FINAL DRAINAGE STUDY
FOR
SAFEWAY DISTRIBUTION CENTER'S
NORTH PARKING AREA
AT 1-70 AND DAHLIA STREET

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This report for the drainage design of the Safeway Distribution Center was prepared by me (or under my supervision) in accordance with the provisions of City and County of Denver Storm Drainage Design and Technical Criteria, and was designed to comply with the provisions thereof. I understand that the City and County of Denver does not, and will not, assume liability for drainage facilities designed by others.

By:_____
Licensed Professional Engineer
State of Colorado No. _____
Affix Seal

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1. GENERAL LOCATION AND DESCRIPTION

A. LOCATION

The Safeway Distribution Center is located in the northwest corner of Section 19, Township 3 South, Range 67 West of the 6th Principal Meridian in the City and County of Denver, State of Colorado. The site is bound to the north by East Stapleton Drive South and Interstate 70, to the south by Smith Road, to the east by Dahlia Street, and to the west by Colorado Boulevard. The site is located in The First Addition to Ramona, East Swansea subdivision.

B. DESCRIPTION OF PROPERTY

The Safeway Distribution Center is an existing active distribution facility with several large warehouse style buildings and outbuildings such as administrative offices. Due to the heavy amount of semi truck traffic within the facility, the majority of the site is paved with little landscaping. This drainage report evaluates the entire 57.1 acre site.

The majority of the existing site currently captures the storm water runoff in inlets and flows undetained to the public storm drain in East Stapleton Drive South, and ultimately flows north under I-70 via a 48" concrete pipe. An exception being a small portion of the site, which is currently detained by an existing detention basin on the south side of the facility.

There are no irrigation facilities on or adjacent to the site.

There has been significant flooding observed on this site. The Master Drainage Report for this area shows significant flooding of the facility.

The area of proposed parking lot reconstruction as addressed in this report does not have easements running through it. The existing storm drain on the far east extent of the proposed storm drain system is currently within a public easement for the conveyance of public flow through the Safeway site.

2. MAJOR DRAINAGE BASINS AND SUB-BASINS

A. MAJOR BASIN DESCRIPTION

The area currently used for parking is deteriorated asphalt and concrete paving with no islands or landscaping. There is a ridge running east to west that divides the runoff. The majority of the storm water runoff from this basin (**Basin P**) currently sheet flows south across the parking areas to inlets located at low points in the concrete paving and enters the storm drain through grated inlets. These

flows are currently conveyed north undetained from the site to storm drains that connect under I-70. The proposed storm drain system has been designed to redirect all flow from this basin into a new detention pond.

All other major basins consist of concrete or asphalt paving with very little pervious area. This includes **Basins A-1**, **A-2**, **B-1**, **C-1**, **C-2**, **D-1**, **E-1** and **E-2**. The majority of storm water runoff from these basins sheet flows across the facility to existing inlets. Runoff from these major basins flows north undetained via storm drains that ultimately connect under I-70. All flow from these basins will remain undetained.

a. Basin P

This basin is approximately 3.04 acres of reconstructed development. This basin was delineated into several sub-basins for the design calculation.

b. Basin A-1

This basin is approximately 17.58 acres of existing development. Flow from this basin is ultimately conveyed to the proposed containment area via an existing 24 inch storm drain. A proposed 22 foot long 42 inch RCP has been designed to redirect this flow before entering the containment area. A time of concentration of 20 minutes was used as an estimate for the design calculations.

c. Basin A-2

This basin is approximately 0.65 acres of reconstructed development. Flow from this basin is conveyed directly to the proposed 24 inch storm drain outlet within the emergency containment area. A time of concentration of 6.68 minutes was calculated using the equations in the UDFCD Criteria Manual.

d. Basin B-1

This basin includes **Sub-Basin B-1a**. The basin is approximately 4.10 acres of existing development and 0.23 acres of reconstructed development delineated as **Sub-Basin B-1a**. Flow from this basin is ultimately conveyed to the proposed storm drain system (**Line B)** via an existing 21 inch storm drain. A time of concentration of 10 minutes was used as an estimate for the design calculations

e. Basin C-1

This basin is approximately 3.91 acres of existing development. All flow from this basin is currently conveyed into an existing detention pond. This flow is eventually released and conveyed to the proposed storm drain system (**Line B)**. This flow was excluded from the design calculations as it was assumed that this flow would have no immediate impact on the overall design.

f. Basin C-2

This basin includes **Sub-Basin C-2a**. The basin is approximately 13.46 acres of existing development and 0.02 acres of reconstructed development delineated as **Sub-Basin C-2a**. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 24 inch storm drain. A time of concentration of 10 minutes was used as an estimate for the design calculations.

g. Basin D-1

This basin includes **Sub-Basin D-1a**. The basin is approximately 2.35 acres of existing development and 0.23 acres of reconstructed development delineated as **Sub-Basin D-1a**. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 18 inch storm drain. A time of concentration of 10 minutes was used as an estimate for the design calculations.

h. Basin E-1

This basin is approximately 10.37 acres of existing development. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 48 inch storm drain. A time of concentration of 10 minutes was used as an estimate for the design calculations.

i. Basin E-2

This basin is approximately 1.61 acres of existing development. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 12 inch storm drain. A time of concentration of 5.64 minutes was used as an estimate for the design calculations.

B. SUB-BASIN DESCRIPTION

The following Sub-Basins were delineated from the Major Basins and only include the areas of reconstructed paving. These Sub-Basins were delineated for calculation purposes. Only Sub-Basins P-1 through P-8 (3.04 acres) will ultimately be detained within the proposed detention pond with the sand filter basin. All flow from the remaining Sub-Basins (1.12 acres) will leave the site undetained. The detention facility will be designed to over-detain, under release to compensate for the undetained release developed areas that cannot be captured, as they are captured by existing inlets. The low flows from the undetained areas will be piped to the contamination pond where there will be some ponding and an opportunity for sediment to settle.

a. Sub-Basin B-1b

This basin is approximately 0.23 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed storm drain

system **Line B** via an existing 21 inch storm drain. This basin was delineated to determine the runoff values for the reconstructed paving.

b. Sub-Basin C-2a

This basin is approximately 0.02 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 24 inch storm drain. This basin was delineated to determine the runoff values for the reconstructed paving.

c. Sub-Basin D-1a

This basin is approximately 0.23 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed storm drain system **Line B** via an existing 18 inch storm drain. This basin was delineated to determine the runoff values for the reconstructed paving.

d. Sub-Basin P-1

This basin is approximately 0.36 acres of reconstructed development. Flow from this basin is conveyed to the proposed outlet structure. A time of concentration of 11.19 minutes was calculated using the equations in the UDFCD Criteria Manual.

e. Sub-Basin P-2

This basin is approximately 0.48 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed sand filter basin via the proposed storm drain **Line A**. A time of concentration of 5.92 minutes was calculated using the equations in the UDFCD Criteria Manual.

f. Sub-Basin P-3

This basin is approximately 0.57 acres of reconstructed development. Flow from this basin is conveyed to the proposed storm drain main **Line A** via a proposed 18 inch RCP. A time of concentration of 5.19 minutes was calculated using the equations in the UDFCD Criteria Manual.

g. Sub-Basin P-4

This basin is approximately 0.12 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed sand filter basin via the proposed storm drain **Line A**. A time of concentration of 5.00 minutes was calculated using the equations in the UDFCD Criteria Manual.

h. Sub-Basin P-5

This basin is approximately 0.58 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed sand filter basin via the proposed storm drain **Line A**. A time of concentration of

7.25 minutes was calculated using the equations in the UDFCD Criteria Manual.

i. Sub-Basin P-6

This basin is approximately 0.40 acres of reconstructed development. Flow from this basin is conveyed to the proposed storm drain **Line A** via a proposed 12 inch RCP. A time of concentration of 5.02 minutes was calculated using the equations in the UDFCD Criteria Manual.

j. Sub-Basin P-7

This basin is approximately 0.36 acres of reconstructed development. Flow from this basin is conveyed to the proposed storm drain **Line A** via a proposed 15 inch RCP. A time of concentration of 5.00 minutes was calculated using the equations in the UDFCD Criteria Manual.

k. Sub-Basin P-8

This basin is approximately 0.17 acres of reconstructed development. Flow from this basin is ultimately conveyed to the proposed sand filter basin via the proposed storm drain **Line A**. A time of concentration of 5.47 minutes was calculated using the equations in the UDFCD Criteria Manual.

See Appendix A for all calculated time of concentration values.

3. DRAINAGE DESIGN CRITERIA

A. REGULATIONS

This drainage report and the drainage design for the site are in accordance with the City and County of Denver Storm Drainage Design & Technical Criteria. Since the existing site does not detain or treat storm water runoff for a majority of the site, every effort has been made to capture a majority of the reconstructed site in a detention pond with a sand filter basin for water quality treatment. The remainder of remaining site flow will be either redirected to a proposed containment area, or bypassed undetained through the proposed release structures. The detention facility will be designed to over-detain, under release to compensate for the undetained release of developed areas that cannot be captured.

B. DEVELOPMENT CRITERIA REFERENCES AND CONSTRAINTS

The drainage design for this project does not alter the locations of ultimate discharge. The only effect to the existing site and surrounding drainage improvements is a decrease in the peak release rate of the site, enhanced water quality through a detention pond with a sand filter basin, and an additional

emergency containment area. Therefore, this project will have no negative impact on the surrounding drainage improvements or the final discharge point.

The major constraints to the drainage design are the flat topography of the existing site, utility conflicts, limited detention area, and the shallow storm drain connections for the detention pond outlet and release structures. These constraints required storm drains with minimal slopes to function with the existing system.

C. HYDROLOGIC CRITERIA

The Rational Method was used to determine peak runoff flows for the 10-yr and 100-yr storms based on Denver's Rainfall Intensity Duration Values, Table 5.3. The detention pond was designed using the Empirical Formulas and Sand Filter Basin spreadsheet provided by Urban Drainage Criteria Manual. The allowable release rates for the detention pond were determined based on the hydrologic soils group for this site and Table SO-1. The detention pond will have a two stage release, for the 10-yr and 100-yr storms. This flow will eventually be released into the proposed emergency containment area. Since the 24 inch outlet pipe within the containment area is too small to convey the remaining site flows, release structures have been designed to convey all excess flows to the existing outlet pipes. All flow entering the emergency containment area that has been restricted by the weirs will be released through the proposed 24 inch outlet pipe to the existing storm drain system in East Stapleton.

D. HYDRAULIC CRITERIA

The storm drains, release structures, and inlets are designed to accommodate the 100-yr storm event. Haestad Method StormCAD was used to determine the appropriate pipe sizes with n values of 0.13, and the Rational Method was used for storm drain routing. StormCAD was also used to verify the routing and runoff computations, the results are presented in the appendix of this report. The headloss coefficients are integral to the design of the system, and the hydraulic grade line is calculated and depicted on the storm drain profiles. The hydraulic grade line depicted on storm drain **Line B** represents the case where all release structures are clogged. The release structures are designed to bypass any potential spills during a 5-year event, and also to accommodate the additional flows that the 24 inch outlet pipe could not handle. All weirs elevations within the release structures were set to a height of the 5-yr hydraulic grade line. Any additional flow will be released over these weirs and through the existing outlet pipes. Details of these structures are shown in the appendices of this report.

E. WATER QUALITY REQUIREMENTS UNDER DENVER'S MUNICIPAL CDPS STORMWATER PERMIT

The Water Quality Capture Volume for the reconstructed site was calculated using the Urban Drainage spreadsheet for Sand Filter Basins. The sand filter basin is a permanent feature of the detention pond. The bottom of the pond will be made up of at least 18 inches of filter sand with an underdrain system that will capture the treated water and release it through the proposed outlet structure. The outlet structure will include a water quality orifice plate with a 12-hour drain time per UDFCD requirements, and a two stage release for the 10-yr and 100-yr storm events. The proposed 18" outlet pipe from the detention pond will include a manual gate to be used in case containment of a spill is required in the downstream containment pond. The emergency containment area will consist of a 2 foot trickle pan and a concrete containment area with sediment forebay. Both the proposed 24" outlet pipe and proposed Manhole # 4 will include a manual gate to be used in case containment is required. Both the detention pond and the emergency containment area will be landscaped per the Landscape Plan. The sand filter bottom will be landscaped with allowable plantings. In addition, both the detention pond and emergency containment area will be lined per recommendations within the Geotechnical Report.

The maintenance requirements for the sand filter basin are included in Appendix D of this report. The operation and maintenance manual for the manual gate to be provided by the manufacture.

F. WAIVERS FROM CRITERIA

No waivers have been requested.

4. DRAINAGE FACILITY DESIGN

A. GENERAL CONCEPT

The reconstruction of the existing parking area will include the construction of two new storm drain systems **Lines A and B**, the construction of a detention pond with a sand filter basin for water quality treatment, and an emergency containment area. Storm drain system **Line A** and the sand filter basin will serve as water quality for the majority of the reconstructed site. In addition, storm drain system **Line B** and the emergency containment area with forebay will provide additional water quality and protection against contaminant spills.

The typical drainage pattern of the reconstructed area is an initial sheet flow to a concentrated gutter flow that is captured by a curb opening inlet. These inlets convey the flow to storm drain **Line A** which empties into the detention pond. The typical drainage pattern of the existing areas is an initial sheet flow, to a concentrated pipe flow. This flow is conveyed to storm drain system **Line B** and emptied into the emergency containment area or released through the release structures.

The detention pond with sand filter basin is sized for a water quality capture volume and detention of the 10-yr and 100-yr storm for the reconstructed site (4.16 acres). Any excess flow beyond the 100-yr event will be conveyed over the emergency spillway to the emergency containment area. Since the total volume of water conveyed to the emergency containment area greatly exceeds the areas actual capacity, several release structures have been designed throughout storm drain **Line B** to release the current peak flows. These release structures will outlet into the existing storm drains that conveyed the original undetained flows from the site. Each release structure will contain a rectangular weir set to the height of the 5-yr hydraulic grade line. The proposed 24 inch outlet pipe for the emergency containment area will release into the existing 24 inch storm drain that conveyed the original undetained flows from the site. In the case where all outlet and diversion structures become clogged, all flows will be conveyed over the 250 foot emergency spillway located in the containment pond.

Additional offsite public flows were not included in the design of the storm drain system. Offsite flow which is currently routed through the existing public storm drain onsite is anticipated to be rerouted through a public storm drain system that has not yet been constructed. The proposed storm drain system will not be connected to the existing public system that conveys this offsite flow until this flow is bypassed. Until that time, the flow from that existing system will remain unaltered. A temporary barrier will be constructed within Diversion Structure D so that all public flow will be unaltered until the future public storm drain has been constructed to bypass this on-site system. Once the existing on-site public system has been made private the temporary barrier will be taken down and the proposed diversion weir will be installed.

The Proposed Drainage Plan included in the back of this report shows the site divided into basins and the associated design points. The charts, figures and tables used to calculate the runoff, inlet and storm drain capacities, water quality and detention volumes were obtained from both the City and County of Denver Storm Drainage Design & Technical Criteria and the Urban Drainage and Flood Control District manuals and spreadsheets. The Rational Method calculations are based on this information. All charts, figures, tables, and spreadsheets are included in the appendix of this report for reference.

B. SPECIFIC DETAILS

The existing onsite storm drain system conveys the runoff to the storm drain in East Stapleton Drive South and ultimately flows to the north underneath I-70. There are no detention facilities currently serving the parking area to be reconstructed. The proposed site will include the construction of two new storm drain systems. One designed to collect the flow from the existing on-site systems **(Line B),** and the other to collect the proposed flow **(Line A)**. The inlets and storm drains for each proposed system are designed to convey the site's 100-yr flows.

The peak runoff to the detention pond during the 100-yr event is approximately 15.64 cfs. The required water quality capture volume for the reconstructed site is 0.08 acre-ft. The 10-yr and 100-yr detention volumes are 0.34 acre-ft and 0.54 acre-ft respectively, including all of the water quality volume for the 10-yr and half the water quality volume for the 100-yr. These values were calculated using the entire 4.16 acres of reconstructed development.

The proposed detention pond is designed to restrict discharge of the required water quality capture volume using a sand filter basin and an orifice plate with a 1-3/8 inch diameter opening. This orifice plate has been designed to release the entire WQCV in 12 hours. The detention pond is also designed with a two stage outlet structure that will release the 10-yr storm event at 0.91 cfs, and the 100-yr storm event at 3.04 cfs. The release rates calculated account for 3.04 acres of reconstructed development that are directed to the detention pond. The remaining 1.12 acres will be directed to the emergency containment area. The bottom of the detention pond will consist of a sand filter basin with a 4-inch underdrain system that will convey the treated water quality volume to the proposed outlet structure. In addition, the pond has an emergency overflow weir designed to accommodate the entire 100-yr peak flow in the event the outlet structure becomes completely clogged. The pond also incorporates a freeboard of one foot above the emergency overflow water surface elevation. In the case where emergency containment is required in either pond, a manual shut-off gate will be installed with the 18 inch outlet pipe.

Storm drain **Line B** and emergency containment area were designed to allow all storm flow to exit the site undetained. In addition, the proposed emergency containment area in conjunction with the release structures has been designed to collect all potential contaminant spills from exiting the site undetained. All containment weir elevations within the release structures were set to a height of the 5-yr hydraulic grade line. The maximum containment ponding volume at the minimum diversion weir elevation is approximately 0.78 acre-ft. In our professional opinion temporary ponding created by the diversion weirs will allow for the 1.12 acres of reconstructed development that was not directed to the sand filter basin an opportunity for sediment to settle. The emergency containment area also includes a trickle pan and concrete sediment forebay. Manual shut-off gates have been installed within the storm drain system to allow for emergency containment. In the case where all outlet and diversion structures become clogged, all flows will be conveyed over the 250 foot emergency spillway into East Stapleton Drive South. The emergency spillway will be reinforced with a permanent turf reinforcement matrix.

The surrounding developments are assumed to be in a fully developed state. It was assumed that there will be no upstream contributions that will affect the proposed storm drain system or detention pond. The reconstruction of the parking area will decrease the existing peak runoff with the use of the detention pond and

additional landscape areas. It will treat the runoff with the use of a sand filter basin. The emergency containment area will provide added protection against contaminant spills. Therefore, this proposed reconstruction will have a positive effect on the downstream system in reduced flows, and result in an enhanced condition of storm water runoff.

The existing site's flat topography, detention area limits, and shallow storm drains were the major constraints in the design of this system. In a situation where the outlets become plugged, it was determined that minor flooding would occur within small portions of the truck travel areas. This flooding would be no greater that 6 inches in depth, and will not cause damage to any buildings.

The entire onsite storm drain system is designed as a private system and will be maintained as such, including the detention pond and emergency containment area. There is a proposed maintenance schedule for the detention basin water quality treatment facility in the appendix of this report. The proposed system is entirely on private land and therefore an easement will be dedicated for the City to access the detention pond and emergency containment area. All design measures for public safety have been taken into account as required by the City and County of Denver Storm Drainage Criteria Manual.

The design of the proposed storm drain system is in strict compliance with the City and County of Denver Storm Drainage Design & Technical Criteria manual as well as the Urban Drainage and Flood Control District's manuals.

To enhance the water quality for this project, a sand filter basin with a permanent water quality orifice plate within the detention pond will serve to treat a majority of the reconstructed site. Manual gates have also been implemented within the 18" outlet pipe, 24" outlet pipe, and Manhole #4 to allow for bypassing the storm drain systems incase containment is required. The Urban Drainage and Flood Control District has done extensive research on the effectiveness of sand filter basins, and it has been shown that this type of treatment greatly reduces the pollutants that are normally washed down stream.

5. CONCLUSIONS

A. COMPLIANCE WITH STANDARDS

The design of the proposed storm drain system, including the detention pond, is in accordance with requirements as provided in the City and County of Denver Storm Drainage Design & Technical Criteria manual, as well as the Urban Drainage and Flood Control District's Manual, Volumes 1-3.

B. DRAINAGE CONCEPT

There are no foreseeable damages that could arise with the implementation of the proposed drainage design assuming that the existing system is of sound construction. The entire system is designed for a 100-yr event, with no flooding of proposed parking areas or pedestrian pathways. In circumstances where all release and outlet structures become clogged, minor flooding of approximately 6 inches in depth may occur within a small portion of the truck travel areas assuming that the perimeter of the property is not flooding. The detention pond has been designed to capture water quality and detain the 10-year and 100-year flows for the reconstructed areas only. In addition, an emergency containment area in conjunction with several release structures has been designed to collect and contain any contaminant spills. The proposed system will have no foreseeable negative impacts on drainage characteristics of the 57.1 acre site or ultimate discharge points.

A Master Drainage Study for 5-yr flows was previously done for this site and surrounding areas. See Master Study Basin 0060-01 included in the appendix of this report. Additional offsite public flows or the master study's flooding scenarios were not included in the design of the proposed storm drain systems.

C. WATER QUALITY

The water quality capture volume required for this site was determined by the Urban Drainage Criteria Manual spreadsheet for a Sand Filter Basin. The WQCV of 0.08 acre-ft will be treated in the proposed sand filter basin. The bottom of the pond will be made up of at least 18 inches of filter sand, through which the storm water will infiltrate. A 4 inch perforated underdain system beneath the sand layer will capture the treated water and release it through the proposed outlet structure. The outlet structure includes a water quality orifice plate to allow for a 12-hour drain time.

6. REFERENCES

Urban Drainage and Flood Control District. 2001 (Revised April 2008).
Drainage Criteria Manual, Volumes 1-3

City and County of Denver, Department of Public Works, Wastewater Management Division, Engineering Division. Revised January 2006
Storm Drainage Design & Technical Criteria

APPENDICES

A. HYDROLOGIC COMPUTATIONS

Imperviousness Calculations

MAJOR BASINS RELEASING UNDETAINED:

DESIGN POINT	AREA DESIGN	A(LAND.) acres	A(PAVED) acres	A(ROOFS) acres	A(TOTAL) acres	COMPOSITE C5	C10	C100	% IMPERVIOUS
12	A-1	0.09	5.08	12.41	17.58	0.79	0.81	0.87	92
11	A-2	0.65	0.00	0.00	0.65	0.15	0.25	0.50	0
14	B-1	0.02	1.36	2.73	4.10	0.80	0.82	0.87	93
15	C-2	0.57	8.48	4.42	13.46	0.82	0.84	0.90	93
16	D-1	0.05	1.27	1.04	2.35	0.82	0.84	0.89	94
18	E-1	0.00	6.35	4.02	10.37	0.84	0.86	0.91	96
17	E-2	0.01	1.57	0.04	1.61	0.89	0.91	0.96	99
				TOTAL AREA:	50.13		WEIGHTED:	0.88	92.36

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

DESIGN POINT	AREA DESIGN	A(LAND.) acres	A(PAVED) acres	A(ROOFS) acres	A(TOTAL) acres	COMPOSITE C5	C10	C100	% IMPERVIOUS
1	P-1	0.36	0.00	0.00	0.36	0.15	0.25	0.50	0
2	P-2	0.05	0.44	0.00	0.48	0.83	0.86	0.92	91
6	P-3	0.05	0.52	0.00	0.57	0.83	0.86	0.92	91
8	P-4	0.05	0.07	0.00	0.12	0.60	0.65	0.78	60
9	P-5	0.08	0.39	0.11	0.58	0.77	0.80	0.87	84
4	P-6	0.04	0.36	0.00	0.40	0.82	0.85	0.91	90
7	P-7	0.01	0.36	0.00	0.36	0.88	0.90	0.95	98
10	P-8	0.03	0.14	0.00	0.17	0.77	0.80	0.88	82
				TOTAL AREA:	3.04		WEIGHTED:	0.86	77.92

TOTAL SITE AREA (acres)=	**53.16**
TOTAL SITE % IMPERVIOUS=	**92**
TOTAL SITE C100=	**0.88**
TOTAL SITE C10=	**0.81**

USDCM Vol 1 Table RO-3 :
Recommended Percentage Imperviousness Values

LAND USE	Percentage Imperviousness
LANDSCAPE	0
ROOFS	90
PAVED	100

USDCM Vol 1 Table RO-5
Runoff Coefficients, C

Percentage Imperviousness	SOIL TYPE: 5-yr	10-yr	D 100-yr
0%	0.15	0.25	0.50
90%	0.75	0.77	0.83
100%	0.90	0.92	0.96

Imperviousness Calculations

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

DESIGN POINT	AREA DESIGN	A(LAND.) acres	A(PAVED) acres	A(ROOFS) acres	A(TOTAL) acres	COMPOSITE C5	C10	C100	% IMPERVIOUS
1	P-1	0.36	0.00	0.00	0.36	0.15	0.25	0.50	0
2	P-2	0.05	0.44	0.00	0.48	0.83	0.86	0.92	91
6	P-3	0.05	0.52	0.00	0.57	0.83	0.86	0.92	91
8	P-4	0.05	0.07	0.00	0.12	0.60	0.65	0.78	60
9	P-5	0.08	0.39	0.11	0.58	0.77	0.80	0.87	84
4	P-6	0.04	0.36	0.00	0.40	0.82	0.85	0.91	90
7	P-7	0.01	0.36	0.00	0.36	0.88	0.90	0.95	98
10	P-8	0.03	0.14	0.00	0.17	0.77	0.80	0.88	82
				TOTAL AREA:	3.04		WEIGHTED:	0.86	77.92

PROPOSED BASINS CONTRIBUTING TO EXISTING STORM DRAINS UNDETAINED:

DESIGN POINT	AREA DESIGN	A(LAND.) acres	A(PAVED) acres	A(ROOFS) acres	A(TOTAL) acres	COMPOSITE C5	C10	C100	% IMPERVIOUS
11	A-2	0.65	0.00	0.00	0.65	0.15	0.25	0.50	0
14	B-1a	0.00	0.23	0.00	0.23	0.90	0.92	0.96	99
15	C-2a	0.00	0.02	0.00	0.02	0.85	0.87	0.93	93
16	D-1a	0.05	0.18	0.00	0.23	0.75	0.78	0.87	80
				TOTAL AREA:	1.12		WEIGHTED:	0.68	38

TOTAL PROPOSED AREA (acres)=	**4.16**
TOTAL WEIGHETD % IMPERVIOUS=	**67**
5-YR WEIGHTED=	**0.65**
10-YR WEIGHTED=	**0.70**
100-YR WEIGHTED=	**0.81**

USDCM Vol 1 Table RO-3 :

Recommended Percentage Imperviousness Values

LAND USE	Percentage Imperviousness
LANDSCAPE	0
ROOFS	90
PAVED	100

USDCM Vol 1 Table RO-5

Runoff Coefficients, C

Percentage Imperviousness	SOIL TYPE: D		
	5-yr	10-yr	100-yr
0%	0.15	0.25	0.50
90%	0.75	0.77	0.83
100%	0.90	0.92	0.96

Time of Concentration Calculations

MAJOR BASINS RELEASING UNDETAINED:

BASIN DATA				INITIAL/OVERLAND TIME (t_i)			CONCENTRATED TRAVEL TIME (t_t)				MINIMUM T_c	URBAN CHECK		**FIRST DESIGN POINT**
DESIGN POINT	BASIN	C5	AREA (acres)	LENGTH (ft)	SLOPE (%)	Ti (min)	LENGTH (ft)	SLOPE (%)	VEL. (fps)	Tt (min)	Tc (min)	Length (ft)	Urban Tc	**BASIN Tc**
12	A-1	0.79	17.58	300	1.00	9.7	2015	0.3	3.26	10.30	19.98	2315	22.9	**20.0**
11	A-2	0.15	0.65	34	25.00	3.5	212	0.3	1.10	3.23	6.68	246	11.4	**6.7**
14	B-1	0.80	4.10	186	1.00	7.5	346	1.0	4.55	1.27	8.72	532	13.0	**8.7**
15	C-2	0.82	13.46	145	1.00	6.1	1333	0.5	4.21	5.28	11.38	1478	18.2	**11.4**
16	D-1	0.82	2.35	215	3.20	5.1	485	0.3	3.26	2.48	7.54	700	13.9	**7.5**
18	E-1	0.84	10.37	133	1.00	5.4	1079	0.5	8.10	2.22	7.59	1212	16.7	**7.6**
17	E-2	0.89	1.61	300	1.50	5.6	0	0.0	0.00	0.00	5.64	300	11.7	**5.6**

Velocity from figure RO-1 and equation RO-4
from the USDCM Volume 1

$$V = C_V * S_W^{.5}$$

Table RO-2 Conveyance Cefficient, Cv

TYPE OF LAND SURFACE	CONVEYANCE COEFFICIENT, Cv
Heavy Meadow	3
Tillage/Field	5
Short Pasture and Lawns	7
Nearly Bare Ground	10
Grassed Waterway	15
Paved areas/shallow paved swales	20

$T_c = T_i + T_t$

Initial Flow Time (eqtn RO-3): $t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}}$

Overland Travel Time: $t_t = \frac{L}{60 * V}$

URBAN Tc (eqtn RO-5): $t_c = \frac{L}{180} + 10$

Minimum Tc= 5 MIN for an Urbanized Basin

Manning's Equation

$$\frac{Q}{A} = V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Time of Concentration Calculations

