DESIGN EXAMPLES—SECTION 2

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2.0 CASE STUDY—STAPLETON REDEVELOPMENT

2.1 Project Setting

The following example illustrates application of this *Manual* for the design of conveyance and detention facilities, including use of computational spreadsheets described in pertinent sections of the *Manual*. Redevelopment of the former Stapleton International Airport in Denver poses significant opportunities and challenges for stormwater management. Like many airports, the site was graded to create gentle grades for runway operations. A formal storm sewer system was installed to control minor storm events, while major 100-year storms were conveyed via sheet flow or by overflow open channels. Consequently, significant drainage infrastructure improvements were needed. The challenge was to strike a balance between conveyance and detention to optimize the reuse of the existing system and minimize grading and demolition.

Figure 1 shows the project location and hydrologic setting for the *Stapleton East-West Linear Park Flood Control Project*. As indicated on Figure 2, the project incorporates a watershed of 104.0 acres that has been delineated into Sub-Basins "031" and "032". The mixture of residential, park, and school uses represents an average surface imperviousness of 44%. This assignment involved providing preliminary-level engineering for a sub-regional detention pond and associated outfall sewer and overflow channel. It is expected to be constructed by 2002 to support redevelopment of the Stapleton site near Yosemite Boulevard and 26th Avenue. The pond had to be designed to meet both detention volume requirements and enable reuse of an existing 54-inch storm sewer that outfalls to Westerly Creek. As a result, the detention volume had to be computed by V=KA, the modified Federal Aviation Administration (FAA) Method and a synthetic unit hydrograph to determine the controlling criteria.

2.2 Project Objectives

A multi-disciplinary team of engineers, landscape architects, planners, and scientists was formed to plan and design facilities to achieve the following objectives:

Provide a detention facility that offers multiple benefits, including park and recreation uses, flood control, water quality enhancement, and educational benefits.

Minimize demolition in and grading of the sub-basin by designing detention facilities to enable a retrofit and reuse of an existing 54-inch storm sewer.

Perform hydraulic engineering to determine the capacity of the existing outfall system and preliminarily size new collection and conveyance systems required to support land development at Stapleton.

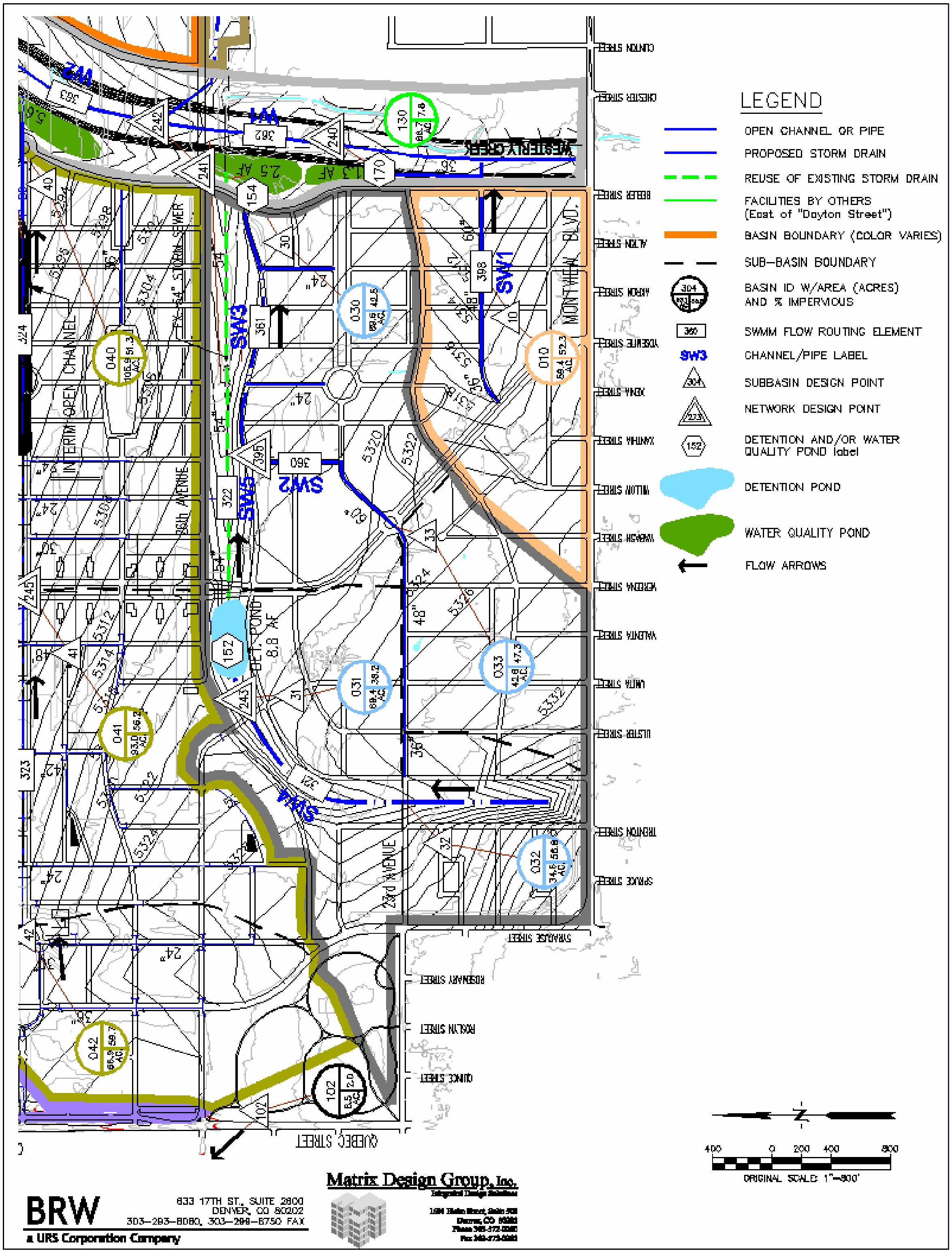


Figure 1—Stapleton Redevelopment Drainage Map

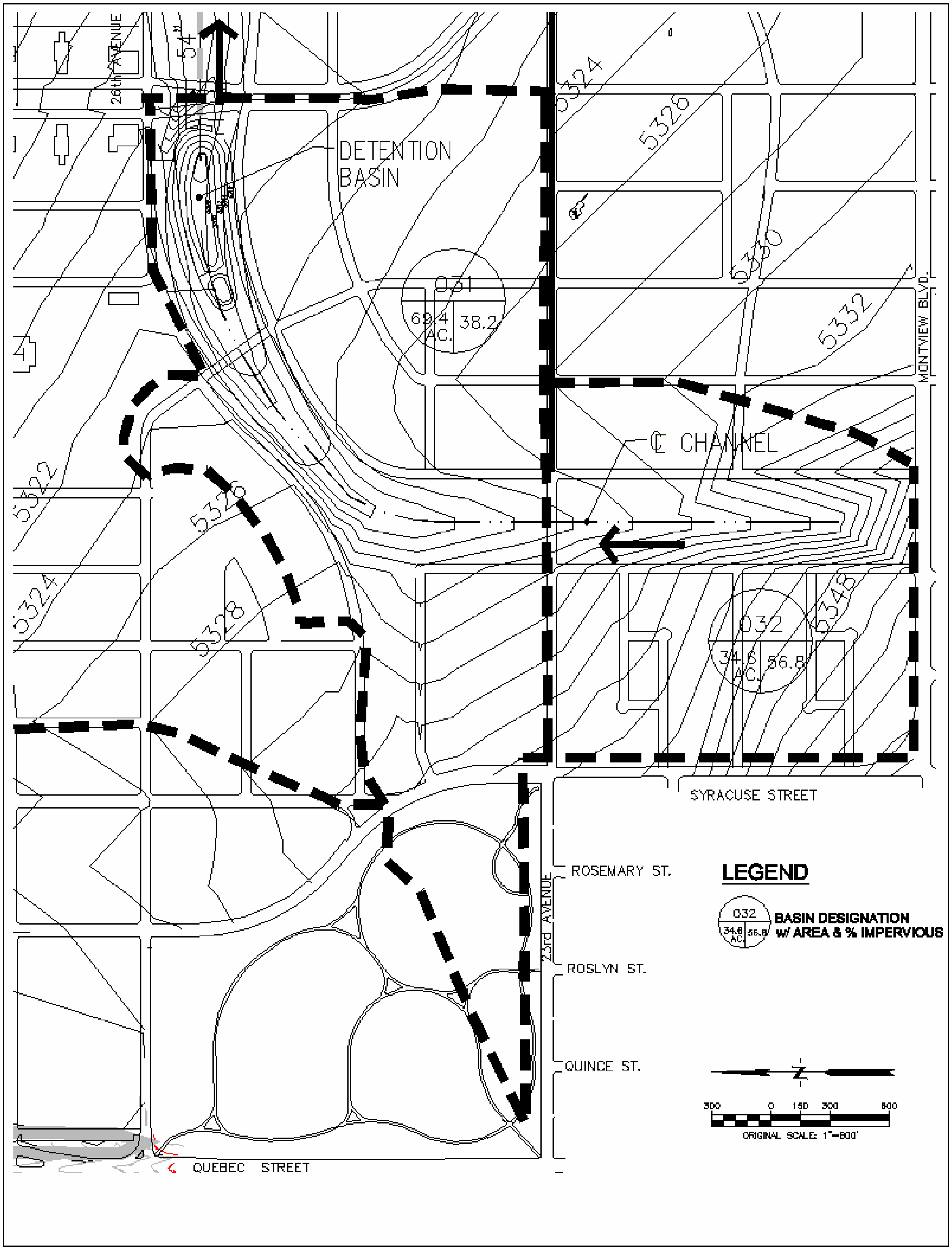


Figure 2—Stapleton Redevelopment Drainage Catchment Map

2.3 Hydrologic Evaluation For Detention Pond Sizing

Three hydrologic methods were used to establish the required detention pond size:

1. The Colorado Urban Hydrograph Procedure (CUHP) and UDSWM
2. The modified FAA Method
3. The V=KA approach

Because of the basin area (greater than 90 acres) and the need to match discharges with the established capacity of an outfall system, the utilization of a more detailed assessment with a synthetic hydrograph generated by CUHP and UDSWM was required. All three methods were used to verify reasonableness of the results and to ensure that appropriate local detention sizing criteria were satisfied.

2.3.1 CUHP and UDSWM

Input data for CUHP and UDSWM are shown in Table 1. Two discharge rates were considered for the pond routing: the allowable release rate and the flow capacity of the 54-inch storm sewer. The allowable release for the 104-acre basin was 88.4 cfs, relating to 0.85 cfs per acre for Type B Soils. The capacity of the 54-inch RCP (n=0.013, slope=0.38%) was 121 cfs and, consequently, the allowable release rate governed the design of the detention volume. Storage characteristics were developed with a preliminary grading plan to enable stage-storage-discharge data to be used in UDSWM routing.

Table 2 presents the modeling results with the required storage volumes for attenuation of flows to the allowable release rate. Figure 3 graphs the inflow and pond discharge hydrographs for the 100-year storm and shows the required minimum detention volume of 8.8 acre-feet.

Table 1—CUHP and UDSWM Input

CUHP Basin Data

Basin	Area (acres)	Imperviousness	Slope	Length (ft)	Time of Concentration (min)	Centroid Length (ft)
031	69.4	38.2%	0.8%	3820	31.2	1600
032	34.6	56.8%	2.0%	1240	16.9	590

Note: Hydrologic Soil Group B Soils are used in this example.

