse selection of low growing, plants that thrive under the specific site, climatic, and watering conditions should be specified. Vegetation whose growing season corresponds to the wet season are preferred. Drought tolerant vegetation should be considered especially for swales that are not part of a regularly irrigated landscaped area.
- The width of the swale should be determined using Manning's Equation using a value of 0.25 for Manning's n.

## **Construction/Inspection Considerations**

- Include directions in the specifications for use of appropriate fertilizer and soil amendments based on soil properties determined through testing and compared to the needs of the vegetation requirements.
- Install swales at the time of the year when there is a reasonable chance of successful establishment without irrigation; however, it is recognized that rainfall in a given year may not be sufficient and temporary irrigation may be used.
- If sod tiles must be used, they should be placed so that there are no gaps between the tiles; stagger the ends of the tiles to prevent the formation of channels along the swale or strip.
- Use a roller on the sod to ensure that no air pockets form between the sod and the soil.
- Where seeds are used, erosion controls will be necessary to protect seeds for at least 75 days after the first rainfall of the season.

## **Performance**

The literature suggests that vegetated swales represent a practical and potentially effective technique for controlling urban runoff quality. While limited quantitative performance data exists for vegetated swales, it is known that check dams, slight slopes, permeable soils, dense grass cover, increased contact time, and small storm events all contribute to successful pollutant removal by the swale system. Factors decreasing the effectiveness of swales include compacted soils, short runoff contact time, large storm events, frozen ground, short grass heights, steep slopes, and high runoff velocities and discharge rates.

Conventional vegetated swale designs have achieved mixed results in removing particulate pollutants. A study performed by the Nationwide Urban Runoff Program (NURP) monitored three grass swales in the Washington, D.C., area and found no significant improvement in urban runoff quality for the pollutants analyzed. However, the weak performance of these swales was attributed to the high flow velocities in the swales, soil compaction, steep slopes, and short grass height.

Another project in Durham, NC, monitored the performance of a carefully designed artificial swale that received runoff from a commercial parking lot. The project tracked 11 storms and concluded that particulate concentrations of heavy metals (Cu, Pb, Zn, and Cd) were reduced by approximately 50 percent. However, the swale proved largely ineffective for removing soluble nutrients.

The effectiveness of vegetated swales can be enhanced by adding check dams at approximately 17 meter (50 foot) increments along their length (See Figure 1). These dams maximize the retention time within the swale, decrease flow velocities, and promote particulate settling. Finally, the incorporation of vegetated filter strips parallel to the top of the channel banks can help to treat sheet flows entering the swale.

Only 9 studies have been conducted on all grassed channels designed for water quality (Table 1). The data suggest relatively high removal rates for some pollutants, but negative removals for some bacteria, and fair performance for phosphorus.

Removal Efficiencies (% Removal)							
Study	TSS	TP	TN	NO <sub>3</sub>	Metals	Bacteria	Type
Caltrans 2002	77	8	67	66	83-90	-33	dry swales
Goldberg 1993	67.8	4.5	-	31.4	42-62	-100	grassed channel
Seattle Metro and Washington Department of Ecology 1992	60	45	-	-25	2-16	-25	grassed channel
Seattle Metro and Washington Department of Ecology, 1992	83	29	-	-25	46-73	-25	grassed channel
Wang et al., 1981	80	-	-	-	70-80	-	dry swale
Dorman et al., 1989	98	18	-	45	37-81	-	dry swale
Harper, 1988	87	83	84	80	88-90	-	dry swale
Kercher et al., 1983	99	99	99	99	99	-	dry swale
Harper, 1988.	81	17	40	52	37-69	-	wet swale
Koon, 1995	67	39	-	9	-35 to 6	-	wet swale

While it is difficult to distinguish between different designs based on the small amount of available data, grassed channels generally have poorer removal rates than wet and dry swales, although some swales appear to export soluble phosphorus (Harper, 1988; Koon, 1995). It is not clear why swales export bacteria. One explanation is that bacteria thrive in the warm swale soils.

### **Siting Criteria**

The suitability of a swale at a site will depend on land use, size of the area serviced, soil type, slope, imperviousness of the contributing watershed, and dimensions and slope of the swale system (Schueler et al., 1992). In general, swales can be used to serve areas of less than 10 acres, with slopes no greater than 5 %. Use of natural topographic lows is encouraged and natural drainage courses should be regarded as significant local resources to be kept in use (Young et al., 1996).

### **Selection Criteria (NCTCOG, 1993)**

- Comparable performance to wet basins
- Limited to treating a few acres
- Availability of water during dry periods to maintain vegetation
- Sufficient available land area

Research in the Austin area indicates that vegetated controls are effective at removing pollutants even when dormant. Therefore, irrigation is not required to maintain growth during dry periods, but may be necessary only to prevent the vegetation from dying.

The topography of the site should permit the design of a channel with appropriate slope and cross-sectional area. Site topography may also dictate a need for additional structural controls. Recommendations for longitudinal slopes range between 2 and 6 percent. Flatter slopes can be used, if sufficient to provide adequate conveyance. Steep slopes increase flow velocity, decrease detention time, and may require energy dissipating and grade check. Steep slopes also can be managed using a series of check dams to terrace the swale and reduce the slope to within acceptable limits. The use of check dams with swales also promotes infiltration.

## **Additional Design Guidelines**

Most of the design guidelines adopted for swale design specify a minimum hydraulic residence time of 9 minutes. This criterion is based on the results of a single study conducted in Seattle, Washington (Seattle Metro and Washington Department of Ecology, 1992), and is not well supported. Analysis of the data collected in that study indicates that pollutant removal at a residence time of 5 minutes was not significantly different, although there is more variability in that data. Therefore, additional research in the design criteria for swales is needed. Substantial pollutant removal has also been observed for vegetated controls designed solely for conveyance (Barrett et al, 1998); consequently, some flexibility in the design is warranted.

Many design guidelines recommend that grass be frequently mowed to maintain dense coverage near the ground surface. Recent research (Colwell et al., 2000) has shown mowing frequency or grass height has little or no effect on pollutant removal.

## **Summary of Design Recommendations**

- 1) The swale should have a length that provides a minimum hydraulic residence time of at least 10 minutes. The maximum bottom width should not exceed 10 feet unless a dividing berm is provided. The depth of flow should not exceed 2/3rds the height of the grass at the peak of the water quality design storm intensity. The channel slope should not exceed 2.5%.
- 2) A design grass height of 6 inches is recommended.
- 3) Regardless of the recommended detention time, the swale should be not less than 100 feet in length.
- 4) The width of the swale should be determined using Manning's Equation, at the peak of the design storm, using a Manning's n of 0.25.
- 5) The swale can be sized as both a treatment facility for the design storm and as a conveyance system to pass the peak hydraulic flows of the 100-year storm if it is located "on-line." The side slopes should be no steeper than 3:1 (H:V).
- 6) Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized for this purpose whenever possible. If flow is to be introduced through curb cuts, place pavement slightly above the elevation of the vegetated areas. Curb cuts should be at least 12 inches wide to prevent clogging.
- 7) Swales must be vegetated in order to provide adequate treatment of runoff. It is important to maximize water contact with vegetation and the soil surface. For general purposes, select fine, close-growing, water-resistant grasses. If possible, divert runoff (other than necessary irrigation) during the period of vegetation

establishment. Where runoff diversion is not possible, cover graded and seeded areas with suitable erosion control materials.

### **Maintenance**

The useful life of a vegetated swale system is directly proportional to its maintenance frequency. If properly designed and regularly maintained, vegetated swales can last indefinitely. The maintenance objectives for vegetated swale systems include keeping up the hydraulic and removal efficiency of the channel and maintaining a dense, healthy grass cover.

Maintenance activities should include periodic mowing (with grass never cut shorter than the design flow depth), weed control, watering during drought conditions, reseeding of bare areas, and clearing of debris and blockages. Cuttings should be removed from the channel and disposed in a local composting facility. Accumulated sediment should also be removed manually to avoid concentrated flows in the swale. The application of fertilizers and pesticides should be minimal.