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

SUB-BASIN DATA				INITIAL/OVERLAND TIME (t_i)			CONCENTRATED TRAVEL TIME (t_t)				MINIMUM T_c	URBAN CHECK		FIRST DESIGN POINT
DESIGN POINT	SUB BASIN	C5	AREA (acres)	LENGTH (ft)	SLOPE (%)	Ti (min)	LENGTH (ft)	SLOPE (%)	VEL. (fps)	Tt (min)	Tc (min)	Length (ft)	Urban Tc	**SUBBASIN Tc**
1	P-1	0.15	0.36	215	3.26	17.03	0	0.0	0.00	0.00	17.03	215	11.19	**11.19**
2	P-2	0.83	0.48	62	1.60	3.29	223	0.5	1.41	2.63	5.92	285	11.58	**5.92**
6	P-3	0.83	0.57	63	1.80	3.16	172	0.5	1.41	2.03	5.19	235	11.31	**5.19**
8	P-4	0.60	0.12	29	3.00	3.38	35	3.3	3.63	0.16	3.54	64	10.36	**5.00**
9	P-5	0.77	0.58	115	1.40	5.75	128	0.5	1.41	1.51	7.25	243	11.35	**7.25**
4	P-6	0.82	0.40	50	1.00	3.53	150	0.7	1.67	1.49	5.02	200	11.11	**5.02**
7	P-7	0.88	0.36	50	1.00	2.78	145	0.5	1.41	1.71	4.49	195	11.08	**5.00**
10	P-8	0.77	0.17	50	1.00	4.23	105	0.5	1.41	1.24	5.47	155	10.86	**5.47**

PROPOSED BASINS CONTRIBUTING TO EXISTING STORM DRAINS UNDETAINED:

SUB-BASIN DATA				INITIAL/OVERLAND TIME (t_i)			CONCENTRATED TRAVEL TIME (t_t)				MINIMUM T_c	URBAN CHECK		FIRST DESIGN POINT
DESIGN POINT	SUB BASIN	C5	AREA (acres)	LENGTH (ft)	SLOPE (%)	Ti (min)	LENGTH (ft)	SLOPE (%)	VEL. (fps)	Tt (min)	Tc (min)	Length (ft)	Urban Tc	**SUBBASIN Tc**
11	A-2	0.15	0.65	34	25.0	3.46	212	0.3	1.10	3.23	6.68	246	11.37	**6.68**
14	B-1a	0.90	0.23	23	1.00	1.76	0	-	-	0.00	1.76	23	10.13	**5.00**
15	C-2a	0.85	0.02	15	1.00	1.75	0	-	-	0.00	1.75	15	10.08	**5.00**
16	D-1a	0.75	0.23	124	1.40	6.35	12	0.5	1.41	0.14	6.50	136	10.76	**6.50**

Velocity from figure RO-1 and equation RO-4
from the USDCM Volume 1

$$V = C_V * S_W^{.5}$$

Table RO-2 Conveyance Cefficient, Cv

TYPE OF LAND SURFACE	CONVEYANCE COEFFICIENT, Cv
Heavy Meadow	3
Tillage/Field	5
Short Pasture and Lawns	7
Nearly Bare Ground	10
Grassed Waterway	15
Paved areas/shallow paved swales	20

$T_c = T_i + T_t$

Initial Flow Time (eqtn RO-3): $t_i = \frac{0.395(1.1 - C_5)\sqrt{L}}{S^{0.33}}$

Overland Travel Time: $t_t = \frac{L}{60 * V}$

Manning's Equation

$$\frac{Q}{A} = V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

URBAN Tc (eqtn RO-5): $t_c = \frac{L}{180} + 10$

Minimum Tc= 5 MIN for an Urbanized Basin

5-Year Flow Calculations

MAJOR BASINS RELEASING UNDETAINED:

BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C5	EFFECTIVE AREA (acres)	BASIN Tc (min)	INTENSITY (in/hr)	**BASIN Q (cfs)**
12	A-1	17.58	0.79	13.90	19.98	2.64	**36.65**
11	A-2	0.65	0.15	0.10	6.68	4.18	**0.40**
14	B-1	4.10	0.80	3.27	8.72	3.82	**12.49**
15	C-2	13.46	0.82	11.03	11.38	3.44	**37.95**
16	D-1	2.35	0.82	1.93	7.54	4.02	**7.74**
18	E-1	10.37	0.84	8.73	7.59	4.01	**35.01**
17	E-2	1.61	0.89	1.44	5.64	4.40	**6.33**

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$

5-Year Flow Calculations

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C5	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
1	P-1	0.36	0.15	0.05	11.19	3.46	**0.19**
2	P-2	0.48	0.83	0.40	5.92	4.34	**1.73**
6	P-3	0.57	0.83	0.47	5.19	4.50	**2.12**
8	P-4	0.12	0.60	0.07	5.00	4.55	**0.31**
9	P-5	0.58	0.77	0.45	7.25	4.07	**1.81**
4	P-6	0.40	0.82	0.33	5.02	4.54	**1.50**
7	P-7	0.36	0.88	0.32	5.00	4.55	**1.46**
10	P-8	0.17	0.77	0.13	5.47	4.44	**0.59**
						TOTAL=	9.71

PROPOSED BASINS CONTRIBUTING TO EXISTING STORM DRAINS UNDETAINED:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C5	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
11	A-2	0.65	0.15	0.10	6.68	4.18	**0.40**
14	B-1a	0.23	0.90	0.20	5.00	4.55	**0.93**
15	C-2a	0.02	0.85	0.02	5.00	4.55	**0.08**
16	D-1a	0.23	0.75	0.17	6.50	4.22	**0.72**
						TOTAL=	2.13

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$

10-Year Flow Calculations

MAJOR BASINS RELEASING UNDETAINED:

BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C10	EFFECTIVE AREA (acres)	BASIN Tc (min)	INTENSITY (in/hr)	**BASIN Q (cfs)**
12	A-1	17.58	0.81	14.26	19.98	3.05	**43.49**
11	A-2	0.65	0.25	0.16	6.68	4.84	**0.78**
14	B-1	4.10	0.82	3.35	8.72	4.42	**14.82**
15	C-2	13.46	0.84	11.34	11.38	3.98	**45.14**
16	D-1	2.35	0.84	1.98	7.54	4.65	**9.19**
18	E-1	10.37	0.86	8.94	7.59	4.64	**41.46**
17	E-2	1.61	0.91	1.47	5.64	5.09	**7.48**

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$

10-Year Flow Calculations

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C10	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
1	P-1	0.36	0.25	0.09	11.19	4.01	**0.36**
2	P-2	0.48	0.86	0.41	5.92	5.02	**2.06**
6	P-3	0.57	0.86	0.49	5.19	5.21	**2.53**
8	P-4	0.12	0.65	0.07	5.00	5.26	**0.39**
9	P-5	0.58	0.80	0.46	7.25	4.71	**2.18**
4	P-6	0.40	0.85	0.34	5.02	5.25	**1.79**
7	P-7	0.36	0.90	0.33	5.00	5.26	**1.73**
10	P-8	0.17	0.80	0.14	5.47	5.13	**0.71**
						TOTAL=	11.77

PROPOSED BASINS CONTRIBUTING TO EXISTING STORM DRAINS UNDETAINED:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C10	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
11	A-2	0.65	0.25	0.16	6.68	4.84	**0.78**
14	B-1a	0.23	0.92	0.21	5.00	5.26	**1.10**
15	C-2a	0.02	0.87	0.02	5.00	5.26	**0.09**
16	D-1a	0.23	0.78	0.18	6.50	4.88	**0.87**
						TOTAL=	2.84

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$

100-Year Flow Calculations

MAJOR BASINS RELEASING UNDETAINED:

BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C100	EFFECTIVE AREA (acres)	BASIN Tc (min)	INTENSITY (in/hr)	**BASIN Q (cfs)**
12	A-1	17.58	0.87	15.23	19.98	5.06	**77.01**
11	A-2	0.65	0.50	0.32	6.68	8.02	**2.59**
14	B-1	4.10	0.87	3.58	8.72	7.32	**26.19**
15	C-2	13.46	0.90	12.09	11.38	6.60	**79.77**
16	D-1	2.35	0.89	2.10	7.54	7.71	**16.20**
18	E-1	10.37	0.91	9.43	7.59	7.69	**72.55**
17	E-2	1.61	0.96	1.54	5.64	8.44	**12.97**

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$

100-Year Flow Calculations

PROPOSED BASINS CONTRIBUTING TO DETENTION POND WITH SAND FILTER BASIN:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C100	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
1	P-1	0.36	0.50	0.18	11.19	6.64	**1.19**
2	P-2	0.48	0.92	0.44	5.92	8.32	**3.66**
6	P-3	0.57	0.92	0.52	5.19	8.63	**4.49**
8	P-4	0.12	0.78	0.09	5.00	8.72	**0.78**
9	P-5	0.58	0.87	0.51	7.25	7.81	**3.95**
4	P-6	0.40	0.91	0.37	5.02	8.71	**3.18**
7	P-7	0.36	0.95	0.35	5.00	8.72	**3.01**
10	P-8	0.17	0.88	0.15	5.47	8.51	**1.30**
						TOTAL=	21.56

PROPOSED BASINS CONTRIBUTING TO EXISTING STORM DRAINS UNDETAINED:

SUB-BASIN DATA					DIRECT RUNOFF		
DESIGN POINT	CONTRIBUTING BASIN	BASIN AREA (acres)	C100	EFFECTIVE AREA (acres)	SUBBASIN Tc (min)	INTENSITY (in/hr)	**SUBBASIN Q (cfs)**
11	A-2	0.65	0.50	0.32	6.68	8.02	**2.59**
14	B-1a	0.23	0.96	0.22	5.00	8.72	**1.91**
15	C-2a	0.02	0.93	0.02	5.00	8.72	**0.17**
16	D-1a	0.23	0.87	0.20	6.50	8.09	**1.59**
						TOTAL=	6.25

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

From Denver Storm Drainage Design and Technical Criteria Equation 5.1:

Rainfall Intensity: $$I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$$

Empirical Volume Calculations

DETENTION VOLUME SIZING USING EMPIRICAL FORMULAS:

Design Return Period (Years)	Soil Type	Imperviousness (%)	Area (Acres)	Allowable Release Rate (cfs)	K Value	**Volume (ac-ft)**
10	D	67	4.16	*	0.06	**0.26**
100	D	67	4.16	*	0.12	**0.49**

* Note: To compensate for the undetained release of Basins A-2, B-1a, C-2a, and D-1a (1.12 acres), the detention volume is being calculated for the developed area of 4.16 acres (3.04 + 1.12 acres), while the release rates are being restricted to 0.91cfs (10-yr) and 3.04cfs (100-yr). See next page for release rate calculations.

Table SO-1 Maximum Unit Flow Release Rates (cfs/acre) from On-Site Detention Facilities

Design Return Period (Years)	NRCS Hydrologic Soil Group		
	A	B	C
2	0.02	0.03	0.04
5	0.07	0.13	0.17
10	0.13	0.23	0.30
25	0.24	0.41	0.52
50	0.33	0.56	0.68
100	0.50	0.85	1.00

Equation SO-1

$$V_i = K_i A$$

Equation SO-2

$$K_{100} = \frac{(1.78 I - 0.002 I^2 - 3.56)}{900}$$

Equation SO-3

$$K_{10} = \frac{(0.95 I - 1.90)}{1000}$$

Allowable Release Rates

RELEASE RATE CALCULATIONS:

Design Return Period (Years)	Soil Type	Imperviousness (%)	Area (Acres)	**Allowable Release Rate (cfs)**
10	D	78	3.04	**0.91**
100	D	78	3.04	**3.04**

Table SO-1 Maximum Unit Flow Release Rates (cfs/acre) from On-Site Detention Facilities

Design Return Period (Years)	NRCS Hydrologic Soil Group		
	A	B	C
2	0.02	0.03	0.04
5	0.07	0.13	0.17
10	0.13	0.23	0.30
25	0.24	0.41	0.52
50	0.33	0.56	0.68
100	0.50	0.85	1.00

Detention Pond Depth vs Volume Calculations

DETENTION POND VOLUME CALCULATIONS:

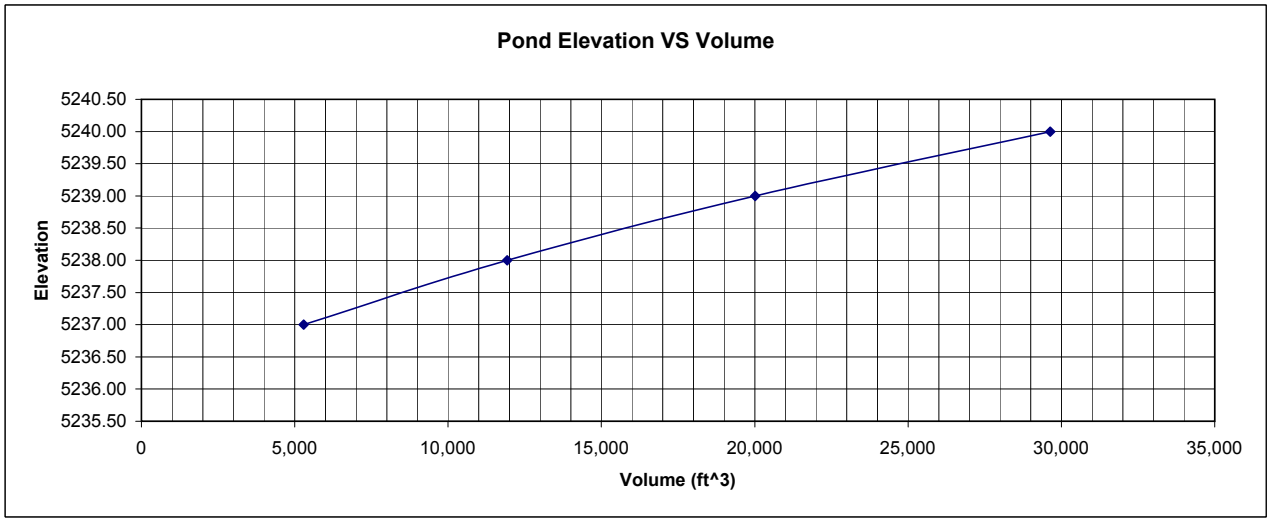
Elevation	Elevation Area	Average Interval Area	Interval Volume	Cumulative Volume	Cumulative Volume (ac-ft)
5236.00	4648.66				0.00
5237.00	5941.14	5294.90	5294.90	5294.90	0.12
5238.00	7334.15	6637.65	6637.65	11932.55	0.27
5239.00	8827.70	8080.93	8080.93	20013.47	0.46
5240.00	10421.78	9624.74	9624.74	29638.21	0.68

WATER SURFACE ELEVATION CALCULATIONS:

	Volume (acre-ft)	WSEL (ft)
Required Water Quality Volume=	0.08	5236.67
Total 10-yr Detention Volume (plus WQCV)=	0.34	5238.35
Total 100-yr Detention Volume (plus 1/2 WQCV)=	0.54	5239.34

Top of Pond

5241.00

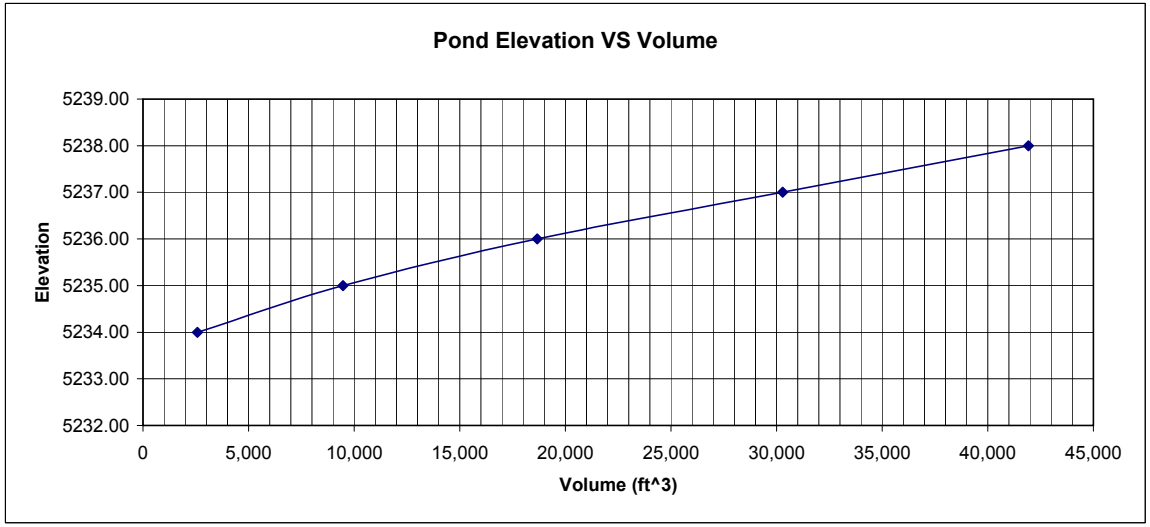


Pond Depth vs Volume Calculations

(Emergency Containment Pond)

EMERGENCY CONTAINMENT AREA VOLUME CALCULATIONS: (NOT INNTENDED FOR DETENTION)

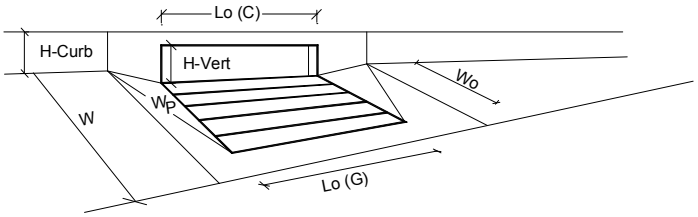
Elevation	Elevation Area	Average Interval Area	Interval Volume	Cumulative Volume	Cumulative Volume (ac-ft)
5233.20	698.41				0.00
5234.00	5757.28	3227.85	2582.28	2582.28	0.06
5235.00	8019.15	6888.22	6888.22	9470.49	0.22
5236.00	10381.55	9200.35	9200.35	18670.84	0.43
5237.00	12844.49	11613.02	11613.02	30283.86	0.70
5238.00	10421.78	11633.14	11633.14	41917.00	0.96
5239.00	10421.78	10421.78	10421.78	52338.78	1.20
5240.00	10421.78	10421.78	10421.78	62760.56	1.44
5240.34	10421.78	10421.78	3543.41	66303.96	1.52



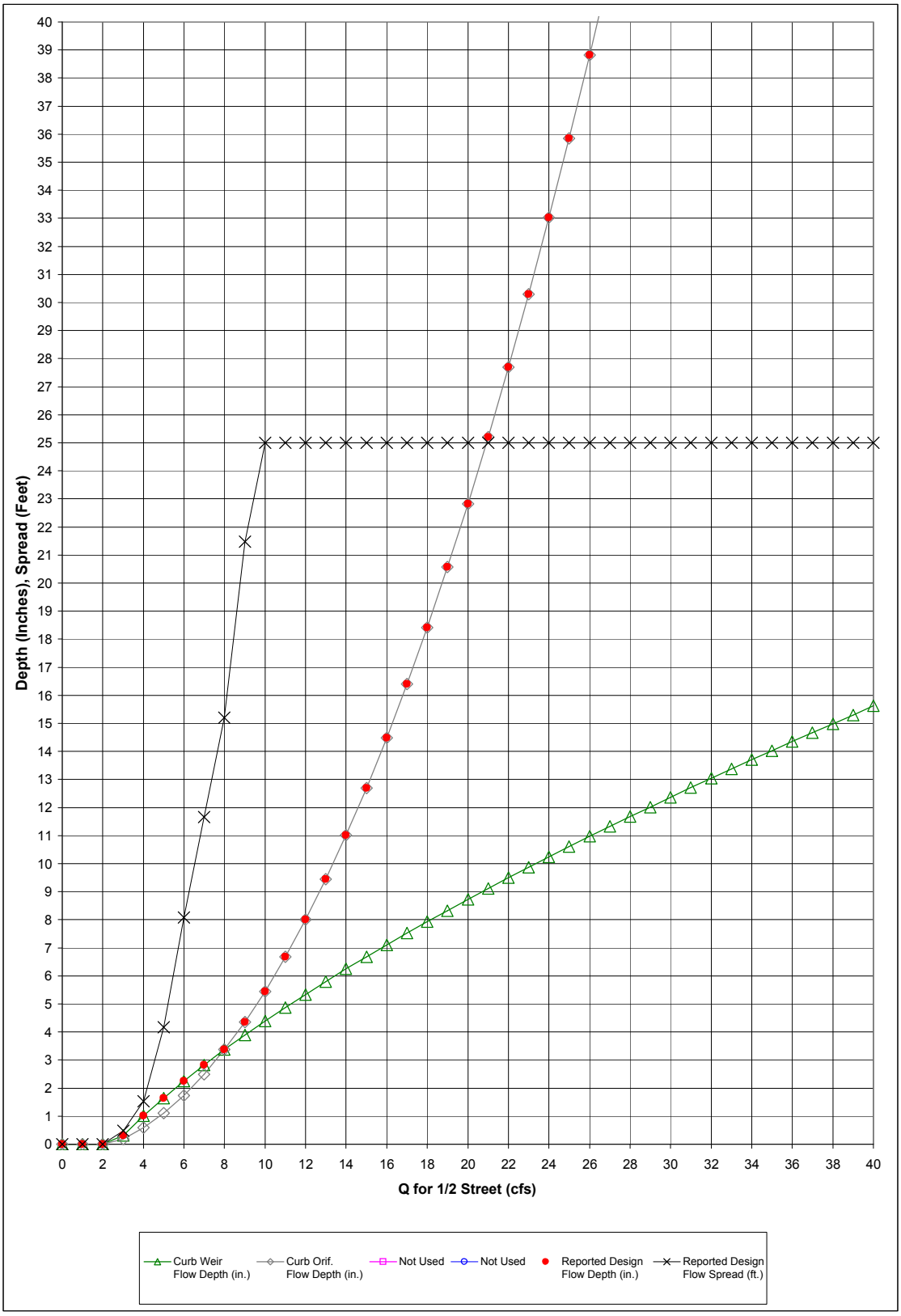
B. HYDRAULIC COMPUTATIONS

INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center
Inlet ID = Denver No. 14 Inlet #1



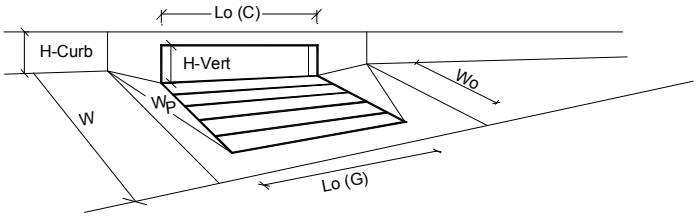
		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 14 Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	6.00	6.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.95	5.95	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.4	63.4	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate as a Weir				
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.73 cfs curb)	d_{wi} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}un}$ =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.73 cfs curb)	d_{wa} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}cl}$ =	N/A	N/A	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.73 cfs curb)	d_{oi} =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.73 cfs curb)	d_{oa} =	N/A	N/A	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**N/A**	**N/A**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.73 cfs curb)	d_{wi} =	2.20	3.62	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.73 cfs curb)	d_{wa} =	2.29	3.78	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.73 cfs curb)	d_{oi} =	2.80	3.29	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.73 cfs curb)	d_{oa} =	2.83	3.44	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.00**	**0.78**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	6.0	6.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	1.7	3.7	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.00**	**0.78**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.0**	**1.2**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**


Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Curb Weir Flow Depth (in.)
Curb Orif. Flow Depth (in.)
Not Used
Not Used
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

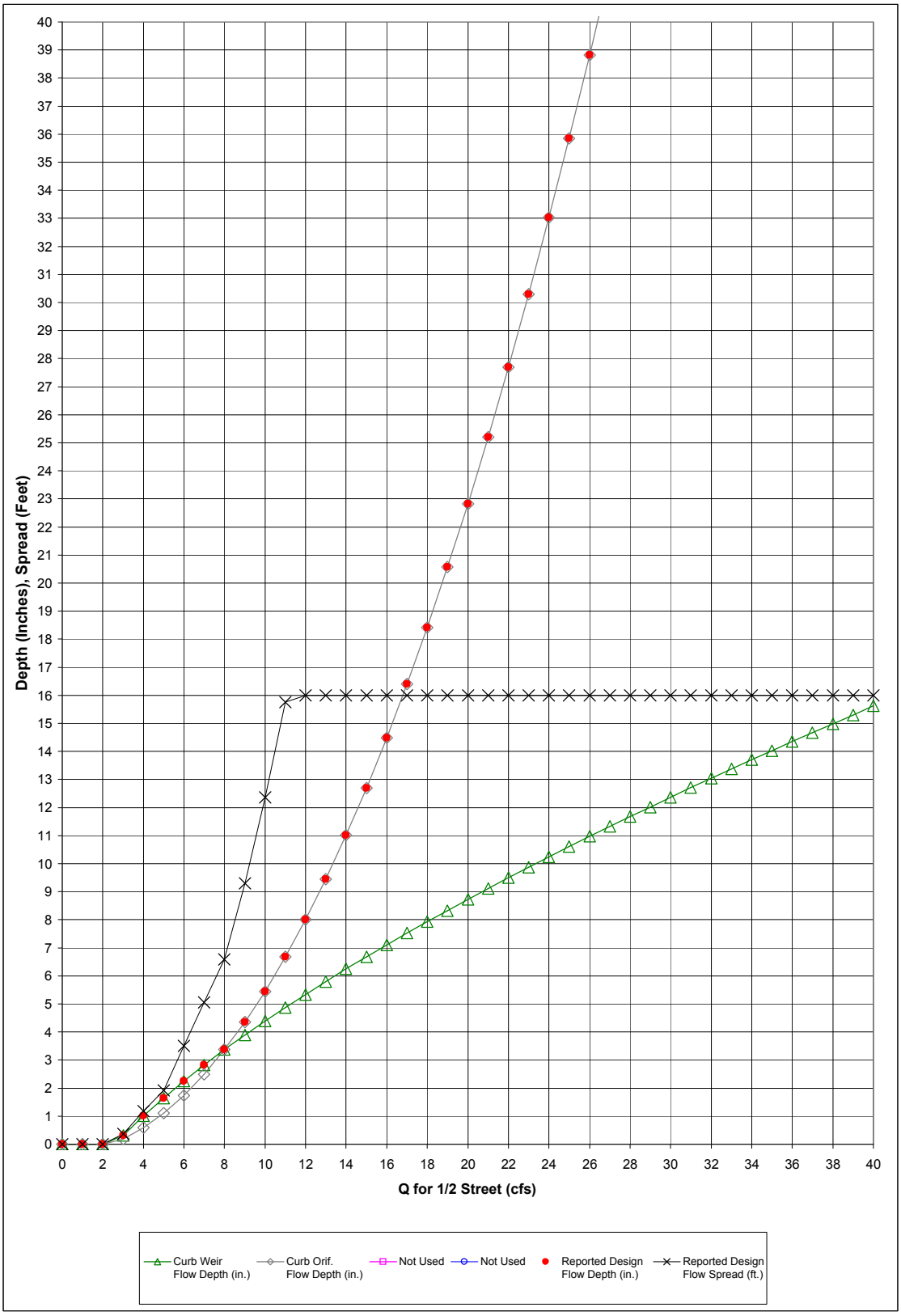
Q Intercepted (cfs)	Curb Weir Flow Depth (in.)	Curb Orif. Flow Depth (in.)	Not Used	Not Used	Reported Design Flow Depth (in.)	Reported Design Flow Spread (ft.)
0.00	0.00	0.00			0.00	0.00
1.00	0.00	0.00			0.00	0.00
2.00	0.00	0.00			0.00	0.00
3.00	0.31	0.18			0.31	0.47
4.00	1.01	0.59			1.01	1.54
5.00	1.65	1.11			1.65	4.17
6.00	2.26	1.74			2.26	8.08
7.00	2.82	2.50			2.82	11.67
8.00	3.37	3.37			3.37	15.19
9.00	3.89	4.35			4.35	21.48
10.00	4.39	5.45			5.45	25.00
11.00	4.87	6.67			6.67	25.00
12.00	5.34	8.00			8.00	25.00
13.00	5.80	9.45			9.45	25.00
14.00	6.25	11.01			11.01	25.00
15.00	6.68	12.69			12.69	25.00
16.00	7.11	14.49			14.49	25.00
17.00	7.52	16.40			16.40	25.00
18.00	7.93	18.42			18.42	25.00
19.00	8.33	20.57			20.57	25.00
20.00	8.73	22.82			22.82	25.00
21.00	9.12	25.20			25.20	25.00
22.00	9.50	27.69			27.69	25.00
23.00	9.87	30.29			30.29	25.00
24.00	10.24	33.02			33.02	25.00
25.00	10.61	35.85			35.85	25.00
26.00	10.97	38.81			38.81	25.00
27.00	11.33	41.88			41.88	25.00
28.00	11.68	45.06			45.06	25.00
29.00	12.02	48.36			48.36	25.00
30.00	12.37	51.78			51.78	25.00
31.00	12.71	55.31			55.31	25.00
32.00	13.04	58.96			58.96	25.00
33.00	13.38	62.72			62.72	25.00
34.00	13.71	66.60			66.60	25.00
35.00	14.03	70.60			70.60	25.00
36.00	14.35	74.71			74.71	25.00
37.00	14.67	78.94			78.94	25.00
38.00	14.99	83.28			83.28	25.00
39.00	15.30	87.74			87.74	25.00
40.00	15.62	92.31			92.31	25.00

INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center
Inlet ID = Denver No. 14 Inlet #2



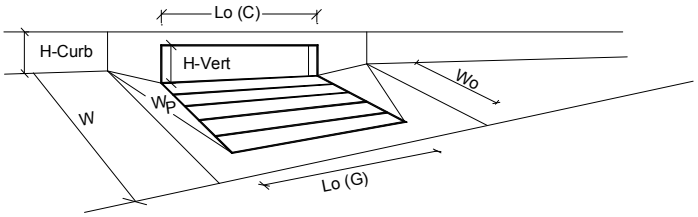
		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 14 Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	6.00	6.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.95	5.95	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.4	63.4	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate as a Weir				
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.31 cfs curb)	d_{wi} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}un}$ =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.31 cfs curb)	d_{wa} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}cl}$ =	N/A	N/A	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.31 cfs curb)	d_{oi} =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.31 cfs curb)	d_{oa} =	N/A	N/A	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**N/A**	**N/A**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.31 cfs curb)	d_{wi} =	0.70	1.29	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.31 cfs curb)	d_{wa} =	0.73	1.35	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.31 cfs curb)	d_{oi} =	2.66	2.69	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.31 cfs curb)	d_{oa} =	2.67	2.70	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.00**	**0.00**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	6.0	6.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	0.3	0.8	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.00**	**0.00**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.0**	**0.0**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**


Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Curb Weir Flow Depth (in.)
Curb Orif. Flow Depth (in.)
Not Used
Not Used
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

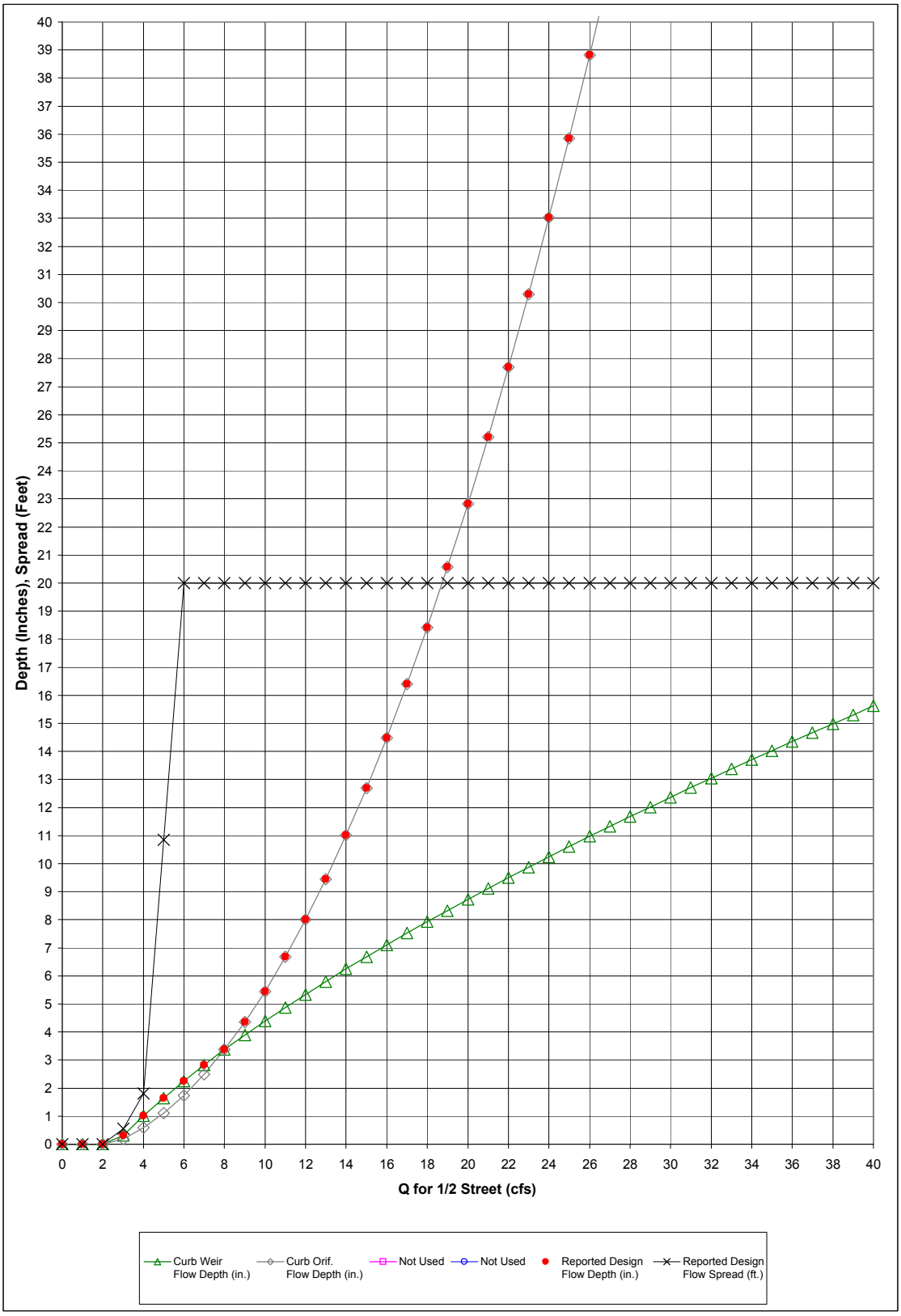
Q Intercepted (cfs)	Curb Weir Flow Depth (in.)	Curb Orif. Flow Depth (in.)	Not Used	Not Used	Reported Design Flow Depth (in.)	Reported Design Flow Spread (ft.)
0.00	0.00	0.00			0.00	0.00
1.00	0.00	0.00			0.00	0.00
2.00	0.00	0.00			0.00	0.00
3.00	0.31	0.18			0.31	0.36
4.00	1.01	0.59			1.01	1.17
5.00	1.65	1.11			1.65	1.92
6.00	2.26	1.74			2.26	3.50
7.00	2.82	2.50			2.82	5.06
8.00	3.37	3.37			3.37	6.58
9.00	3.89	4.35			4.35	9.31
10.00	4.39	5.45			5.45	12.36
11.00	4.87	6.67			6.67	15.75
12.00	5.34	8.00			8.00	16.00
13.00	5.80	9.45			9.45	16.00
14.00	6.25	11.01			11.01	16.00
15.00	6.68	12.69			12.69	16.00
16.00	7.11	14.49			14.49	16.00
17.00	7.52	16.40			16.40	16.00
18.00	7.93	18.42			18.42	16.00
19.00	8.33	20.57			20.57	16.00
20.00	8.73	22.82			22.82	16.00
21.00	9.12	25.20			25.20	16.00
22.00	9.50	27.69			27.69	16.00
23.00	9.87	30.29			30.29	16.00
24.00	10.24	33.02			33.02	16.00
25.00	10.61	35.85			35.85	16.00
26.00	10.97	38.81			38.81	16.00
27.00	11.33	41.88			41.88	16.00
28.00	11.68	45.06			45.06	16.00
29.00	12.02	48.36			48.36	16.00
30.00	12.37	51.78			51.78	16.00
31.00	12.71	55.31			55.31	16.00
32.00	13.04	58.96			58.96	16.00
33.00	13.38	62.72			62.72	16.00
34.00	13.71	66.60			66.60	16.00
35.00	14.03	70.60			70.60	16.00
36.00	14.35	74.71			74.71	16.00
37.00	14.67	78.94			78.94	16.00
38.00	14.99	83.28			83.28	16.00
39.00	15.30	87.74			87.74	16.00
40.00	15.62	92.31			92.31	16.00

INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center
Inlet ID = Denver No. 14 Inlet #3



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 14 Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	6.00	6.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.95	5.95	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.4	63.4	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate as a Weir				
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.59 cfs curb)	d_{wi} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}un}$ =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.59 cfs curb)	d_{wa} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}cl}$ =	N/A	N/A	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.59 cfs curb)	d_{oi} =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.59 cfs curb)	d_{oa} =	N/A	N/A	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**N/A**	**N/A**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.59 cfs curb)	d_{wi} =	1.07	1.82	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.59 cfs curb)	d_{wa} =	1.12	1.90	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 0.59 cfs curb)	d_{oi} =	2.68	2.74	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 0.59 cfs curb)	d_{oa} =	2.68	2.76	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.00**	**0.00**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	6.0	6.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	0.6	1.3	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.00**	**0.00**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.0**	**0.0**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**

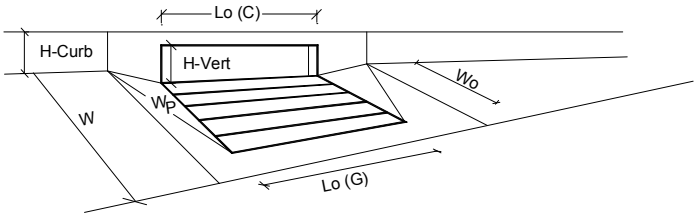

Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Curb Weir Flow Depth (in.)
Curb Orif. Flow Depth (in.)
Not Used
Not Used
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

Q Intercepted (cfs)	Curb Weir Flow Depth (in.)	Curb Orif. Flow Depth (in.)	Not Used	Not Used	Reported Design Flow Depth (in.)	Reported Design Flow Spread (ft.)
0.00	0.00	0.00			0.00	0.00
1.00	0.00	0.00			0.00	0.00
2.00	0.00	0.00			0.00	0.00
3.00	0.31	0.18			0.31	0.55
4.00	1.01	0.59			1.01	1.80
5.00	1.65	1.11			1.65	10.84
6.00	2.26	1.74			2.26	20.00
7.00	2.82	2.50			2.82	20.00
8.00	3.37	3.37			3.37	20.00
9.00	3.89	4.35			4.35	20.00
10.00	4.39	5.45			5.45	20.00
11.00	4.87	6.67			6.67	20.00
12.00	5.34	8.00			8.00	20.00
13.00	5.80	9.45			9.45	20.00
14.00	6.25	11.01			11.01	20.00
15.00	6.68	12.69			12.69	20.00
16.00	7.11	14.49			14.49	20.00
17.00	7.52	16.40			16.40	20.00
18.00	7.93	18.42			18.42	20.00
19.00	8.33	20.57			20.57	20.00
20.00	8.73	22.82			22.82	20.00
21.00	9.12	25.20			25.20	20.00
22.00	9.50	27.69			27.69	20.00
23.00	9.87	30.29			30.29	20.00
24.00	10.24	33.02			33.02	20.00
25.00	10.61	35.85			35.85	20.00
26.00	10.97	38.81			38.81	20.00
27.00	11.33	41.88			41.88	20.00
28.00	11.68	45.06			45.06	20.00
29.00	12.02	48.36			48.36	20.00
30.00	12.37	51.78			51.78	20.00
31.00	12.71	55.31			55.31	20.00
32.00	13.04	58.96			58.96	20.00
33.00	13.38	62.72			62.72	20.00
34.00	13.71	66.60			66.60	20.00
35.00	14.03	70.60			70.60	20.00
36.00	14.35	74.71			74.71	20.00
37.00	14.67	78.94			78.94	20.00
38.00	14.99	83.28			83.28	20.00
39.00	15.30	87.74			87.74	20.00
40.00	15.62	92.31			92.31	20.00

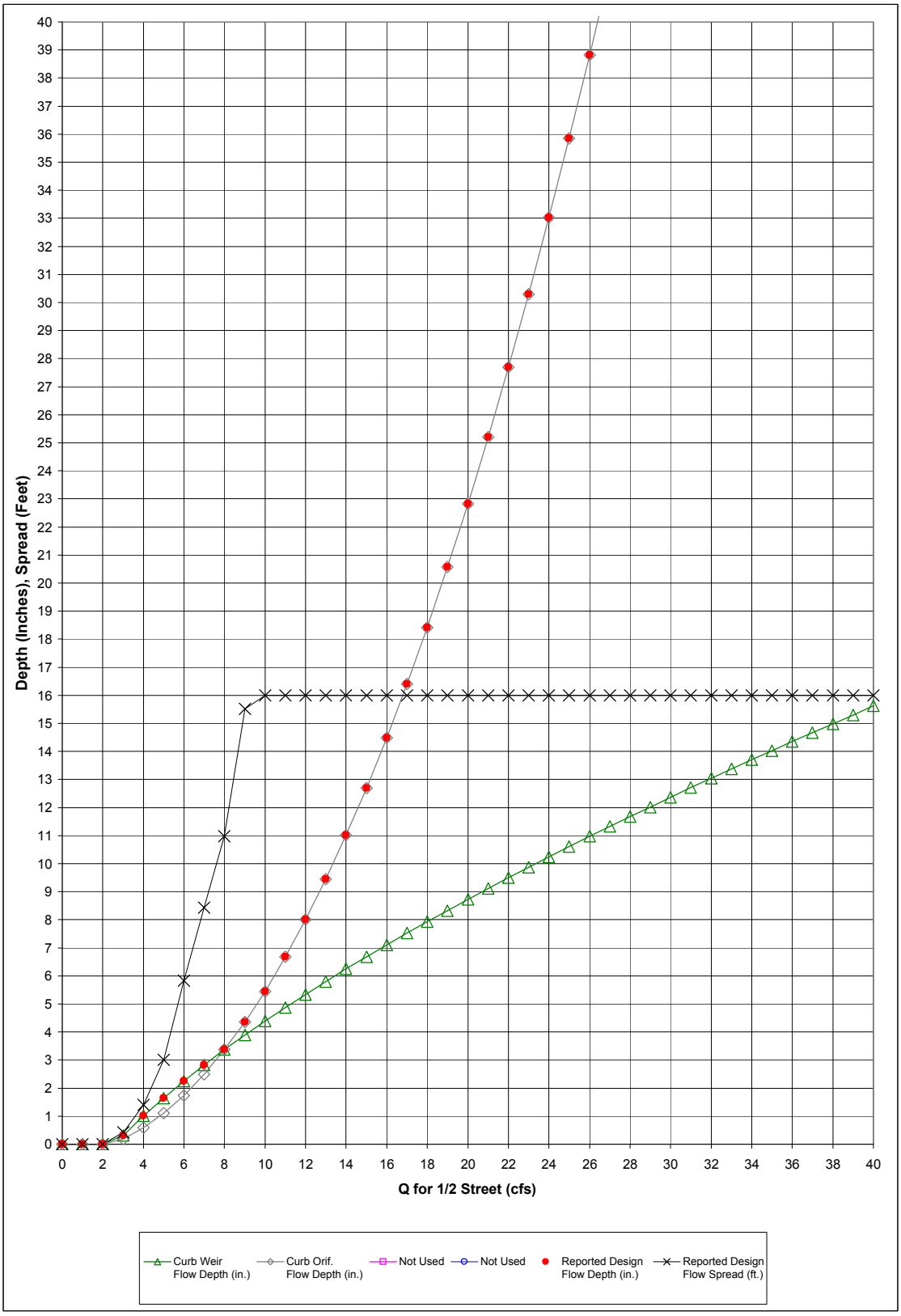
INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center

Inlet ID = Denver No.14 Inlet #4



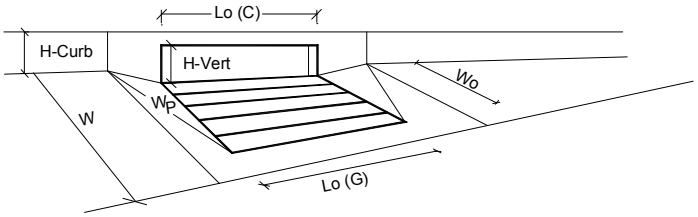
		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 14 Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	6.00	6.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.95	5.95	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.4	63.4	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate as a Weir				
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.81 cfs curb)	d_{wi} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}un}$ =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.81 cfs curb)	d_{wa} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}cl}$ =	N/A	N/A	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.81 cfs curb)	d_{oi} =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.81 cfs curb)	d_{oa} =	N/A	N/A	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**N/A**	**N/A**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.81 cfs curb)	d_{wi} =	2.26	3.81	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.81 cfs curb)	d_{wa} =	2.36	3.98	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.81 cfs curb)	d_{oi} =	2.81	3.39	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.81 cfs curb)	d_{oa} =	2.85	3.56	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.00**	**0.98**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	6.0	6.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	1.8	4.0	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.00**	**0.98**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.0**	**1.4**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**


Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Curb Weir Flow Depth (in.)
Curb Orif. Flow Depth (in.)
Not Used
Not Used
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

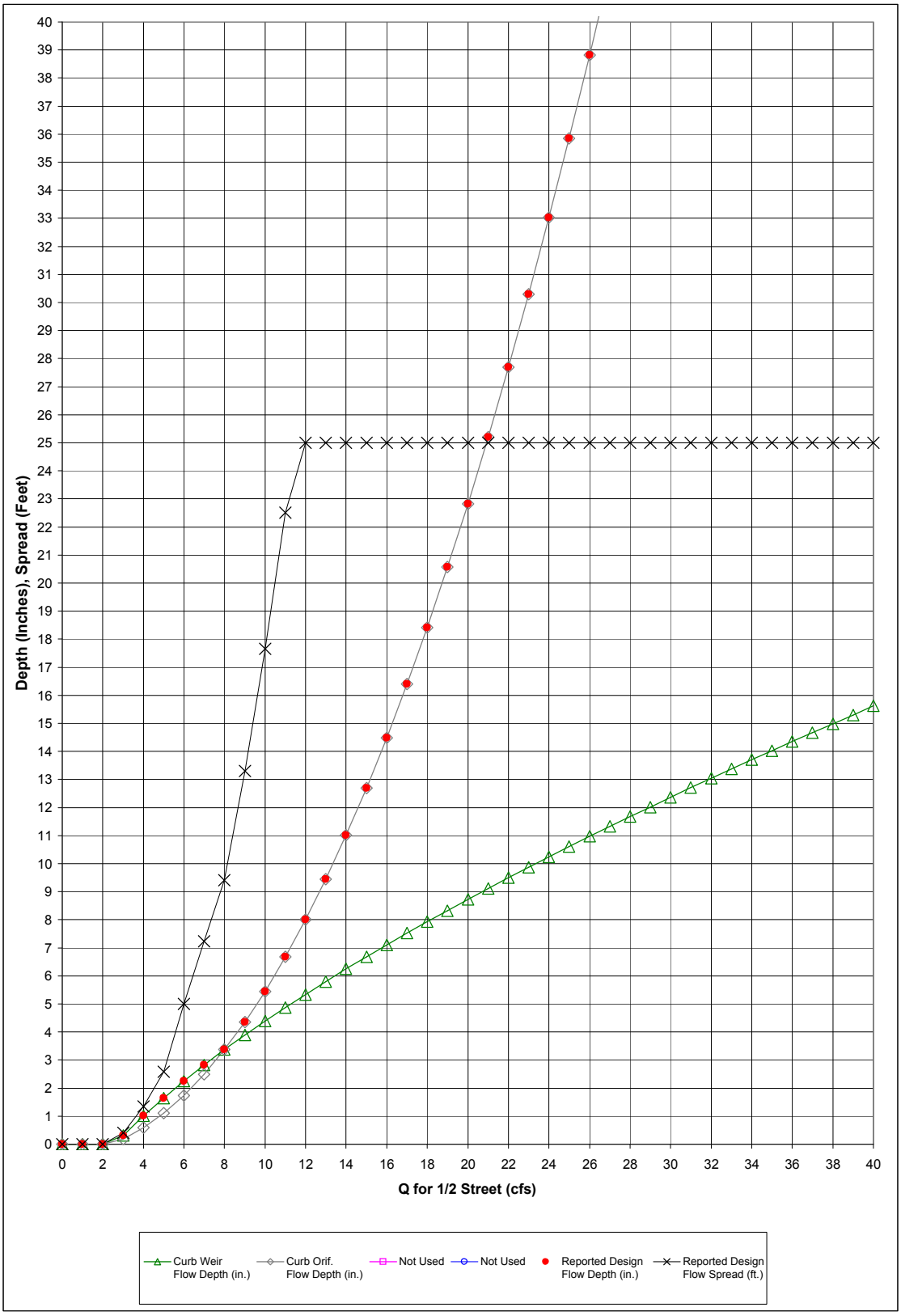
Q Intercepted (cfs)	Curb Weir Flow Depth (in.)	Curb Orif. Flow Depth (in.)	Not Used	Not Used	Reported Design Flow Depth (in.)	Reported Design Flow Spread (ft.)
0.00	0.00	0.00			0.00	0.00
1.00	0.00	0.00			0.00	0.00
2.00	0.00	0.00			0.00	0.00
3.00	0.31	0.18			0.31	0.43
4.00	1.01	0.59			1.01	1.41
5.00	1.65	1.11			1.65	3.01
6.00	2.26	1.74			2.26	5.84
7.00	2.82	2.50			2.82	8.43
8.00	3.37	3.37			3.37	10.97
9.00	3.89	4.35			4.35	15.51
10.00	4.39	5.45			5.45	16.00
11.00	4.87	6.67			6.67	16.00
12.00	5.34	8.00			8.00	16.00
13.00	5.80	9.45			9.45	16.00
14.00	6.25	11.01			11.01	16.00
15.00	6.68	12.69			12.69	16.00
16.00	7.11	14.49			14.49	16.00
17.00	7.52	16.40			16.40	16.00
18.00	7.93	18.42			18.42	16.00
19.00	8.33	20.57			20.57	16.00
20.00	8.73	22.82			22.82	16.00
21.00	9.12	25.20			25.20	16.00
22.00	9.50	27.69			27.69	16.00
23.00	9.87	30.29			30.29	16.00
24.00	10.24	33.02			33.02	16.00
25.00	10.61	35.85			35.85	16.00
26.00	10.97	38.81			38.81	16.00
27.00	11.33	41.88			41.88	16.00
28.00	11.68	45.06			45.06	16.00
29.00	12.02	48.36			48.36	16.00
30.00	12.37	51.78			51.78	16.00
31.00	12.71	55.31			55.31	16.00
32.00	13.04	58.96			58.96	16.00
33.00	13.38	62.72			62.72	16.00
34.00	13.71	66.60			66.60	16.00
35.00	14.03	70.60			70.60	16.00
36.00	14.35	74.71			74.71	16.00
37.00	14.67	78.94			78.94	16.00
38.00	14.99	83.28			83.28	16.00
39.00	15.30	87.74			87.74	16.00
40.00	15.62	92.31			92.31	16.00

INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center
Inlet ID = Denver No. 14 Inlet #5



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 14 Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	N/A	N/A	feet
Width of a Unit Grate	W_o =	N/A	N/A	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	N/A	N/A	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	6.00	6.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.95	5.95	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	63.4	63.4	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	N/A	N/A	
Clogging Factor for Multiple Units	Clog =	N/A	N/A	
Grate as a Weir				
Flow Depth at Local Depression without Clogging (0 cfs grate, 2.12 cfs curb)	d_{wi} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}un}$ =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 2.12 cfs curb)	d_{wa} =	N/A	N/A	inches
This Row Used for Combination Inlets Only	$d_{curb\text{-}cl}$ =	N/A	N/A	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 2.12 cfs curb)	d_{oi} =	N/A	N/A	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 2.12 cfs curb)	d_{oa} =	N/A	N/A	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**N/A**	**N/A**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 2.12 cfs curb)	d_{wi} =	2.52	4.15	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 2.12 cfs curb)	d_{wa} =	2.63	4.33	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 2.12 cfs curb)	d_{oi} =	2.87	3.61	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 2.12 cfs curb)	d_{oa} =	2.92	3.83	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.00**	**1.33**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	6.0	6.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	2.1	4.5	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.00**	**1.33**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.0**	**1.8**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**

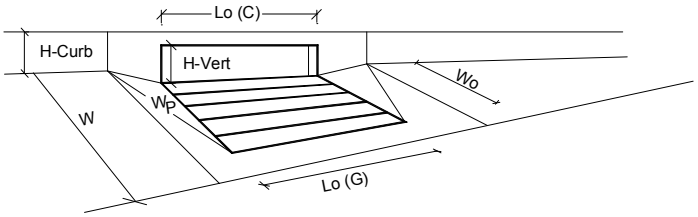

Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Curb Weir Flow Depth (in.)
Curb Orif. Flow Depth (in.)
Not Used
Not Used
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

Q Intercepted (cfs)	Curb Weir Flow Depth (in.)	Curb Orif. Flow Depth (in.)	Not Used	Not Used	Reported Design Flow Depth (in.)	Reported Design Flow Spread (ft.)
0.00	0.00	0.00			0.00	0.00
1.00	0.00	0.00			0.00	0.00
2.00	0.00	0.00			0.00	0.00
3.00	0.31	0.18			0.31	0.41
4.00	1.01	0.59			1.01	1.34
5.00	1.65	1.11			1.65	2.58
6.00	2.26	1.74			2.26	5.00
7.00	2.82	2.50			2.82	7.22
8.00	3.37	3.37			3.37	9.41
9.00	3.89	4.35			4.35	13.30
10.00	4.39	5.45			5.45	17.66
11.00	4.87	6.67			6.67	22.50
12.00	5.34	8.00			8.00	25.00
13.00	5.80	9.45			9.45	25.00
14.00	6.25	11.01			11.01	25.00
15.00	6.68	12.69			12.69	25.00
16.00	7.11	14.49			14.49	25.00
17.00	7.52	16.40			16.40	25.00
18.00	7.93	18.42			18.42	25.00
19.00	8.33	20.57			20.57	25.00
20.00	8.73	22.82			22.82	25.00
21.00	9.12	25.20			25.20	25.00
22.00	9.50	27.69			27.69	25.00
23.00	9.87	30.29			30.29	25.00
24.00	10.24	33.02			33.02	25.00
25.00	10.61	35.85			35.85	25.00
26.00	10.97	38.81			38.81	25.00
27.00	11.33	41.88			41.88	25.00
28.00	11.68	45.06			45.06	25.00
29.00	12.02	48.36			48.36	25.00
30.00	12.37	51.78			51.78	25.00
31.00	12.71	55.31			55.31	25.00
32.00	13.04	58.96			58.96	25.00
33.00	13.38	62.72			62.72	25.00
34.00	13.71	66.60			66.60	25.00
35.00	14.03	70.60			70.60	25.00
36.00	14.35	74.71			74.71	25.00
37.00	14.67	78.94			78.94	25.00
38.00	14.99	83.28			83.28	25.00
39.00	15.30	87.74			87.74	25.00
40.00	15.62	92.31			92.31	25.00

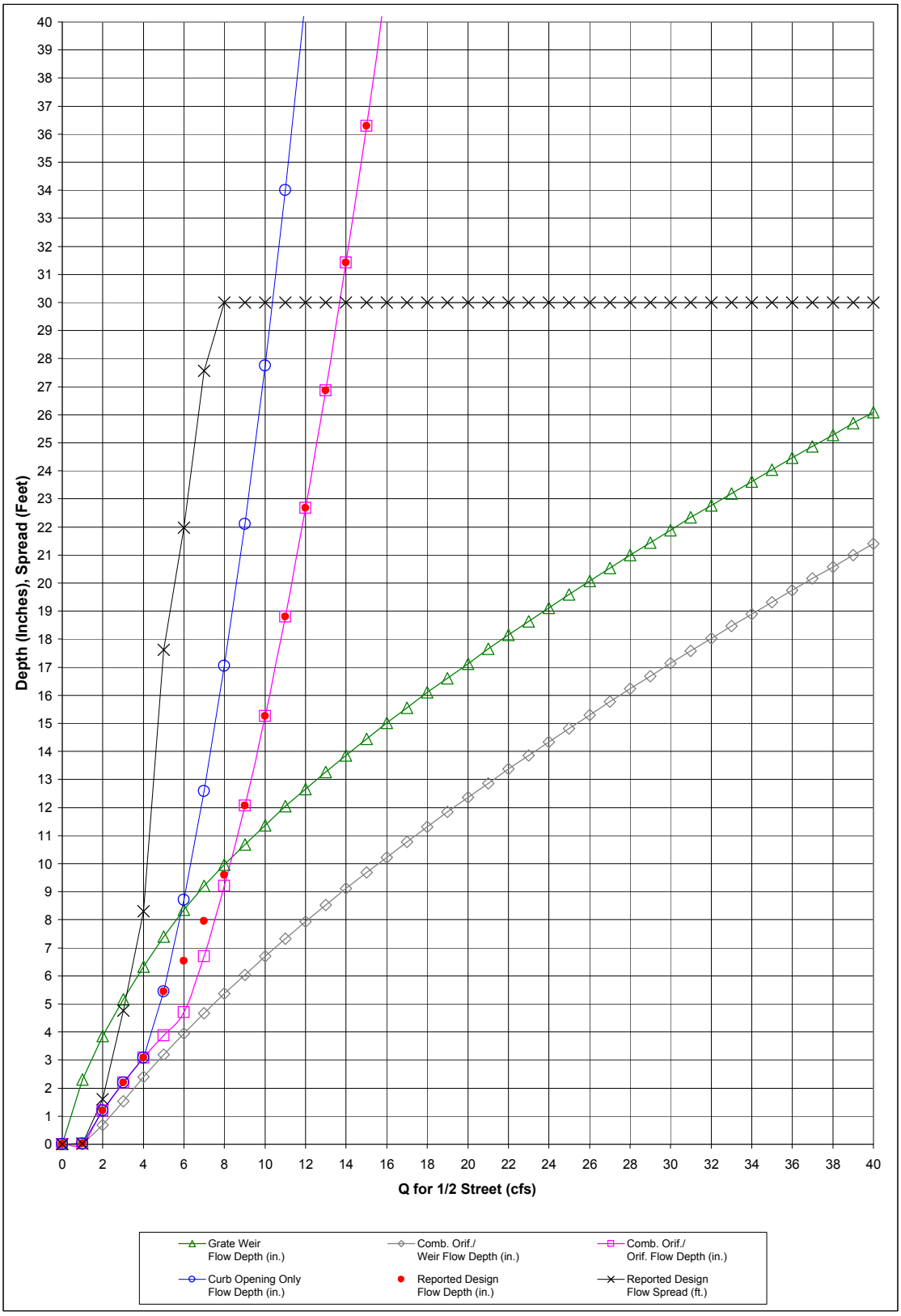
INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center