UDSWM Pond Routing Data

Elevation (Feet)	Depth (Feet)	Storage (Acre-feet)	Discharge (cfs)
5308.7	0.0	0.00	0.0
5310.0	1.3	1.99	0.1
5310.0	1.3	2.00	20.0
5312.2	3.5	4.50	23.9
5312.3	3.6	4.60	88.4
5314.0	5.3	8.78	88.4
5314.1	5.4	8.80	90.0
5316.0	7.3	20.00	5000.0

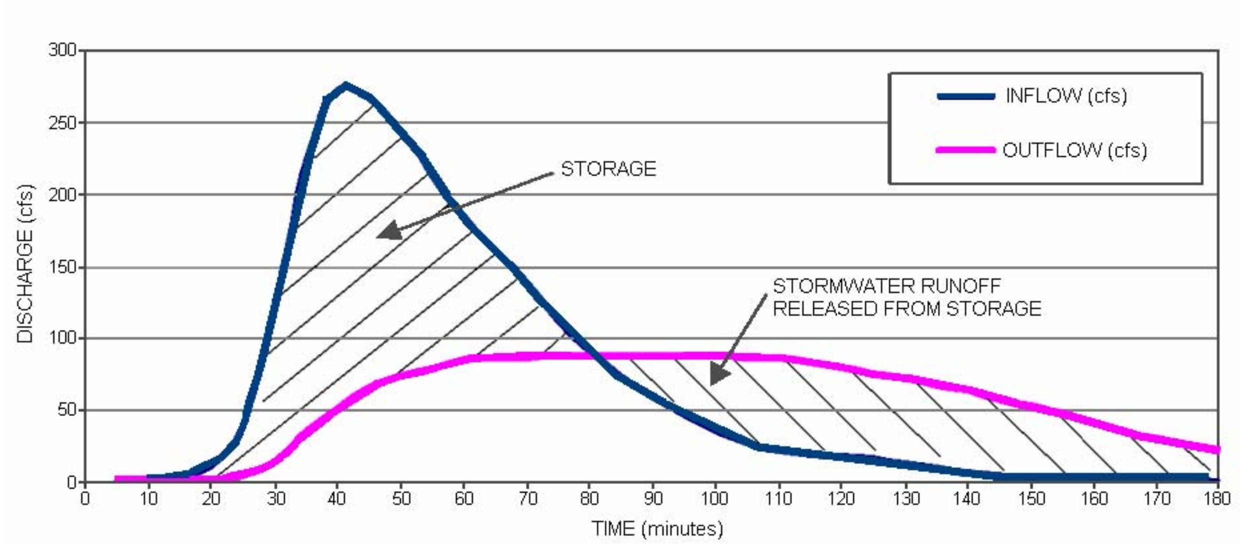


Figure 3—Detention Pond Inflow/Outflow Hydrographs

Table 2—CUHP and UDSWM Modeling Results

Return Period	Q_{in} **(cfs)**	Q_{out} **(cfs)**	**Detention Storage Volume (acre-feet)**
2	44	20	2.1
5	83	22	3.3
10	106	24	4.3
50	222	88	7.0
100	273	88	8.8

2.3.2 Rational Method Hydrology

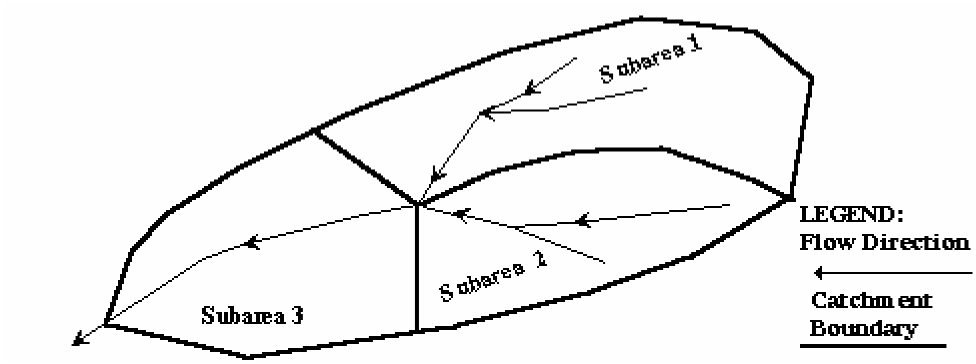
For purposes of this design example, the basin was also analyzed using the Rational Method. Figures 4 and 5 are spreadsheets used to determine the composite runoff coefficients for the basin; they show the 10-year composite runoff coefficient to be 0.55 and the 100-year composite runoff coefficient to be 0.65. By evaluating the basin runoff coefficients, overland flow path, and concentrated flow path, the resulting time of concentration is 35 minutes.

The time of concentration is related to rainfall intensity for use in the Rational Method. By inputting the basin area, runoff coefficients, and rainfall intensity into the Rational Method equation, Q=CIA. Figures 6 and 7 show the 10-year and 100-year peak discharges into the detention pond from the 104-acre drainage basin to be 131 cfs and 250 cfs, respectively.

Area-Weighting for Runoff Coefficient Calculation

Project Title =	Stapleton Redevelopment Area
Catchment ID =	31.1, 31 and 32
Return Period =	10yr (initial event), 100yr (major event)

Illustration



Instructions: For each catchment Sub area, enter values for A and C.

(10-yr Event)

Subarea ID input	Area acres A input	Runoff Coeff C input	Product CA output
31.1A	5.23	0.50	2.62
31.1B	1.10	0.60	0.66
31.1C	1.19	0.50	0.60
31.1D	0.26	0.50	0.13
31.1E	0.42	0.50	0.21
31	61.20	0.50	30.60
32	34.60	0.65	22.49
Sum:	**104.00**	**Sum:**	**57.30**

(100-yr Event)

Subarea ID input	Area acres A input	Runoff Coeff C input	Product CA output
31.1A	5.23	0.60	3.14
31.1B	1.10	0.70	0.77
31.1C	1.19	0.60	0.71
31.1D	0.26	0.60	0.16
31.1E	0.42	0.60	0.25
31	61.20	0.60	36.72
32	34.60	0.75	25.95
Sum:	**104.00**	**Sum:**	**67.70**

Weighted Runoff Coeffecient

(sum CA / sum A) = **0.55** (10-yr Event) **0.65** (100-yr Event)

Figures 4 & 5—Area-Weighting for Runoff Coefficient Calculation

CALCULATION OF A PEAK RUNOFF USING RATIONAL METHOD

I. Catchment Hydrologic Data

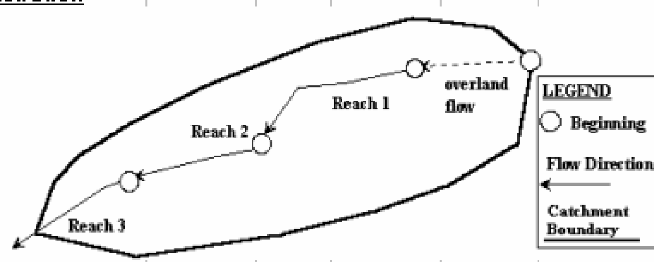
(10-yr Event)			(100-yr Event)	
Catchment ID =	31, 32	(input)	Catchment ID	31, 32
Area (A) =	104.00	(input)	Area (A) =	104.00
Runoff Coeff (C)=	0.55	(input)	Runoff Coeff (C	0.65

II. Rainfall Information I (inch/hr) = C1 * P1 /(C2 + Td)^ C3

Tr =	10	years	(input)	Tr =	100	years
C1 =	28.50		(input)	C1 =	28.50	
C2=	10.00		(input)	C2=	10.00	
C3=	0.786		(input)	C3=	0.786	
P1=	1.61	inches	(input)	P1=	2.60	inches

III. Analysis of Flow Time (Time of Concentration) for a Catchment

Illustration



SCS Type	Heavy Meadow	Tillage Field	Forest Woods	Short Pasture	Bare Soil	Grass Swale	Paved Flow
Conveyance	2.50	5.00	5.00	7.00	10.00	15.00	20.00

Calculations:

Reach ID	Slope S ft/ft input	Length L ft input	5-yr Runoff Coeff C-5 input	SCS Convey-ance input	Flow Velocity V fps output	Flow Time Tf minutes output
Overland	0.0050	60.00	0.45		0.09	11.43
1	0.0060	550.00		20.00	1.55	5.92
2	0.0200	1000.00		20.00	2.83	5.89
3	0.0084	3000.00		20.00	1.83	27.28
4						
5						
Sum		4610.00				50.51
					Regional Tc =	35.61
					Min value of Regional Tc and Flow Time is Recommended Tc =	35.61

IV. Peak Runoff Prediction

(10-yr Event)			(100-yr Event)		
I =	2.28	inch/hr	I =	3.68	inch/hr
Qp =	130.56	cfs	Qp =	249.10	cfs

Figures 6 and 7—Calculation of a Peak Runoff Using Rational Method

2.3.3 FAA Method

The modified FAA Method utilizes the Rational Method to estimate detention volumes using a mass diagram. It is appropriate for basins smaller than 160 acres without multiple detention ponds or unusual watershed storage characteristics. Table 3 highlights key input data for use of the FAA Method.

Table 3—FAA Method Input Data

	Area (acres)	Runoff Coefficient C	SCS Soil Type	T_c (min)	Release Rate (cfs/acre)	1-Hour Precip. (in)
10-Year	104	0.55	B	35	0.23	1.60
100-Year	104	0.65	B	35	0.85	2.60

Figure 8 shows the computation of the 10-year storage volume using the FAA method. The plot of mass inflow versus mass outflow is depicted on Figure 9. Figures 10 and 11 show the corresponding information for the 100-year storage volume. The vertical difference between the plots of the 100-year inflow and modified outflow relates to a minimum detention volume of 382,399 cubic feet (8.8 acre-feet).

2.3.4 Denver Regression Equation

For checking purposes, the use of the formula V=KA is required in the Denver Metropolitan area. The formulae for the coefficient, K, for initial and major storm events are stated below.

$K_{10} = (0.95I – 1.90)/1000$

$K_{100} = (1.78I – 0.002[I]^2 – 3.56)/1000$

where I = Basin Imperviousness (%)

For a 104-acre basin with an imperviousness of 44%, the corresponding detention volumes are as shown below in Table 4.

Table 4—Detention Volume

	BASIN 031		BASIN 032		TOTAL	
Area =	69.40	acres	34.60	acres	**104.00**	**acres**
Imp. =	38%		57%		**44.4%**	
K_{10} =	0.034		0.052		**0.040**	
K_{100} =	0.062		0.091		**0.072**	
VOL_{10} =	2.387	acre-feet	1.801	acre-feet	**4.188**	**acre-feet**
VOL_{100} =	4.269	acre-feet	3.152	acre-feet	**7.421**	**acre-feet**

For catchments less than 160 acres only. For larger catchments, use hydrograph routing methods.
(Note: for catchments larger than 90 acres, CUHP hydrograph and routing are recommended).

10-YEAR

Design Information (Input)

Catchment Drainage Area	A =	104.00	acres
Runoff Coefficient	C =	0.55	
Predevelopment NRCS Soil Group	Type =	B	B
Return Period for Detention Control	T =	10	10
Time of concentration of Watershed	Tc =	35	minutes
Allowable Unit Release Rate (See Table A)	q =	0.23	Default
One-hour Precipitation	P1 =	1.60	inches

Design Rainfall IDF Formula I = C1* P1/(C2+Td)^C3

Coefficient one	C1 =	28.50	Click Here to Accept Denver Area Default Values
Coefficient two	C2 =	10.00	
Coefficient three	C3 =	0.79	

Determination of Average Outflow from the Basin (Calculated)

Inflow Peak Runoff	Qp-in =	128.92	cfs
Allowable Peak Outflow Rate	Qp-out =	23.92	cfs
Ratio of Qp-out/Qp-in	Ratio =	0.19	

Recommended Unit Flow Release Rate in cfs/acre of tributary catchment within UDFCD boundaries.

Design Frequency	NRCS (SCS) Hydrologic Soil Group		
	A	B	C & D
2-year	0.02	0.03	0.04
5-year	0.07	0.13	0.17
10-year	0.13	0.23	0.30
25-year	0.24	0.41	0.52
50-year	0.33	0.56	0.68
100-year	0.50	0.85	1.00

100-YEAR

Design Information (Input)

Catchment Drainage Area	A =	104.00	acres
Runoff Coefficient	C =	0.65	
Predevelopment NRCS Soil Group	Type =	B	B
Return Period for Detention Control	T =	100	10
Time of concentration of Watershed	Tc =	35	minutes
Allowable Unit Release Rate (See T	q =	0.85	Default
One-hour Precipitation	P1 =	2.60	inches

Design Rainfall IDF Formula I = C1* P1/(C2+Td)^C3

Coefficient one	C1 =	28.50	Click Here to Accept Denver Area Default Values
Coefficient two	C2 =	10.00	
Coefficient three	C3 =	0.79	

Determination of Average Outflow from the Basin (Calculated)

Inflow Peak Runoff	Qp-in =	247.59	cfs
Allowable Peak Outflow Rate	Qp-out =	88.40	cfs
Ratio of Qp-out/Qp-in	Ratio =	0.36	

Recommended Unit Flow Release Rate in cfs/acre of tributary catchme within UDFCD boundaries.

Design Frequency	NRCS (SCS) Hydrologic Soil Group		
	A	B	C & D
2-year	0.02	0.03	0.04
5-year	0.07	0.13	0.17
10-year	0.13	0.23	0.30
25-year	0.24	0.41	0.52
50-year	0.33	0.56	0.68
100-year	0.50	0.85	1.00

Determination of Detention Volume Using Modified FAA Method

Rainfall duration must be entered in an increasing order.