Another aspect of a good maintenance plan is repairing damaged areas within a channel. For example, if the channel develops ruts or holes, it should be repaired utilizing a suitable soil that is properly tamped and seeded. The grass cover should be thick; if it is not, reseed as necessary. Any standing water removed during the maintenance operation must be disposed to a sanitary sewer at an approved discharge location. Residuals (e.g., silt, grass cuttings) must be disposed in accordance with local or State requirements. Maintenance of grassed swales mostly involves maintenance of the grass or wetland plant cover. Typical maintenance activities are summarized below:

- Inspect swales at least twice annually for erosion, damage to vegetation, and sediment and debris accumulation preferably at the end of the wet season to schedule summer maintenance and before major fall runoff to be sure the swale is ready for winter. However, additional inspection after periods of heavy runoff is desirable. The swale should be checked for debris and litter, and areas of sediment accumulation.
- Grass height and mowing frequency may not have a large impact on pollutant removal. Consequently, mowing may only be necessary once or twice a year for safety or aesthetics or to suppress weeds and woody vegetation.
- Trash tends to accumulate in swale areas, particularly along highways. The need for litter removal is determined through periodic inspection, but litter should always be removed prior to mowing.
- Sediment accumulating near culverts and in channels should be removed when it builds up to 75 mm (3 in.) at any spot, or covers vegetation.
- Regularly inspect swales for pools of standing water. Swales can become a nuisance due to mosquito breeding in standing water if obstructions develop (e.g. debris accumulation, invasive vegetation) and/or if proper drainage slopes are not implemented and maintained.

## **Cost**

### ***Construction Cost***

Little data is available to estimate the difference in cost between various swale designs. One study (SWRPC, 1991) estimated the construction cost of grassed channels at approximately \$0.25 per ft<sup>2</sup>. This price does not include design costs or contingencies. Brown and Schueler (1997) estimate these costs at approximately 32 percent of construction costs for most stormwater management practices. For swales, however, these costs would probably be significantly higher since the construction costs are so low compared with other practices. A more realistic estimate would be a total cost of approximately \$0.50 per ft<sup>2</sup>, which compares favorably with other stormwater management practices.

Table 2 Swale Cost Estimate (SEWRPC, 1991)

Component	Unit	Extent	Unit Cost			Total Cost		
			Low	Moderate	High	Low	Moderate	High
Mobilization / Demobilization-Light	Swale	1	\$107	\$274	\$441	\$107	\$274	\$441
Site Preparation	Acre	0.5	\$2,200	\$3,800	\$5,400	\$1,100	\$1,900	\$2,700
Clearing <sup>a</sup> .....	Acre	0.25	\$3,800	\$5,200	\$6,600	\$950	\$1,300	\$1,650
Grubbing <sup>b</sup> .....	Yd <sup>3</sup>	372	\$2.10	\$3.70	\$5.30	\$781	\$1,376	\$1,972
General Excavation <sup>d</sup> .....	Yd <sup>2</sup>	1,210	\$0.20	\$0.35	\$0.50	\$242	\$424	\$605
Level and Till <sup>e</sup> .....								
Sites Development	Yd <sup>2</sup>	1,210	\$0.40	\$1.00	\$1.60	\$484	\$1,210	\$1,938
Salvaged Topsoil	Yd <sup>2</sup>	1,210	\$1.20	\$2.40	\$3.60	\$1,452	\$2,904	\$4,356
Seed, and Mulch <sup>f</sup> ..								
Sod <sup>g</sup> .....								
<b>Subtotal</b>	--	--	--	--	--	\$5,116	\$9,388	\$13,660
Contingencies	Swale	1	25%	25%	25%	\$1,279	\$2,347	\$3,415
<b>Total</b>	--	--	--	--	--	\$6,395	\$11,735	\$17,075

Source: (SEWRPC, 1991)

- Note: Mobilization/demobilization refers to the organization and planning involved in establishing a vegetative swale.
- <sup>a</sup> Swale has a bottom width of 1.0 foot, a top width of 10 feet with 1:3 side slopes, and a 1,000-foot length.
- <sup>b</sup> Area cleared = (top width + 10 feet) x swale length.
- <sup>c</sup> Area grubbed = (top width x swale length).
- <sup>d</sup> Volume excavated = (0.67 x top width x swale depth) x swale length (parabolic cross-section).
- <sup>e</sup> Area filled = (top width +  $\frac{8(\text{swale depth})^2}{3(\text{top width})}$ ) x swale length (parabolic cross-section).
- <sup>f</sup> Area seeded = area cleared x 0.5.
- <sup>g</sup> Area sodded = area cleared x 0.5.

# Vegetated Swale

TC-30

Table 3 Estimated Maintenance Costs (SEWRPC, 1991)

Component	Unit Cost	Swale Size (Depth and Top Width)		Comment
		1.5 Foot Depth, One-Foot Bottom Width, 10-Foot Top Width	3-Foot Depth, 3-Foot Bottom Width, 21-Foot Top Width	
Lawn Mowing	\$0.85 / 1,000 ft <sup>2</sup> / mowing	\$0.14 / linear foot	\$0.21 / linear foot	Lawn maintenance area = (top width + 10 feet) x length. Mow eight times per year
General Lawn Care	\$9.00 / 1,000 ft <sup>2</sup> / year	\$0.18 / linear foot	\$0.28 / linear foot	Lawn maintenance area = (top width + 10 feet) x length
Swale Debris and Litter Removal	\$0.10 / linear foot / year	\$0.10 / linear foot	\$0.10 / linear foot	-
Grass Reseeding with Mulch and Fertilizer	\$0.30 / yd <sup>2</sup>	\$0.01 / linear foot	\$0.01 / linear foot	Area revegetated equals 1% of lawn maintenance area per year
Program Administration and Swale Inspection	\$0.15 / linear foot / year, plus \$25 / inspection	\$0.15 / linear foot	\$0.15 / linear foot	Inspect four times per year
<b>Total</b>	<b>**</b>	<b>\$0.56 / linear foot</b>	<b>\$ 0.75 / linear foot</b>	<b>-</b>

**Maintenance Cost**

Caltrans (2002) estimated the expected annual maintenance cost for a swale with a tributary area of approximately 2 ha at approximately \$2,700. Since almost all maintenance consists of mowing, the cost is fundamentally a function of the mowing frequency. Unit costs developed by SEWRPC are shown in Table 3. In many cases vegetated channels would be used to convey runoff and would require periodic mowing as well, so there may be little additional cost for the water quality component. Since essentially all the activities are related to vegetation management, no special training is required for maintenance personnel.

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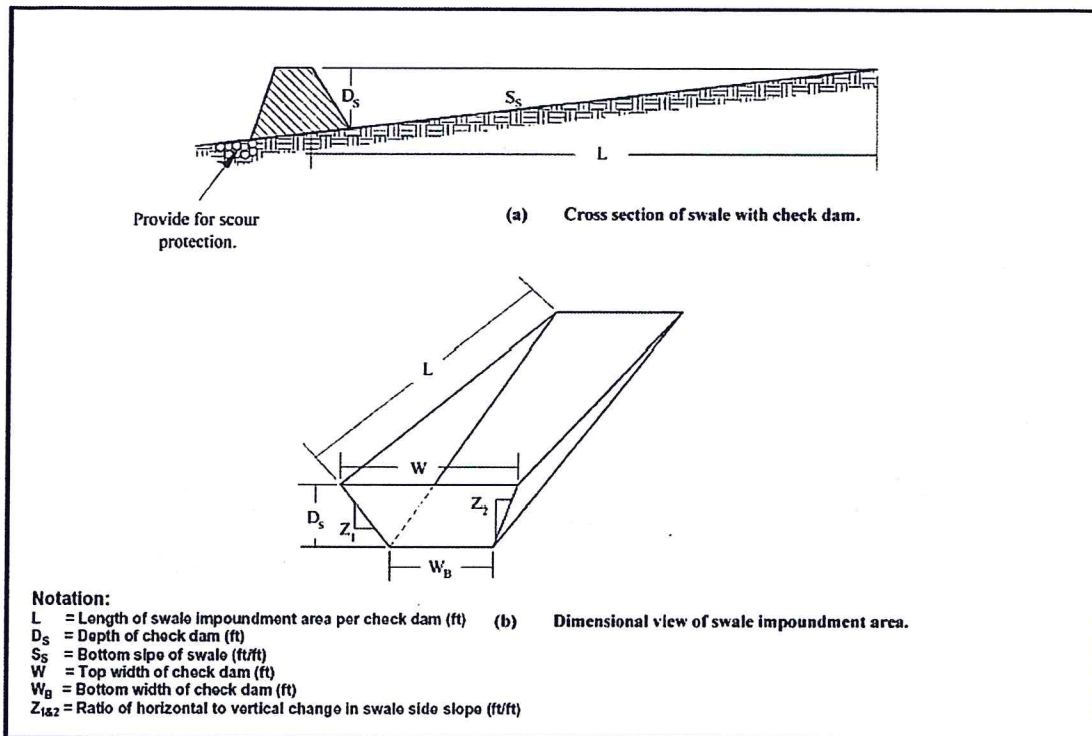
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## Description

Grassed buffer strips (vegetated filter strips, filter strips, and grassed filters) are vegetated surfaces that are designed to treat sheet flow from adjacent surfaces. Filter strips function by slowing runoff velocities and allowing sediment and other pollutants to settle and by providing some infiltration into underlying soils. Filter strips were originally used as an agricultural treatment practice and have more recently evolved into an urban practice. With proper design and maintenance, filter strips can provide relatively high pollutant removal. In addition, the public views them as landscaped amenities and not as stormwater infrastructure. Consequently, there is little resistance to their use.