Inlet ID = Denver No. 16 Combo Inlet #1



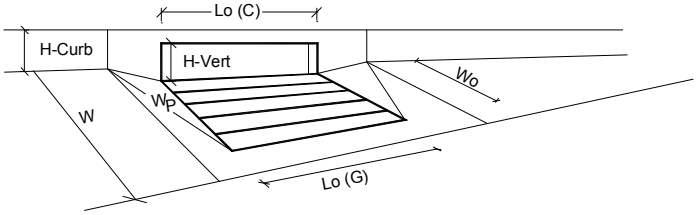
		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 16 Combination		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	2.00	2.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	3.00	3.00	feet
Width of a Unit Grate	W_o =	1.73	1.73	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	0.15	0.15	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	0.50	0.50	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	3.00	3.00	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	0.67	0.67	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	3.00	3.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	8.50	8.50	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.25	5.25	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	0.0	0.0	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.50	0.50	
Grate as a Weir: **The Controlling Factor Will Be:**		Curb Opening as Weir	Curb Opening As Weir	
Flow Depth at Local Depression without Clogging (1.5 cfs grate, 0 cfs curb)	d_{wi} =	4.97	7.12	inches
Flow Depth (Curb Opening Only) without Clogging (0 cfs grate, 1.5 cfs curb)	$d_{curb\text{-}un}$ =	2.56	4.23	inches
Flow Depth at Local Depression with Clogging (1.5 cfs grate, 0 cfs curb)	d_{wa} =	6.91	10.32	inches
Flow Depth (Curb Opening Only) with Clogging (0 cfs grate, 1.5 cfs curb)	$d_{curb\text{-}cl}$ =	2.65	4.37	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.5 cfs curb)	d_{oi} =	2.58	4.25	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.5 cfs curb)	d_{oa} =	2.77	4.56	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**0.77**	**2.56**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.5 cfs curb)	d_{wi} =	2.58	4.25	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.5 cfs curb)	d_{wa} =	2.77	4.56	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0 cfs grate, 1.5 cfs curb)	d_{oi} =	2.56	4.23	inches
Flow Depth at Local Depression with Clogging (0 cfs grate, 1.5 cfs curb)	d_{oa} =	2.65	4.37	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.77**	**2.56**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	3.0	3.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	1.5	3.2	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.65**	**2.37**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.9**	**5.4**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**


Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Grate Weir Flow Depth (in.)
Comb. Orif./ Weir Flow Depth (in.)
Comb. Orif./ Orif. Flow Depth (in.)
Curb Opening Only Flow Depth (in.)
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

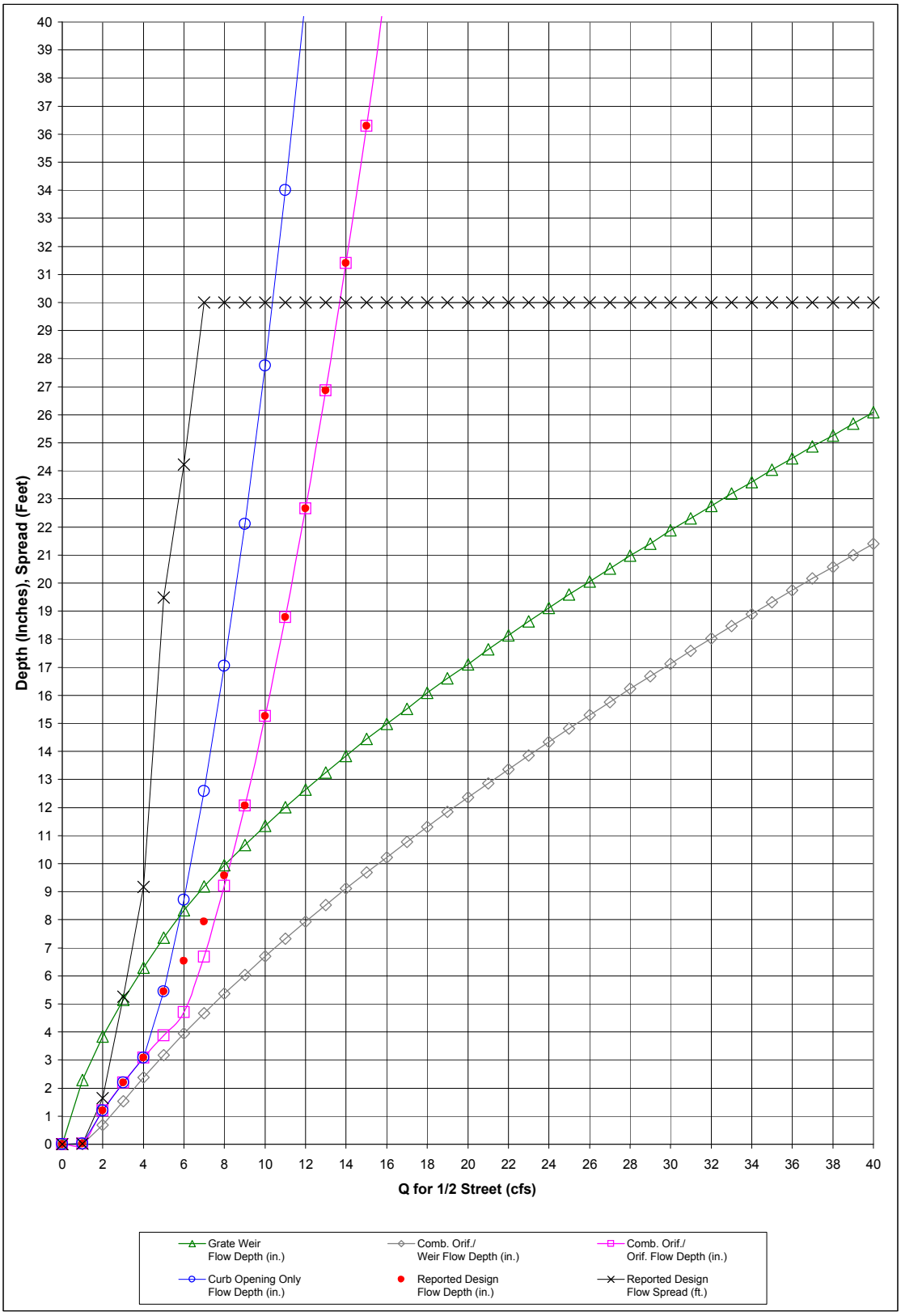
Q Intercepted (cfs)	Grate Weir □Flow Depth (in.)	Comb. Orif./ □Weir Flow Depth (in.)	Comb. Orif./ □Orif. Flow Depth (in.)	Curb Opening Only □Flow Depth (in.)	Reported Design □Flow Depth (in.)	Reported Design□Flow Spread (ft.)
0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	2.31	0.00	0.02	0.02	0.02	0.03
2.00	3.86	0.69	1.20	1.20	1.20	1.60
3.00	5.16	1.54	2.20	2.20	2.20	4.76
4.00	6.32	2.39	3.09	3.09	3.09	8.30
5.00	7.39	3.19	3.87	5.44	5.44	17.62
6.00	8.36	3.95	4.71	8.71	6.54	21.97
7.00	9.20	4.67	6.69	12.58	7.95	27.56
8.00	9.97	5.37	9.21	17.04	9.59	30.00
9.00	10.69	6.04	12.07	22.10	12.07	30.00
10.00	11.37	6.69	15.26	27.75	15.26	30.00
11.00	12.04	7.32	18.80	34.00	18.80	30.00
12.00	12.65	7.93	22.67	40.84	22.67	30.00
13.00	13.26	8.53	26.87	48.28	26.87	30.00
14.00	13.85	9.11	31.42	56.31	31.42	30.00
15.00	14.44	9.68	36.30	64.94	36.30	30.00
16.00	15.01	10.23	41.52	74.16	41.52	30.00
17.00	15.55	10.78	47.07	83.98	47.07	30.00
18.00	16.10	11.31	52.96	94.39	52.96	30.00
19.00	16.61	11.84	59.18	105.40	59.18	30.00
20.00	17.13	12.36	65.75	117.00	65.75	30.00
21.00	17.65	12.86	72.65	129.20	72.65	30.00
22.00	18.16	13.37	79.88	141.99	79.88	30.00
23.00	18.64	13.86	87.45	155.38	87.45	30.00
24.00	19.12	14.34	95.37	169.37	95.37	30.00
25.00	19.60	14.82	103.62	183.94	103.62	30.00
26.00	20.08	15.30	112.20	199.12	112.20	30.00
27.00	20.54	15.77	121.12	214.89	121.12	30.00
28.00	20.99	16.23	130.37	231.25	130.37	30.00
29.00	21.43	16.68	139.97	248.21	139.97	30.00
30.00	21.88	17.14	149.90	265.76	149.90	30.00
31.00	22.34	17.58	160.17	283.91	160.17	30.00
32.00	22.76	18.02	170.77	302.65	170.77	30.00
33.00	23.20	18.46	181.71	321.99	181.71	30.00
34.00	23.62	18.89	192.99	341.92	192.99	30.00
35.00	24.04	19.32	204.60	362.45	204.60	30.00
36.00	24.46	19.75	216.55	383.57	216.55	30.00
37.00	24.88	20.17	228.84	405.29	228.84	30.00
38.00	25.28	20.58	241.46	427.61	241.46	30.00
39.00	25.71	21.00	254.42	450.51	254.42	30.00
40.00	26.08	21.41	267.71	474.02	267.71	30.00

INLET IN A SUMP OR SAG LOCATION

Project = Safeway Distribution Center
Inlet ID = Denver No. 16 Combo Inlet # 2



		MINOR	MAJOR	
Design Information (Input)		MINOR	MAJOR	
Type of Inlet	Type =	Denver No. 16 Combination		
Local Depression (additional to continuous gutter depression 'a' from 'Q-Allow')	a_{local} =	2.00	2.00	inches
Number of Unit Inlets (Grate or Curb Opening)	No =	1	1	
Grate Information		MINOR	MAJOR	
Length of a Unit Grate	L_o (G) =	3.00	3.00	feet
Width of a Unit Grate	W_o =	1.73	1.73	feet
Area Opening Ratio for a Grate (typical values 0.15-0.90)	A_{ratio} =	0.15	0.15	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)	C_f (G) =	0.50	0.50	
Grate Weir Coefficient (typical value 3.00)	C_w (G) =	3.00	3.00	
Grate Orifice Coefficient (typical value 0.67)	C_o (G) =	0.67	0.67	
Curb Opening Information		MINOR	MAJOR	
Length of a Unit Curb Opening	L_o (C) =	3.00	3.00	feet
Height of Vertical Curb Opening in Inches	H_{vert} =	8.50	8.50	inches
Height of Curb Orifice Throat in Inches	H_{throat} =	5.25	5.25	inches
Angle of Throat (see USDCM Figure ST-5)	Theta =	0.0	0.0	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)	W_p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)	C_f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.30-3.00)	C_w (C) =	2.30	2.30	
Curb Opening Orifice Coefficient (typical value 0.67)	C_o (C) =	0.67	0.67	
Resulting Gutter Flow Depth for Grate Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.50	0.50	
Grate as a Weir: **The Controlling Factor Will Be:**		Curb Opening as Weir	Curb Opening As Weir	
Flow Depth at Local Depression without Clogging (1.46 cfs grate, 0 cfs curb)	d_{wi} =	3.79	5.11	inches
Flow Depth (Curb Opening Only) without Clogging (0 cfs grate, 1.46 cfs curb)	$d_{curb\text{-}un}$ =	2.52	4.08	inches
Flow Depth at Local Depression with Clogging (1.46 cfs grate, 0 cfs curb)	d_{wa} =	5.04	7.15	inches
Flow Depth (Curb Opening Only) with Clogging (0 cfs grate, 1.46 cfs curb)	$d_{curb\text{-}cl}$ =	2.60	4.21	inches
Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (1.46 cfs grate, 0 cfs curb)	d_{oi} =	2.52	4.08	inches
Flow Depth at Local Depression with Clogging (0.98 cfs grate, 0.48 cfs curb)	d_{oa} =	2.60	4.21	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Grate}$ =**	**0.60**	**2.21**	**inches**
Resulting Gutter Flow Depth for Curb Opening Inlet Capacity in a Sump		MINOR	MAJOR	
Clogging Coefficient for Multiple Units	Coef =	1.00	1.00	
Clogging Factor for Multiple Units	Clog =	0.10	0.10	
Curb as a Weir, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (0.61 cfs grate, 0.85 cfs curb)	d_{wi} =	1.77	2.86	inches
Flow Depth at Local Depression with Clogging (0.42 cfs grate, 1.04 cfs curb)	d_{wa} =	2.18	3.54	inches
Curb as an Orifice, Grate as an Orifice		MINOR	MAJOR	
Flow Depth at Local Depression without Clogging (1.46 cfs grate, 0 cfs curb)	d_{oi} =	2.52	4.08	inches
Flow Depth at Local Depression with Clogging (0.98 cfs grate, 0.48 cfs curb)	d_{oa} =	2.60	4.21	inches
Resulting Gutter Flow Depth Outside of Local Depression	**$d_{a\text{-}Curb}$ =**	**0.60**	**2.21**	**inches**
Resultant Street Conditions		MINOR	MAJOR	
Total Inlet Length	L =	3.0	3.0	feet
Total Inlet Interception Capacity (Design Discharge from *Q-Peak*)	Q_a =	1.5	3.0	cfs
Resultant Gutter Flow Depth (based on sheet *Q-Allow* geometry)	**d =**	**0.60**	**2.21**	**inches**
Resultant Street Flow Spread (based on sheet *Q-Allow* geometry)	**T =**	**0.8**	**5.3**	**feet**
Resultant Flow Depth at Street Crown	**d_{CROWN} =**	**0.00**	**0.00**	**inches**


Depth (Inches), Spread (Feet)
Q for 1/2 Street (cfs)
Grate Weir Flow Depth (in.)
Comb. Orif./ Weir Flow Depth (in.)
Comb. Orif./ Orif. Flow Depth (in.)
Curb Opening Only Flow Depth (in.)
Reported Design Flow Depth (in.)
Reported Design Flow Spread (ft.)

Q Intercepted (cfs)	Grate Weir □Flow Depth (in.)	Comb. Orif./ □Weir Flow Depth (in.)	Comb. Orif./ □Orif. Flow Depth (in.)	Curb Opening Only □Flow Depth (in.)	Reported Design □Flow Depth (in.)	Reported Design□Flow Spread (ft.)
0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	2.28	0.00	0.02	0.02	0.02	0.03
2.00	3.84	0.68	1.20	1.20	1.20	1.65
3.00	5.14	1.53	2.20	2.20	2.20	5.26
4.00	6.30	2.38	3.09	3.09	3.09	9.17
5.00	7.37	3.18	3.87	5.44	5.44	19.48
6.00	8.34	3.95	4.71	8.71	6.53	24.23
7.00	9.18	4.67	6.68	12.58	7.93	30.00
8.00	9.95	5.37	9.20	17.04	9.58	30.00
9.00	10.67	6.04	12.06	22.10	12.06	30.00
10.00	11.35	6.69	15.26	27.75	15.26	30.00
11.00	12.02	7.32	18.79	34.00	18.79	30.00
12.00	12.64	7.93	22.66	40.84	22.66	30.00
13.00	13.24	8.53	26.87	48.28	26.87	30.00
14.00	13.84	9.11	31.41	56.31	31.41	30.00
15.00	14.44	9.68	36.29	64.94	36.29	30.00
16.00	14.99	10.23	41.51	74.16	41.51	30.00
17.00	15.52	10.78	47.06	83.98	47.06	30.00
18.00	16.08	11.31	52.95	94.39	52.95	30.00
19.00	16.60	11.84	59.18	105.40	59.18	30.00
20.00	17.11	12.36	65.74	117.00	65.74	30.00
21.00	17.63	12.86	72.64	129.20	72.64	30.00
22.00	18.14	13.36	79.88	141.99	79.88	30.00
23.00	18.63	13.86	87.45	155.38	87.45	30.00
24.00	19.12	14.34	95.37	169.37	95.37	30.00
25.00	19.59	14.82	103.61	183.94	103.61	30.00
26.00	20.06	15.30	112.19	199.12	112.19	30.00
27.00	20.52	15.76	121.11	214.89	121.11	30.00
28.00	20.97	16.23	130.37	231.25	130.37	30.00
29.00	21.41	16.68	139.97	248.21	139.97	30.00
30.00	21.88	17.13	149.90	265.76	149.90	30.00
31.00	22.31	17.58	160.16	283.91	160.16	30.00
32.00	22.74	18.02	170.76	302.65	170.76	30.00
33.00	23.19	18.46	181.71	321.99	181.71	30.00
34.00	23.60	18.89	192.98	341.92	192.98	30.00
35.00	24.04	19.32	204.59	362.45	204.59	30.00
36.00	24.44	19.75	216.54	383.57	216.54	30.00
37.00	24.88	20.17	228.83	405.29	228.83	30.00
38.00	25.26	20.58	241.45	427.61	241.45	30.00
39.00	25.69	21.00	254.41	450.51	254.41	30.00
40.00	26.08	21.41	267.71	474.02	267.71	30.00

Emergency Overflow Weir Sizing Calculations

EMERGENCY OVERFLOW CALCULATIONS:

Elevation (ft)	Water Surface Elevation	Top of Berm Elevation (ft)	Flow Height (ft)	Flow (cfs)	Width (ft)
5239.34	5240.00	5241.00	0.66	15.91	8.81

PROPOSED TOTAL SITE FLOW CALCULATIONS:

Design Period (yr)	Time of Concentration (min)	Area (acres)	C100	Intensity (in/hour)	Total Flow (cfs)
100	13.50	3.04	0.86	6.13	15.91

STORMCAD FLOW CALCULATIONS:

Design Period (yr)	Total Flow (cfs)
100.00	15.34

Weir Equation

$$Q = C_W \times b \times H^{3/2}$$

Weir Coefficient = 3.37

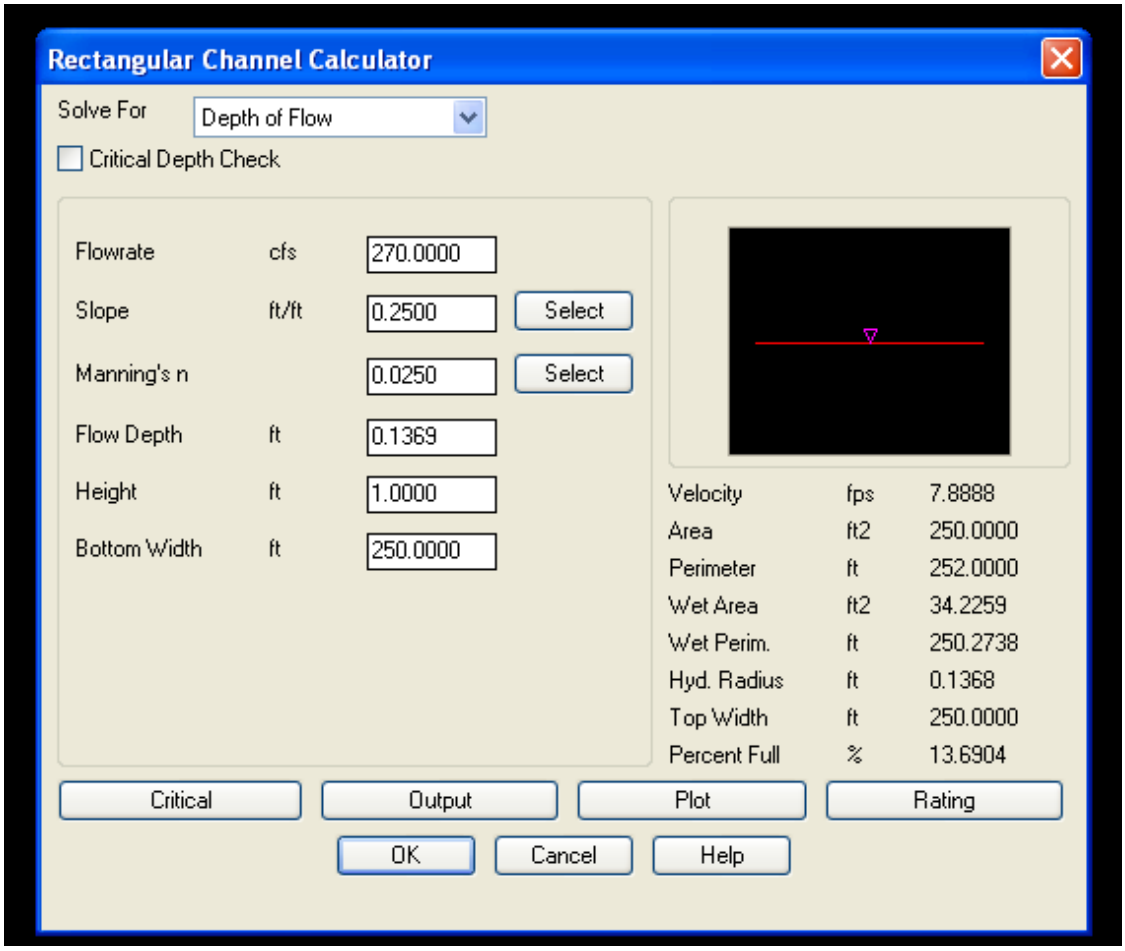
$$I = \frac{28.5 * P1}{(10 + T_C)^{.786}}$$

From Denver Storm Drainage Design and Technical Criteria Table 5.1 :

ONE-HOUR POINT RAINFALL DEPTHS

RETURN PERIOD	ONE-HR POINT RAINFALL (inches)
2-YEAR	0.95
5-YEAR	1.34
10-YEAR	1.55
50-YEAR	2.25
100-YEAR	2.57

Emergency Spillway Calculations:



Job No: CO1176F
Project: Safeway Distribution Center
Date: 7/7/2011

Riprap Basin Calculations

Pd=(v^2+gd)^1/2 g= 32.2 ft/sec^2
T = 1.75D(50)
larger of L = 4D or (D)^1/2(V/2) D = Diameter of the pipe(ft)
W = 4D d = design depth of flow in the pipe(ft)
v = velocity(ft)

Riprap sized per Urban Drainage Hydraulic Structures Section 3.4.3.2

Outlet 30" Pipe

v =	4.23					
d =	1.33					
D =	2.5					
Pd =	7.79					
use type	L	**riprap**	D(50)=	9	(in)	
T =	15.75	**(in)**				
L =	10	**(ft)**	**or**	3.34	**(ft)**	**Use L= 10.00 (ft)**
W =	10	**(ft)**				

Outlet 18" Pipe

v =	2.83					
d =	0.88					
D =	1.5					
Pd =	6.03					
use type	L	**riprap**	D(50)=	9	(in)	
T =	15.75	**(in)**				
L =	6	**(ft)**	**or**	1.73	**(ft)**	**Use L= 6.00 (ft)**
W =	6	**(ft)**				

STORMCAD CALCULATIONS

Calculation Detailed Summary

Element Details			
ID	13	Notes	
Label	Base Calculation Options		

Hydraulic Summary			
Flow Profile Method	Backwater Analysis	Average Velocity Method	Actual Uniform Flow Velocity
Number of Flow Profile Steps	5	Minimum Structure Headloss	0.00 ft
Hydraulic Grade Convergence Test	0.001 ft	Minimum Time of Concentration	5.000 min

Inlets			
Neglect Side Flow?	False	Active Components for Combination Inlets In Sag	Grate and Curb
Neglect Gutter Cross Slope For Side Flow?	True	Active Components for Combination Inlets on Grade	Grate and Curb

HEC-22			
Elevations Considered Equal Within	0.50 ft	Depressed Unsubmerged	1.000
Consider Non-Piped Plunging Flow	False	Half Bench Submerged	0.950
Flat Submerged	1.000	Half Bench Unsubmerged	0.150
Flat Unsubmerged	1.000	Full Bench Submerged	0.750
Depressed Submerged	1.000	Full Bench Unsubmerged	0.070

AASHTO			
Expansion, Ke	0.350	Shaping Adjustment, Cs	0.500
Contraction, Kc	0.250	Non-Piped Flow Adjustment, Cn	1.300

Bend Angle vs. Bend Loss Curve

Bend Angle (degrees)	Bend Loss Coefficient, Kb
0.00	0.000
15.00	0.190
30.00	0.350
45.00	0.470
60.00	0.560
75.00	0.640
90.00	0.700

Generic Structure Loss	
Governing Upstream Pipe Selection Method	Pipe with Maximum QV

Calculation Detailed Summary

Catchment Summary

Label	Area (acres)	Time of Concentration (min)	Rational C	Catchment CA (acres)	Catchment Intensity (in/hr)	Catchment Rational Flow (ft^3/s)
BASIN A1	17.580	20.000	0.870	15.295	5.060	78.01
BASIN B1	4.100	10.000	0.870	3.567	6.950	24.99
BASIN C2	13.460	10.000	0.900	12.114	6.950	84.87
BASIN D1	2.350	10.000	0.890	2.092	6.950	14.65
BASIN E1	10.370	10.000	0.910	9.437	6.950	66.11
BASIN E2	1.610	5.640	0.960	1.546	8.493	13.23
BASIN P-2	0.480	5.920	0.920	0.442	8.394	3.74
BASIN P-6	0.400	5.020	0.910	0.364	8.713	3.20
BASIN P-7	0.360	5.000	0.950	0.342	8.720	3.01
BASIN P-3	0.570	5.190	0.920	0.524	8.653	4.57
BASIN P-8	0.170	5.470	0.880	0.150	8.554	1.29
BASIN P-4	0.120	5.000	0.780	0.094	8.720	0.82
BASIN P-5	0.580	7.250	0.870	0.505	7.924	4.03

Conduit Summary

Label	Conduit Description	Conduit Shape	Branch ID	Subnetwork Outfall	Flow (ft^3/s)	Velocity (Average) (ft/s)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Depth (In) (ft)	Depth (Out) (ft)
CO-6	Circular Pipe - 21.0 in	Circular Pipe	6	OF-1	24.99	10.39	5,240.98	5,240.04	6.60	5.89
CO-7	Circular Pipe - 24.0 in	Circular Pipe	5	OF-1	84.87	27.01	5,248.61	5,241.71	12.46	6.36
CO-8	Circular Pipe - 15.0 in	Circular Pipe	2	OF-1	14.65	11.94	5,245.53	5,242.33	8.29	5.28
CO-5	Elliptical Pipe - 68.0 x 43.5 in	Elliptical Pipe	4	OF-1	66.11	3.79	5,242.23	5,242.20	6.03	6.06
CO-9	Circular Pipe - 12.0 in	Circular Pipe	3	OF-1	13.23	16.85	5,248.27	5,242.20	9.82	3.85
CO-11	Circular Pipe - 27.0 in	Circular Pipe	1	OF-2	78.01	19.62	5,243.58	5,237.11	9.90	3.79
CO-10	Circular Pipe - 42.0 in	Circular Pipe	1	OF-2	77.85	8.09	5,236.41	5,236.01	3.09	2.76
CO-2	Circular Pipe - 60.0 in	Circular Pipe	2	OF-1	163.13	8.31	5,241.18	5,240.04	6.43	5.96
CO-1	Circular Pipe - 60.0 in	Circular Pipe	2	OF-1	182.44	9.29	5,239.23	5,237.22	5.15	3.87
CO-4	Elliptical Pipe - 68.0 x 43.5 in	Elliptical Pipe	3	OF-1	76.63	4.39	5,242.64	5,242.33	6.50	6.67
CO-3	Circular Pipe - 60.0 in	Circular Pipe	2	OF-1	88.96	4.53	5,242.18	5,241.71	6.51	6.96
CO-19	Elliptical Pipe - 23.0 x 14.0 in	Elliptical Pipe	7	OF-3	4.03	2.12	5,241.26	5,241.17	1.30	1.54
CO-17	Circular Pipe - 21.0 in	Circular Pipe	7	OF-3	5.83	2.40	5,241.03	5,240.67	1.91	2.11
CO-16	Circular Pipe - 24.0 in	Circular Pipe	7	OF-3	11.82	3.62	5,240.54	5,239.74	2.59	2.38
CO-15	Circular Pipe - 24.0 in	Circular Pipe	7	OF-3	13.68	4.24	5,239.60	5,239.42	2.24	2.15
CO-14	Circular Pipe - 24.0 in	Circular Pipe	7	OF-3	13.60	4.21	5,239.25	5,239.06	2.08	1.99
CO-13	Circular Pipe - 27.0 in	Circular Pipe	7	OF-3	16.52	4.04	5,239.06	5,237.82	2.09	1.66
CO-12	Circular Pipe - 30.0 in	Circular Pipe	7	OF-3	15.64	4.23	5,237.59	5,237.33	1.53	1.34
CO-18	Elliptical Pipe - 23.0 x 14.0 in	Elliptical Pipe	7	OF-3	5.18	2.70	5,241.17	5,241.03	1.64	1.81
CO-21	Circular Pipe - 15.0 in	Circular Pipe	9	OF-3	3.01	2.45	5,240.76	5,240.67	2.73	2.72
CO-22	Circular Pipe - 18.0 in	Circular Pipe	8	OF-3	4.57	2.59	5,240.74	5,240.67	2.71	2.72
CO-20	Circular Pipe - 12.0 in	Circular Pipe	10	OF-3	3.20	4.07	5,240.03	5,239.74	1.99	1.88

Calculation Detailed Summary

Node Summary

Label	Element Type	Subnetwork Outfall	Flow (Total Surface) (ft^3/s)	Flow (Total Out) (ft^3/s)	Elevation (Ground) (ft)	Elevation (Invert) (ft)	Energy Grade Line (In) (ft)	Energy Grade Line (Out) (ft)
CB-3	Catch Basin	OF-2	78.01	78.01	5,241.28	5,233.68	5,247.26	5,247.26
MH-1	Manhole	OF-1	0.00	182.44	5,241.80	5,234.08	5,241.38	5,240.57
MH-7	Manhole	OF-1	84.87	84.87	5,241.96	5,236.15	5,260.10	5,253.30
MH-2	Manhole	OF-1	0.00	163.13	5,241.89	5,234.75	5,242.79	5,242.25
MH-5	Manhole	OF-1	66.11	66.11	5,242.30	5,236.20	5,242.65	5,242.49
MH-3	Manhole	OF-1	0.00	88.96	5,242.96	5,235.67	5,242.65	5,242.49
CB-1	Catch Basin	OF-1	14.65	14.65	5,241.54	5,237.24	5,243.76	5,243.76
CB-2	Catch Basin	OF-1	13.23	13.23	5,241.45	5,238.45	5,245.86	5,245.86
MH-4	Manhole	OF-1	0.00	76.63	5,242.20	5,236.14	5,242.55	5,242.55
MH-8	Manhole	OF-2	0.00	77.85	5,241.80	5,233.32	5,238.27	5,237.57
MH-6	Manhole	OF-1	24.99	24.99	5,241.98	5,234.38	5,243.67	5,242.66
CB-10	Catch Basin	OF-3	4.03	4.03	5,242.11	5,239.96	5,241.34	5,241.34
CB-8	Catch Basin	OF-3	0.82	5.77	5,241.85	5,239.12	5,241.12	5,241.12
MH-12	Manhole	OF-3	0.00	11.38	5,242.30	5,237.95	5,240.89	5,240.76
MH-11	Manhole	OF-3	0.00	13.32	5,242.35	5,237.36	5,240.04	5,239.89
MH-10	Manhole	OF-3	0.00	13.24	5,242.03	5,237.17	5,239.71	5,239.54
CB-4	Catch Basin	OF-3	3.74	16.08	5,240.70	5,236.97	5,239.34	5,239.34
MH-9	Manhole	OF-3	0.00	15.17	5,240.50	5,236.06	5,238.20	5,237.97
CB-9	Catch Basin	OF-3	1.29	5.12	5,241.81	5,239.53	5,241.30	5,241.30
CB-6	Catch Basin	OF-3	3.01	3.01	5,240.95	5,238.03	5,240.85	5,240.85
CB-7	Catch Basin	OF-3	4.57	4.57	5,241.00	5,238.03	5,240.85	5,240.85
CB-5	Catch Basin	OF-3	3.20	3.20	5,240.71	5,238.04	5,240.28	5,240.28