10-YEAR

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
5.00	5.37	92,123	1.00	23.92	7,176	84,947
10.00	4.28	146,791	1.00	23.92	14,352	132,439
15.00	3.59	184,600	1.00	23.92	21,528	163,072
20.00	3.10	213,116	1.00	23.92	28,704	184,412
25.00	2.75	235,851	1.00	23.92	35,880	199,971
30.00	2.47	254,687	1.00	23.92	43,056	211,631
35.00	2.25	270,734	1.00	23.92	50,232	220,502
40.00	2.07	284,699	0.94	22.43	53,820	230,879
45.00	1.92	297,056	0.89	21.26	57,408	239,648
50.00	1.80	308,136	0.85	20.33	60,996	247,140
55.00	1.69	318,180	0.82	19.57	64,584	253,596
60.00	1.59	327,368	0.79	18.94	68,172	259,196
65.00	1.51	335,836	0.77	18.40	71,760	264,076
70.00	1.43	343,692	0.75	17.94	75,348	268,344
75.00	1.36	351,020	0.73	17.54	78,936	272,084
80.00	1.30	357,891	0.72	17.19	82,524	275,367
85.00	1.25	364,359	0.71	16.88	86,112	278,247
90.00	1.20	370,471	0.69	16.61	89,700	280,771
95.00	1.15	376,267	0.68	16.37	93,288	282,979
100.00	1.11	381,779	0.68	16.15	96,876	284,903
105.00	1.07	387,035	0.67	15.95	100,464	286,571
110.00	1.04	392,059	0.66	15.77	104,052	288,007
115.00	1.01	396,873	0.65	15.60	107,640	289,233
120.00	0.97	401,493	0.65	15.45	111,228	290,265
125.00	0.95	405,937	0.64	15.31	114,816	291,121
130.00	0.92	410,218	0.63	15.18	118,404	291,814
135.00	0.89	414,348	0.63	15.06	121,992	292,356
140.00	0.87	418,339	0.63	14.95	125,580	292,759
145.00	0.85	422,200	0.62	14.85	129,168	293,032
150.00	0.83	425,940	0.62	14.75	132,756	293,184
155.00	0.81	429,568	0.61	14.66	136,344	293,224
160.00	0.79	433,089	0.61	14.58	139,932	293,157
165.00	0.77	436,512	0.61	14.50	143,520	292,992
170.00	0.75	439,841	0.60	14.42	147,108	292,733
175.00	0.74	443,082	0.60	14.35	150,696	292,386
180.00	0.72	446,241	0.60	14.29	154,284	291,957
185.00	0.71	449,321	0.59	14.22	157,872	291,449

Stormwater Detention Volume (Cubic Feet) = 293,224
Stormwater Detention Volume (Acre Feet) = 6.731

100-YEAR

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
5.00	8.72	176,919	1.00	88.40	26,520	150,399
10.00	6.95	281,905	1.00	88.40	53,040	228,865
15.00	5.83	354,515	1.00	88.40	79,560	274,955
20.00	5.05	409,280	1.00	88.40	106,080	303,200
25.00	4.47	452,942	1.00	88.40	132,600	320,342
30.00	4.02	489,114	1.00	88.40	159,120	329,994
35.00	3.66	519,932	1.00	88.40	185,640	334,292
40.00	3.37	546,751	0.94	82.88	198,900	347,851
45.00	3.13	570,482	0.89	78.58	212,160	358,322
50.00	2.92	591,761	0.85	75.14	225,420	366,341
55.00	2.74	611,051	0.82	72.33	238,680	372,371
60.00	2.58	628,695	0.79	69.98	251,940	376,755
65.00	2.45	644,958	0.77	68.00	265,200	379,758
70.00	2.32	660,044	0.75	66.30	278,460	381,584
75.00	2.22	674,119	0.73	64.83	291,720	382,399
80.00	2.12	687,313	0.72	63.54	304,980	382,333
85.00	2.03	699,735	0.71	62.40	318,240	381,495
90.00	1.95	711,473	0.69	61.39	331,500	379,973
95.00	1.88	722,604	0.68	60.48	344,760	377,844
100.00	1.81	733,189	0.68	59.67	358,020	375,169
105.00	1.75	743,283	0.67	58.93	371,280	372,003
110.00	1.69	752,932	0.66	58.26	384,540	368,392
115.00	1.63	762,176	0.65	57.65	397,800	364,376
120.00	1.58	771,049	0.65	57.09	411,060	359,989
125.00	1.54	779,583	0.64	56.58	424,320	355,263
130.00	1.49	787,804	0.63	56.10	437,580	350,224
135.00	1.45	795,736	0.63	55.66	450,840	344,896
140.00	1.41	803,400	0.63	55.25	464,100	339,300
145.00	1.38	810,816	0.62	54.87	477,360	333,456
150.00	1.34	817,999	0.62	54.51	490,620	327,379
155.00	1.31	824,965	0.61	54.18	503,880	321,085
160.00	1.28	831,728	0.61	53.87	517,140	314,588
165.00	1.25	838,301	0.61	53.58	530,400	307,901
170.00	1.23	844,695	0.60	53.30	543,660	301,035
175.00	1.20	850,920	0.60	53.04	556,920	294,000
180.00	1.17	856,985	0.60	52.79	570,180	286,805
185.00	1.15	862,900	0.59	52.56	583,440	279,460

Stormwater Detention Volume (Cubic Feet) = 382,399
Stormwater Detention Volume (Acre Feet) = 8.779

Figures 8 and 9—Detention Volume by Modified FAA Method

(See Chapter 5-Runoff of this *Manual* for description of method)

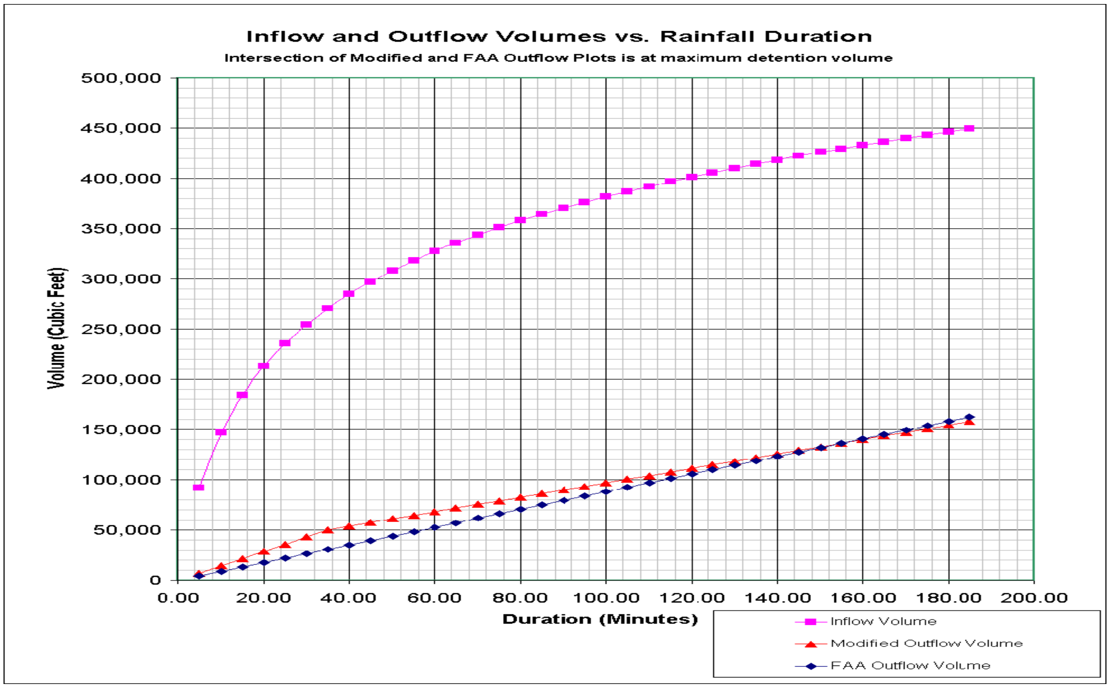


Figure 10—10-Year Modified FAA Method

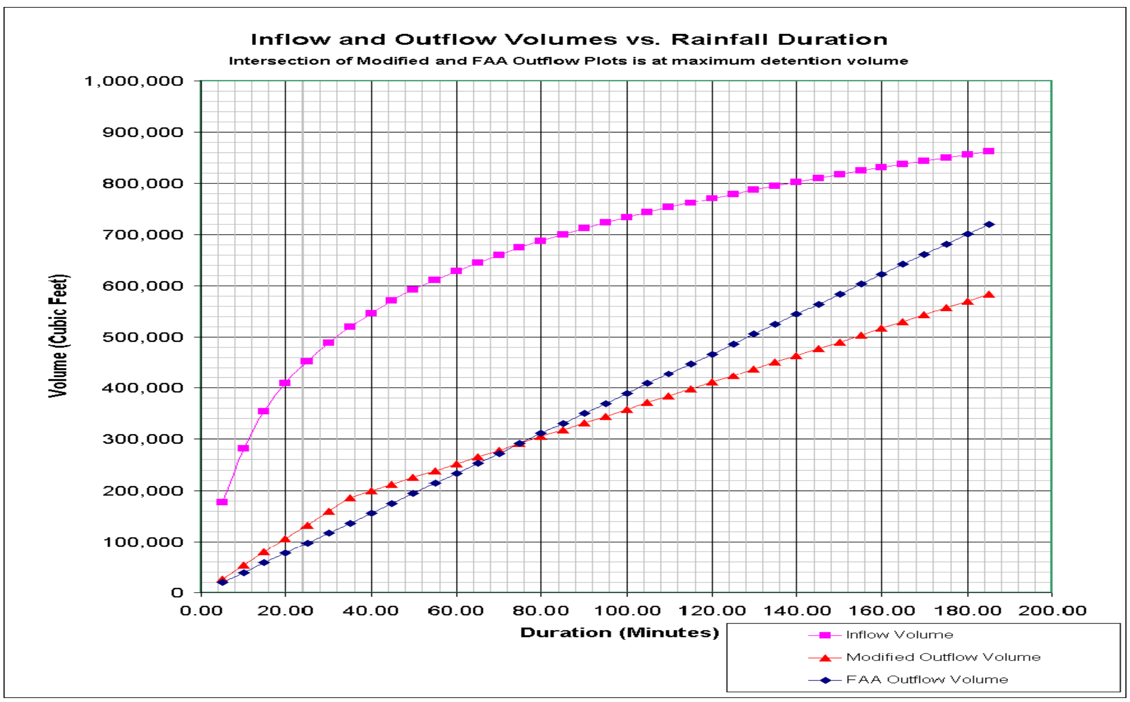


Figure 11—100-Year Modified FAA

2.3.5 Comparison of the Sizing Methodologies

Table 5 offers a comparison of the modeling results for detention sizing.

Table 5—Summary Comparison of Sizing Methodologies

	V=KA (Acre-Feet)	FAA Method (Acre-Feet)	CUHP/SWM (Acre-Feet)
10-Year	4.2	6.7	4.3
100-Year	7.4	8.8	8.8

For the purposes of this design, the results of the CUHP/UDSWM analysis were used with a required storage volume of 8.8 acre-feet.

2.4 Detention Pond Outlet Configuration

A more detailed grading plan and storm sewer layouts for the detention pond area and adjacent roadways are illustrated on Figure 12. In order to prepare a design for the detention pond, it was necessary to confirm the adequacy of pond volume and establish related water surface depths. The outlet had to be designed to restrict discharges to the design criteria for each storm event and corresponding depth (and hydraulic head) condition. Additionally, the water quality capture volume (WQCV) had to be computed and included in the design volume.

Other objectives of the pond design included:

- For aesthetic purposes, the landscape architect determined that a more elongated and contoured shape was desirable.
- In order to provide for safety and to address the potential risk associated with the adjacent elementary school site, a dry detention pond scheme was selected. A maximum depth of 6 ft was provided and a more flatly graded perimeter area was chosen as a safety shelf.
- A multi-stage outlet was designed to control discharges of the WQCV, 10-year, and 100-year events.
- An overflow spillway and overland channel to Westerly Creek had to be provided for events greater than the 100-year storm and emergency operations.
- Due to the embankment height of less than 10 feet, the Colorado State Engineer did not regulate the pond and a Probable Maximum Flood (PMF) analysis was not required. However, in final design the emergency spillway must be designed for the un-attenuated inflow peak 100-year flow rate of 273 cfs or more and the embankment stability checked for a total flow of 273 cfs.