## California Experience

Caltrans constructed and monitored three vegetated buffer strips in southern California and is currently evaluating their performance at eight additional sites statewide. These strips were generally effective in reducing the volume and mass of pollutants in runoff. Even in the areas where the annual rainfall was only about 10 inches/yr, the vegetation did not require additional irrigation. One factor that strongly affected performance was the presence of large numbers of gophers at most of the southern California sites. The gophers created earthen mounds, destroyed vegetation, and generally reduced the effectiveness of the controls for TSS reduction.

## Advantages

- Buffers require minimal maintenance activity (generally just erosion prevention and mowing).
- If properly designed, vegetated, and operated, buffer strips can provide reliable water quality benefits in conjunction with high aesthetic appeal.

## Design Considerations

- Tributary Area
- Slope
- Water Availability
- Aesthetics

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	■
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	▲
<input checked="" type="checkbox"/>	Metals	■
<input checked="" type="checkbox"/>	Bacteria	●
<input checked="" type="checkbox"/>	Oil and Grease	■
<input checked="" type="checkbox"/>	Organics	▲

### Legend (Removal Effectiveness)

- |          |        |
|----------|--------|
| ● Low    | ■ High |
| ▲ Medium |        |



- Flow characteristics and vegetation type and density can be closely controlled to maximize BMP effectiveness.
- Roadside shoulders act as effective buffer strips when slope and length meet criteria described below.

### **Limitations**

- May not be appropriate for industrial sites or locations where spills may occur.
- Buffer strips cannot treat a very large drainage area.
- A thick vegetative cover is needed for these practices to function properly.
- Buffer or vegetative filter length must be adequate and flow characteristics acceptable or water quality performance can be severely limited.
- Vegetative buffers may not provide treatment for dissolved constituents except to the extent that flows across the vegetated surface are infiltrated into the soil profile.
- This technology does not provide significant attenuation of the increased volume and flow rate of runoff during intense rain events.

### **Design and Sizing Guidelines**

- Maximum length (in the direction of flow towards the buffer) of the tributary area should be 60 feet.
- Slopes should not exceed 15%.
- Minimum length (in direction of flow) is 15 feet.
- Width should be the same as the tributary area.
- Either grass or a diverse selection of other low growing, drought tolerant, native vegetation should be specified. Vegetation whose growing season corresponds to the wet season is preferred.

### **Construction/Inspection Considerations**

- Include directions in the specifications for use of appropriate fertilizer and soil amendments based on soil properties determined through testing and compared to the needs of the vegetation requirements.
- Install strips at the time of the year when there is a reasonable chance of successful establishment without irrigation; however, it is recognized that rainfall in a given year may not be sufficient and temporary irrigation may be required.
- If sod tiles must be used, they should be placed so that there are no gaps between the tiles; stagger the ends of the tiles to prevent the formation of channels along the strip.
- Use a roller on the sod to ensure that no air pockets form between the sod and the soil.

- Where seeds are used, erosion controls will be necessary to protect seeds for at least 75 days after the first rainfall of the season.

## Performance

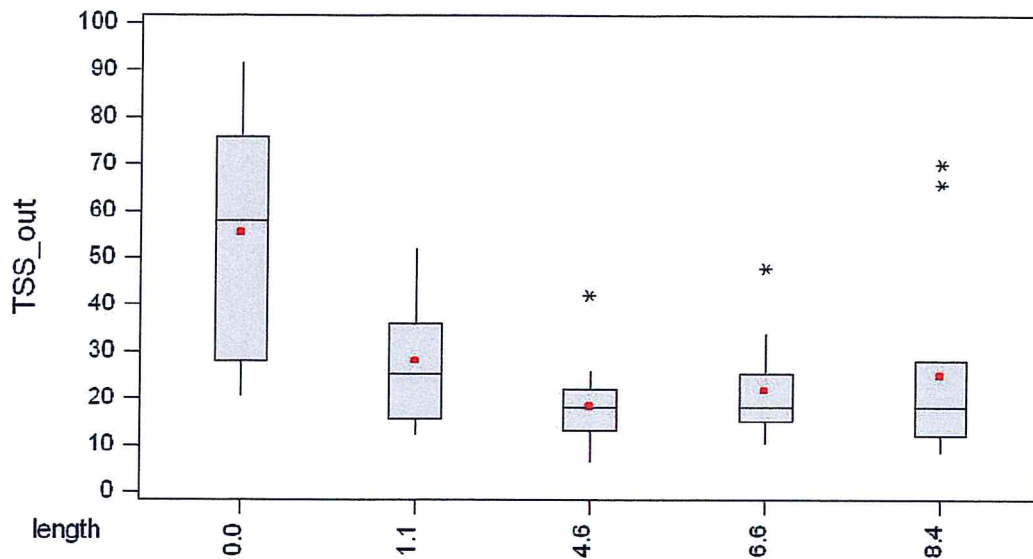
Vegetated buffer strips tend to provide somewhat better treatment of stormwater runoff than swales and have fewer tendencies for channelization or erosion. Table 1 documents the pollutant removal observed in a recent study by Caltrans (2002) based on three sites in southern California. The column labeled "Significance" is the probability that the mean influent and effluent EMCs are not significantly different based on an analysis of variance.

The removal of sediment and dissolved metals was comparable to that observed in much more complex controls. Reduction in nitrogen was not significant and all of the sites exported phosphorus for the entire study period. This may have been the result of using salt grass, a warm weather species that is dormant during the wet season, and which leaches phosphorus when dormant.

Another Caltrans study (unpublished) of vegetated highway shoulders as buffer strips also found substantial reductions often within a very short distance of the edge of pavement. Figure 1 presents a box and whisker plot of the concentrations of TSS in highway runoff after traveling various distances (shown in meters) through a vegetated filter strip with a slope of about 10%. One can see that the TSS median concentration reaches an irreducible minimum concentration of about 20 mg/L within 5 meters of the pavement edge.

**Table 1 Pollutant Reduction in a Vegetated Buffer Strip**

Constituent	Mean EMC		Removal %	Significance P
	Influent (mg/L)	Effluent (mg/L)		
TSS	119	31	74	<0.000
NO <sub>3</sub> -N	0.67	0.58	13	0.367
TKN-N	2.50	2.10	16	0.542
Total N <sup>a</sup>	3.17	2.68	15	-
Dissolved P	0.15	0.46	-206	0.047
Total P	0.42	0.62	-52	0.035
Total Cu	0.058	0.009	84	<0.000
Total Pb	0.046	0.006	88	<0.000
Total Zn	0.245	0.055	78	<0.000
Dissolved Cu	0.029	0.007	77	0.004
Dissolved Pb	0.004	0.002	66	0.006
Dissolved Zn	0.099	0.035	65	<0.000



Filter strips also exhibit good removal of litter and other floatables because the water depth in these systems is well below the vegetation height and consequently these materials are not easily transported through them. Unfortunately little attenuation of peak runoff rates and volumes (particularly for larger events) is normally observed, depending on the soil properties. Therefore it may be prudent to follow the strips with another practice than can reduce flooding and channel erosion downstream.

### Siting Criteria

The use of buffer strips is limited to gently sloping areas where the vegetative cover is robust and diffuse, and where shallow flow characteristics are possible. The practical water quality benefits can be effectively eliminated with the occurrence of significant erosion or when flow concentration occurs across the vegetated surface. Slopes should not exceed 15 percent or be less than 1 percent. The vegetative surface should extend across the full width of the area being drained. The upstream boundary of the filter should be located contiguous to the developed area. Use of a level spreading device (vegetated berm, sawtooth concrete border, rock trench, etc) to facilitate overland sheet flow is not normally recommended because of maintenance considerations and the potential for standing water.