Inlet Summary

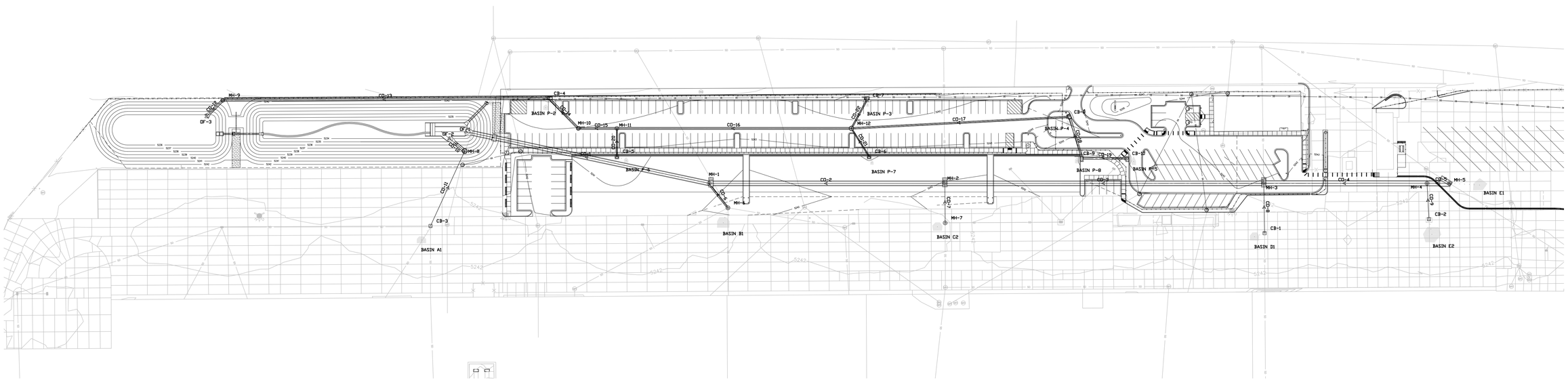
Label	Inlet Type	Inlet Type (Inlet)	Inlet	Flow (Total Intercepted) (ft^3/s)	Flow (Total Bypassed) (ft^3/s)	Bypass Target	Capture Efficiency (Calculated) (%)	Gutter Depth (in)	Gutter Spread (ft)
CB-3	Percent Capture	(N/A)	(N/A)	78.01	0.00	(N/A)	100.0	0.0	0.0
CB-1	Percent Capture	(N/A)	(N/A)	14.65	0.00	(N/A)	100.0	0.0	0.0
CB-2	Percent Capture	(N/A)	(N/A)	13.23	0.00	(N/A)	100.0	0.0	0.0
CB-10	Percent Capture	(N/A)	(N/A)	4.03	0.00	(N/A)	100.0	0.0	0.0
CB-8	Percent Capture	(N/A)	(N/A)	0.82	0.00	(N/A)	100.0	0.0	0.0
CB-4	Percent Capture	(N/A)	(N/A)	3.74	0.00	(N/A)	100.0	0.0	0.0
CB-9	Percent Capture	(N/A)	(N/A)	1.29	0.00	(N/A)	100.0	0.0	0.0
CB-6	Percent Capture	(N/A)	(N/A)	3.01	0.00	(N/A)	100.0	0.0	0.0
CB-7	Percent Capture	(N/A)	(N/A)	4.57	0.00	(N/A)	100.0	0.0	0.0
CB-5	Percent Capture	(N/A)	(N/A)	3.20	0.00	(N/A)	100.0	0.0	0.0

Calculation Detail Summary
(5-yr Flows)

Label	Start Node	Invert (Upstream) (ft)	Stop Node	Invert (Downstream) (ft)	Diameter (in)	Rise (in)	Span (in)	Upstream CA (acres)	System CA (acres)	System Intensity (in/hr)	Flow (ft^3/s)	Velocity (Average) (ft/s)	Slope (Calculated) (ft/ft)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
CO-1	MH-1	5,234.08	OF-1	5,233.35	60.0	(N/A)	(N/A)	28.755	28.755	3.334	96.62	7.11	0.002	5,237.31	5,236.15
CO-2	MH-2	5,234.75	MH-1	5,234.08	60.0	(N/A)	(N/A)	25.188	25.188	3.417	86.75	6.87	0.002	5,238.14	5,237.79
CO-3	MH-2	5,234.75	MH-3	5,235.67	60.0	(N/A)	(N/A)	13.074	13.074	3.548	46.75	5.93	-0.002	5,238.55	5,238.43
CO-4	MH-3	5,235.67	MH-4	5,236.14	(N/A)	43.5	68.0	10.982	10.982	3.619	40.07	5.55	-0.002	5,238.72	5,238.67
CO-5	MH-5	5,236.20	MH-4	5,236.14	(N/A)	43.5	68.0	9.437	9.437	3.630	34.53	5.15	0.002	5,238.73	5,238.72
CO-6	MH-6	5,234.38	MH-1	5,234.15	21.0	(N/A)	(N/A)	3.567	3.567	3.630	13.05	5.43	0.006	5,238.05	5,237.79
CO-7	MH-7	5,236.15	MH-2	5,235.35	24.0	(N/A)	(N/A)	12.114	12.114	3.630	44.33	14.11	0.016	5,240.31	5,238.43
CO-8	CB-1	5,237.24	MH-3	5,237.06	15.0	(N/A)	(N/A)	2.092	2.092	3.630	7.65	6.24	0.003	5,239.54	5,238.67
CO-9	CB-2	5,238.45	MH-4	5,238.35	12.0	(N/A)	(N/A)	1.546	1.546	4.432	6.91	8.79	0.002	5,240.99	5,239.32
CO-10	MH-8	5,233.32	OF-2	5,233.25	42.0	(N/A)	(N/A)	15.295	15.295	2.630	40.55	6.09	0.003	5,235.46	5,235.23
CO-11	CB-3	5,233.68	MH-8	5,233.32	27.0	(N/A)	(N/A)	15.295	15.295	2.640	40.70	10.24	0.004	5,237.63	5,235.86
CO-12	MH-9	5,236.06	OF-3	5,236.00	30.0	(N/A)	(N/A)	2.420	2.420	3.273	7.98	3.65	0.002	5,237.13	5,236.94
CO-13	CB-4	5,236.97	MH-9	5,236.16	27.0	(N/A)	(N/A)	2.420	2.420	3.489	8.51	3.67	0.002	5,238.24	5,237.28
CO-14	MH-10	5,237.17	CB-4	5,237.07	24.0	(N/A)	(N/A)	1.978	1.978	3.519	7.02	3.40	0.002	5,238.37	5,238.24
CO-15	MH-11	5,237.36	MH-10	5,237.27	24.0	(N/A)	(N/A)	1.978	1.978	3.547	7.07	3.40	0.002	5,238.59	5,238.49
CO-16	MH-12	5,237.95	MH-11	5,237.36	24.0	(N/A)	(N/A)	1.614	1.614	3.763	6.12	3.39	0.002	5,239.09	5,238.69
CO-17	CB-8	5,239.12	MH-12	5,238.56	21.0	(N/A)	(N/A)	0.748	0.748	4.049	3.05	2.88	0.002	5,239.91	5,239.20
CO-18	CB-9	5,239.53	CB-8	5,239.22	(N/A)	14.0	23.0	0.654	0.654	4.090	2.70	3.95	0.006	5,240.05	5,239.91
CO-19	CB-10	5,239.96	CB-9	5,239.63	(N/A)	14.0	23.0	0.505	0.505	4.136	2.10	3.65	0.006	5,240.42	5,240.05
CO-20	CB-5	5,238.04	MH-11	5,237.86	12.0	(N/A)	(N/A)	0.364	0.364	4.546	1.67	3.47	0.005	5,238.74	5,238.69
CO-21	CB-6	5,238.03	MH-12	5,237.95	15.0	(N/A)	(N/A)	0.342	0.342	4.550	1.57	2.38	0.002	5,239.22	5,239.20
CO-22	CB-7	5,238.03	MH-12	5,237.95	18.0	(N/A)	(N/A)	0.524	0.524	4.515	2.39	2.67	0.002	5,239.21	5,239.20

NOTE: The 5-yr Hydraulic Grade Line (In) values shown on this page were used to set the height of the weirs in each of the proposed diversion structures.

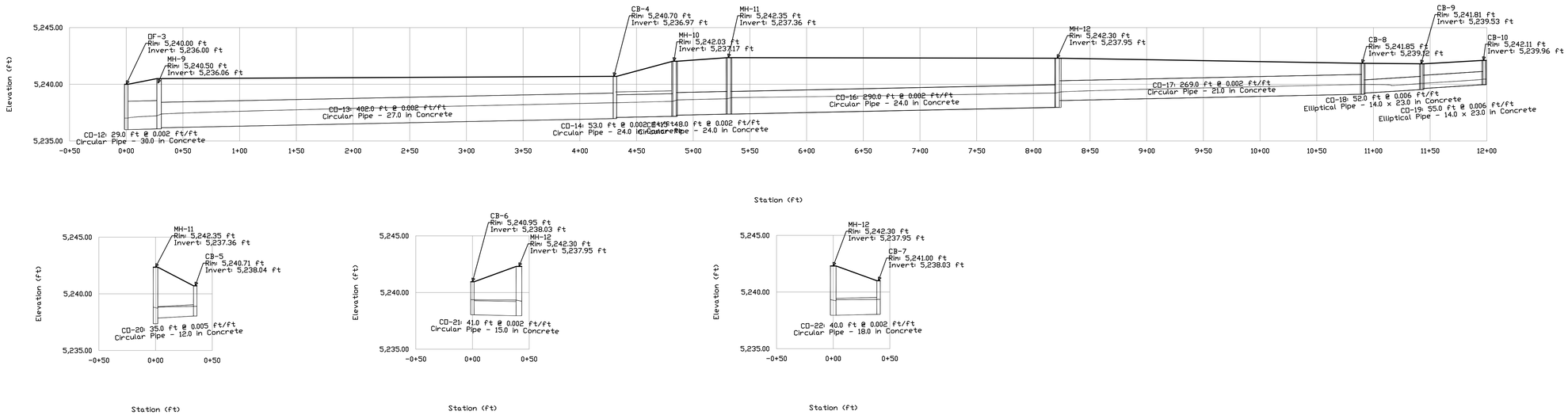
MH-1 = Diversion Structure A --> HGL=5237.31
MH-2 = Diversion Structure B --> HGL=5238.14
MH-3 = Diversion Structure C --> HGL=5238.55
MH-5 = Diversion Structure D --> HGL=5238.73

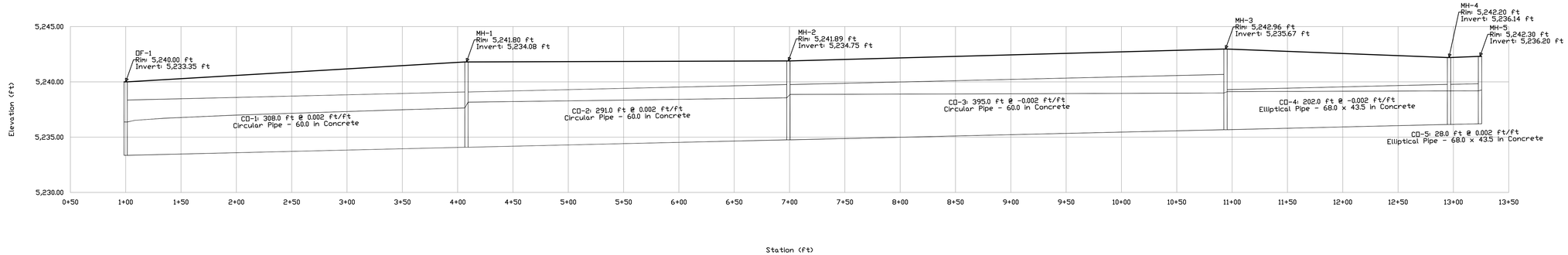


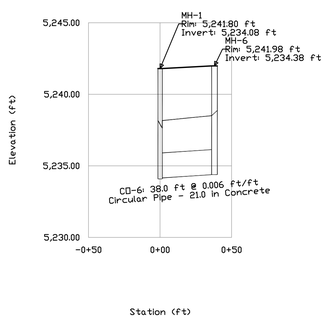
MH-9
CO-13
CB-4
BASIN P-2
MH-10
CO-15
MH-11
CO-16
CB-7
BASIN P-3
MH-12
CO-17
BASIN P-4
CB-8
CB-5
BASIN P-6
CB-6
BASIN P-7
CB-9
CO-19
CB-10
BASIN P-8
BASIN P-5
MH-1
CO-2
MH-2
MH-3
CO-4
MH-4
MH-5
BASIN E1
CB-3
BASIN A1
BASIN B1
MH-7
BASIN C2
CB-1
BASIN D1
CB-2
BASIN E2

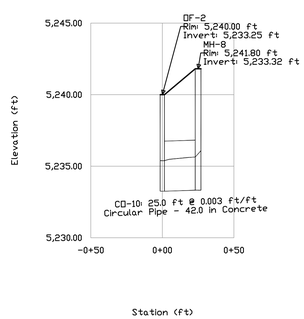
StormCAD 10-yr Flow Profiles

1"=50' Horizontal
1"=5' Vertical



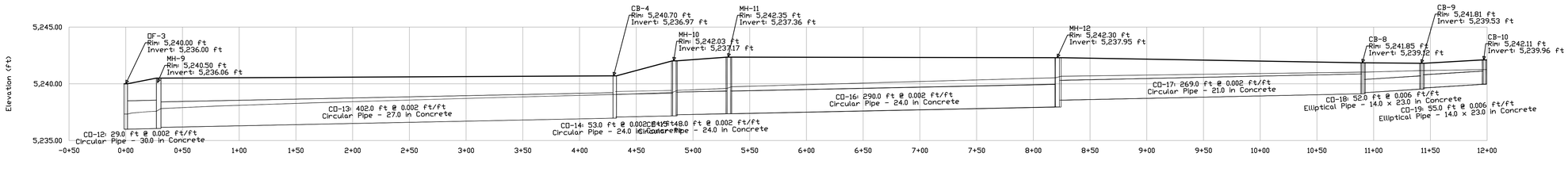


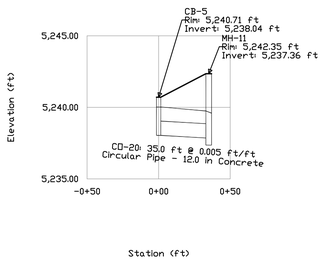


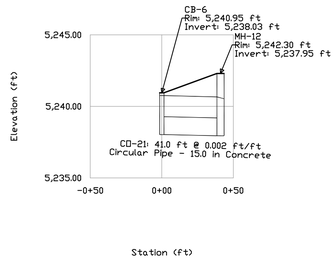


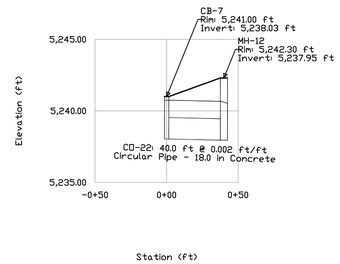
StormCAD 100-yr Flow Profiles

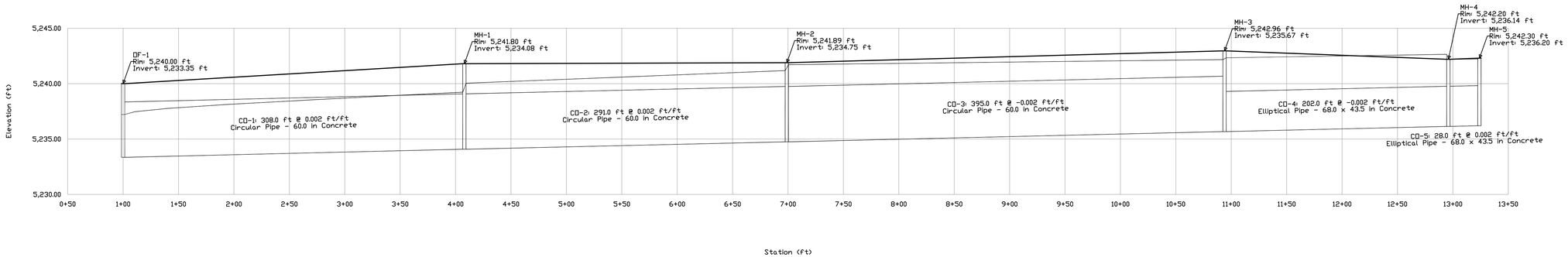
1"=50' Horizontal
1"=5' Vertical.

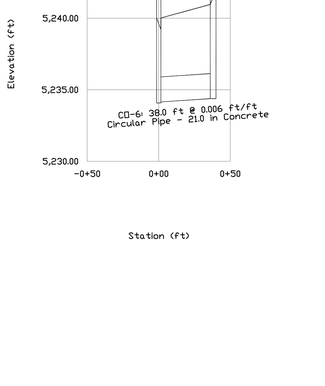


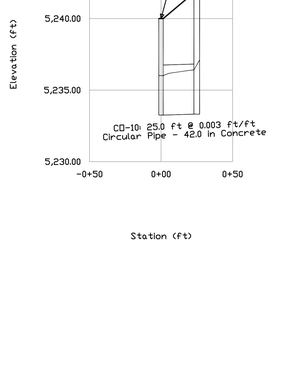












C. WATER QUALITY ENHANCEMENT BMP'S

Design Procedure Form: Sand Filter (SF)

Designer:	**DAP**
Company:	**Tait and Associates, Inc.**
Date:	**July 27, 2011**
Project:	**Safeway Distribution Center**
Location:	**Denver, CO**

1. Basin Storage Volume	
A) Effective Imperviousness of Tributary Area, I_a (100% if all paved and roofed areas upstream of sand filter)	I_a = 67.2 %
B) Tributary Area's Imperviousness Ratio ($i = I_a/100$)	i = 0.672
C) Water Quality Capture Volume (WQCV) Based on 24-hour Drain Time $WQCV = 0.9 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i)$	WQCV = 0.24 watershed inches
D) Contributing Watershed Area (including sand filter area)	Area = 181,125 sq ft
E) Water Quality Capture Volume (WQCV) Design Volume $V_{WQCV} = WQCV / 12 * Area$	V_{WQCV} = 3,570 cu ft =0.08 acres
F) For Watersheds Outside of the Denver Region, Depth of Average Runoff Producing Storm	d_6 = _____ in
G) For Watersheds Outside of the Denver Region, Water Quality Capture Volume (WQCV) Design Volume	$V_{WQCV\ OTHER}$ = _____ cu ft
H) User Input of Water Quality Capture Volume (WQCV) Design Volume (Only if a different WQCV Design Volume is desired)	$V_{WQCV\ USER}$ = _____ cu ft
2. Basin Geometry	
A) WQCV Depth	D_{WQCV} = 0.7 ft
B) Sand Filter Side Slopes (Horizontal distance per unit vertical, 4:1 or flatter preferred). Use "0" if sand filter has vertical walls.	Z = 4.00 ft / ft
C) Mimimum Filter Area (Flat Surface Area)	A_{Min} = 793 sq ft
D) Actual Filter Area	A_{Actual} = 4649 sq ft
E) Volume Provided	V_T = 3570 cu ft
3. Filter Material	Choose One: (●) 18" CDOT Class C Filter Material; ( ) Other (Explain): _____
4. Underdrain System	
A) Are underdrains provided?	Choose One: (●) YES; ( ) NO
B) Underdrain system orifice diameter for 12 hour drain time	
i) Distance From Lowest Elevation of the Storage Volume to the Center of the Orifice	y = 2.1 ft
ii) Volume to Drain in 12 Hours	Vol_{12} = 3,570 cu ft
iii) Orifice Diameter, 3/8" Minimum	D_O = 1.37 in =1-3/8"

Design Procedure Form: Sand Filter (SF)

Designer: DAP
Company: Tait and Associates, Inc.
Date: July 27, 2011
Project: Safeway Distribution Center
Location: Denver, CO

5. Impermeable Geomembrane Liner and Geotextile Separator Fabric A) Is an impermeable liner provided due to proximity of structures or groundwater contamination?	Choose One (●) YES ( ) NO **PROVIDE A 30 MIL (MIN) PVC GEOMEMBRANE PER TABLE SF-4 WITH SEPARATOR FABRIC (PER TABLE SF-3) ABOVE IT. PROVIDE SEPARATOR FABRIC BELOW THE GEOMEMBRANE AS WELL IF SUBGRADE IS ANGULAR OR COULD OTHERWISE PUNCTURE THE GEOMEMBRANE.**
6-7. Inlet / Outlet Works A) Describe the type of energy dissipation at inlet points and means of conveying flows in excess of the WQCV through the outlet	Type L riprap protection. All excess flow will be conveyed to another detention pond via a 18" outlet pipe or emergency overflow.

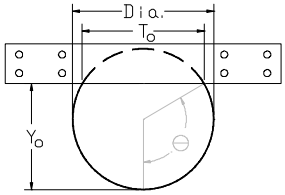
Notes:

RESTRICTOR PLATE SIZING FOR CIRCULAR VERTICAL ORIFICES

Project: **CO1176F Safeway Distribution Center**

Basin ID:

x



		10-yr	100-yr	
		#1 Vertical Orifice	#2 Vertical Orifice	
Sizing the Restrictor Plate for Circular Vertical Orifices or Pipes (Input)				
Water Surface Elevation at Design Depth	Elev: WS =	5,238.35	5,239.34	feet
Pipe/Vertical Orifice Entrance Invert Elevation	Elev: Invert =	5,233.89	5,233.89	feet
Required Peak Flow through Orifice at Design Depth	Q =	0.91	3.04	cfs
Pipe/Vertical Orifice Diameter (inches)	Dia =	6.0	18.0	inches
Orifice Coefficient	C_o =	0.61	0.61	
Full-flow Capacity (Calculated)				
Full-flow area	Af =	0.20	1.77	sq ft
Half Central Angle in Radians	Theta =	3.14	3.14	rad
Full-flow capacity	Qf =	2.0	18.8	cfs
	Percent of Design Flow =	216%	617%	
Calculation of Orifice Flow Condition				
Half Central Angle (0<Theta<3.1416)	Theta =	1.50	0.95	rad
Flow area	A_o =	0.09	0.27	sq ft
Top width of Orifice (inches)	T_o =	5.98	14.67	inches
Height from Invert of Orifice to Bottom of Plate (feet)	Y_o =	0.23	0.32	feet
Elevation of Bottom of Plate	Elev Plate Bottom Edge =	5,234.12	5,234.21	feet
Resultant Peak Flow Through Orifice at Design Depth	Q_o =	0.9	3.1	cfs
Width of Equivalent Rectangular Vertical Orifice	**Equivalent Width =**	**0.39**	**0.84**	**feet**

Trash Rack Sizing

TRASH RACK CALCULATIONS:

Outlet Pipe Size (in)	Area Outlet Pipe (in^2)	A_t (in^2)	A_t (ft^2)	A_t/A_{ot}
18.00	254.47	2102.71	14.60	8.26

Use At (ft^2)= 14.60

T-12 **Outlet Structures**

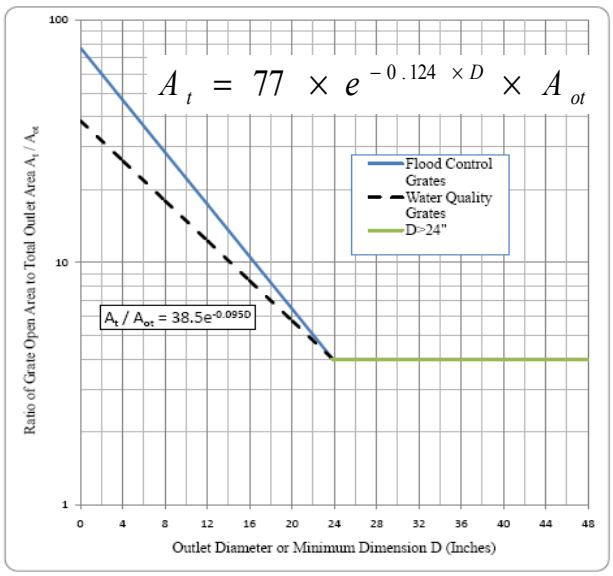
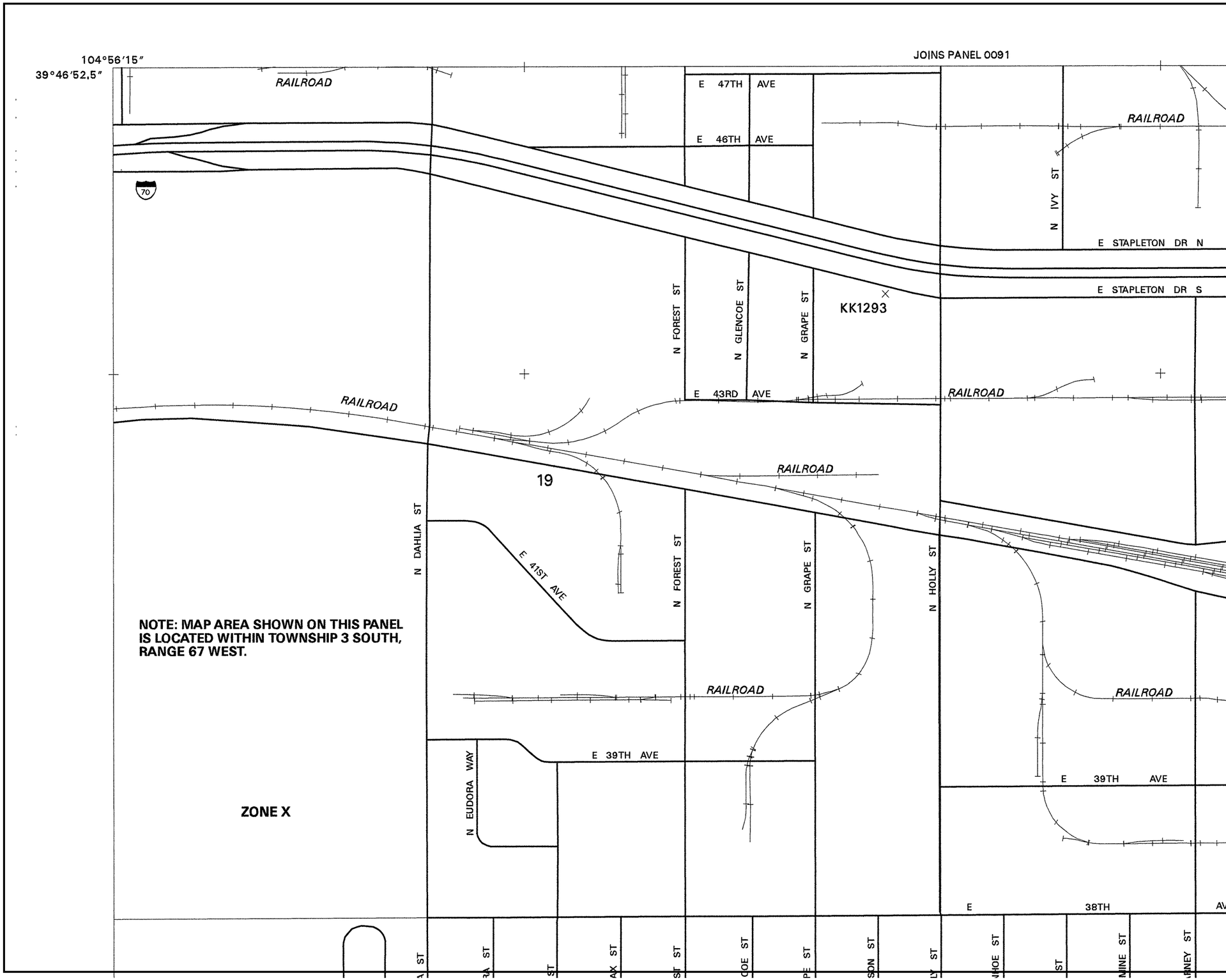


Figure OS-1. Trash Rack Sizing

D. EXCERPTS FROM SUPPORTING DOCUMENTS



NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 3 SOUTH, RANGE 67 WEST.