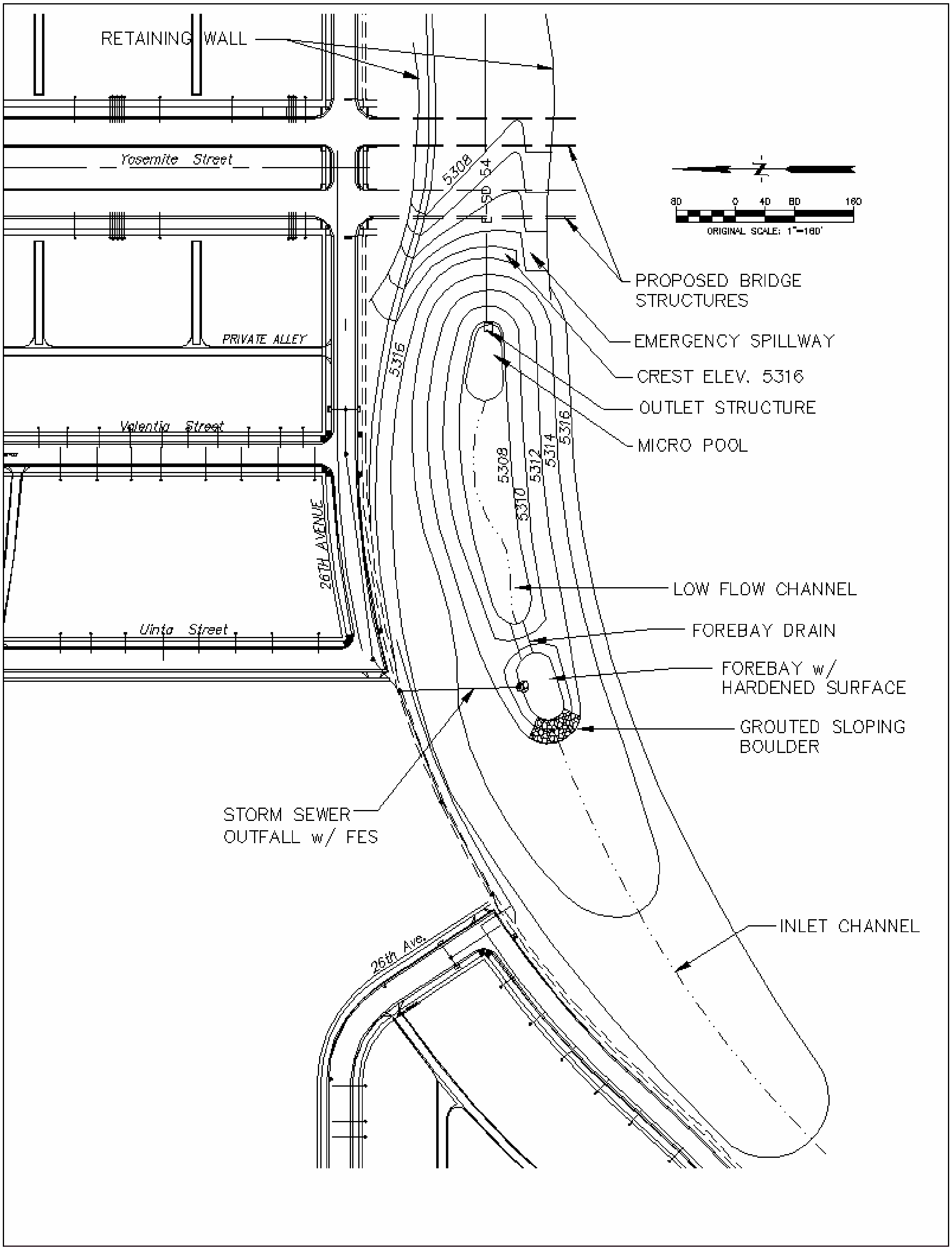


Figure 12—Stapleton Redevelopment Detention Pond Detail

2.4.1 Stage-Storage Relationships

To properly size the outlet works, it is important to develop depth versus cumulative storage volume relationships for the final detention pond configuration, as shown on Table 6. Figure 13 graphically shows the rating curve for the pond.

Table 6—Stapleton East-West Detention Pond Cumulative Volume Analysis

Contour (feet)	Area (sq. ft.)	Avg Area (sq. ft.)	Volume (cu. ft.)	Cum. Vol. (cu. ft.)	Cum. Vol. (ac-ft)
5306	2,788				
		10,992	21,984	21,984	0.50
5308	22,303				
		28,992	57,983	79,967	1.84
5310	36,242				
		52,065	104,131	184,098	4.23
5312	69,696				
		102,551	205,102	389,200	8.93
5314	139,392				
		188,602	377,203	766,403	17.59
5316	242,542				

2.4.2 Water Quality Volume Requirements

The WQCV must also be determined and incorporated into the pond design. Figure 14 (3 pages) shows the computation of the WQCV from the **Extended Dry Detention Spreadsheet** of Volume 3 of this *Manual.* This computation includes the analysis of the perforated plate, trash rack, forebay, micro-pool and outlet structure components for proper operation. As indicated on line 1(D), a volume of 1.99 acre-feet will be required. Figure 15 is the same analysis of the perforated plate for WQCV using the newly developed spreadsheet from Volumes 1 and 2 of this *Manual*. This computation shows a total of 20 holes (1.50-inch diameter with 5 columns and 4 rows) that will release runoff at the appropriate rate for water quality treatment. Figure 16 is the analysis of the 10-year pond outlet orifice to accomplish the desired release rate of 0.23 cfs/acre (Type B soils), or 24 cfs for a 104-acre drainage basin. Figure 17 is the computation form for the 100-year release rate of 0.80 cfs/acre (Type B soils), or 88 cfs for the drainage catchment area.

2.4.3 Final Pond Outlet Configuration

The final recommended outlet configuration is shown in plan and section view in Figure 18. As shown the WQCV of 2.0 acre-feet will require a ponded depth of 1.3 feet. The 100-year detention volume of 8.8 acre-feet will pond to a depth of 5.3 feet (excluding the micro-pond). These include the WQCV released over a 40-hour period. A horizontal grate at elevation 5313 controls the 100-year event.

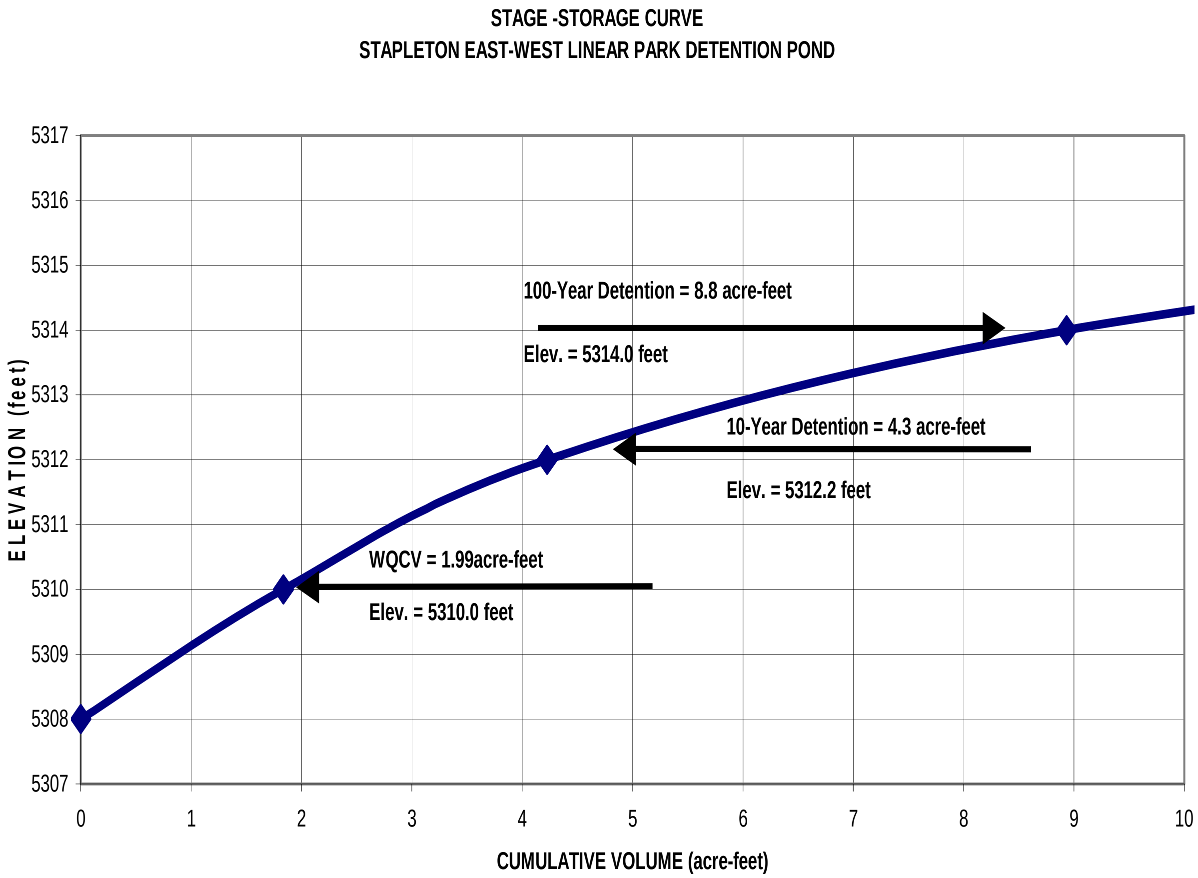


Figure 13—Stage-Storage Curve Stapleton East-West Linear Park Detention Pond

Sheet 1 of 3

Figure 14

Designer:	
Company:	
Date:	**February 9, 2001**
Project:	**UDFCD Example**
Location:	**Stapleton Redevelopment**

1. Basin Storage Volume			
	I_a =	44.40	%
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	i =	0.44	
B) Contributing Watershed Area (Area)	Area =	104.00	acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV =	0.19	watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol =	1.990	acre-feet
2. Outlet Works			
A) Outlet Type (Check One)	X	Orifice Plate	
		Perforated Riser Pipe	
		Other:	
B) Depth at Outlet Above Lowest Perforation (H)	H =	1.30	feet
C) Required Maximum Outlet Area per Row, (A_o)	A_o =	9.57	square inches
D) Perforation Dimensions **(enter one only)**:			
i) Circular Perforation Diameter **OR**	D =	1.5000	inches, **OR**
ii) 2" Height Rectangular Perforation Width	W =		inches
E) Number of Columns (nc, See Table 6a-1 For Maximum)	nc =	5	number
F) Actual Design Outlet Area per Row (A_o)	A_o =	8.84	square inches
G) Number of Rows (nr)	nr =	4	number
H) Total Outlet Area (A_{ot})	A_{ot} =	34.46	square inches
3. Trash Rack			
A) Needed Open Area: A_t = 0.5 * (Figure 7 Value) * A_{ot}	A_t =	1,102	square inches
B) Type of Outlet Opening (Check One)	X	≤ 2" Diameter **Round**	
		2" High **Rectangular**	
		Other:	
C) For 2", or Smaller, **Round Opening** (Ref.: Figure 6a):			
i) Width of Trash Rack and Concrete Opening (W_{conc}) from Table 6a-1	W_{conc} =	60	inches
ii) Height of Trash Rack Screen (H_{TR})	H_{TR} =	40	inches

Figure 14—Design Procedure For Extended Detention Basin Sedimentation Facility

Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 2 of 3

Designer:
Company:
Date: **February 9, 2001**
Project: **UDFCD Example**
Location: **Stapleton Redevelopment**

Figure 14

iii) Type of Screen (Based on Depth H), Describe if "Other"	**X**	S.S. #93 VEE Wire (US Filter)	
		Other:	
iv) Screen Opening Slot Dimension, Describe if "Other"	**X**	0.139" (US Filter)	
		Other:	
v) Spacing of Support Rod (O.C.)	1.00	inches	
Type and Size of Support Rod (Ref.: Table 6a-2)	**TE 0.074 in. x 1.00 in.**		
vi) Type and Size of Holding Frame (Ref.: Table 6a-2)	**1.25 in. x 1.50 in. angle**		
D) For 2" High **Rectangular Opening** (Refer to Figure 6b):			
I) Width of Rectangular Opening (W)	W =		inches
ii) Width of Perforated Plate Opening (W_{conc} = W + 12")	W_{conc} =		inches
iii) Width of Trashrack Opening ($W_{opening}$) from Table 6b-1	$W_{opening}$ =		inches
iv) Height of Trash Rack Screen (H_{TR})	H_{TR} =		inches
v) Type of Screen (based on depth H) (Describe if "Other")		Klemp™ KPP Series Aluminum	
		Other:	
vi) Cross-bar Spacing (Based on Table 6b-1, Klemp™ KPP Grating). Describe if "Other"		inches	
		Other:	
vii) Minimum Bearing Bar Size (Klemp™ Series, Table 6b-2) (Based on depth of WQCV surcharge)			
4. Detention Basin length to width ratio		4.00	(L/W)
5 Pre-sedimentation Forebay Basin - Enter design values			
A) Volume (5 to 10% of the Design Volume in 1D)		0.199	acre-feet
B) Surface Area		0.199	acres
C) Connector Pipe Diameter (Size to drain this volume in 5-minutes under inlet control)		24	inches
D) Paved/Hard Bottom and Sides		**Y**	yes/no

Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 3 of 3

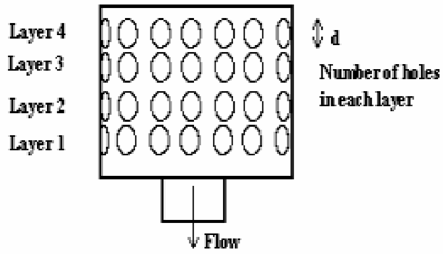
Figure 14

Designer:
Company:
Date: **February 9, 2001**
Project: **UDFCD Example**
Location: **Stapleton Redevelopment**