Filter strips are applicable in most regions, but are restricted in some situations because they consume a large amount of space relative to other practices. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer or as pretreatment to a structural practice. In arid areas, however, the cost of irrigating the grass on the practice will most likely outweigh its water quality benefits, although aesthetic considerations may be sufficient to overcome this constraint. Filter strips are generally impractical in ultra-urban areas where little pervious surface exists.

Some cold water species, such as trout, are sensitive to changes in temperature. While some treatment practices, such as wet ponds, can warm stormwater substantially, filter strips do not

are not expected to increase stormwater temperatures. Thus, these practices are good for protection of cold-water streams.

Filter strips should be separated from the ground water by between 2 and 4 ft to prevent contamination and to ensure that the filter strip does not remain wet between storms.

## **Additional Design Guidelines**

Filter strips appear to be a minimal design practice because they are basically no more than a grassed slope. In general the slope of the strip should not exceed 15fc% and the strip should be at least 15 feet long to provide water quality treatment. Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion. The top of the strip should be installed 2-5 inches below the adjacent pavement, so that vegetation and sediment accumulation at the edge of the strip does not prevent runoff from entering.

A major question that remains unresolved is how large the drainage area to a strip can be. Research has conclusively demonstrated that these are effective on roadside shoulders, where the contributing area is about twice the buffer area. They have also been installed on the perimeter of large parking lots where they performed fairly effectively; however much lower slopes may be needed to provide adequate water quality treatment.

The filter area should be densely vegetated with a mix of erosion-resistant plant species that effectively bind the soil. Native or adapted grasses, shrubs, and trees are preferred because they generally require less fertilizer and are more drought resistant than exotic plants. Runoff flow velocities should not exceed about 1 fps across the vegetated surface.

For engineered vegetative strips, the facility surface should be graded flat prior to placement of vegetation. Initial establishment of vegetation requires attentive care including appropriate watering, fertilization, and prevention of excessive flow across the facility until vegetation completely covers the area and is well established. Use of a permanent irrigation system may help provide maximal water quality performance.

In cold climates, filter strips provide a convenient area for snow storage and treatment. If used for this purpose, vegetation in the filter strip should be salt-tolerant (e.g., creeping bentgrass), and a maintenance schedule should include the removal of sand built up at the bottom of the slope. In arid or semi-arid climates, designers should specify drought-tolerant grasses to minimize irrigation requirements.

## **Maintenance**

Filter strips require mainly vegetation management; therefore little special training is needed for maintenance crews. Typical maintenance activities and frequencies include:

- Inspect strips at least twice annually for erosion or damage to vegetation, preferably at the end of the wet season to schedule summer maintenance and before major fall run-off to be sure the strip is ready for winter. However, additional inspection after periods of heavy run-off is most desirable. The strip should be checked for debris and litter and areas of sediment accumulation.
- Recent research on biofiltration swales, but likely applicable to strips (Colwell et al., 2000), indicates that grass height and mowing frequency have little impact on pollutant removal;

consequently, mowing may only be necessary once or twice a year for safety and aesthetics or to suppress weeds and woody vegetation.

- Trash tends to accumulate in strip areas, particularly along highways. The need for litter removal should be determined through periodic inspection but litter should always be removed prior to mowing.
- Regularly inspect vegetated buffer strips for pools of standing water. Vegetated buffer strips can become a nuisance due to mosquito breeding in level spreaders (unless designed to dewater completely in 48-72 hours), in pools of standing water if obstructions develop (e.g. debris accumulation, invasive vegetation), and/or if proper drainage slopes are not implemented and maintained.

### **Cost**

#### **Construction Cost**

Little data is available on the actual construction costs of filter strips. One rough estimate can be the cost of seed or sod, which is approximately 30¢ per ft<sup>2</sup> for seed or 70¢ per ft<sup>2</sup> for sod. This amounts to between \$13,000 and \$30,000 per acre of filter strip. This cost is relatively high compared with other treatment practices. However, the grassed area used as a filter strip may have been seeded or sodded even if it were not used for treatment. In these cases, the only additional cost is the design. Typical maintenance costs are about \$350/acre/year (adapted from SWRPC, 1991). This cost is relatively inexpensive and, again, might overlap with regular landscape maintenance costs.

The true cost of filter strips is the land they consume. In some situations this land is available as wasted space beyond back yards or adjacent to roadsides, but this practice is cost-prohibitive when land prices are high and land could be used for other purposes.

#### **Maintenance Cost**

Maintenance of vegetated buffer strips consists mainly of vegetation management (mowing, irrigation if needed, weeding) and litter removal. Consequently the costs are quite variable depending on the frequency of these activities and the local labor rate.

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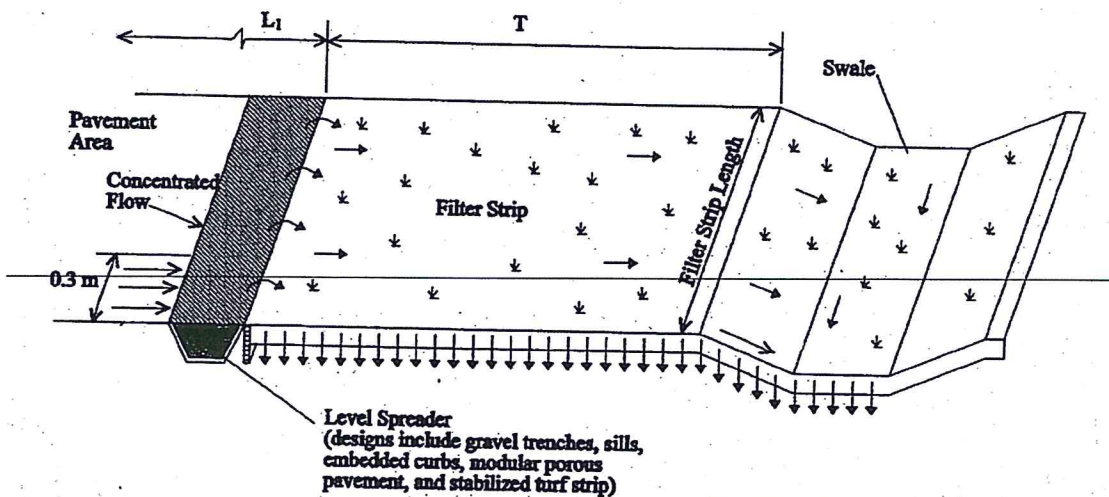
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Yu, S., S. Barnes and V. Gerde. 1993. *Testing of Best Management Practices for Controlling Highway Runoff*. FHWA/VA 93-R16. Virginia Transportation Research Council, Charlottesville, VA.

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## Description

Water quality inlets (WQIs), also commonly called trapping catch basins, oil/grit separators or oil/water separators, consist of one or more chambers that promote sedimentation of coarse materials and separation of free oil (as opposed to emulsified or dissolved oil) from stormwater. Some WQIs also contain screens to help retain larger or floating debris, and many of the newer designs also include a coalescing unit that helps promote oil/water separation. A typical WQI, as shown in the schematic, consists of a sedimentation chamber, an oil separation chamber, and a discharge chamber.

These devices are appropriate for capturing hydrocarbon spills, but provide very marginal sediment removal and are not very effective for treatment of stormwater runoff. WQIs typically capture only the first portion of runoff for treatment and are generally used for pretreatment before discharging to other best management practices (BMPs).

## California Experience

Caltrans investigated the use of coalescing plate oil/water separators at maintenance stations in Southern California. Twenty-two maintenance stations were originally considered for implementation of this technology; however, only one site appeared to have concentrations that were sufficiently high to warrant installation of an oil-water separator. Concentrations of free oil in stormwater runoff observed during the course of the study even from this site were too low for effective operation of this technology, and no free oil was ever captured by the device.

## Advantages

- Can provide spill control.

## Limitations

- WQIs generally provide limited hydraulic and residuals storage. Due to the limited storage, WQIs do not provide substantial stormwater improvement.
- Standing water in the devices can provide a breeding ground for mosquitoes.
- Certain designs maintain permanent sources of standing water where mosquito and other vector breeding may occur.

## Design and Sizing Guidelines

- Water quality inlets are most effective for spill control and should be sized accordingly.