ZONE X

MAP SCALE 1" = 500'

250 0 500 1000 FEET

METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0093G

FIRM

FLOOD INSURANCE RATE MAP

CITY AND COUNTY OF DENVER, COLORADO

PANEL 93 OF 300

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
DENVER, CITY AND COUNTY OF	080046	0093	G

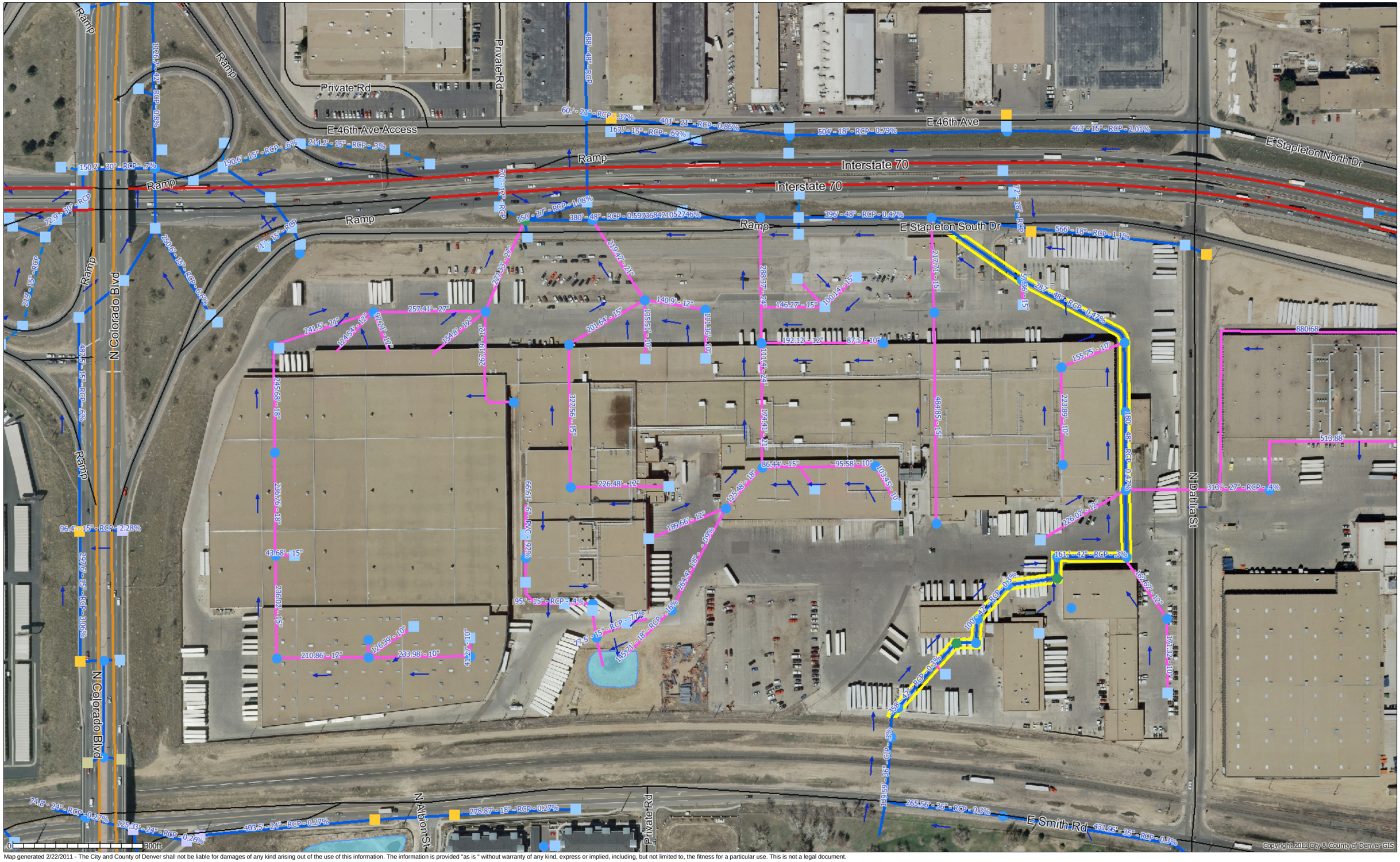
Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER 0800460093G

MAP REVISED: NOVEMBER 17, 2005

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov



Map generated 2/22/2011 - The City and County of Denver shall not be liable for damages of any kind arising out of the use of this information. The information is provided "as is " without warranty of any kind, express or implied, including, but not limited to, the fitness for a particular use. This is not a legal document.

Basin 0060-01 (I-70 & Colorado Boulevard)

Existing System Description:
This basin includes a mix of industrial and residential land uses, and consists of 1,279 acres (1.99 square miles). It is fully built-out with older neighborhood residential in the upper reaches and commercial in the lower reaches. ***Blueprint Denver*** shows the region downstream (northwest) of Vasquez Boulevard as an "Area of Change". The upper reaches of Basin 0060-01 are shown as "Areas of Stability". All drainage from this basin outfalls to the South Platte River.

Drainage Deficiencies:
The existing drainage system has capacity to convey less than the 1-year storm event to the outfall. Rainfall runoff in excess of the storm drain capacity has a history of ponding in the sump and flat areas. Major drainage problems have been experienced along the Smith Road corridor and along Vasquez Boulevard near Sand Creek. Specific drainage deficiencies include:

- The existing Union Pacific Railroad (UPPR) right-of-way creates a major barrier for surface flow and causes broad flooding of the Swansea Park area.
- Improved conveyance (storm sewer) is needed in York Street above Brighton Boulevard and in the industrial zones above Vasquez Boulevard.

Potential Ponding Areas:
An extensive area of potential ponding has been identified within Basins 0060-01 and 4400-02, related primarily to ponding behind railroad tracks, I-70, and other sumps in the lower portions of each basin. A total of 716 structures with a structure value (excluding land value) of $120.6M (per 2003 Denver Assessors' records) lie within the 501-acre zone most severely impacted by the flooding.

Proposed Capital Improvements:
Drainage improvements in these basins should be viewed primarily to achieve the minimum performance objectives for residential and commercial areas (2- and 5-year system capacities) established for the Denver stormwater program, primarily due to the high cost of retrofitting new storm sewer in existing streets. Proposed capital improvements include:

Project F1: Lower Park Hill Outfall A new 5-year storm drain system is needed in Dahlia Street. This proposed project will connect downstream to the planned 84" at Dahlia St and Smith Road (currently in design) and project 4400-02 F2 upstream.

Project G: 50th Avenue Interceptor A new storm drain system is needed in 50th Avenue. This proposed project will connect to the existing storm drain system at 50th Avenue and Colorado Blvd.

Project H: Columbine and York Upgrades A 5-year storm drain system is needed in Brighton Boulevard from York Street, extending south in Brighton Boulevard, York Street and Columbine Street.

Project I: Riverside Outfall A 5-year 78-inch outfall storm sewer is proposed within York Street from Brighton Boulevard to outfall to the South Platte River. This proposed project would provide an outfall system for 0060-01-H upstream.

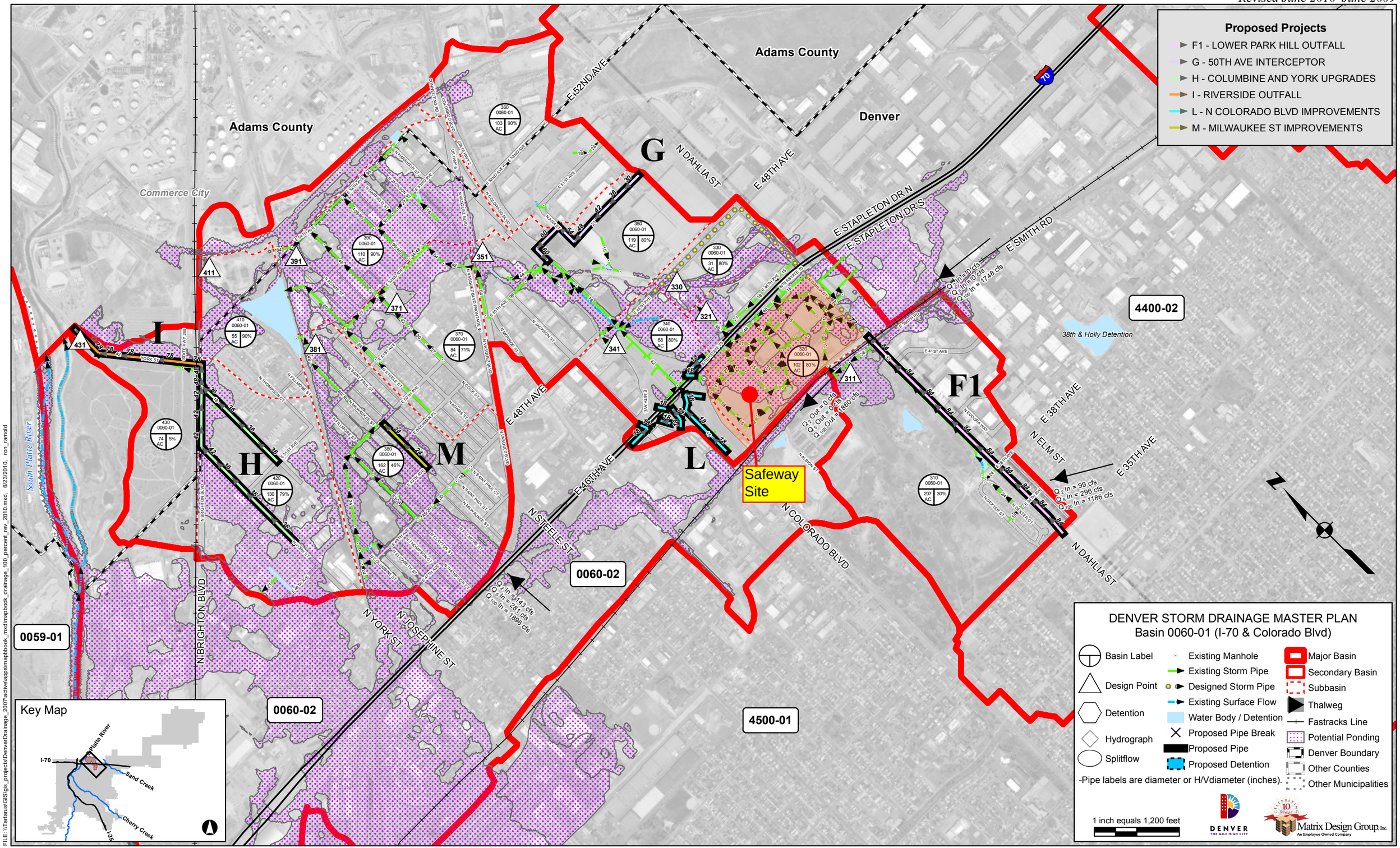
Project L: N Colorado Blvd Improvements This project includes the addition of storm sewers in local roads to improve the interception of the minor storm events.

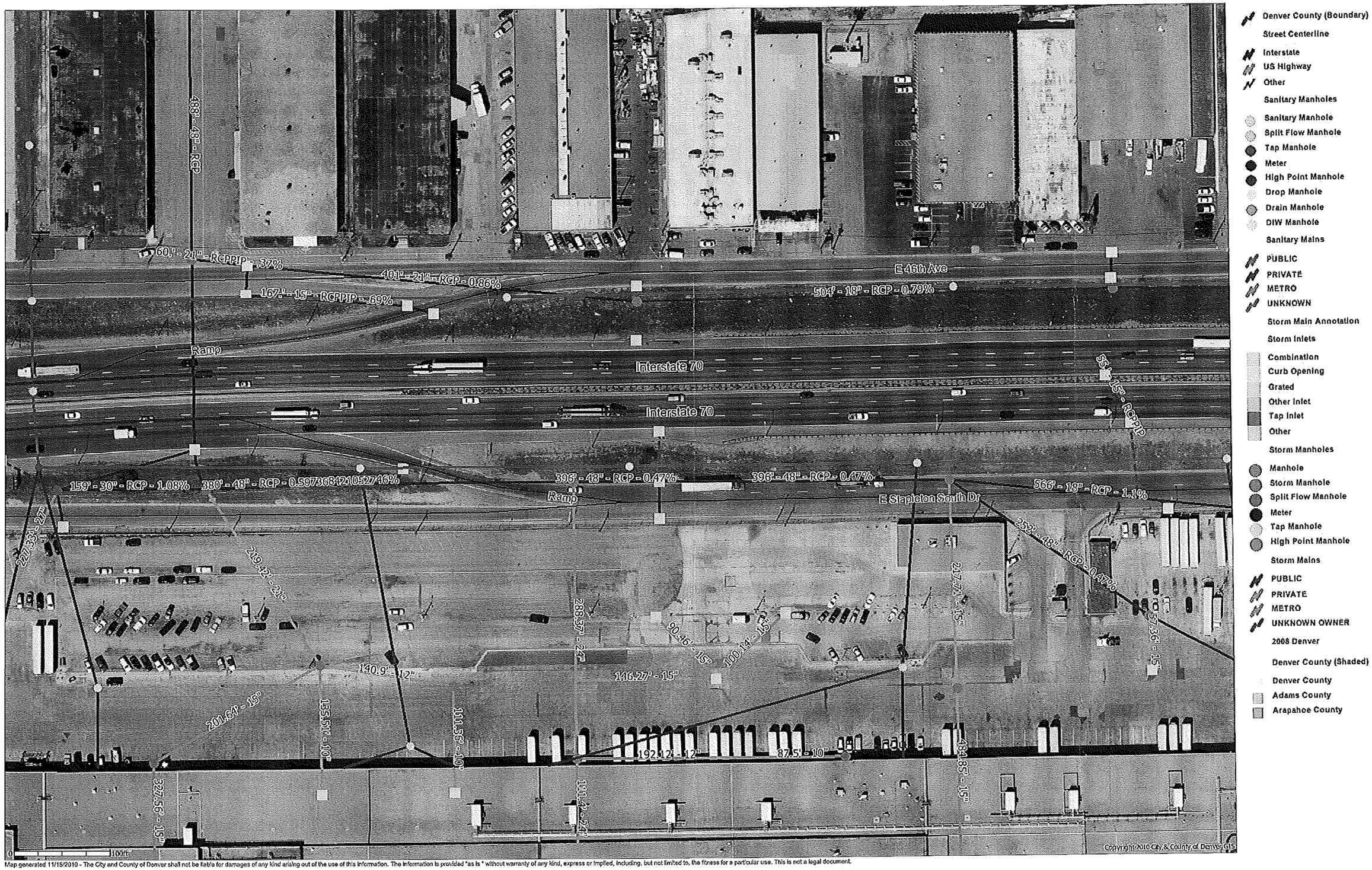
Project M: Milwaukee St Improvements A new 24-inch storm drain is needed in Milwaukee St from 50th Avenue to 49th Avenue.

Existing Hydrology:

Design Point / Flow Element	Contributing Basins and Split Flow Routing Elements	Basin Parameters	Discharge		
		Area	Q2-yr	Q5-yr	Q100-yr
		(acres)	(cfs)	(cfs)	(cfs)
310	310	207	59	140	450
311	310, 1615*, 1627*	207	98	308	2581
320	320	102	149	224	490
321	320, 1635*	102	149	224	490
330	330	31	38	57	125
331	330	31	38	57	125
340	340	68	80	120	266
341	340, 1330*, 1331*	408	275	466	1182
350	350	120	169	253	554
351	350, 1340*	527	329	515	1174
360	360	103	170	248	509
370	370	84	89	142	328
371	370	84	89	142	328
380	380	162	80	153	427
381	380, 1651*	246	145	304	1861
390	390	110	173	252	519
391	390, 1350*	883	430	620	1416
410	410	55	79	115	238
411	1390	938	714	1170	3298
420	420	130	168	254	559
421	420	130	168	254	559
430	430	74	4	41	173
431	420,430	204	110	210	609
441	500, 510	split	93	176	445

*See map in Technical Appendix for locations of all split flow routing elements.


Proposed Projects
F1 - LOWER PARK HILL OUTFALL
G - 50TH AVE INTERCEPTOR
H - COLUMBINE AND YORK UPGRADES
I - RIVERSIDE OUTFALL
L - N COLORADO BLVD IMPROVEMENTS
M - MILWAUKEE ST IMPROVEMENTS
Adams County
Denver
Commerce City
South Platte River
38th & Holly Detention
Safeway Site
4400-02
0060-02
0059-01
4500-01
DENVER STORM DRAINAGE MASTER PLAN
Basin 0060-01 (I-70 & Colorado Blvd)
Basin Label
Design Point
Detention
Hydrograph
Splitflow
Existing Manhole
Existing Storm Pipe
Designed Storm Pipe
Existing Surface Flow
Water Body / Detention
Proposed Pipe Break
Proposed Pipe
Proposed Detention
Major Basin
Secondary Basin
Subbasin
Thalweg
Fastracks Line
Potential Ponding
Denver Boundary
Other Counties
Other Municipalities
-Pipe labels are diameter or H/Vdiameter (inches).
1 inch equals 1,200 feet
Key Map
I-70
Platte River
Sand Creek
Cherry Creek
I-25
FILE: \\Tartarus\GIS\gis_projects\DenverDrainage_2007\active\apps\mapbook_mxd\mapbook_drainage_100_percent_rev_2010.mxd, 6/23/2010, ron_ramold



Denver County (Boundary)

Street Centerline

Interstate

US Highway

Other

Sanitary Manholes

Sanitary Manhole

Split Flow Manhole

Tap Manhole

Meter

High Point Manhole

Drop Manhole

Drain Manhole

DIW Manhole

Sanitary Mains

PUBLIC

PRIVATE

METRO

UNKNOWN

Storm Main Annotation

Storm Inlets

Combination

Curb Opening

Grated

Other Inlet

Tap Inlet

Other

Storm Manholes

Manhole

Storm Manhole

Split Flow Manhole

Meter

Tap Manhole

High Point Manhole

Storm Mains

PUBLIC

PRIVATE

METRO

UNKNOWN OWNER

2008 Denver

Denver County (Shaded)

Denver County

Adams County

Arapahoe County

Map generated 11/15/2010 - The City and County of Denver shall not be liable for damages of any kind arising out of the use of this information. The information is provided "as is" without warranty of any kind, express or implied, including, but not limited to, the fitness for a particular use. This is not a legal document.

CITY AND COUNTY OF DENVER

STORM DRAINAGE DESIGN & TECHNICAL CRITERIA

DEPARTMENT OF PUBLIC WORKS
WASTEWATER MANAGEMENT DIVISION
ENGINEERING DIVISION

REVISED JANUARY 2006

5.0 RAINFALL

5.1 Introduction

The design rainfall data to be used to complete hydrologic analyses described in the RUNOFF chapter of these DENVER CRITERIA are presented in this section. More specifically, this chapter provides: 1) point precipitation values for Denver, 2) information on the Colorado Urban Hydrograph Procedure (CUHP), and 3) an intensity-duration-frequency table for use with the Rational Method. All hydrological analyses within Denver shall use the rainfall data presented herein for calculating storm runoff. There may be cases where the designer needs to consider events more extreme than the 100-year storm (e.g., for public safety).

The design storms and intensity-frequency-duration tables for Denver were developed using the rainfall data and procedures presented in the DISTRICT MANUAL and are presented herein for convenience.

5.2 Rainfall Depth-Duration-Frequency Values

A review of the isopluvial maps presented in the *Precipitation-Frequency Atlas of the Western United States, Volume III-Colorado* (National Oceanic and Atmospheric Administration [NOAA] Atlas) shows that all of Denver can be included in one rainfall zone. The precipitation values for various return periods and duration storms were found to have minimal variation.

The 1-hour point rainfall is necessary for use with both the Rational Method and CUHP and is also the basis for deriving durations less than one hour. For watersheds greater than 10 square miles, the 3-hour rainfall depth is required, and for watersheds 20 square miles and larger, the 6-hour rainfall depth is required for use with CUHP. One-hour point rainfall values are summarized in Table 5.1. To obtain durations less than 1 hour, the factors in Table 5.2 are applied to the 1-hour point rainfall.

Table 5.1. One-hour Point Rainfall Depths

Return Period	One-hour Point Rainfall (inches)
2-Year	0.95
5-Year	1.34
10-Year	1.55
50-Year	2.25
100-Year	2.57
Date: July, 1992 Revised:	Reference: Wastewater Management Division, 1987, as determined based on NOAA Atlas 2, Volume IIII.

Table 5.3. Rainfall Intensity Duration Values for Use with the Rational Method

Time	Rainfall Intensity in Inches per Hour					Time	Rainfall Intensity in Inches per Hour				
Min.	2-yr	5-yr	10-yr	50-yr	100-yr	Min.	2-yr	5-yr	10-yr	50-yr	100-yr
5	3.22	4.55	5.26	7.63	8.72	35	1.36	1.92	2.22	3.22	3.68
10	2.57	3.63	4.19	6.09	6.95	36	1.34	1.88	2.18	3.16	3.61
11	2.47	3.49	4.04	5.86	6.69	37	1.31	1.85	2.14	3.11	3.55
12	2.38	3.36	3.89	5.65	6.45	38	1.29	1.82	2.11	3.06	3.49
13	2.30	3.25	3.76	5.45	6.23	39	1.27	1.79	2.07	3.01	3.44
14	2.23	3.14	3.63	5.27	6.02	40	1.25	1.76	2.04	2.96	3.38
15	2.16	3.04	3.52	5.11	5.83	41	1.23	1.74	2.01	2.92	3.33
16	2.09	2.95	3.41	4.95	5.66	42	1.21	1.71	1.98	2.87	3.28
17	2.03	2.86	3.31	4.81	5.49	43	1.19	1.69	1.95	2.83	3.23
18	1.97	2.78	3.22	4.67	5.34	44	1.18	1.66	1.92	2.79	3.19
19	1.92	2.71	3.13	4.55	5.19	45	1.16	1.64	1.89	2.75	3.14
20	1.87	2.64	3.05	4.43	5.06	46	1.14	1.61	1.87	2.71	3.10
21	1.82	2.57	2.97	4.31	4.93	47	1.13	1.59	1.84	2.67	3.05
22	1.78	2.51	2.90	4.21	4.81	48	1.11	1.57	1.82	2.64	3.01
23	1.73	2.45	2.83	4.11	4.69	49	1.10	1.55	1.79	2.60	2.97
24	1.69	2.39	2.76	4.01	4.58	50	1.08	1.53	1.77	2.57	2.93
25	1.66	2.34	2.70	3.92	4.48	51	1.07	1.51	1.75	2.53	2.89
26	1.62	2.28	2.64	3.84	4.38	52	1.06	1.49	1.72	2.50	2.86
27	1.58	2.24	2.59	3.75	4.29	53	1.04	1.47	1.70	2.47	2.82
28	1.55	2.19	2.53	3.68	4.20	54	1.03	1.45	1.68	2.44	2.79
29	1.52	2.14	2.48	3.60	4.11	55	1.02	1.44	1.66	2.41	2.75
30	1.49	2.10	2.43	3.53	4.03	56	1.01	1.42	1.64	2.38	2.72
31	1.46	2.06	2.39	3.46	3.95	57	0.99	1.40	1.62	2.35	2.69
32	1.43	2.02	2.34	3.40	3.88	58	0.98	1.39	1.60	2.33	2.66
33	1.41	1.99	2.30	3.34	3.81	59	0.97	1.37	1.58	2.30	2.63
34	1.38	1.95	2.26	3.28	3.74	60	0.96	1.35	1.57	2.27	2.60

Date: Jan 2006 Calculated based on Equation 5.1.
Revised:

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(Volume 1)

June 2001

Revised April 2008

Urban Drainage and Flood Control District

L = length of overland flow (500 ft maximum for non-urban land uses, 300 ft maximum for urban land uses)

S = average basin slope (ft/ft)

Equation RO-3 is adequate for distances up to 500 feet. Note that, in some urban watersheds, the overland flow time may be very small because flows quickly channelize.

2.4.2 Overland Travel Time

For catchments with overland and channelized flow, the time of concentration needs to be considered in combination with the overland travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the overland travel time, t_t, can be estimated with the help of Figure RO-1 or the following equation (Guo 1999):

$$V = C_v S_w^{0.5} \qquad \text{(RO-4)}$$

in which:

V = velocity (ft/sec)

C_v = conveyance coefficient (from Table RO-2)

S_w = watercourse slope (ft/ft)

Table RO-2—Conveyance Coefficient, C_v

Type of Land Surface	Conveyance Coefficient, C_v
Heavy meadow	2.5
Tillage/field	5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

The time of concentration, t_c, is then the sum of the initial flow time, t_i, and the travel time, t_t, as per Equation RO-2.

2.4.3 First Design Point Time of Concentration in Urban Catchments

Using this procedure, the time of concentration at the first design point (i.e., initial flow time, t_i) in an urbanized catchment should not exceed the time of concentration calculated using Equation RO-5.

$$t_c = \frac{L}{180} + 10 \qquad \text{(RO-5)}$$

in which:

t_c = maximum time of concentration at the first design point in an urban watershed (minutes)

Table RO-3—Recommended Percentage Imperviousness Values

Land Use or Surface Characteristics	Percentage Imperviousness
Business:	
Commercial areas	95
Neighborhood areas	85
Residential:	
Single-family	*
Multi-unit (detached)	60
Multi-unit (attached)	75
Half-acre lot or larger	*
Apartments	80
Industrial:	
Light areas	80
Heavy areas	90
Parks, cemeteries	5
Playgrounds	10
Schools	50
Railroad yard areas	15
Undeveloped Areas:	
Historic flow analysis	2
Greenbelts, agricultural	2
Off-site flow analysis (when land use not defined)	45
Streets:	
Paved	100
Gravel (packed)	40
Drive and walks	90
Roofs	90
Lawns, sandy soil	0
Lawns, clayey soil	0

* See Figures RO-3 through RO-5 for percentage imperviousness.

$$C_A = K_A + \left(1.31i^3 - 1.44i^2 + 1.135i - 0.12\right) \text{ for } C_A \geq 0\text{, otherwise } C_A = 0 \qquad \text{(RO-6)}$$

$$C_{CD} = K_{CD} + \left(0.858i^3 - 0.786i^2 + 0.774i + 0.04\right) \qquad \text{(RO-7)}$$

$$C_B = \left(C_A + C_{CD}\right)/2$$

Table RO-5— Runoff Coefficients, *C*

Percentage Imperviousness	Type C and D NRCS Hydrologic Soil Groups					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.04	0.15	0.25	0.37	0.44	0.50
5%	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.11	0.21	0.30	0.41	0.47	0.53
15%	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.17	0.26	0.34	0.44	0.50	0.55
25%	0.20	0.28	0.36	0.46	0.51	0.56
30%	0.22	0.30	0.38	0.47	0.52	0.57
35%	0.25	0.33	0.40	0.48	0.53	0.57
40%	0.28	0.35	0.42	0.50	0.54	0.58
45%	0.31	0.37	0.44	0.51	0.55	0.59
50%	0.34	0.40	0.46	0.53	0.57	0.60
55%	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.41	0.46	0.51	0.57	0.60	0.63
65%	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.60	0.63	0.66	0.70	0.72	0.74
85%	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.73	0.75	0.77	0.80	0.82	0.83
95%	0.80	0.82	0.84	0.87	0.88	0.89
100%	0.89	0.90	0.92	0.94	0.95	0.96
	TYPE B NRCS HYDROLOGIC SOILS GROUP					
0%	0.02	0.08	0.15	0.25	0.30	0.35
5%	0.04	0.10	0.19	0.28	0.33	0.38
10%	0.06	0.14	0.22	0.31	0.36	0.40
15%	0.08	0.17	0.25	0.33	0.38	0.42
20%	0.12	0.20	0.27	0.35	0.40	0.44
25%	0.15	0.22	0.30	0.37	0.41	0.46
30%	0.18	0.25	0.32	0.39	0.43	0.47
35%	0.20	0.27	0.34	0.41	0.44	0.48
40%	0.23	0.30	0.36	0.42	0.46	0.50
45%	0.26	0.32	0.38	0.44	0.48	0.51
50%	0.29	0.35	0.40	0.46	0.49	0.52
55%	0.33	0.38	0.43	0.48	0.51	0.54
60%	0.37	0.41	0.46	0.51	0.54	0.56
65%	0.41	0.45	0.49	0.54	0.57	0.59
70%	0.45	0.49	0.53	0.58	0.60	0.62
75%	0.51	0.54	0.58	0.62	0.64	0.66
80%	0.57	0.59	0.63	0.66	0.68	0.70
85%	0.63	0.66	0.69	0.72	0.73	0.75
90%	0.71	0.73	0.75	0.78	0.80	0.81
95%	0.79	0.81	0.83	0.85	0.87	0.88
100%	0.89	0.90	0.92	0.94	0.95	0.96

TABLE RO-5 (Continued)—Runoff Coefficients, *C*

Percentage Imperviousness	Type A NRCS Hydrologic Soils Group					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.00	0.00	0.05	0.12	0.16	0.20
5%	0.00	0.02	0.10	0.16	0.20	0.24
10%	0.00	0.06	0.14	0.20	0.24	0.28
15%	0.02	0.10	0.17	0.23	0.27	0.30
20%	0.06	0.13	0.20	0.26	0.30	0.33
25%	0.09	0.16	0.23	0.29	0.32	0.35
30%	0.13	0.19	0.25	0.31	0.34	0.37
35%	0.16	0.22	0.28	0.33	0.36	0.39
40%	0.19	0.25	0.30	0.35	0.38	0.41
45%	0.22	0.27	0.33	0.37	0.40	0.43
50%	0.25	0.30	0.35	0.40	0.42	0.45
55%	0.29	0.33	0.38	0.42	0.45	0.47
60%	0.33	0.37	0.41	0.45	0.47	0.50
65%	0.37	0.41	0.45	0.49	0.51	0.53
70%	0.42	0.45	0.49	0.53	0.54	0.56
75%	0.47	0.50	0.54	0.57	0.59	0.61
80%	0.54	0.56	0.60	0.63	0.64	0.66
85%	0.61	0.63	0.66	0.69	0.70	0.72
90%	0.69	0.71	0.73	0.76	0.77	0.79
95%	0.78	0.80	0.82	0.84	0.85	0.86
100%	0.89	0.90	0.92	0.94	0.95	0.96

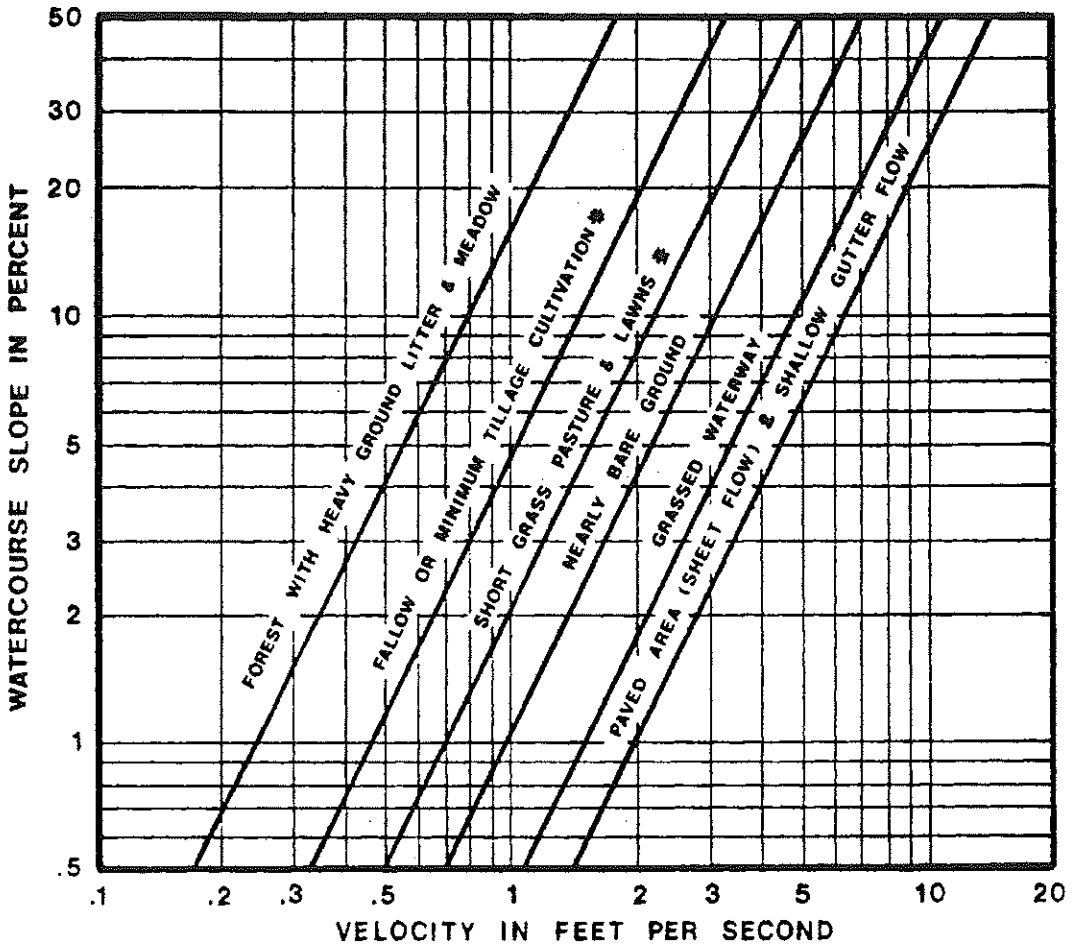

Figure RO-1—Estimate of Average Overland Flow Velocity for Use With the Rational Formula

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June 2001

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Urban Drainage and Flood Control District

the calculated 100-year volume. Others require that the 100-year volume be added to the WQCV. All jurisdictions require the WQCV be added to the 5- or 10-year volume. When clear written local criteria on this matter are absent, the District recommends that no less than 50% of the WQCV be added to the calculated 100-year volume for 100-year volumes obtained using empirical equations and the FAA Method. However, unless the local jurisdiction requires adding all or part of the WQCV to the 100-year volume obtained using the simplified *Full Spectrum Detention* design; District does not recommend adding any part of the WQCV to the 100-year volume. When the analysis is done using hydrograph routing methods, each level of controls needs to be accounted for and the resultant 100-year control volume used in final design.