6. Two-Stage Design			
A) Top Stage (D_{WQ} = 2' Minimum)	D_{WQ} =	2.00	feet
	Storage=	1.493	acre-feet
B) Bottom Stage (D_{BS} = D_{WQ} + 1.5' Minimum, D_{WQ} + 3.0' Maximum, Storage = 5% to 15% of Total WQCV)	D_{BS} =	3.50	feet
	Storage=	0.299	acre-feet
	Surf. Area=	0.085	acres
C) Micro Pool (Minimum Depth = the Larger of 0.5 * Top Stage Depth or 2.5 Feet)	Depth=	2.50	feet
	Storage=	0.214	acre-feet
	Surf. Area=	0.086	acres
D) Total Volume: Vol_{tot} = Storage from 5A + 6A + 6B Must be ≥ Design Volume in 1D	Vol_{tot} =	1.990	acre-feet
7. Basin Side Slopes (Z, horizontal distance per unit vertical) Minimum Z = 4, Flatter Preferred	Z =	4.00	(horizontal/vertical)
8. Dam Embankment Side Slopes (Z, horizontal distance per unit vertical) Minimum Z = 4, Flatter Preferred	Z =	4.00	(horizontal/vertical)
9. Vegetation (Check the method or describe "Other")	X	Native Grass	
		Irrigated Turf Grass	
		Other:	

Notes:

Flow Capacity of a Riser (Inlet Control)



1 Description of Riser

Diameter of holes	d =	1.500	in
Number of holes per layer	n =	5	
Vertical Distance Between Holes	h =	4	in
Orifice Discharge Coefficient	C_o =	0.60	
Total opening area at each layer	A_o =	8.8358	sq in
		0.0614	sq ft

2 Calculation of Collection Capacity

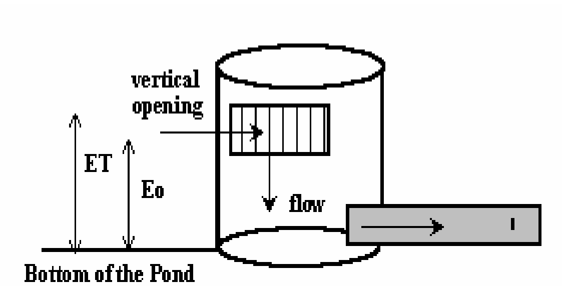
The starting elevation of water surface must be >= central elevation of the first layer.

Starting with Layer 1, enter the central elevation of each layer of holes (up to 10 layers) to calculate flow rates.

Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7	Layer 8	Layer 9	Layer 10	Flow Rate cfs
	Central Elevations of Layers of Holes in feet	5308.76	5309.09	5309.43	5309.76							
		Collection Capacity for Each Layer of Holes in cfs										
start	5308.80	0.06	0.00	0.00	0.00							**0.06**
	5308.90	0.11	0.00	0.00	0.00							**0.11**
	5309.00	0.14	0.00	0.00	0.00							**0.14**
	5309.10	0.17	0.02	0.00	0.00							**0.20**
	5309.20	0.20	0.10	0.00	0.00							**0.29**
	5309.30	0.22	0.13	0.00	0.00							**0.35**
	5309.40	0.24	0.16	0.00	0.00							**0.40**
	5309.50	0.25	0.19	0.08	0.00							**0.52**
	5309.60	0.27	0.21	0.12	0.00							**0.60**
	5309.70	0.29	0.23	0.15	0.00							**0.67**
	5309.80	0.30	0.25	0.18	0.06							**0.79**
	5309.90	0.32	0.27	0.20	0.11							**0.89**
	5310.00	0.33	0.28	0.22	0.14							**0.98**
	5311.00	0.44	0.41	0.37	0.33							**1.55**
	5312.00	0.53	0.50	0.47	0.44							**1.95**
	5313.00	0.61	0.58	0.56	0.53							**2.28**
	5314.00	0.68	0.65	0.63	0.61							**2.57**
	5315.00	0.74	0.72	0.70	0.68							**2.83**
	5316.00	0.79	0.78	0.76	0.74							**3.07**

Figure 15—Flow Capacity of a Riser (Inlet Control)



1 Description of Vertical Orifice

Net Opening Area	A_o =	4.2	sq ft
Orifice Coefficient	C_o =	0.65	
Top Elevation of Orifice Opening Area	E_t =	5312.00	ft
Center Elevation of Orifice Opening	E_o =	5311.00	ft

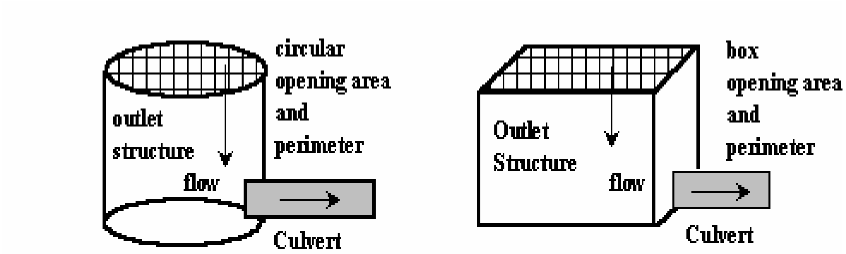
2 Calculation of Collection Capacity

The starting elevation of water surface >= top of the orifice opening.

Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Collection Capacity cfs (output)
start	5312.00	21.91
	5312.10	22.98
	5312.20	24.00
	5312.30	24.98
	5312.40	25.92
	5312.50	26.83
	5312.60	27.71
	5312.70	28.56
	5312.80	29.39
	5312.90	30.20
	5313.00	30.98

Figure 16—Collection Capacity of Vertical Orifice (Inlet Control)



1 Description of Horizontal Orifice

Net Opening Area (after Trash Rack Reduction)	A_o =	50.0	sq ft
Net Perimeter as Weir Length	L_w =	30.0	ft
Orifice Coefficient	C_o =	0.560	
Weir Coefficient	C_w =	3.000	
Center Elevation of Orifice Opening	E_o =	5313.00	ft

2 Calculation of Collection Capacity

The starting elevation of water surface must be >= Eo
Elevations of water surface must be entered in an increasing order.

	Water Surface Elevation ft (input)	Weir Flow cfs (output)	Orifice Flow cfs (output)	Collection Capacity cfs (output)
start	5313.00	0.00	0.00	0.00
	5313.10	2.85	71.06	2.85
	5313.20	8.05	100.49	8.05
	5313.30	14.79	123.07	14.79
	5313.40	22.77	142.11	22.77
	5313.50	31.82	158.89	31.82
	5313.60	41.83	174.05	41.83
	5313.70	52.71	188.00	52.71
	5313.80	64.40	200.98	64.40
	5313.90	76.84	213.17	76.84
	5314.00	90.00	224.70	90.00

Figure 17—Collection Capacity of Horizontal Orifice (Inlet Control)

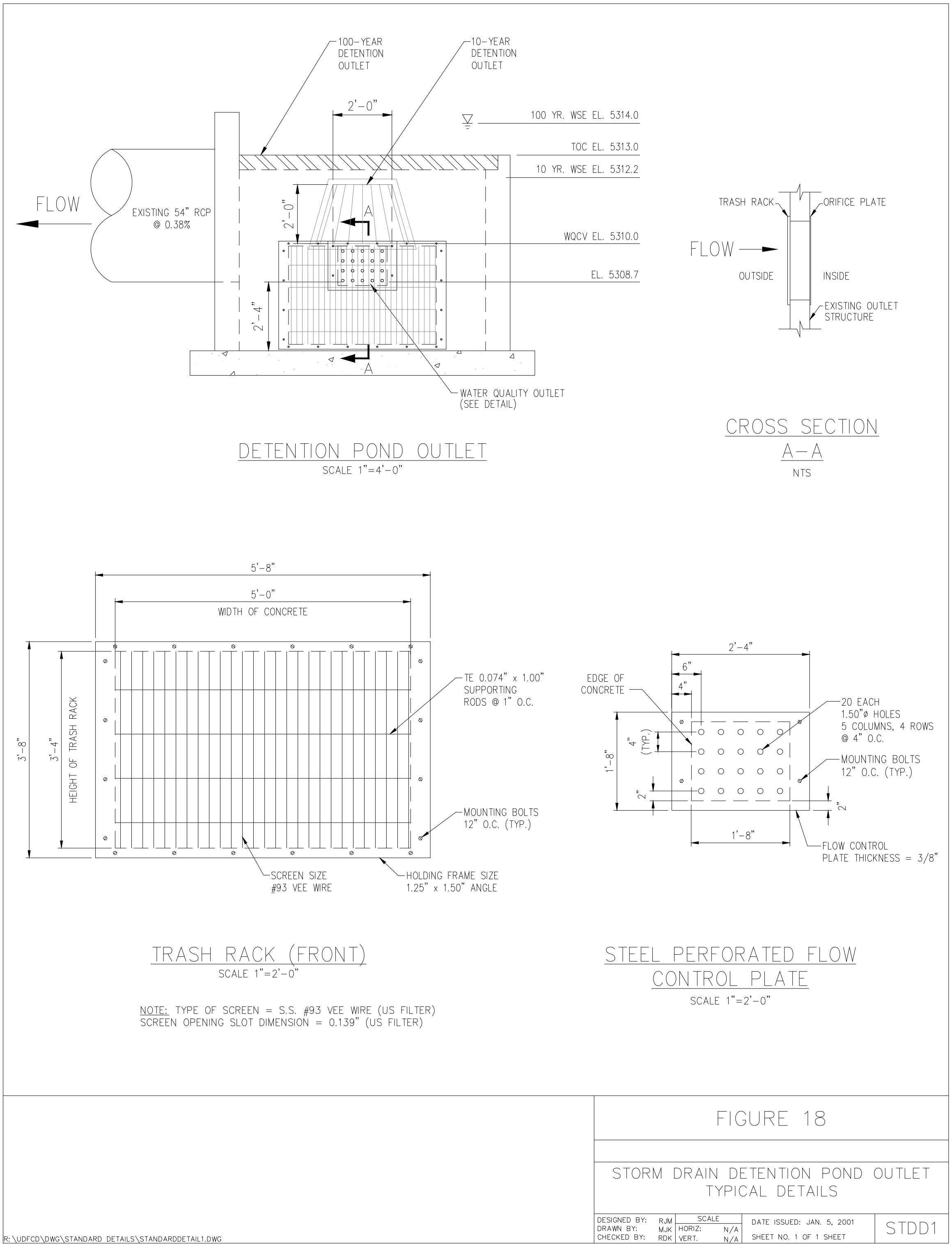


Figure 18—Detention Pond Outlet

2.5 Hydraulic Analysis And Capacity Verification Of The Existing Outfall

The capacity of the existing 54-inch storm sewer is a critical consideration in the design of the East-West Linear Park drainage system. Because the system outfalls to a major drainageway (Westerly Creek) that may create a tailwater control during peak flood flow conditions, a more detailed standard-step backwater analysis was performed. Figure 19 presents the profile of the existing pipeline.

The standard-step backwater is based on Manning's Equation to compute friction losses. Minor (form) losses should also be accounted for using the equations and factors described in the STREETS/INLETS/ STORM SEWERS chapter of this *Manual*. Figure 20 tabulates the computational process for the 100-year storm and a discharge rate of 88.4 cfs. The 100-year Westerly Creek floodplain elevation at the outfall of 5,304 ft is used as the beginning water surface elevation. Figure 21 provides a plot of the computed hydraulic grade line (HGL) and energy grade line (EGL) for the system. As indicated by an HGL above the crown of the pipe, a pressure flow condition exists for the 100-year storm. Because the 100-year HGL at the inlet is below the crown of pipe (outlet controlled), the allowable release rate of 88.4 cfs was used in the design of a multi-stage outlet (versus a restricting pipe capacity).