## Design Considerations

- Area Required

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	●
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	▲
<input checked="" type="checkbox"/>	Metals	●
<input checked="" type="checkbox"/>	Bacteria	●
<input checked="" type="checkbox"/>	Oil and Grease	▲
<input checked="" type="checkbox"/>	Organics	●

### Legend (Removal Effectiveness)

- Low      ■ High  
▲ Medium



- Designs that utilize covered sedimentation and filtration basins should be accessible to vector control personnel via access doors to facilitate vector surveillance and controlling the basins if needed.

### Performance

WQIs are primarily utilized to remove sediment from stormwater runoff. Grit and sediment are partially removed by gravity settling within the first two chambers. A WQI with a detention time of 1 hour may expect to have 20 to 40 percent removal of sediments. Hydrocarbons associated with the accumulated sediments are also often removed from the runoff through this process. The WQI achieves slight, if any, removal of nutrients, metals and organic pollutants other than free petroleum products (Schueler, 1992).

A 1993 MWCOG study found that an average of less than 5 centimeters (2 inches) of sediments (mostly coarse-grained grit and organic matter) were trapped in the WQIs. Hydrocarbon and total organic carbon (TOC) concentrations of the sediments averaged 8,150 and 53,900 milligrams per kilogram, respectively. The mean hydrocarbon concentration in the WQI water column was 10 milligrams per liter. The study also indicated that sediment accumulation did not increase over time, suggesting that the sediments become re-suspended during storm events. The authors concluded that although the WQI effectively separates oil and grease from water, re-suspension of the settled matter appears to limit removal efficiencies. Actual removal only occurs when the residuals are removed from the WQI (Schueler 1992).

A 1990 report by API found that the efficiency of oil and water separation in a WQI is inversely proportional to the ratio of the discharge rate to the unit's surface area. Due to the small capacity of the WQI, the discharge rate is typically very high and the detention time is very short. For example, the MWCOG study found that the average detention time in a WQI is less than 0.5 hour. This can result in minimal pollutant settling (API, 1990). However, the addition of coalescing units in many current WQI units may increase oil/water separation efficiency. Most coalescing units are designed to achieve a specific outlet concentration of oil and grease (for example, 10-15 mg/L oil and grease).

Pollutant removal in stormwater inlets can be somewhat improved using inserts, which are promoted for removal of oil and grease, trash, debris, and sediment. Some inserts are designed to drop directly into existing catch basins, while others may require extensive retrofit construction.

### Siting Criteria

Oil/water separation units are often utilized in specific industrial areas, such as airport aprons, equipment washdown areas, or vehicle storage areas. In these instances, runoff from the area of concern will usually be diverted directly into the unit, while all other runoff is sent to the storm drain downstream from the oil/water separator. Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and resuspending settled pollutants.

### Additional Design Guidelines

Prior to WQI design, the site should be evaluated to determine if another BMP would be more cost-effective in removing the pollutants of concern. WQIs should be used when no other BMP is feasible. The WQI should be constructed near a storm drain network so that flow can be easily diverted to the WQI for treatment (NVPDC, 1992). Any construction activities within the

drainage area should be completed before installation of the WQI, and the drainage area should be revegetated so that the sediment loading to the WQI is minimized.

WQIs are most effective for small drainage areas. Drainage areas of 0.4 hectares (1 acre) or less are often recommended. WQIs are typically used in an off-line configuration (i.e., portions of runoff are diverted to the WQI), but they can be used as on-line units (i.e., receive all runoff). Generally, off-line units are designed to handle the first 1.3 centimeters (0.5 inches) of runoff from the drainage areas. Upstream isolation/diversion structures can be used to divert the water to the off-line structure (Schueler, 1992). On-line units receive higher flows that will likely cause increased turbulence and resuspension of settled material, thereby reducing WQI performance.

Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and resuspending settled pollutants. WQIs are available as pre-manufactured units or can be cast in place. Reinforced concrete should be used to construct below-grade WQIs. The WQIs should be water tight to prevent possible ground water contamination.

## **Maintenance**

Typical maintenance of WQIs includes trash removal if a screen or other debris capturing device is used, and removal of sediment using a vacuum truck. Operators need to be properly trained in WQI maintenance. Maintenance should include keeping a log of the amount of sediment collected and the date of removal. Some cities have incorporated the use of GIS systems to track sediment collection and to optimize future catch basin cleaning efforts.

One study (Pitt, 1985) concluded that WQIs can capture sediments up to approximately 60 percent of the sump volume. When sediment fills greater than 60 percent of their volume, catch basins reach steady state. Storm flows can then resuspend sediments trapped in the catch basin, and will bypass treatment. Frequent clean-out can retain the volume in the catch basin sump available for treatment of stormwater flows.

At a minimum, these inlets should be cleaned at least twice during the wet season. Two studies suggest that increasing the frequency of maintenance can improve the performance of catch basins, particularly in industrial or commercial areas. One study of 60 catch basins in Alameda County, California, found that increasing the maintenance frequency from once per year to twice per year could increase the total sediment removed by catch basins on an annual basis (Mineart and Singh, 1994). Annual sediment removed per inlet was 54 pounds for annual cleaning, 70 pounds for semi-annual and quarterly cleaning, and 160 pounds for monthly cleaning. For catch basins draining industrial uses, monthly cleaning increased total annual sediment collected to six times the amount collected by annual cleaning (180 pounds versus 30 pounds). These results suggest that, at least for industrial uses, more frequent cleaning of catch basins may improve efficiency.

BMPs designed with permanent water sumps, vaults, and/or catch basins (frequently installed below-ground) can become a nuisance due to mosquito and other vector breeding. Preventing mosquito access to standing water sources in BMPs (particularly below-ground) is the best prevention plan, but can prove challenging due to multiple entrances and the need to maintain the hydraulic integrity of the system. BMPs that maintain permanent standing water may require routine inspections and treatments by local mosquito and vector control agencies to

suppress mosquito production. Standing water in oil/water separators may contain sufficient floating hydrocarbons to prevent mosquito breeding, but this is not a reliable control alternative to vector exclusion or chemical treatment.

### Cost

A typical pre-cast catch basin costs between \$2,000 and \$3,000; however, oil/water separators can be much more expensive. The true pollutant removal cost associated with catch basins, however, is the long-term maintenance cost. A vactor truck, the most common method of catch basin cleaning, costs between \$125,000 and \$150,000. This initial cost may be high for smaller Phase II communities. However, it may be possible to share a vactor truck with another community. Typical vactor trucks can store between 10 and 15 cubic yards of material, which is enough storage for three to five catch basins. Assuming semi-annual cleaning, and that the vactor truck could be filled and material disposed of twice in one day, one truck would be sufficient to clean between 750 and 1,000 catch basins. Another maintenance cost is the staff time needed to operate the truck. Depending on the regulations within a community, disposal costs of the sediment captured in catch basins may be significant.

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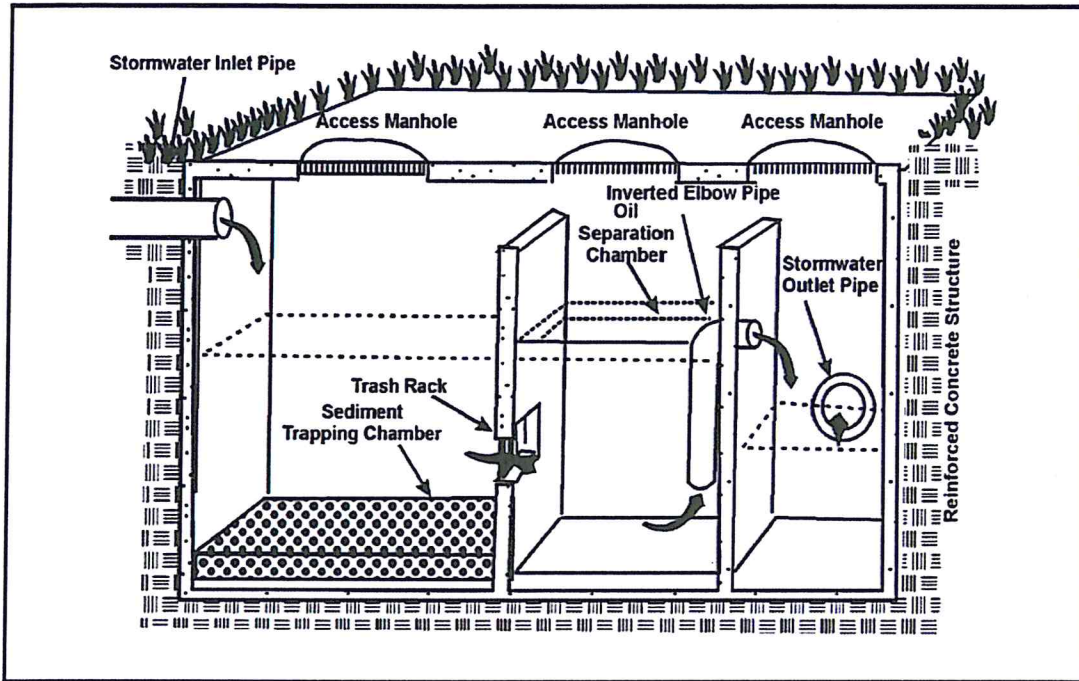
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## Description

A multiple treatment system uses two or more BMPs in series. Some examples of multiple systems include: settling basin combined with a sand filter; settling basin or biofilter combined with an infiltration basin or trench; extended detention zone on a wet pond.