3.2 Sizing of On-Site Detention Facilities

3.2.1 Maximum Allowable Unit Release Rates for On-Site Facilities

The maximum allowable unit release rates in the Denver area per acre of tributary catchment for on-site detention facilities for various design return periods are listed in Table SO-1. These maximum releases rates will apply for all on-site detention facilities unless other rates are recommended in a District-approved master plan. For regional facilities see Section 3.2.5.

Allowable unit release rates in Table SO-1 for each a soil group in the tributary catchment shall be area-weighted to composite the allowable unit release rate for the total catchment. Multiply this rate by the total tributary catchment's area to obtain the design release rates in cubic feet per second (cfs). Whenever Natural Resources Conservation Service (NRCS) soil surveys are not available, approximate their equivalent types using results of detailed soil investigations at the site.

Table SO-1—Maximum Unit Flow Release Rates (cfs/acre) from On-Site Detention Facilities

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

3.2.2 Empirical Equations for the Sizing of On-Site Detention Storage Volumes

Urbonas and Glidden (1983), as part of the District's ongoing hydrologic research, conducted studies that evaluated peak storm runoff flows along major drainageways. The following set of empirical equations provided preliminary estimates of on-site detention facility sizing for areas within the District. They are

intended for single return period control and not for use when off-site inflows are present or when multi-stage controls are to be used (e.g., 10- and 100-year peak control). In addition, these equations are not intended to replace detailed hydrologic and flood routing analysis, or even the analysis using the Rational Formula-based FAA method for the sizing of detention storage volumes. The District does not promote the use of these empirical equations. It does not object, however, to their use by local governments who have adopted them or want to adopt them as minimum requirements for the sizing of on-site detention for small catchments within their jurisdiction. If the District has a master plan that contains specific guidance for detention storage or sizing of on-site detention facilities, those guidelines should be followed instead. The empirical equations for NRCS Soil types B, C and D are as follows:

$$V_i = K_i A \qquad \text{(SO-1)}$$

for the 100-year:

$$K_{100} = \frac{(1.78\,I - 0.002\,I^2 - 3.56)}{900} \qquad \text{(SO-2)}$$

for the 10-year:

$$K_{10} = \frac{(0.95\,I - 1.90)}{1{,}000} \qquad \text{(SO-3)}$$

for the 5-year:

$$K_5 = \frac{(0.77I - 2.65)}{1{,}000} \qquad \text{(SO-4)}$$

For Soil Type A, Equations SO-1 and SO-2 tend to underestimate the needed 100-year detention volume. Instead, Equation SO-5 needs to be used to estimate the 100-year detention volume for Type A Soils (i.e., V_{100A}):

$$V_{100A} = \left(-0.00005501 \cdot I^2 + 0.030148 \cdot I - 0.12\right) \cdot \frac{A}{12} \qquad \text{(SO-5)}$$

in which:

V_i = required volume where subscript *i* = 100-, 10- or 5-year storm, as appropriate (acre-feet)

K_i = empirical volume coefficient where subscript *i* = 100-, 10- or 5-year storm, as appropriate

I = fully developed tributary catchment imperviousness (%)

A = tributary catchment area (acres)

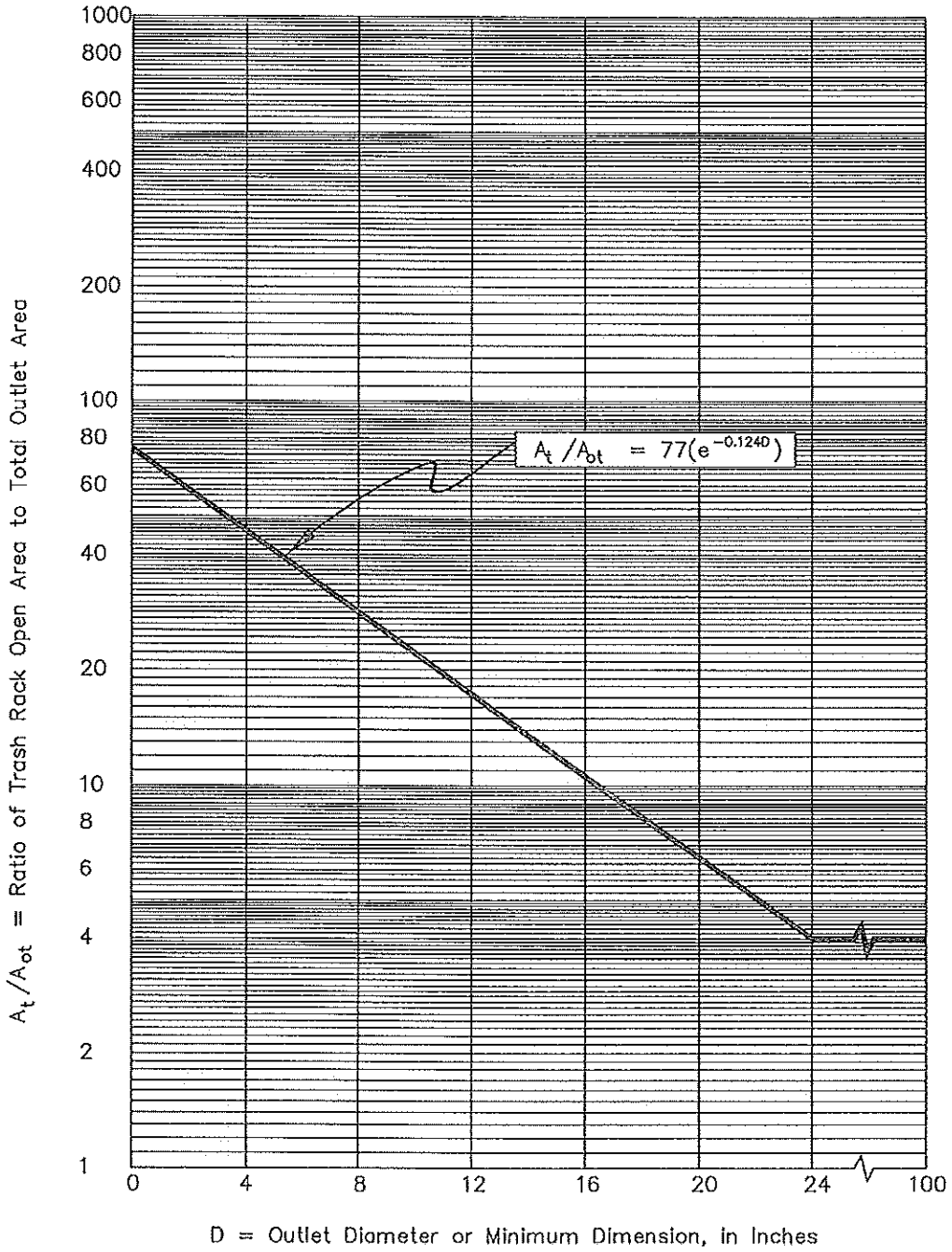

Figure SO-7—Minimum Trash Rack Open Area—Extended Range

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Updated September 1, 1999

(Latest Revision – April 2008)

Urban Drainage and Flood Control District
2480 W. 26th Avenue, Suite 156B
Denver, CO 80211, USA
www.udfcd.org

Description

Photograph SF-1. The vegetation on the surface of the filter and the shallow design depth of this sand filter, help the BMP blend into its surroundings. Photo courtesy of Fred Bromberger.

A sand filter is a filtering or infiltrating BMP that consists of a surcharge zone underlain by a sand bed with an underdrain system (when necessary). During a storm, accumulated runoff collects in the surcharge zone and gradually infiltrates into the underlying sand bed, filling the void spaces of the sand. The underdrain gradually dewaters the sand bed and discharges the runoff to a nearby channel, swale, or storm sewer. It is similar to a BMP designed for bioretention in that it utilizes filtering, but differs in that it is not specifically designed for vegetative growth. For this reason, it can have a greater depth and be designed for a larger contributing area. A sand filter is also similar to an extended detention basin (EDB) in that it is a dry basin, which can be easily designed to include the flood control volume above the WQCV or EURV. However, a sand filter does not require a forebay or micropool because the solids that would be deposited in these components in an EDB will be retained on the surface of the sand bed in a sand filter. Sand filters can be vegetated with species that will tolerate both wet and dry conditions and occasional inundation. The rain garden growing media is recommended for sand filters where vegetation is desired. Sand filters can also be placed in a vault. Underground sand filters have additional requirements. See Fact Sheet T-11 for additional discussion on underground BMPs.

Sand/Media Filter	
Functions	
LID/Volume Red.	Yes
WQCV Capture	Yes
WQCV+Flood Control	Yes
Fact Sheet Includes EURV Guidance	No
Typical Effectiveness for Targeted Pollutants[3]	
Sediment/Solids	Very Good[1]
Nutrients	Good
Total Metals	Good
Bacteria	Moderate
Other Considerations	
Life-cycle Costs[4]	Moderate

[1] Not recommended for watersheds with high sediment yields (unless pretreatment is provided).

[3] Based primarily on data from the International Stormwater BMP Database (www.bmpdatabase.org).

[4] Based primarily on BMP-REALCOST available at www.udfcd.org. Analysis based on a single installation (not based on the maximum recommended watershed tributary to each BMP).

Site Selection

Sand filters require a stable watershed. When the watershed includes phased construction, sparsely vegetated areas, or steep slopes in sandy soils, consider another BMP or provide pretreatment before runoff from these areas reach the rain garden.

When sand filters (and other BMPs used for infiltration) are located adjacent to buildings or pavement areas, protective measures should be implemented to avoid adverse impacts to these structures. Oversaturated subgrade soil underlying a structure can cause the structure to settle or result in moisture-related problems. Wetting of expansive soils or bedrock can cause swelling, resulting in structural movements. A geotechnical engineer should evaluate the potential impact of the BMP on

adjacent structures based on an evaluation of the subgrade soil, groundwater, and bedrock conditions at the site. Additional minimum requirements include:

- In locations where subgrade soils do not allow infiltration, the filter layer should be underlain by an underdrain system.
- Where infiltration can adversely impact adjacent structures, the filter layer should be underlain by an underdrain system designed to divert water away from the structure.
- In locations where potentially expansive soils or bedrock exist, placement of a sand filter adjacent to structures and pavement should only be considered if the BMP includes an underdrain designed to divert water away from the structure, and is lined with an impermeable geomembrane liner designed to restrict seepage.

Benefits

- Filtering BMPs provide effective water quality enhancement including phosphorus removal.

Limitations

- This BMP may clog and require maintenance if a moderate to high level of silts and clays are allowed to flow into the facility.
- This BMP should not be located within 10 feet of a building foundation without an impermeable membrane. See *Bioretention* (BMP Fact Sheet T-3) of this manual for additional information.
- The sand filter should not be put into operation while construction or major landscaping activities are taking place in the watershed.

Designing for Maintenance

Recommended maintenance practices for all BMPs are provided in Chapter 6 of this manual. During design the following should be considered to ensure ease of maintenance over the long-term:

- Do not put a filter sock on the underdrain. This is not necessary and can cause the BMP to clog.
- Install cleanouts. Cleanouts can be used for inspection (by camera) immediately following construction to ensure that the underdrain pipe was not crushed during construction. They can also be used to for ongoing maintenance practices. Consider locating cleanouts in the side slopes of the basin and above the depth of ponding.
- Provide vegetated side slopes to pre-treat runoff by filtering (straining). This will reduce the frequency of maintenance.

Design Procedure and Criteria

The following steps outline the design procedure and criteria for a sand filter.

1. **Basin Storage Volume**: Provide a storage volume above the sand bed of the basin equal to the WQCV based on a 24-hour drain time. Although the BMP will be designed to drain in 12 hours, sizing the basin for a longer drain time will ensure containment of the WQCV as infiltration through the filter layer slows over time.

- Determine the imperviousness of the tributary area (or effective imperviousness where LID techniques are implemented). Determine the required WQCV (watershed inches of runoff) using Figure 3-2 in Chapter 3 of this manual. The volume should be based on a drain time of 24 hours.

- Calculate the design volume as follows:

$$V = \left[\frac{\text{WQCV}}{12}\right] A \qquad \text{Equation SF-1}$$

Where:

V = design volume (ft^3)

A = watershed area tributary to the sand filter (ft^2)

2. **Basin Geometry:** Use equation SF-2 to calculate the minimum filter area, which is the flat surface of the sand filter. Sediment will reside on the filter area of the sand filter. Therefore, if the filter area is too small, the filter may clog prematurely. If this is of particular concern, increasing the filter area will decrease the frequency of maintenance. The following equation provides the minimum filter area allowing for some of the volume to be stored beyond the area of the filter. **Note that the total volume must also equal or exceed the design volume.**

   The side slopes of the basin should be stable and maintainable. For vegetated side slopes, a 4:1 (horizontal: vertical) minimum slope is recommended. Use vertical walls where side slopes are steeper than 3:1

$$A = \frac{2\text{V}}{9} \qquad \text{Equation SF-2}$$

Where:

A = minimum filter area (flat surface area) (ft^2)

V = design volume (ft^3)

3. **Filter Material**: Provide, at a minimum, an 18-inch layer of CDOT Class C filter material (see Table SF-1). Maintain a flat surface on the top of the sand bed.

Table SF-1. Gradation Specifications for CDOT Class C Filter Material
(Source: CDOT Table 703-7)

Sieve Size	Mass Percent Passing Square Mesh Sieves
19.0 mm (3/4")	100
4.75 mm (No. 4)	60 – 100
300 µm (No. 50)	10 – 30
150 µm (No. 100)	0 – 10
75 µm (No. 200)	0 - 3

4. **Underdrain System**: Underdrains are typically required for sand filters and should be provided if infiltration tests show rates slower than 2 times that required to drain the WQCV over 12 hours, or where required to divert water away from structures as determined by a professional engineer. Percolation tests should be performed or supervised by a licensed professional engineer and conducted at a minimum depth equal to the bottom of the sand filter. Additionally, underdrains are required where impermeable membranes are used. There are three basic types of sand filters:

   - **No-Infiltration:** This section includes an underdrain and an impermeable liner that does not allow for infiltration of stormwater into the subgrade soils. It is appropriate to use a no-infiltration system when any of the following is true:
     - Land use or activities could contaminate groundwater when stormwater is allowed to infiltrate, or
     - The BMP is located over potentially expansive soils or bedrock or is adjacent (within 10 feet) to structures.
   - **Partial Infiltration:** This section does not include an impermeable liner, allowing for some infiltration. Stormwater that does not infiltrate will be collected and removed by an underdrain system.
   - **Full Infiltration:** This section is designed to infiltrate all of the water stored into the subgrade below. UDFCD recommends a minimum infiltration rate of 2 times the rate needed to drain the WQCV over 12 hours.

   When using an underdrain system, provide a control orifice sized to drain the design volume in approximately 12 hours or more (see Equation SF-3). Use a minimum orifice size of 3/8 inch to avoid clogging. This will provide detention and slow release of the WQCV to offset hydromodification. Provide cleanouts to allow inspection of the drainpipe system during and after construction to ensure that the pipe was not crushed or disconnected during construction and to allow for maintenance of the underdrain. Space underdrain pipes a maximum of 20 feet on-center.

$$D_{12\text{ hour drain time}} = \sqrt{\frac{V}{1414\, y^{0.41}}} \qquad \text{Equation SF-3}$$

   Where:

   D = orifice diameter (in)

   y = distance from the lowest elevation of the storage volume (ft) (i.e., surface of the filter) to the center of the orifice

   V = volume to drain in 12 hours (WQCV) (ft^3)

   In previous versions of this manual, UDFCD recommended that the underdrain be placed in an aggregate layer and that a geotextile (separator fabric) be placed between this aggregate and the growing medium. This version of the manual replaces that section with materials that, when used together, eliminate the need for a separator fabric.

The underdrain system should be placed below the 18-inch (minimum) filter layer. The underdrain system should be placed within an 5-inch-thick section of CDOT Class C filter material meeting the gradation in Table SF-1. Areas of the underdrain layer may be deeper due to the slope of the underdrain. If no underdrain is required, the minimum section can be reduced to the 18-inch filter layer. Use slotted pipe that meets the slot dimensions provided in Table SF-2.

Table SF-2. Dimensions for Slotted Pipe

Pipe Size	Slot Length[1]	Maximum Slot Width	Slot Centers[1]	Open Area[1] (per foot)
4"	1-1/16"	0.032"	0.413"	1.90 in^2
6"	1-3/8"	0.032"	0.516"	1.98 in^2

[1] Some variation in these values is acceptable and is expected from various pipe manufacturers. Be aware that both increased slot length and decreased slot centers will be beneficial to hydraulics but detrimental to the structure of the pipe.

Table SF-3. Physical Requirements for Separator Fabric[1]

Property	Class B		Test Method
	Elongation < 50%[2]	Elongation > 50%[2]	
Grab Strength, N (lbs)	800 (180)	510 (115)	ASTM D 4632
Puncture Resistance, N (lbs)	310 (70)	180 (40)	ASTM D 4833
Trapezoidal Tear Strength, N (lbs)	310 (70)	180 (40)	ASTM D 4533
Apparent Opening Size, mm (US Sieve Size)	AOS < 0.3mm (US Sieve Size No. 50)		ASTM D 4751
Permittivity, sec^{-1}	0.02 default value, must also be greater than that of soil		ASTM D 4491
Permeability, cm/sec	k fabric > k soil for all classes		ASTM D 4491
Ultraviolet Degradation at 500 hours	50% strength retained for all classes		ASTM D 4355

[1] Strength values are in the weaker principle direction
[2] As measured in accordance with ASTM D 4632

5. **Impermeable Geomembrane Liner and Geotextile Separator Fabric:** For no-infiltration sections, install a minimum 30-mil thick PVC geomembrane liner, per Table SF-4, on the bottom and sides of the basin, extending up at least to the top of the underdrain layer. Provide at least 9 inches (12 inches if possible) of cover over the membrane where it is attached to the wall to protect the membrane from UV deterioration. The geomembrane should be field-seamed using a dual track welder, which allows for non-destructive testing of almost all field seams. A small amount of single track and/or adhesive seaming should be allowed in limited areas to seam around pipe perforations, to patch seams removed for destructive seam testing, and for limited repairs. The liner should be installed with slack to prevent tearing due to backfill, compaction, and settling. Place CDOT Class B geotextile separator fabric above the geomembrane to protect it from being punctured during the placement of the filter material above the liner. If the subgrade contains angular rocks or other material that could puncture the geomembrane, smooth-roll the surface to create a suitable surface. If smooth-rolling the surface does not provide a suitable surface, also place the separator fabric between the geomembrane and the underlying subgrade. This should only be done when necessary because fabric placed under the geomembrane can increase seepage losses through pinholes or other geomembrane defects. Connect the geomembrane to perimeter concrete walls around the basin perimeter, creating a watertight seal between the geomembrane and the walls using a continuous batten bar and anchor connection (see Figure SF-3). Where the need for the impermeable membrane is not as critical, the membrane can be attached with a nitrile-based vinyl adhesive. Use watertight PVC boots for underdrain pipe penetrations through the liner (see Figure SF-2).

Table SF-4. Physical Requirements for Geomembrane

Property	Thickness 0.76 mm (30 mil)	Test Method
Thickness, % Tolerance	±5	ASTM D 1593
Tensile Strength, kN/m (lbs/in) width	12.25 (70)	ASTM D 882, Method B
Modulus at 100% Elongation, kN/m (lbs/in)	5.25 (30)	ASTM D 882, Method B
Ultimate Elongation, %	350	ASTM D 882, Method A
Tear Resistance, N (lbs)	38 (8.5)	ASTM D 1004
Low Temperature Impact, °C (°F)	-29 (-20)	ASTM D 1790
Volatile loss, % max.	0.7	ASTM D 1203, Method A
Pinholes, No. Per 8 m^2 (No. per 10 sq. yds.) max.	1	N/A
Bonded Seam Strength, % of tensile strength	80	N/A

6. **Inlet Works**: Provide energy dissipation at all inlet points into the sand filter. Use an impact basin for pipes and a baffle chute or grouted sloping boulder drop if a channel or swale is used, or install a Type VL or L riprap basin underlain with geotextile fabric at the inlet (see Figure SF-1). Fill all rock voids with the filter material specified in Table SF-1.

7. **Outlet Works**: Slope the underdrain into a larger outlet structure. As discussed in Step 4, use an orifice plate to drain the WQCV over approximately 12 hours. Flows exceeding the WQCV should also drain into the outlet structure. Additional flow restrictions may be incorporated to provide Full Spectrum Detention, as discussed in the *Storage* chapter of Volume 2, or peak reduction for other specific storm events.

   For Full Spectrum Detention, perform reservoir routing calculations to design the outlet structure. The *UD-Detention* workbook, available at www.udfcd.org, can be used for this purpose. The design could include a second orifice located at the WQCV elevation or could include a downstream point of control designed to drain the full excess urban runoff volume (EURV) in approximately 72 hours.

Construction Considerations

Proper construction of sand filters involves careful attention to material specifications and construction details. For a successful project, do the following:

- Protect area from excessive sediment loading during construction. The portion of the site draining to the sand filter must be stabilized before allowing flow into the sand filter.
- When using an impermeable liner, ensure enough slack in the liner to allow for backfill, compaction, and settling without tearing the liner.