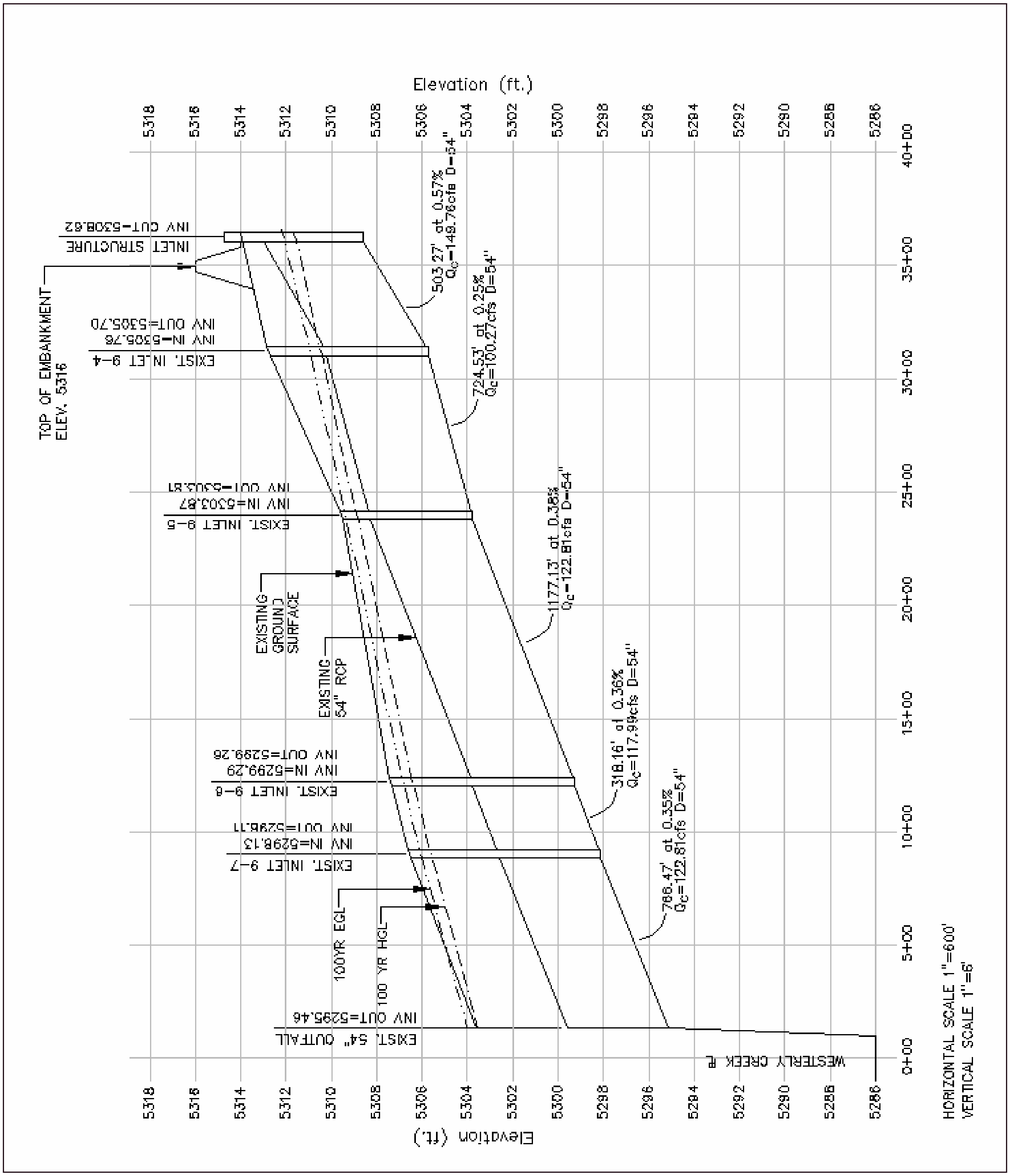


Figure 19—54" Pipe Outfall Profile

HYDRAULIC DESIGN OF STORM SEWER SYSTEMS

STANDARD STEP BACKWATER ANALYSIS FOR FULL PIPE GEOMETRY

PROJECT: Stapleton East-West Linear Park Outfall

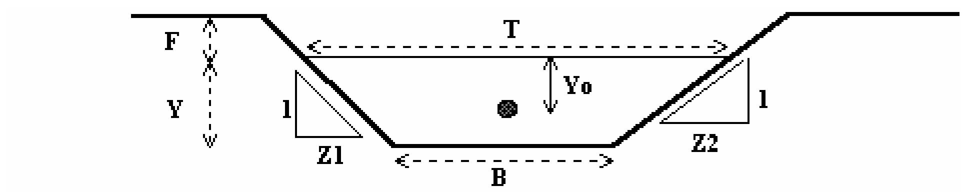
Manning's N-Value = 0.013 Full Flow Factor = 0.9

NOTES:
1 Computed values shown in Italics. All other values are required input
2 Freeboard criteria: HGL at or below rim or grnd.
3 Starting EGL set at Westerly Creek 100-Year floodplain elevation, assuming velocity head in Westerly Creek is negligible at culvert entrance

Design Point	Rim or Grnd. Elev.	Inv.	Sewer Grade	E.G.L.	U/S pipe dia.	Area	Q	Vel.	Vel. Hd.	H.G.L	Friction Slope	Pipe Length	Frict. Loss	Junction Loss		Exit/Form Loss		Total Losses		Freel
									Hv		Sf	L	Hf	Km	Hm	Ke	He	frict.	other	HGL
	(ft)	(ft)	%	(ft)	(in)	(sq.ft)	(cfs)	(fps)	(ft)	(ft)	(ft/ft)	(ft)	(ft)		(ft)		(ft)	(ft)	(ft)	(ft)
Westerly Creek	5305.0	5295.45		5304.00	54	*15.90*	*88.4*	*5.6*	*0.48*	5303.52										1.5
			0.35								0.00201	766.5	*1.54*	0	*0.00*	1	*0.48*	*1.54*	*0.48*	
Inlet #9-7, d/s	5306.6	5298.11		5306.02	54	*15.90*	*88.4*	*5.6*	*0.48*	5305.54										1.0
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-7, u/s	5306.6	5298.13		5306.17	54	*15.90*	*88.4*	*5.6*	*0.48*	5305.69										0.9
			0.36								0.00201	318.2	*0.64*	1	*0.00*		*0.00*	*0.64*	*0.00*	
Inlet #9-6, d/s	5307.4	5299.26		5306.80	54	*15.90*	*88.4*	*5.6*	*0.48*	5306.33										1.1
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-6, u/s	5307.4	5299.29		5306.95	54	*15.90*	*88.4*	*5.6*	*0.48*	5306.47										0.9
			0.38								0.00201	1177.1	*2.37*	1	*0.00*		*0.00*	*2.37*	*0.00*	
Inlet #9-5, d/s	5309.5	5303.81		5309.32	54	*15.90*	*88.4*	*5.6*	*0.48*	5308.84										0.7
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-5, u/s	5309.5	5303.87		5309.46	54	*15.90*	*88.4*	*5.6*	*0.48*	5308.98										0.5
			0.25								0.00201	724.5	*1.46*	1	*0.00*		*0.00*	*1.46*	*0.00*	
Inlet #9-4, d/s	5312.8	5305.70		5310.92	54	*15.90*	*88.4*	*5.6*	*0.48*	5310.44										2.4
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-4, u/s	5312.8	5305.76		5311.06	54	*15.90*	*88.4*	*5.6*	*0.48*	5310.58										2.2
			0.57								0.00201	503.3	*1.01*	1	*0.00*		*0.00*	*1.01*	*0.00*	
Inlet #9-3, d/s	5314.0	5308.62		5312.07	54	*15.90*	*88.4*	*5.6*	*0.48*	5311.59										2.4
			n/a								0.00201	0.1	*0.00*	0.75	*0.12*	0.05	*0.02*	*0.00*	*0.14*	
Inlet #9-3, u/s	5314.0	5308.77		5312.22	54	*15.90*	*88.4*	*5.6*	*0.48*	5311.74										2.2

Figure 20—Hydraulic Design of Storm Sewer Systems

Project =	STAPLETON REDEVELOPMENT
Channel ID =	DETENTION POND EMERGENCY OVERFLOW CHANNEL



Design overflow channel for 100-year peak inflow without attenuation (273 cfs).

Design Information (Input)			
Channel Invert Slope	So =	0.0030	ft/ft
Channel Manning's N	N =	0.038	
Bottom Width	B =	30.0	ft
Left Side Slope	Z1 =	4.0	ft/ft
Right Side Slope	Z2 =	4.0	ft/ft
Freeboard Height	F =	1.0	ft
Design Water Depth	Y =	**2.25**	ft

Normal Flow Condition (Calculated)			
Discharge	**Q =**	**279.6**	cfs
Froude Number	**Fr =**	**0.42**	
Flow Velocity	**V =**	**3.2**	ft
Flow Area	A =	87.8	ft
Top Width	T =	48.0	sq ft
Wetted Perimeter	P =	48.6	ft
Hydraulic Radius	R =	1.8	fps
Hydraulic Depth	D =	1.8	ft
Specific Energy	Es =	2.4	ft
Centroid of Flow Area	Yo =	1.0	ft
Specific Force	Fs =	7.4	klb's

Figure 21—Normal Flow Analysis - Trapezoidal Channel

2.6 Local Storm Sewer Design

The detention facility will adequately provide subregional storage for sub-basins 031 and 032 to protect downstream structures and control discharges to Westerly Creek. It will be essential to provide a conveyance system within the local sub-basins to collect and safely transport stormwater to the detention pond. Similar to most drainage systems, the Stapleton East-West Linear Park Flood Control Project utilizes a combination of roadway, open channel, and formal storm sewers for these purposes.

Figure 22 illustrates local basin 031 with further delineation of tributary areas (031.1A through 031.1E) to allow computation of hydrologic and hydraulic conditions at major intersections and inlet locations. An enlarged view of the storm sewer layout is shown on Figure 23, including an initial set of inlets at the intersection of 24th and 26th Avenues and installation of 24-inch RCP for conveyance to the detention pond.

2.6.1 Determination of Allowable Street Capacity

Inlets are provided to drain intersections without excessive encroachment and at street locations where needed to maintain allowable inundation depths for the initial and major storm events. Figure 24 shows computation of street capacity for the initial storm (2-year) with a normal depth, Y, to the top of curb. The corresponding capacity, Q_{max}, is 7.06 cfs. A similar calculation is performed in Figure 25 for the major storm for the specific roadway cross-section being constructed using Manning's Equation and the allowable depths indicated in this *Manual*. The corresponding capacity, Q_{max}, is 87.5 cfs.

2.6.2 Determination of Inlet Hydrology

The Rational Method is used to determine peak discharges for the local tributary area to each inlet. Figure 26 shows computation of the 2-year discharge for sub-basin 0.31.1B and the corresponding flow rate of 1.06 cfs. A check of the flow conditions in the street is provided on Figure 27 for 1.1 cfs and computation of the V_sD (velocity times depth product) to be 0.61 ft^2/sec.

2.6.3 Inlet Capacity Calculations

Figure 28 demonstrates use of the **UDINLET** spreadsheet for a **Curb Opening Inlet in a Sump** for inlet 26-5A. For the 2-year discharge of 1.1 cfs, a 6-foot curb opening in a sump condition will provide full capture (with a maximum capacity of 6.8 cfs).

2.6.4 Street and Storm Sewer Conveyance Computations

To determine the appropriate combination of inlet, storm sewer, and street conveyance capacity, a detailed hydrologic and hydraulic analysis must be performed for each tributary area under initial (2-year) and major (100-year) conditions. The computational spreadsheets shown on Figures 29 and 30 present these analyses for the local street and storm sewer system.