## California Experience

The research wetlands at Fremont, California are a combination of wet ponds, wetlands, and vegetated controls.

## Advantages

- BMPs that are less sensitive to high pollutant loadings, especially solids, can be used to pretreat runoff for sand filters and infiltration devices where the potential for clogging exists.
- BMPs which target different constituents can be combined to provide treatment for all constituents of concern.
- BMPs which use different removal processes (sedimentation, filtration, biological uptake) can be combined to improve the overall removal efficiency for a given constituent.
- BMPs in series can provide redundancy and reduce the likelihood of total system failure.

## Limitations

- Capital costs of multiple systems are higher than for single devices.
- Space requirements are greater than that required for a single technology.

## Design and Sizing Guidelines

Refer to individual treatment control BMP fact sheets.

## Performance

- Be aware that placing multiple BMPs in series does not necessarily result in combined cumulative increased performance. This is because the first BMP may already achieve part of the gain normally achieved by the second BMP. On the other hand, picking the right combination can often help optimize performance of the second BMP since the influent to the second BMP is of more consistent water quality, and thus more consistent performance, thereby allowing the BMP to achieve its highest performance.

## Design Considerations

- Area Required
- Slope
- Water Availability
- Hydraulic Head
- Environmental Side-effects

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	■
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	■
<input checked="" type="checkbox"/>	Metals	■
<input checked="" type="checkbox"/>	Bacteria	▲
<input checked="" type="checkbox"/>	Oil and Grease	■
<input checked="" type="checkbox"/>	Organics	■

### Legend (Removal Effectiveness)

- Low                      ■ High  
▲ Medium



- When addressing multiple constituents through multiple BMPs, one BMP may optimize removal of a particular constituent, while another BMP optimizes removal of a different constituent or set of constituents. Therefore, selecting the right combination of BMPs can be very constructive in collectively removing multiple constituents.

**Siting Criteria**

Refer to individual treatment control BMP fact sheets.

**Additional Design Guidelines**

- When using two or more BMPs in series, it may be possible to reduce the size of BMPs.
- Existing pretreatment requirements may be able to be avoided when using some BMP combinations.

**Maintenance**

Refer to individual treatment control BMP fact sheets.

**Cost**

Refer to individual treatment control BMP fact sheets.

**Resources and Sources of Additional Information**

Refer to individual treatment control BMP fact sheets.

## Description

Vortex separators: (alternatively, swirl concentrators) are gravity separators, and in principle are essentially wet vaults. The difference from wet vaults, however, is that the vortex separator is round, rather than rectangular, and the water moves in a centrifugal fashion before exiting. By having the water move in a circular fashion, rather than a straight line as is the case with a standard wet vault, it is possible to obtain significant removal of suspended sediments and attached pollutants with less space. Vortex separators were originally developed for combined sewer overflows (CSOs), where it is used primarily to remove coarse inorganic solids. Vortex separation has been adapted to stormwater treatment by several manufacturers.

## California Experience

There are currently about 100 installations in California.

## Advantages

- May provide the desired performance in less space and therefore less cost.
- May be more cost-effective pre-treatment devices than traditional wet or dry basins.
- Mosquito control may be less of an issue than with traditional wet basins.

## Limitations

- As some of the systems have standing water that remains between storms, there is concern about mosquito breeding.
- It is likely that vortex separators are not as effective as wet vaults at removing fine sediments, on the order 50 to 100 microns in diameter and less.
- The area served is limited by the capacity of the largest models.
- As the products come in standard sizes, the facilities will be oversized in many cases relative to the design treatment storm, increasing the cost.
- The non-steady flows of stormwater decreases the efficiency of vortex separators from what may be estimated or determined from testing under constant flow.
- Do not remove dissolved pollutants.

## Design Considerations

- Service Area
- Settling Velocity
- Appropriate Sizing
- Inlet Pipe Diameter

## Targeted Constituents

<input checked="" type="checkbox"/>	Sediment	▲
<input checked="" type="checkbox"/>	Nutrients	●
<input checked="" type="checkbox"/>	Trash	
<input checked="" type="checkbox"/>	Metals	●
	Bacteria	
<input checked="" type="checkbox"/>	Oil and Grease	
<input checked="" type="checkbox"/>	Organics	

### Legend (Removal Effectiveness)

- Low
- High
- ▲ Medium



- A loss of dissolved pollutants may occur as accumulated organic matter (e.g., leaves) decomposes in the units.

### Design and Sizing Guidelines

The stormwater enters, typically below the effluent line, tangentially into the basin, thereby imparting a circular motion in the system. Due to centrifugal forces created by the circular motion, the suspended particles move to the center of the device where they settle to the bottom. There are two general types of vortex separation: free vortex and dampened (or impeded) vortex. Free vortex separation becomes dampened vortex separation by the placement of radial baffles on the weir-plate that impede the free vortex-flow pattern.

It has been stated with respect to CSOs that the practical lower limit of vortex separation is a particle with a settling velocity of 12 to 16.5 feet per hour (0.10 to 0.14 cm/s). As such, the focus for vortex separation in CSOs has been with settleable solids generally 200 microns and larger, given the presence of the lighter organic solids. For inorganic sediment, the above settling velocity range represents a particle diameter of 50 to 100 microns. Head loss is a function of the size of the target particle. At 200 microns it is normally minor but increases significantly if the goal is to remove smaller particles.

The commercial separators applied to stormwater treatment vary considerably with respect to geometry, and the inclusion of radial baffles and internal circular chambers. At one extreme is the inclusion of a chamber within the round concentrator. Water flows initially around the perimeter between the inner and outer chambers, and then into the inner chamber, giving rise to a sudden change in velocity that purportedly enhances removal efficiency. The opposite extreme is to introduce the water tangentially into a round manhole with no internal parts of any kind except for an outlet hood. Whether the inclusion of chambers and baffles gives better performance is unknown. Some contend that free vortex, also identified as swirl concentration, creates less turbulence thereby increasing removal efficiency. One product is unique in that it includes a static separator screen.

- Sizing is based on the peak flow of the design treatment event as specified by local government.
- If an in-line facility, the design peak flow is four times the peak of the design treatment event.
- If an off-line facility, the design peak flow is equal to the peak of the design treatment event.
- Headloss differs with the product and the model but is generally on the order of one foot or less in most cases.

### Construction/Inspection Considerations

No special considerations.

### Performance

Manufacturer's differ with respect to performance claims, but a general statement is that the manufacturer's design and rated capacity (cfs) for each model is based on and believed to achieve an aggregate reduction of 90% of all particles with a specific gravity of 2.65 (glacial sand) down to 150 microns, and to capture the floatables, and oil and grease. Laboratory tests of

two products support this claim. The stated performance expectation therefore implies that a lesser removal efficiency is obtained with particles less than 150 microns, and the lighter, organic settleables. Laboratory tests of one of the products found about 60% removal of 50 micron sand at the expected average operating flow rate

Experience with the use of vortex separators for treating combined sewer overflows (CSOs), the original application of this technology, suggests that the lower practical limit for particle removal are particles with a settling velocity of 12 feet per hour (Sullivan, 1982), which represents a particle diameter of 100 to 200 microns, depending on the specific gravity of the particle. The CSO experience therefore seems consistent with the limited experience with treating stormwater, summarized above

Traditional treatment technologies such as wet ponds and extended detention basins are generally believed to be more effective at removing very small particles, down to the range of 10 to 20 microns. Hence, it is intuitively expected that vortex separators do not perform as well as the traditional wet and dry basins, and filters. Whether this matters depends on the particle size distribution of the sediments in stormwater. If the distribution leans towards small material, there should be a marked difference between vortex separators and, say, traditional wet vaults. There are little data to support this conjecture

In comparison to other treatment technologies, such as wet ponds and grass swales, there are few studies of vortex separators. Only two of manufactured products currently available have been field tested. Two field studies have been conducted. Both achieved in excess of 80% removal of TSS. However, the test was conducted in the Northeast (New York state and Maine) where it is possible the stormwater contained significant quantities of deicing sand. Consequently, the influent TSS concentrations and particle size are both likely considerably higher than is found in California stormwater. These data suggest that if the stormwater particles are for the most part fine (i.e., less than 50 microns), vortex separators will not be as efficient as traditional treatment BMPs such as wet ponds and swales, if the latter are sized according to the recommendations of this handbook.