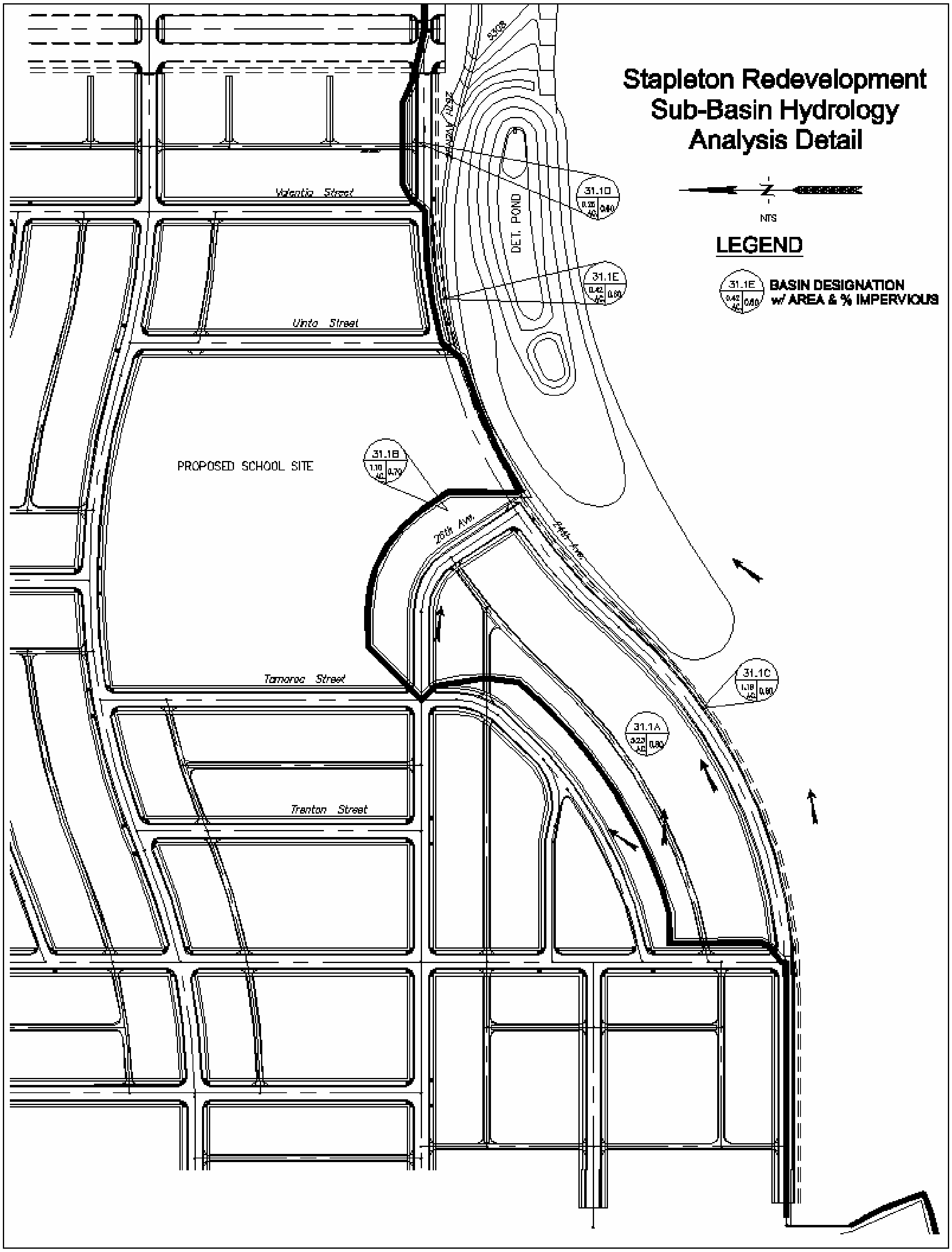


Figure 22—Sub-Basin Hydrology Analysis Detail

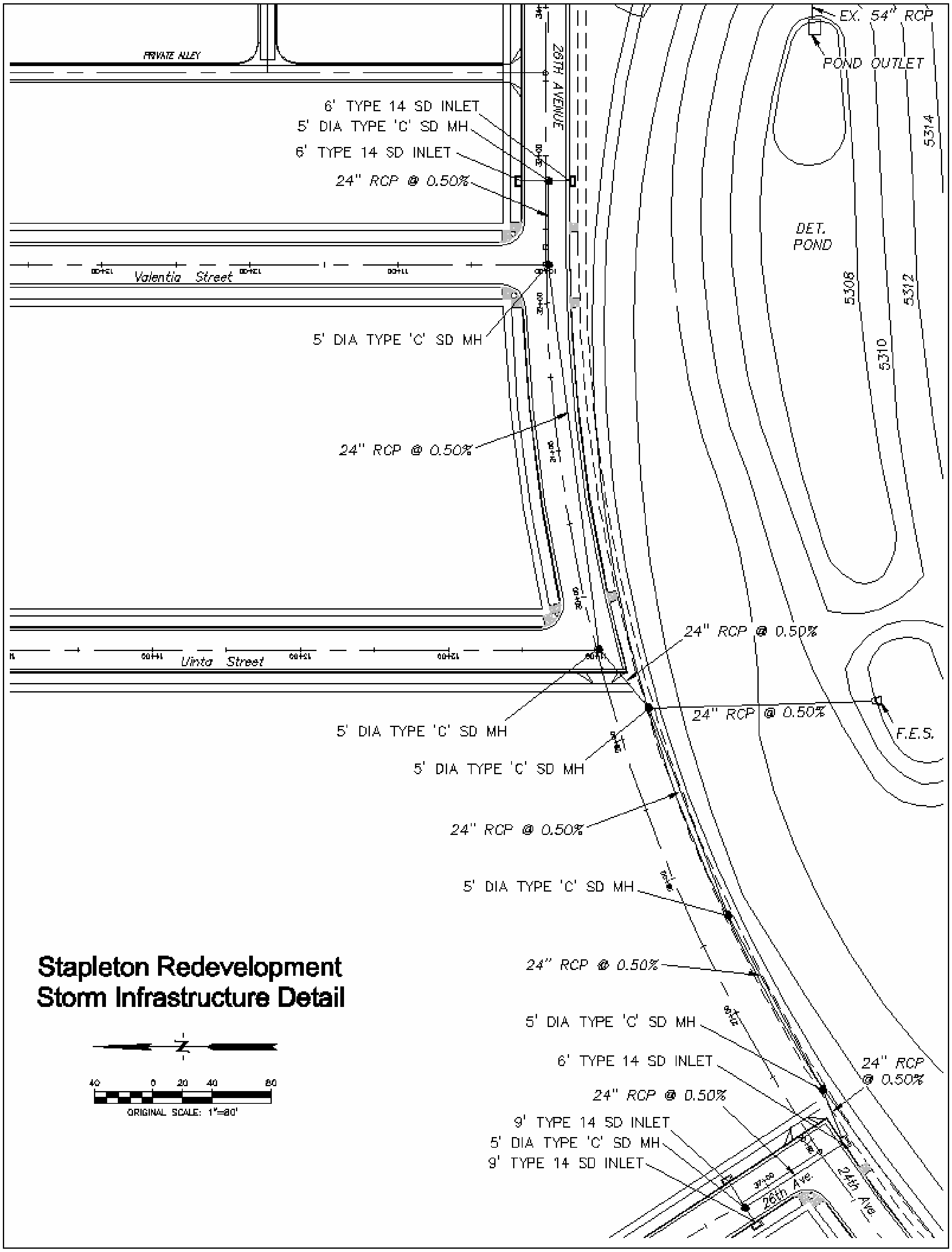
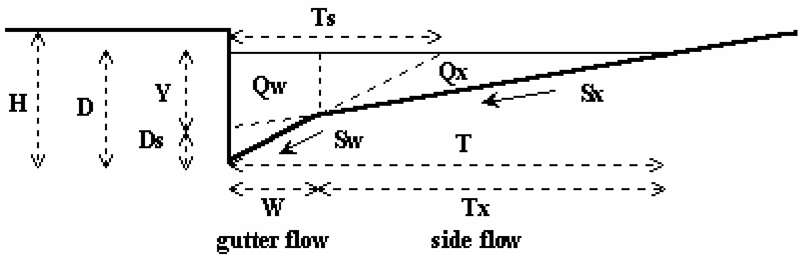


Figure 23—Storm Infrastructure Detail

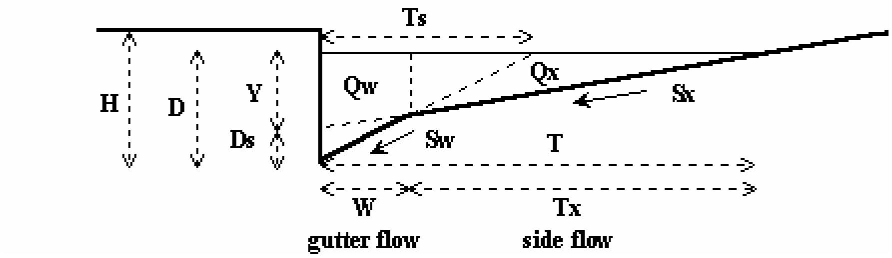
Project = Stapleton Redevelopment
Street ID = 26th Avenue (32' Fl - Fl Local Street)



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0050	ft/ft
Gutter Cross Slope:	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Major Event	T =	16.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Water Depth without Gutter Depression	Y =	0.32	ft
Water Depth with a Gutter Depression	D =	0.49	ft
Spread for Side Flow on the Street	Tx =	14.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	5.84	ft
Flowrate Carried by Width Ts	Qws =	4.3	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.4	cfs
Gutter Flow	Qw =	2.9	cfs
Side Flow	Qx =	4.1	cfs
Maximum Spread Capacity	**Q-Tm =**	**7.1**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	16.67	ft
Spread for Gutter Flow along Gutter Slope	Ts =	6.00	ft
Flowrate Carried by Width Ts	Qws =	4.7	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.6	cfs
Gutter Flow	Qw =	3.1	cfs
Side Flow	Qx =	6.6	cfs
Gutter Full Capacity	**Q-full =**	**9.7**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**7.1**	**cfs**

Figure 24—Gutter Stormwater Conveyance Capacity for Initial Event

Project = Stapleton Redevelopment
Street ID = 26th Avenue (32' Fl - Fl Local Street)



Gutter Geometry

Curb Height	H =	12.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0050	ft/ft
Gutter Cross Slope	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Maximum Water Spread for Major Event	T =	16.00	ft

Gutter Conveyance Capacity Based On Maximum Water Spread

Water Depth without Gutter Depression	Y =	0.32	ft
Water Depth with a Gutter Depression	D =	0.49	ft
Spread for Side Flow on the Street	Tx =	14.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	5.84	ft
Flowrate Carried by Width Ts	Qws =	4.3	cfs
Flowrate Carried by Width (Ts - W)	Qww =	1.4	cfs
Gutter Flow	Qw =	2.9	cfs
Side Flow	Qx =	4.1	cfs
Maximum Spread Capacity	**Q-Tm =**	**7.1**	**cfs**

Gutter Full Conveyance Capacity Based on Curb Height

Spread for Side Flow on the Street	Tx =	41.67	ft
Spread for Gutter Flow along Gutter Slope	Ts =	12.00	ft
Flowrate Carried by Width Ts	Qws =	29.7	cfs
Flowrate Carried by Width (Ts - W)	Qww =	18.3	cfs
Gutter Flow	Qw =	11.4	cfs
Side Flow	Qx =	76.1	cfs
Gutter Full Capacity	**Q-full =**	**87.5**	**cfs**

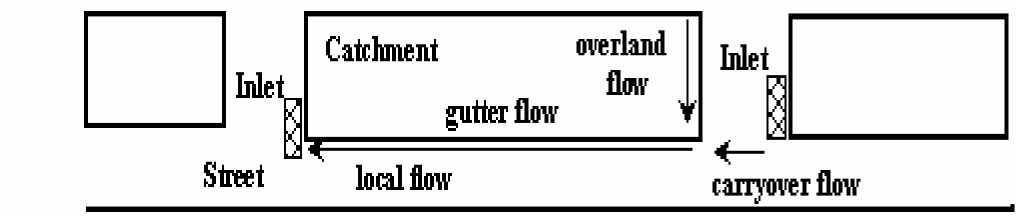
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)

Reduction Factor for Major Event	R-maj =	1.00	
Gutter Design Conveyance Capacity for Major Event	**Q-maj =**	**7.1**	**cfs**

Figure 25—Gutter Stormwater Conveyance Capacity for Major Event

Design Flow = Local Flow + Carryover Flow

Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl-Fl Local Street)
Return Period	2 year (Basin 31.1B)



A.LOCAL FLOW ANALYSIS

Area (A) =	1.10	acres	(input)
Runoff Coeff (C) =	0.45		(input)

Rainfall Information I (inch/hr) = 28.5 * P1 /(10 + Td)^0.786

P1 =	0.95	inches	(input one-hr precipitation)

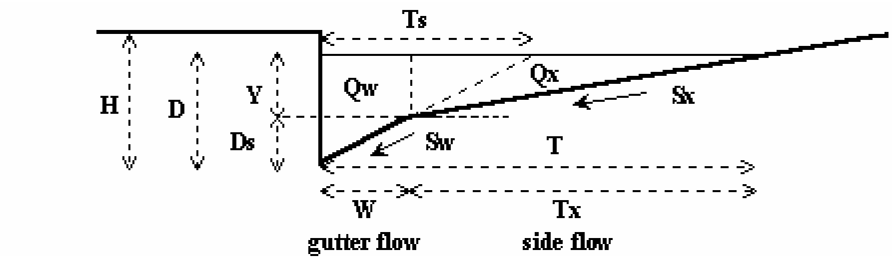
Calculations of Time of Concentration

Reach **ID**	**Slope** **ft/ft** input	**Length** **ft** input	**5-yr Runoff Coeff** input	**Flow Velocity fps** output	**Flow Time minutes** output
Overland Flow	0.0150	50.00	0.50	0.12	6.70
Gutter Flow	0.0050	900.00		1.41	10.61
Sum		950.00			17.31
	Regional Tc =	15.28	minutes		
	Recommended Tc =	15.28	minutes		
	Enter Design Tc =	15.28	minutes		

B.LOCAL PEAK FLOW				
Design Rainfall	I =	2.14	inch/hr	(output)
Local Peak Flow	Qp =	1.06	cfs	(output)
C.CARRYOVER FLOW	Qco =	0.00	cfs	(input)
D.DESIGN PEAK FLOW	Qs =	**1.06**	cfs	(output)

Figure 26—Determination Of Design Peak Flow On The Street

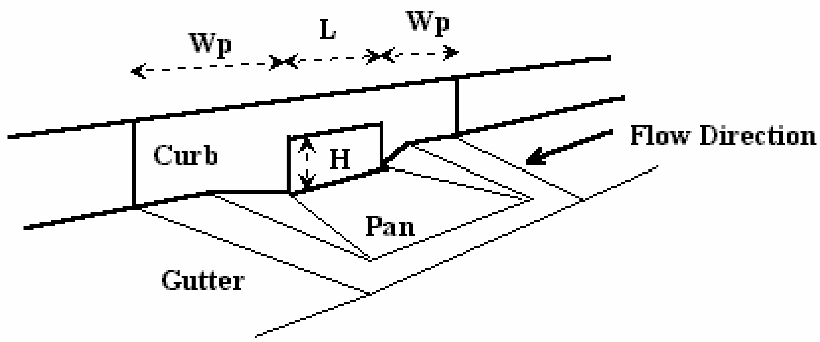
Project =	Stapleton Redevelopment
Street ID =	26th Avenue (32' Fl-Fl Local Street)