There are no equations that provide a straightforward determination of efficiency as a function of unit configuration and size. Design specifications of commercial separators are derived from empirical equations that are unique and proprietary to each manufacturer. However, some general relationships between performance and the geometry of a separator have been developed. CSO studies have found that the primary determinants of performance of vortex separators are the diameters of the inlet pipe and chamber with all other geometry proportional to these two.

Sullivan et al. (1982) found that performance is related to the ratios of chamber to inlet diameters,  $D_2/D_1$ , and height between the inlet and outlet and the inlet diameter,  $H_1/D_1$ , shown in Figure 3. The relationships are: as  $D_2/D_1$  approaches one, the efficiency decreases; and, as the  $H_1/D_1$  ratio decreases, the efficiency decreases. These relationships may allow qualitative comparisons of the alternative designs of manufacturers. Engineers who wish to apply these concepts should review relevant publications presented in the References.

## Siting Criteria

There are no particularly unique siting criteria. The size of the drainage area that can be served by vortex separators is directly related to the capacities of the largest models.

**Additional Design Guidelines**

Vortex separators have two capacities if positioned as in-line facilities, a treatment capacity and a hydraulic capacity. Failure to recognize the difference between the two may lead to significant under sizing; i.e., too small a model is selected. This observation is relevant to three of the five products. These three technologies all are designed to experience a unit flow rate of about 24 gallons/square foot of separator footprint at the peak of the design treatment event. This is the horizontal area of the separator zone within the container, not the total footprint of the unit. At this unit flow rate, laboratory tests by these manufacturers have established that the performance will meet the general claims previously described. However, the units are sized to handle 100 gallons/square foot at the peak of the hydraulic event. Hence, in selecting a particular model the design engineer must be certain to match the peak flow of the design event to the stated treatment capacity, not the hydraulic capacity. The former is one-fourth the latter. If the unit is positioned as an off-line facility, the model selected is based on the capacity equal to the peak of the design treatment event.

**Maintenance**

Maintenance consists of the removal of accumulated material with an eductor truck. It may be necessary to remove and dispose the floatables separately due to the presence of petroleum product.

***Maintenance Requirements***

Remove all accumulated sediment, and litter and other floatables, annually, unless experience indicates the need for more or less frequent maintenance.

**Cost**

Manufacturers provide costs for the units including delivery. Installation costs are generally on the order of 50 to 100 % of the manufacturer's cost. For most sites the units are cleaned annually.

***Cost Considerations***

The different geometry of the several manufactured separators suggests that when comparing the costs of these systems to each other, that local conditions (e.g., groundwater levels) may affect the relative cost-effectiveness.

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## Description

Drain inserts are manufactured filters or fabric placed in a drop inlet to remove sediment and debris. There are a multitude of inserts of various shapes and configurations, typically falling into one of three different groups: socks, boxes, and trays. The sock consists of a fabric, usually constructed of polypropylene. The fabric may be attached to a frame or the grate of the inlet holds the sock. Socks are meant for vertical (drop) inlets. Boxes are constructed of plastic or wire mesh. Typically a polypropylene "bag" is placed in the wire mesh box. The bag takes the form of the box. Most box products are one box; that is, the setting area and filtration through media occur in the same box. Some products consist of one or more trays or mesh grates. The trays may hold different types of media. Filtration media vary by manufacturer. Types include polypropylene, porous polymer, treated cellulose, and activated carbon.

## California Experience

The number of installations is unknown but likely exceeds a thousand. Some users have reported that these systems require considerable maintenance to prevent plugging and bypass.

## Advantages

- Does not require additional space as inserts as the drain inlets are already a component of the standard drainage systems.
- Easy access for inspection and maintenance.
- As there is no standing water, there is little concern for mosquito breeding.
- A relatively inexpensive retrofit option.

## Limitations

Performance is likely significantly less than treatment systems that are located at the end of the drainage system such as ponds and vaults. Usually not suitable for large areas or areas with trash or leaves than can plug the insert.

## Design and Sizing Guidelines

Refer to manufacturer's guidelines. Drain inserts come in many configurations but can be placed into three general groups: socks, boxes, and trays. The sock consists of a fabric, usually constructed of polypropylene. The fabric may be attached to a frame or the grate of the inlet holds the sock. Socks are meant for vertical (drop) inlets. Boxes are constructed of plastic or wire mesh. Typically a polypropylene "bag" is placed in the wire mesh box. The bag takes the form of the box. Most box products are

## Design Considerations

- Use with other BMPs
- Fit and Seal Capacity within Inlet

## Targeted Constituents

- Sediment
- Nutrients
- Trash
- Metals
- Bacteria
- Oil and Grease
- Organics

## Removal Effectiveness

See New Development and Redevelopment Handbook-Section 5.



one box; that is, the setting area and filtration through media occurs in the same box. One manufacturer has a double-box. Stormwater enters the first box where setting occurs. The stormwater flows into the second box where the filter media is located. Some products consist of one or more trays or mesh grates. The trays can hold different types of media. Filtration media vary with the manufacturer: types include polypropylene, porous polymer, treated cellulose, and activated carbon.

**Construction/Inspection Considerations**

Be certain that installation is done in a manner that makes certain that the stormwater enters the unit and does not leak around the perimeter. Leakage between the frame of the insert and the frame of the drain inlet can easily occur with vertical (drop) inlets.

**Performance**

Few products have performance data collected under field conditions.

**Siting Criteria**

It is recommended that inserts be used only for retrofit situations or as pretreatment where other treatment BMPs presented in this section area used.

**Additional Design Guidelines**

Follow guidelines provided by individual manufacturers.

**Maintenance**

Likely require frequent maintenance, on the order of several times per year.

**Cost**

- The initial cost of individual inserts ranges from less than \$100 to about \$2,000. The cost of using multiple units in curb inlet drains varies with the size of the inlet.
- The low cost of inserts may tend to favor the use of these systems over other, more effective treatment BMPs. However, the low cost of each unit may be offset by the number of units that are required, more frequent maintenance, and the shorter structural life (and therefore replacement).

**References and Sources of Additional Information**

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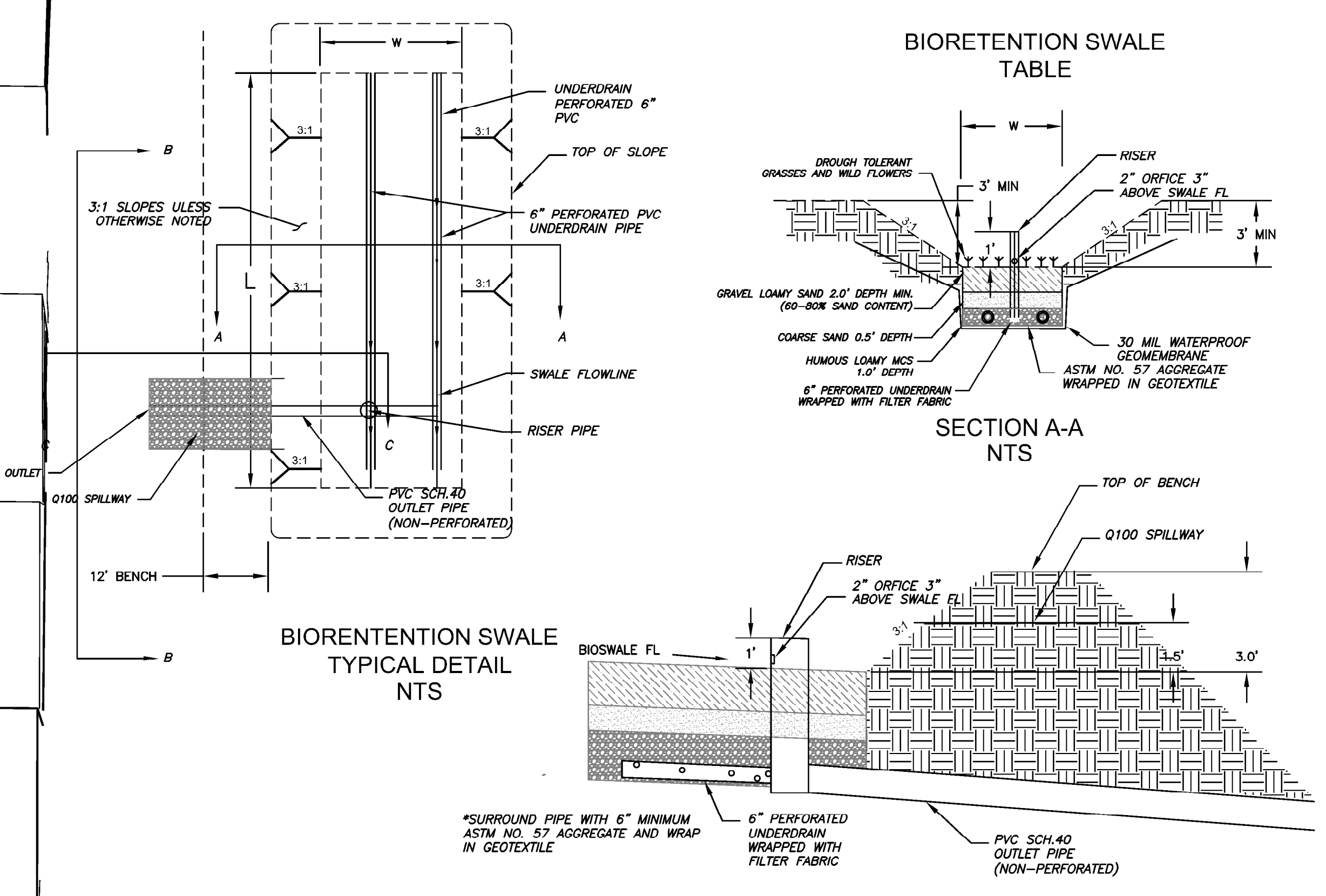
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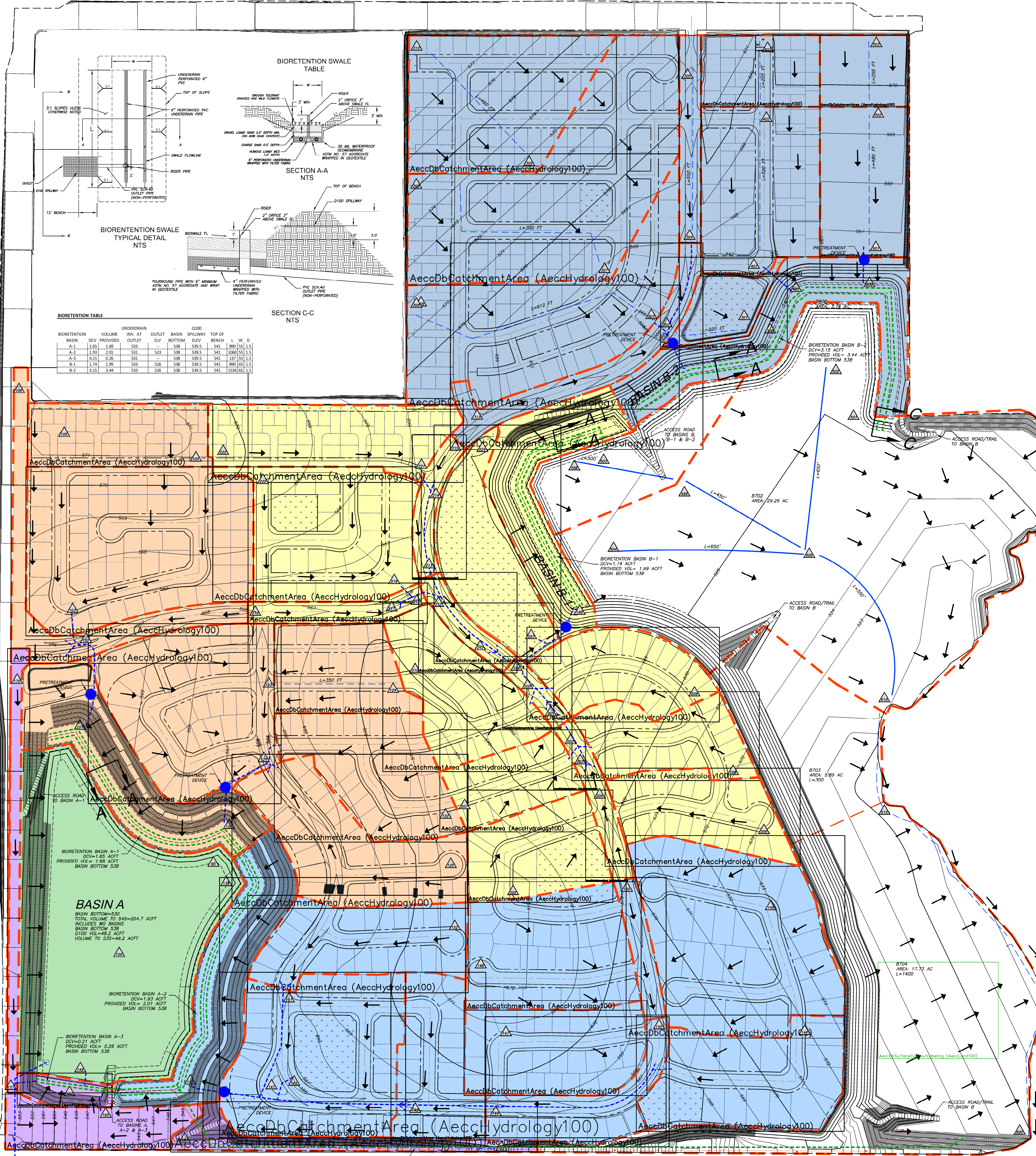
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**BIORETENTION TABLE**

BIORETENTION BASIN	DCV PROVIDED	UNDERDRAIN VOLUME	OUTLET ELEV.	Q100 SPILLWAY BOTTOM ELEV.	Q100 SPILLWAY TOP OF BENCH ELEV.	L	W	D
A-1	1.65	1.69	533	538	539.5	541	800	55
A-2	1.93	2.01	531	523	538	539.5	541	1000
A-3	0.21	0.26	533	526	538	539.5	541	137
B-1	1.74	1.99	533	526	538	539.5	541	890
B-2	3.15	3.44	533	526	538	539.5	541	1536



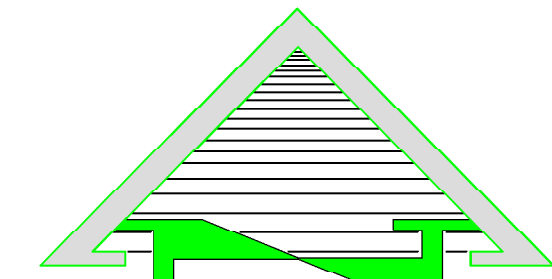
BASIN A OUTLET DAYLIGHT TO CUCAMONGA

BIORETENTION BASINS A-1, A-2, A-3 LOWFLOW OUTLET

West Basin - BMP Low Flow Pipe Outlet

**LEGEND**

- LINK NODE
- FLOW DIRECTION
- BIORETENTION UNDERDRAIN AND LOWFLOW OUTLET
- DRAINAGE AREA BOUNDARY
- PROPOSED STORM DRAIN PIPE
- DRAINAGE PATH LENGTH / SUB AREA DRAINAGE PATH
- PRETREATMENT VORTEXES UNIT
- TRIBUTARY TO WATER QUALITY BASIN A-1
- TRIBUTARY TO WATER QUALITY BASIN A-2
- TRIBUTARY TO WATER QUALITY BASIN A-3
- TRIBUTARY TO WATER QUALITY BASIN B-1
- TRIBUTARY TO WATER QUALITY BASIN B-2



GRAPHIC SCALE  
( IN FEET )  
1 inch = 100 ft.

10/14/2015

DATE: 12-21-15

WQMP EXHIBIT