Street Geometry (Input)			
Design Discharge in the Gutter	**Qo =**	**1.1**	**cfs**
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Gutter Cross Slope	Sw =	0.0833	ft/ft
Manning's Roughness	N =	0.016	
Gutter Conveyance Capacity			
Water Spread Width	T =	4.32	ft
Water Depth without Gutter Depression	Y =	0.09	ft
Water Depth with a Gutter Depression	D =	0.25	ft
Spread for Side Flow on the Street	Tx =	2.32	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.04	ft
Flowrate Carried by Width Ts	Qws =	1.07	cfs
Flowrate Carried by Width (Ts - W)	Qww =	0.06	cfs
Gutter Flow	Qw =	1.01	cfs
Side Flow	Qx =	0.05	cfs
Total Flow (Check against Qo)	**Qs =**	**1.1**	**cfs**
Gutter Flow to Design Flow Ratio	Eo =	0.95	
Equivalent Slope for the Street	Se =	0.10	
Flow Area	As =	0.35	sq ft
Flow Velocity	Vs =	3.00	fps
VsD product	**VsD =**	**0.76**	**ft^2/s**

Figure 27—Gutter Conveyance Capacity

Project =	Stapleton Redevelopment
Inlet ID =	6' Type 14 (Basin 31.1B)



Design Information (Input)			
Design discharge on the street (from Street Hy)	**Qo =**	**1.1**	**cfs**
Length of a unit inlet	Lu =	6.00	ft
Side Width for Depression Pan	Wp =	2.00	ft
Clogging Factor for a Single Unit	Co =	0.20	
Height of Curb Opening	H =	0.50	ft
Orifice Coefficient	Cd =	0.65	
Weir Coefficient	Cw =	2.30	
Water Depth for the Design Condition	Yd =	0.55	ft
Angle of Throat	Theta =	1.05	rad
Number of Curb Opening Inlets	N =	1	

Curb Opening Inlet Capacity in a Sump			
As a Weir			
Total Length of Curb Opening Inlet	L =	6.00	ft
Capacity as a Weir without Clogging	Qwi =	9.0	cfs
Clogging Coefficient for Multiple Units	Clog-Coeff =	1.00	
Clogging Factor for Multiple Units	Clog =	0.20	
Capacity as a Weir with Clogging	Qwa =	7.9	cfs
As an Orifice			
Capacity as an Orifice without Clogging	Qoi =	9.0	cfs
Capacity as an Orifice with Clogging	Qoa =	7.2	cfs
Capacity for Design with Clogging	**Qa =**	**7.2**	**cfs**
Capture %age for this inlet = Qa/Qs =	**C% =**	**682.80**	**%**

Figure 28—Curb Opening Inlet In A Sump

STORM DRAINAGE SYSTEM COMPUTATION FORM | **Design Storm: 2 yr**

Location: Stapleton Filing No 2

Computed by:

Checked by:

									Overland Time (ti)			Travel Time (tt)				tc Check			Runoff		
Basin ID	Inlet No.	Area	Cumulative Area	**Coefficient "C2"**	Coefficient "C5"	Coefficient "C100"	CA	Cumulative CA	Length (300' max)	Slope	ti	Length	Slope	Velocity (Fig. 3-2)	tt	tc=ti+tt	Total length	tc = (l/180+10)	Final tc	Intensity "I"	Total Peak Discharge "Q"
		acres	acres						ft	%	min.	ft	%	fps	min	min	ft	min	min	in/hr	cfs
31.1A	26-5A	5.23		0.40	0.45	0.60	2.09		60	0.5%	11.4	325	0.5%	1.5	3.6	15.0	385	12.1	12.1	2.37	5.0
31.1B	26-5B	1.10		0.45	0.50	0.70	0.50		70	0.5%	11.4	1017	0.5%	1.5	11.3	22.7	1087	16.0	16.0	2.09	1.0
PIPE			6.33	0.41				2.59											16.0	2.09	5.4
31.1C	26-5C	1.19		0.87	0.89	0.91	1.04		50	1.5%	2.3	1017	0.5%	1.5	11.3	13.6	1067	15.9	13.6	2.25	2.3
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
PIPE			7.52	0.48				3.62											16.0	2.09	7.6
31.1D	26-10B	0.26		0.40	0.45	0.60	0.10		50	0.5%	10.4	220	0.5%	1.5	2.4	12.9	270	11.5	11.5	2.43	0.3
31.1E	26-10A	0.42		0.87	0.89	0.91	0.37		50	0.5%	3.3	810	0.5%	1.5	9.0	12.3	860	14.8	12.3	2.36	0.9
PIPE			0.68	0.69				0.47											12.3	2.36	1.1
PIPE			0.68	0.69				0.47											12.3	2.36	1.1
PIPE			8.20	0.50				4.09											16.0	2.09	8.5
			8.20	**0.50**				**4.09**											**16.0**	**2.09**	**8.5**

Sub-Basin Data		Street				Inlet		System		Pipe											
Basin ID	Slope	Allowable Capacity (half of street)	Bypassed Flow	New Flow	Total Street Flow	Length	Allowable Capacity	Intercepted Flow	Street Flow	Pipe Identification (Upstream - Downstream)	Length	Slope	Size	Allowable Capacity (0.80 Capacity)	Pipe Flow	q/Q(0.8 Full)	v/V(Full) (from Fig 8-1)	Velocity	Pipe Flow Time	Enough Capacity?	Remarks
	%	cfs	cfs	cfs	cfs	ft	cfs	cfs	cfs		(ft)	%	in	cfs	cfs			fps	min		
31.1A	0.50%	5.8	0.0	5.0	5.0	9	8.6	5.0	0.0	26-5A 26-5	20	0.50%	18	5.9	5.0	0.84	1.00	3.4	0.1	no	
31.1B	0.63%	8.1	0.0	1.0	1.0	9	8.6	1.0	0.0	26-5B + 26-5	12	0.83%	18	7.7	1.0	0.14	0.58	2.5	0.1	no	
PIPE								5.4	0.0	26-5 + 26-5C	79	0.50%	24	12.8	5.4	0.42	0.80	3.3	0.4	no	
31.1C	0.50%	5.8	0.0	2.3	2.3	6	2.9	2.3	0.0	26-5C + 26-6	35	0.50%	24	12.8	2.3	0.18	0.63	2.6	0.2	no	
PIPE								7.6	0.0	26-6 + 26-7	117	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.5	no	
PIPE								7.6	0.0	26-7 + 26-8	150	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.7	no	
PIPE								7.6	0.0	26-8 + 26-9	54	0.50%	24	12.8	7.6	0.59	0.89	3.6	0.2	no	
31.1D	0.53%	6.5	0.0	0.3	0.3	6	5.8	0.3	0.0	26-11B + 26-11	18	0.60%	18	6.5	0.3	0.04	0.45	1.7	0.2	no	
31.1E	0.53%	6.5	0.0	0.9	0.9	6	5.8	0.9	0.0	26-11A + 26-11	14	0.79%	18	7.5	0.9	0.12	0.56	2.4	0.1	no	
PIPE								1.1	0.0	26-11 + 26-10	262	0.50%	24	12.8	1.1	0.09	0.52	2.1	2.1	no	
PIPE								1.1	0.0	26-10 + 26-9	57	0.50%	24	12.8	1.1	0.09	0.52	2.1	0.4	no	
PIPE								8.5	0.0	26-9 + OUTLET	180	0.61%	30	25.6	8.5	0.33	0.75	3.9	0.8	no	
	-	-						**8.5**	**0.0**			**0.61%**	**30**	**25.6**	**8.5**					yes	OUTLET TO DETENTION BASIN

Figure 29—Storm Drainage System Computation Form—2 Year

STORM DRAINAGE SYSTEM COMPUTATION FORM **Design Storm: 100 yr**

Location: Stapleton Filing No 2

Computed by:

Checked by:

Sub-Basin Data									Overland Time (ti)			Travel Time (tt)					tc Check		Runoff		
Basin ID	Inlet No.	Area	Cumulative Area	Coefficient "C2"	Coefficient "C5"	**Coefficient "C100"**	CA	Cumulative CA	Length (300' max)	Slope	ti	Length	Slope	Velocity (Fig. 3-2)	tt	tc=ti+tt	Total length	tc = (l/180+10)	Final tc	Intensity "I"	Total Peak Discharge "Q"
		acres	acres						ft	%	min.	ft	%	fps	min	min	ft	min	min	in/hr	cfs
31.1A	26-5A	5.23		0.40	0.45	0.60	3.14		60	0.5%	11.4	325	0.5%	1.5	3.6	15.0	385	12.1	12.1	6.49	20.4
31.1B	26-5B	1.10		0.45	0.50	0.70	0.77		70	0.5%	11.4	1017	0.5%	1.5	11.3	22.7	1087	16.0	16.0	5.72	4.4
PIPE			6.33			0.62		3.91											16.0	5.72	22.3
31.1C	26-5C	1.19		0.87	0.89	0.91	1.08		50	1.5%	2.3	1017	0.5%	1.5	11.3	13.6	1067	15.9	13.6	6.17	6.7
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
PIPE			7.52			0.66		4.99											16.0	5.72	28.5
31.1D	26-10B	0.26		0.40	0.45	0.60	0.16		50	0.5%	10.4	220	0.5%	1.5	2.4	12.9	270	11.5	11.5	6.65	1.0
31.1E	26-10A	0.42		0.87	0.89	0.91	0.38		50	0.5%	3.3	810	0.5%	1.5	9.0	12.3	860	14.8	12.3	6.45	2.5
PIPE			0.68			0.79		0.54											12.3	6.45	3.5
PIPE			0.68			0.79		0.54											12.3	6.45	3.5
PIPE			8.2			0.67		5.53											16.0	5.72	31.6
			8.20			**0.67**		**5.53**											**16.04**	**5.72**	**31.6**

Sub-Basin Data		Street				Inlet					Pipe										
Basin ID	Slope	Allowable Capacity*	Bypassed Flow (Negative flows indicates bypass flow to another DP system (See Remarks))	New Flow	Total Street Flow	Length	Allowable Capacity	Intercepted Flow (if inlet is in series, less intercepted flow is possible -- see remarks)	Bypassed Street Flow	Pipe Identification (Upstream-Downstream)	Length	Slope	Size	Allowable Capacity	Pipe Flow	q/Q(Full)	v/V(Full)	Velocity	Pipe Flow Time	Enough Capacity (Street + Storm Sewer)?	Remarks
	%	cfs	cfs	cfs	cfs	ft	cfs	cfs	cfs		(ft)	%	in	cfs	cfs			fps	(min)		
31.1A	0.50%	23.3	0.0	20.4	20.4	9	12.2	7.4	13.0	26-5A 26-5	20	0.5%	18	7.4	7.4	1.0	1.01	4.3	0.1	yes	
31.1B	0.63%	32.1	13.0	4.4	17.4	9	12.2	9.6	7.8	26-5B + 26-5	12	0.8%	18	9.6	9.6	1.0	1.01	5.5	0.0	yes	
PIPE				22.3				16.0	6.3	26-5 + 26-5C	79	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.3	yes	
31.1C	0.50%	23.3	6.3	6.7	13.0	6	12.2	12.2	0.8	26-5C + 26-6	35	0.5%	24	16.0	12.2	0.8	0.98	3.8	0.2	yes	
PIPE				28.5				16.0	12.5	26-6 + 26-7	117	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.4	yes	
PIPE				28.5				16.0	12.5	26-7 + 26-8	150	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.5	yes	
PIPE				28.5				16.0	12.5	26-8 + 26-9	54	0.5%	24	16.0	16.0	1.0	1.01	5.2	0.2	yes	
31.1D	0.53%	25.8	0.0	1.0	1.0	6	8.2	1.0	0.0	26-11B + 26-1	18	0.6%	18	8.1	1.0	0.1	0.57	0.3	0.9	yes	
31.1E	0.53%	25.8	0.0	2.5	2.5	6	8.2	2.5	0.0	26-11A + 26-1	14	0.8%	18	9.3	2.5	0.3	0.70	1.0	0.2	yes	
PIPE				3.5				3.5	0.0	26-11 + 26-10	262	0.5%	24	16.0	3.5	0.2	0.66	0.7	5.9	yes	
PIPE				3.5				3.5	0.0	26-10 + 26-9	57	0.5%	24	16.0	3.5	0.2	0.66	0.7	1.3	yes	BYPASS FLOW TO LOW POINT @ INLET
PIPE				31.6				19.5	0.0	26-9 + OUTLET	180	0.6%	30	32.0	19.5	0.6	0.90	3.6	0.8	yes	
				31.6				**19.5**	**0.0**			**0.6%**	**30**	**32.0**	**19.5**						OUTLET TO DETENTION BASIN

Figure 30—Storm Drainage System Computation Form—100 Year