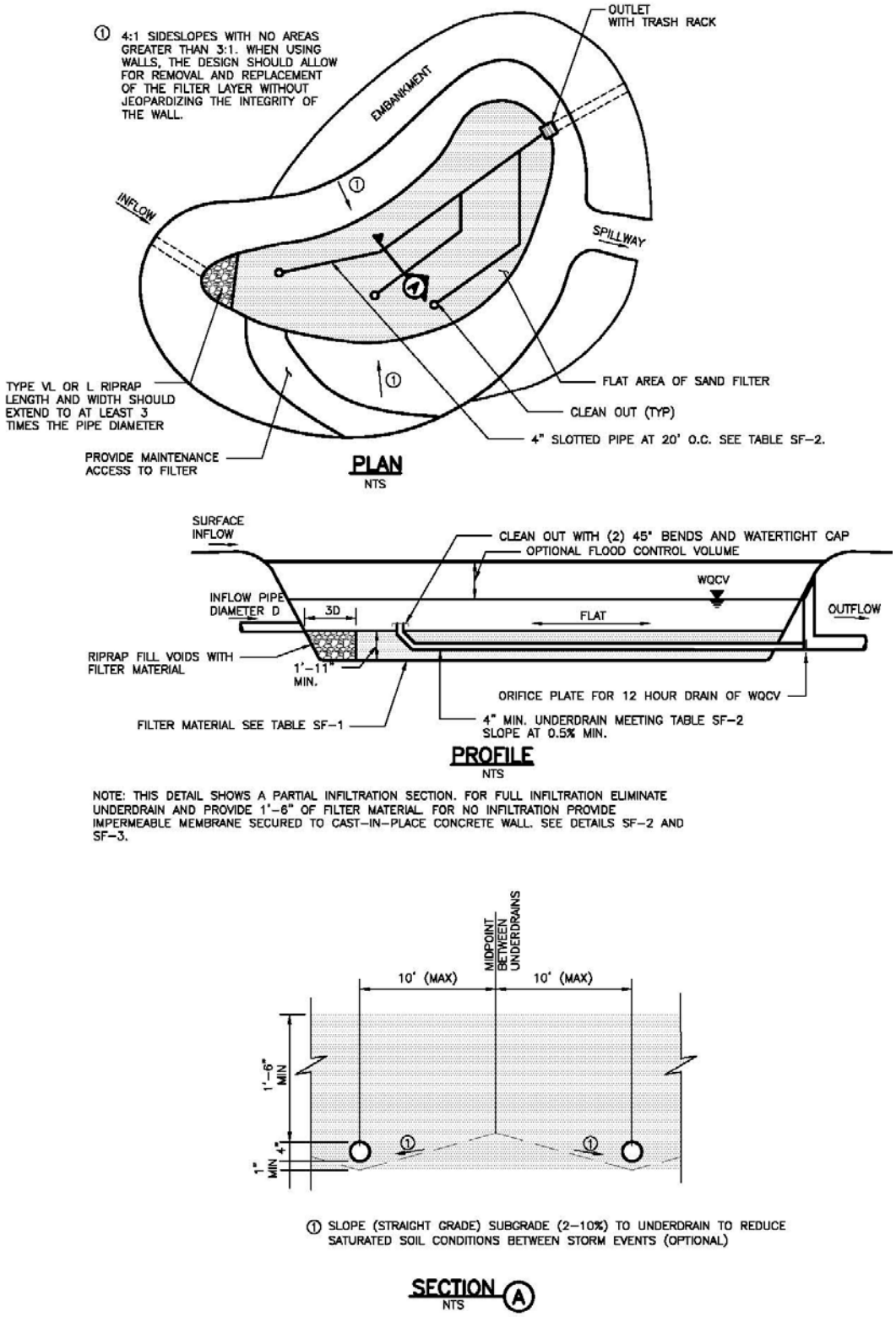


Figure SF-1. Sand Filter Plan and Sections

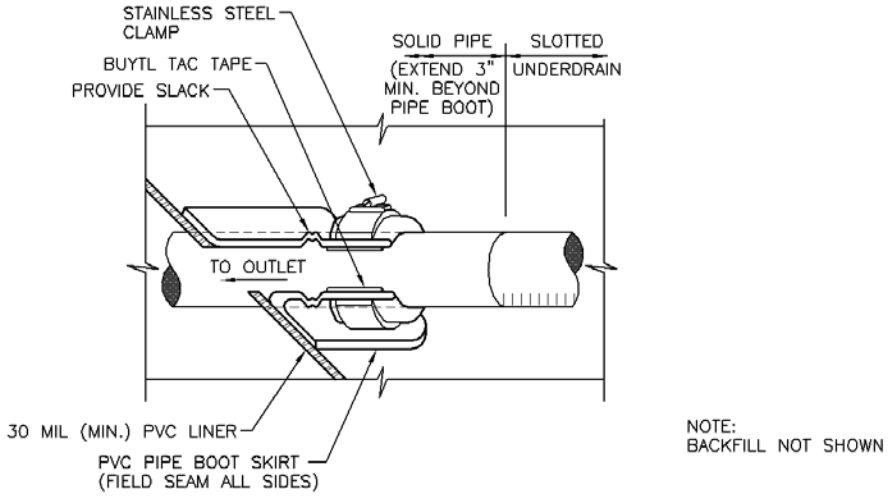


Figure SF-2. Geomembrane Liner/Underdrain Penetration Detail

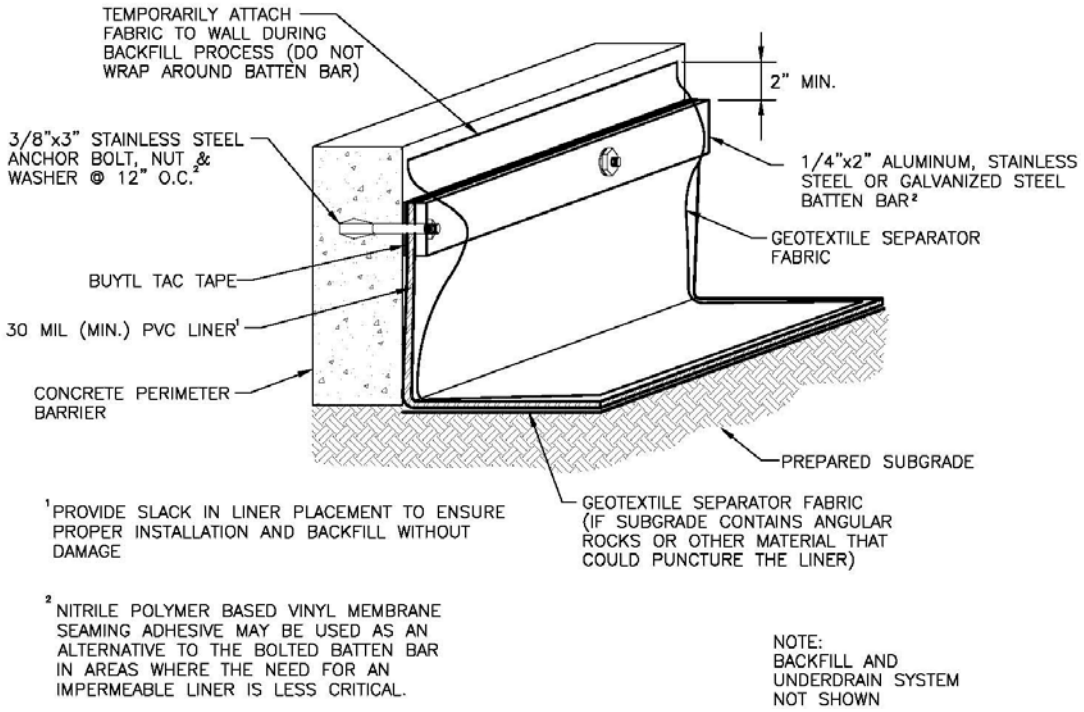


Figure SF-3. Geomembrane Liner/Concrete Connection Detail

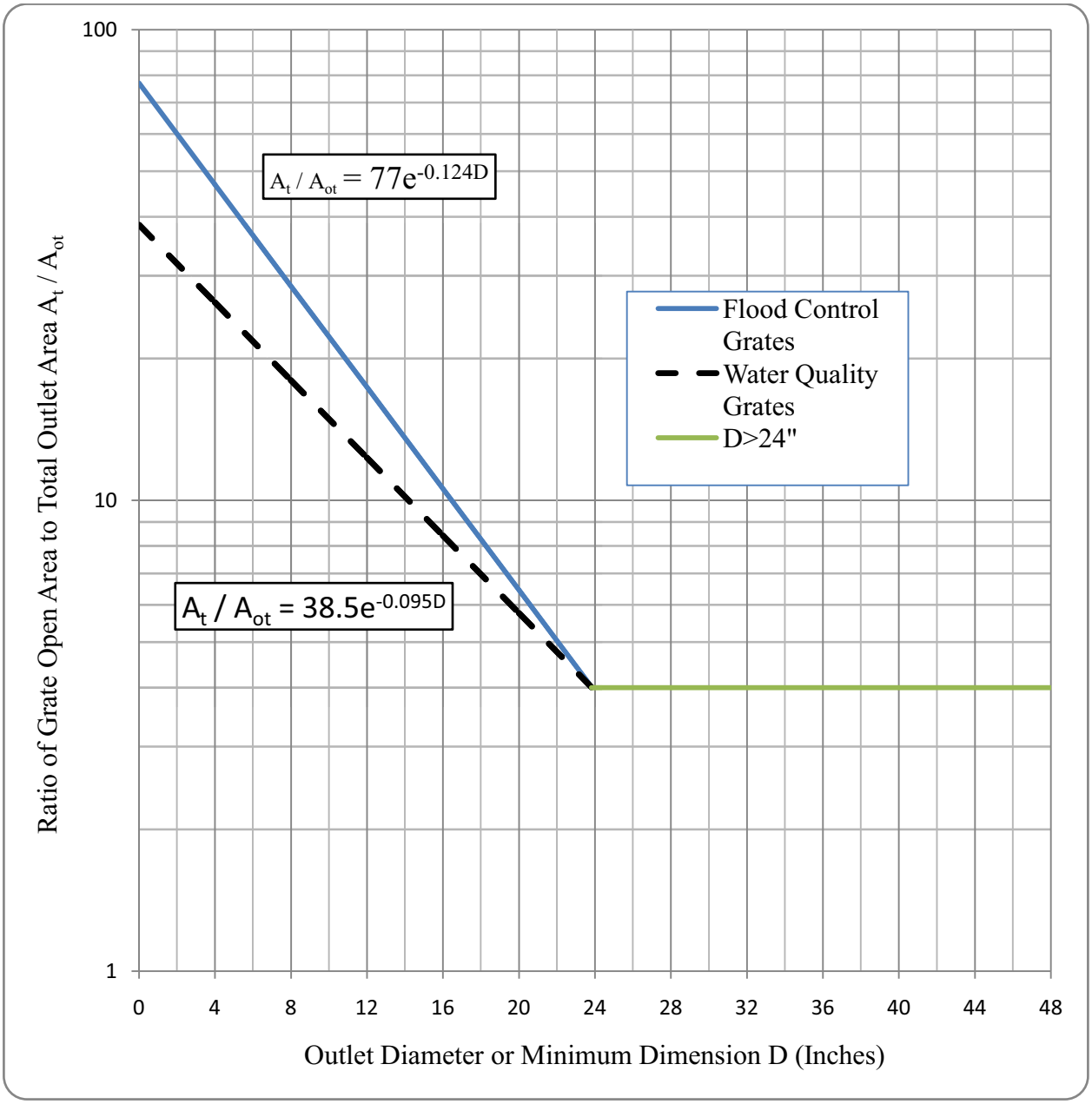


Figure OS-1. Trash Rack Sizing

Table SQ-6—BMP Stormwater Quality Influent and Effluent Event Mean Concentrations Based on Analysis of 200 BMP Sites in United States and Canada

Constituents	Point of Discharge	Extended Detention	Wet Pond	Wetland Basin	Biofilter	Media Filter
Suspended Solids (mg/l	*Influent*	**87.7** (48.4-159)	**88.8** (48.9-156)	**82.1** (65.7-103)	**52.0** (22-123)	**61.1** (45.4-82.4)
	Effluent	**41.4** (30.8-55.5)	**19.0** (12.9-28.0)	**19.7** (16.6-23.4)	**24.6** (15.0-40.3)	**25.5** (14.7-44.3)
Total Copper (ug/l)	*Influent*	**32.3** (22.7-46)	**18.0** (7.4-43)	xx	**21.8** (11.6-40.9)	**15.3** (12.4-18.8)
	Effluent	**18.9** (16.6-21.5)	**6.92** (4.7-10.3)	xx	**10.0** (5.6-17.9)	**9.81** (8.1-11.8)
Dissolved Copper (ug/l)	*Influent*	**12.1** (8-18.3)	**8.87** (5.4-14.6)	xx	12.3 (6.5-23.4)	8.83 (6.7-11.6)
	Effluent	**14.7** (10.4-20.9)	**5.09** (3.1-8.3)	xx	**7.66** (4.7-12.5)	**7.95** (6.6-9.7)
Total Lead (ug/l)	*Influent*	**69.2** (33.6-143)	**33.3** (10.2-109)	**12.6** (3.8-42)	**19.6** (7.4-51.6)	**15.6** (9.3-26.1)
	Effluent	**15.0** (9.5-23.8)	**6.68** (2.9-15.6)	**3.25** (1.9-5.6)	**6.95** (4.2-11.7)	**5.5** (3.5-8.6)
Dissolved Lead (ug/l)	*Influent*	**3.4** (2-5.8)	**9.48** (0.9-101.4)	xx	**2.5** (0.9-6.9)	**2.18** (1.6-3.1)
	Effluent	**2.33** (1.7-3.3)	**4.16** (2.0-8.9)	xx	**1.35** (0.5-3.6)	**1.42** (1.0-1.9)
Total Zinc (ug/l)	*Influent*	**274** (178-422)	**75.3** (44-128.9)	**164** (54.6-494)	**129** (57.3-291)	**122** (72.6-204)
	Effluent	**85.3** (50.6-143.7)	**28.6** (21.4-38.3)	**119** (32.8-429)	**39.4** (28.2-55.2)	**65.0** (45.3-93.2)
Dissolved Zinc (ug/l)	*Influent*	xx	**57.4** (20.1-163)	xx	**67.4** (33.8-134)	**71.7** (41.3-124)
	Effluent	xx	**16.9** (2.6-109)	xx	**32.0** (26.7-38.3)	**57.1** (37.7-86.6)
Total Phosphorus (mg/l)	*Influent*	**0.4** (0.3-0.5)	**0.53** (0.3-0.9)	**2.91** (1.9-4.6)	**0.19** (0.1-0.4)	**0.25** (0.2-0.4)
	Effluent	**0.3** (0.2-0.44)	**0.16** (0.12-0.21)	**0.15** (0.07-0.33)	**0.32** (0.24-0.43)	**0.14** (0.11-0.17)
Total Nitrogen (mg/l)	*Influent*	xx	**1.49** (0.6-3.6)	**2.56** (1.6-4)	**0.58** (0.3-1)	xx
	Effluent	xx	**1.17** (0.77-1.78)	**2.42** (1.46-4.0)	**0.69** (0.37-1.29)	xx
TKN (mg/l)	*Influent*	**1.99** (1.6-2.5)	**1.06** (0.8-1.4)	**1.23** (1-1.6)	**2.27** (1.8-2.9)	**2.2** (1.7-2.9)
	Effluent	**1.87** (1.46-2.39)	**0.84** (0.68-1.04)	**1.33** (0.84-2.11)	**1.6** (1.42-1.8)	**1.79** (1.45-2.2)

Does not meet 95% confidence test for change in the mean between inflow and outflow
Meets 95% confidence test for change in mean concentration between inflow and outflow

Cautionary Note: This table presents statistics of site mean EMCs without weighting the sites for data density. Each site has equal weight in the analysis.
Some sites had very little data, but have same weight as sites with much data.

Legend: **mm.m** - Mean of all site mean EMCs
(ll.ll - uu.uu) = (Lower – Upper) values of the 95% confidence test of the mean.

An effective strategy for reducing stormwater pollution loads is to use multiple BMPs, including nonstructural measures, source controls, and structural BMPs. A single practice and/or facility cannot generally provide significant reductions in stormwater pollutant loads because these pollutants come from many sources within a municipality. Also, multiple BMPs can provide complementary water quality enhancement to achieve desired results. A multilevel BMP approach, schematically depicted in Figure SQ-1, deals with the many pollutant and runoff sources throughout a watershed and shows that whenever feasible, combining the most effective BMPs in a series can be an effective strategy to reduce pollutant loads being transported to the receiving waters by stormwater.

AC-31 ALUMINUM CANAL GATE

SIZES 12" - 84"

- **A SUPERBLY ENGINEERED GATE EXPRESSLY SUITABLE FOR CORROSION-RESISTANT, RUST PROOF DEMANDS.**
- **RUGGED, HEAVY DUTY CONSTRUCTION - YET LIGHTWEIGHT AND EASY TO INSTALL.**
- **ALUMINUM GATE ELIMINATES ELECTROLYSIS USUALLY FOUND IN CAST IRON GATE TO ALUMINUM PIPE CONNECTIONS.**
- **MINIMAL LEAKAGE - TIGHT CLOSURE**

FEATURES:

SEATING HEADS TO 30 FEET
UNSEATING HEADS TO 20 FEET

TAPERED SLIDE:
Provides wedge-type seating and low friction opening.

NEOPRENE J-BULB SEAL:
Virtual leakproof closure. Easily replaceable.

GUIDE INSERTS:
Reduced slide friction. Ultra high molecular weight (UHMW) polyethylene, dovetailed.

INSTALLATION OPTIONS:
Directly onto CMP - self-contained spigot.
Onto thimble - circular flangeback.
Onto wall - flatback.

STAINLESS STEEL STEM & HARDWARE

BRONZE LIFT NUT

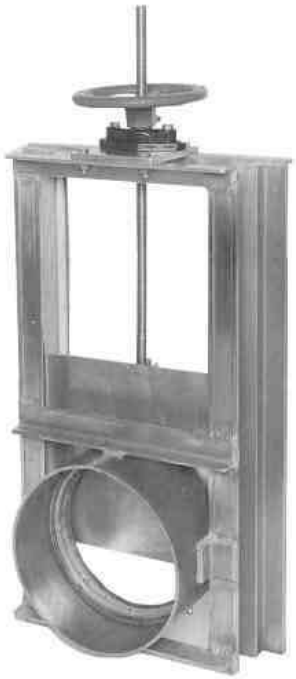
Recommended Maximum Seating Heads*

8" - 30"	30 Feet
36" - 42"	20 Feet
48" - 60"	15 Feet
66" - 84"	10 Feet

*** These recommendations are based upon recent design studies and modifications of this valve and upon years of installation evaluations.**

AC-31sb FRONT VIEW

AC-31 Flatback w/Torque Tube (Rising Stem Extension)



Back View Showing Spigot Connection and Neoprene Seal

Waterman INDUSTRIES, INC.

TYPICAL SPECIFICATIONS
FOR MODEL AC-31

Aluminum Slide Gates, where shown on the plans or indicated in the specifications shall be Model AC-31 Aluminum Slide Gates with resilient seal as manufactured by Waterman Industries, Inc. or equal.

The gates will be self-contained, rising stem of the spigotback design for attaching to corrugated pipe [or flatback design for attachment to head wall with anchor bolts], [or flange design with drilling suitable for attachment to 25 or 125 lb. ASI companion flange].

Guide frames shall be of extruded aluminum shape, of sufficient section to carry the operating forces of the gate, and shall have UHMW polyethylene inserts on which the slide assembly travels to minimize friction. A spigot ring will be welded to the guides and have a seating surface at minimum 1½° angle to which a resilient J-Bulb seal shall be attached or held in place by a substantial section ring.

Dual Headrails (Yokes) shall be welded to the guide rails and so positioned that the slide is removable from the gate. The slide shall be aluminum plate suitable reinforced with extruded structural shapes for the head requirements specified and will not deflect more than 1/360 of the gate width under the design head. Suitable side guides will be welded to the slide, which will travel within the guides, and place the cover in an angle corresponding to the seat surface of the guide frame assembly. A rising stainless steel stem having modified acme type threads shall attach to the slide with a clevis and pin arrangement.

The stem will be designed to have a L/r of 200 or less and to withstand in compression at least twice the rated output of the lift at 40 lb. pull. A suitable handwheel or gear type operator should be mounted on the headrails (yoke) of the gate and will require a maximum 40 lb. pull on the handwheel rim or crank handle to operate the gate.

Flatback gates shall be attached to headwalls with anchor bolts or expansion anchors. Gates shall be installed on the anchors with nuts both behind and on the flange face so as to position the gate without stress or distortion. See manufacturer's installation instructions. Sealing between headwall and gate flange will be by dry-pack non-shrink grout or other suitable mastic sealant.

Spigotback gates will be attached to corrugated metal pipe by field drilling the pipe with 7/16" holes and secured with 3/8" stainless steel bolts, maximum spacing to be 8" on circumference of pipe. Sealing between pipe and gate will be made with a suitable mastic sealing material to assure water tightness at this joint.

The gate, when installed, shall have no more than 0.01 gpm leakage per foot of sealing periphery for seating or unseating heads up to 15 feet.

Aluminum and stainless steel will be mill finish. Paint for lifts will be manufacturer's standard [or prime paint as specified elsewhere.]

MATERIAL

Aluminum Plate & Shapes - ASTM-B211 Alloy 6061-T6

J-Bulb Seal - Neoprene ASTM-CB610-625, D-2000

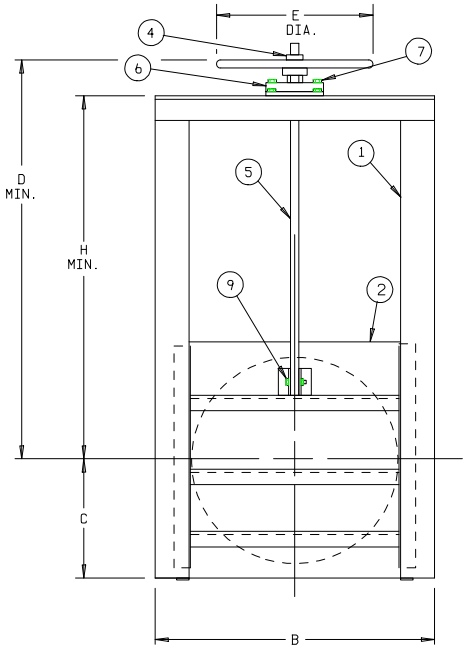
Fasteners - ASTM F593 & F594 Type 304 or 316

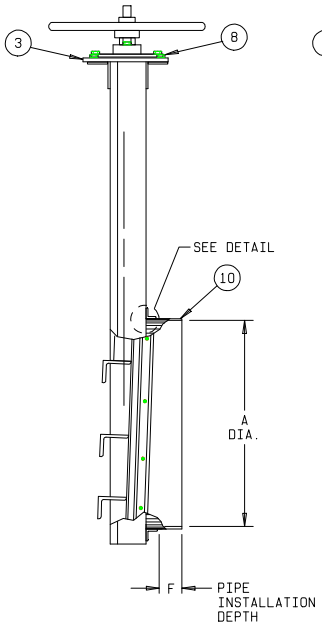
Stem - ASTM A-276 Type 304 & 316

Handwheel Lift - Cast iron ASTM A-126 CL B W/manganese bronze lift nut per ASTM B-584 alloy 865

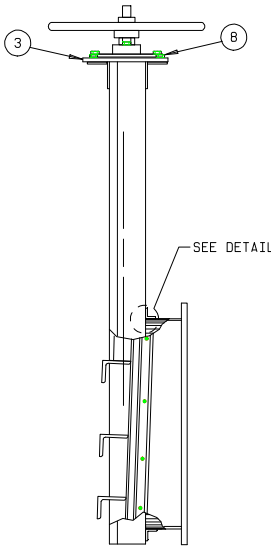
Enclosed Gear Lift - Cast iron ASTM A-126 CL B W/manganese bronze lift nut per ASTM B-584 alloy 865

TYPICAL DIMENSIONS AC-31 ALUMINUM CANAL GATE

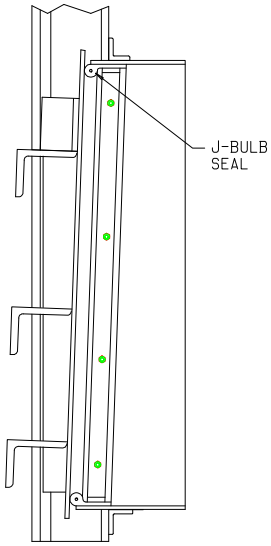




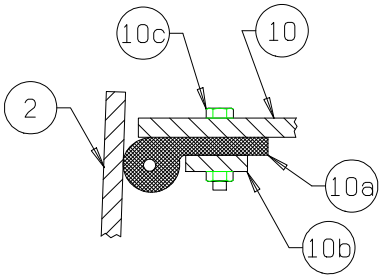
SPIGOTBACK



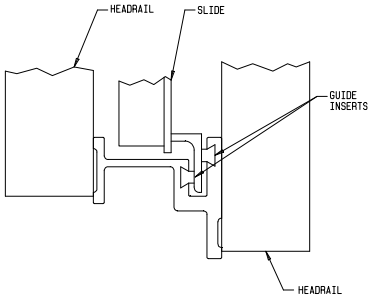
FLATBACK/
FLANGEBACK



DRAWING REPRESENTS SIDE VIEW
OF TAPERED SLIDE AGAINST
J-BULB SEATING



FRAME ASSEMBLY DETAIL
(OPTIONAL FLANGE)



DRAWING REPRESENTS TOP VIEW
OF UHMW POLYETHYLENE GUIDE INSERTS

PARTS LIST	
No.	**Name**
1	Frame (Aluminum,extruded)
2	Cover (Aluminum plate)
3	Top Plate (Aluminum)
4	Handwheel
5	Stem (Stainless steel,threaded)
6	Lift Housing and Nut
7	Lift Housing Hardware (Stainless steel)
8	Top Plate Hardware (Stainless steel)
9	Stem to Cover Attaching Hardware (Stainless steel)
10	Aluminum Spigot
10a	J-Bulb Seat Ring (Neoprene Rubber)
10b	Retainer Ring (Aluminum)
10c	Mounting Hardware (Stainless steel)

AC-31 ALUMINUM CANAL GATE

DIMENSION IN INCHES

GATE SIZE (DIA)	A DIA.	B	C	D*	E**	F	H
12	13⅛	21	8	26	10	4¼	22
15	16⅛	25	9½	32½	10	4¼	28½
18	19⅛	28	11	37½	12	4¼	33½
24	25⅛	34	14	46½	12	4¼	42½
30	31⅛	40	18	58½	15	4¼	54½
36	37⅛	46	21	67½	15	4¼	63½
42	43⅛	52	24	76½	18	5½	72½
48	49⅛	57⅞	28½	93*	15R**	5	87
54	55⅛	63⅞	31½	101*	15R**	6	95
60	61⅛	69⅞	34½	114¼*	15R**	6	108
66	67⅛	75⅞	37½	123¼*	15R**	6	117
72	73⅛	81⅞	40½	132¼*	15R**	6	126
84	85⅛	94⅛	46½	152¾*	15R**	7	146½

* Dimension from gate centerline to centerline of lift input shaft
** 15" Handcrank

NOTE: FOR PRELIMINARY DESIGN PURPOSES ONLY
DO NOT USE FOR INSTALLATION
UNLESS PART OF CERTIFIED & APPROVED SUBMITTAL

CONTECH TRM C-51

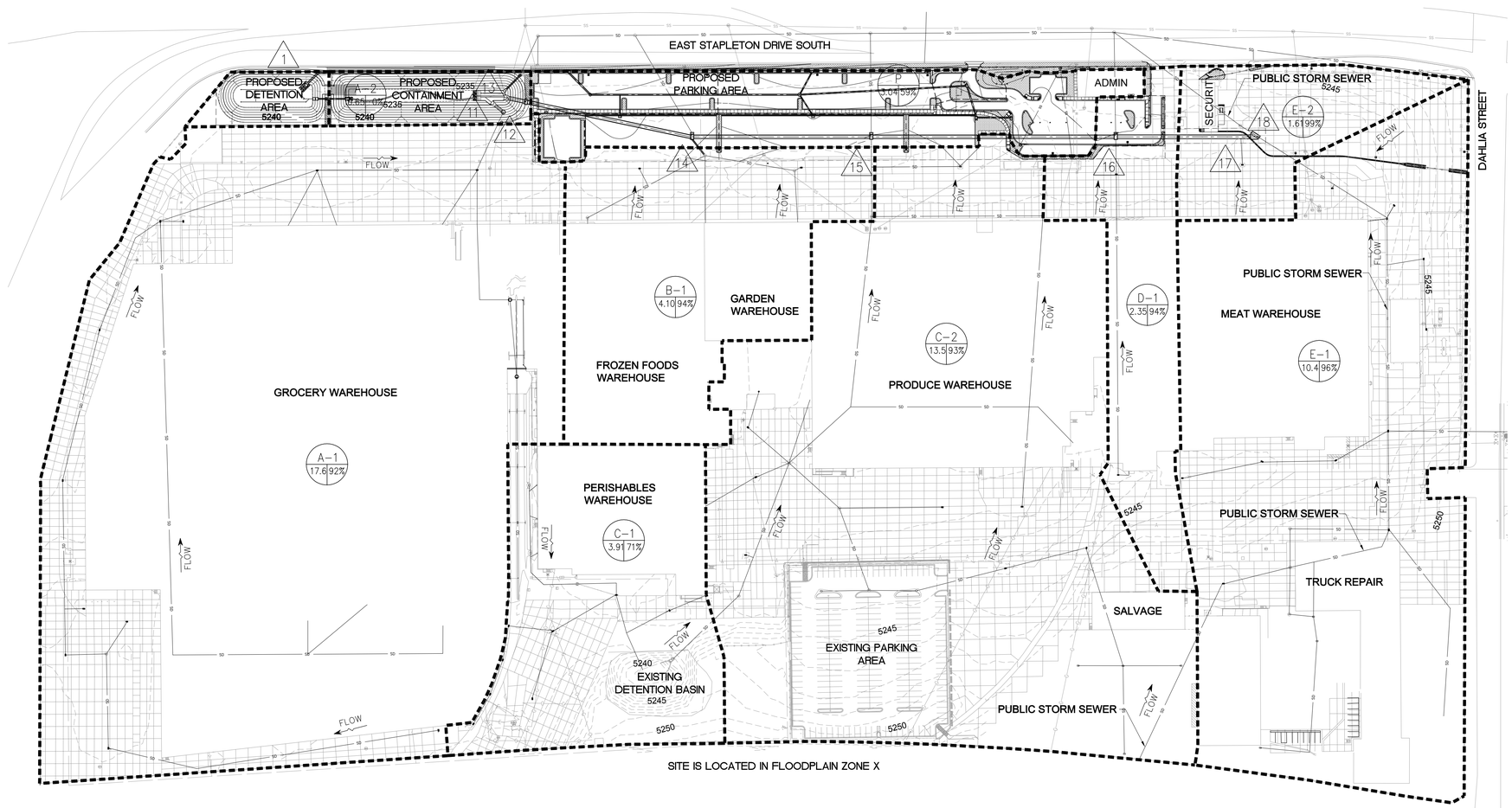
The Turf Reinforcement Matrix (TRM) shall consist of a lofty web of polyolefin fibers positioned between two high strength, biaxially oriented nets and mechanically bound together by parallel stitching with polyolefin thread. The matrix shall possess strength and elongation properties to limit stretching and shall be maintained in a water-saturated condition. Every component of the matrix shall be stabilized against ultraviolet degradation and inert to chemicals normally encountered in a natural soil environment. The product shall have the following properties:

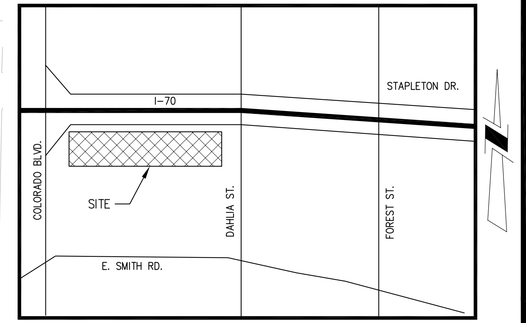
		MARV[2]	
Property	**Test Method**	**English**	**Metric**
Physical			
Mass/Unit Area	ASTM D6566	14.0 oz/sy	475g/m^3
Thickness	ASTM D6525	0.40 in	10.1 mm
Light Penetration (% Passing)	ASTM D6567	5%	5%
Color	Visual	Tan	
Mechanical			
Tensile Strength	ASTM D6818	300 x 225 lb/ft	4.3 x 3.2 kN/m
Tensile Elongation	ASTM D6818	85% (max)	85% (max)
Resiliency	ASTM D6524	80%	80%
Flexibility	ASTM D6575	0.022 in-lb (ave)	25,000 mg-cm (ave)
Durability			
UV Resistance @ 1000 hrs	ASTM D4355	80%	80%
Performance			
Shear Stress [3]	ASTM D6460	8 psf	383 Pa
Manning's "n"[4]	Calculated	0.026	0.026
Seedling Emergence[5]	ECTC Draft Method #4	–	–
Roll Size		6.5 ft / 138.5 ft	2.0 m x 42.2 m

NOTES:

1. The property values listed are effective 3/1/2004 and are subject to change without notice.
2. MARV indicates minimum average roll value, calculates as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the value reported.
3. Maximum permissible shear stress has been obtained through vegetated testing programs featuring specific soil types, vegetation classes, flow conditions, and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers.
4. Calculated as typical values from large-scale flexible channel lining test programs with a flow depth of 6 to 12 inches.
5. Calculated as average plant height obtained with tall fescue grass in sand 14 days after seeding.

E. DRAINAGE MAPS





VICINITY MAP
1"=1000'

0 100 150 200
(IN FEET)
1 INCH = 100 FEET

LEGEND

5245	EXISTING MAJOR CONTOUR
SS	EXISTING SANITARY SEWER
W	EXISTING WATERLINE
SD	EXISTING STORM SEWER
OE	EXISTING OVERHEAD ELECTRIC
5245	PROPOSED MAJOR CONTOUR
SS	PROPOSED SANITARY SEWER
	PROPOSED STORM DRAIN

MAJOR BASIN TABLE:

DESIGN POINT	CONTRIBUTING BASIN	BASIN SIZE (acres)	5-YR RUNOFF (cfs)	100-YR RUNOFF (cfs)
1	P	3.0	9.71	21.56
12	A-1	17.6	36.7	77.0
11	A-2	0.7	0.4	2.6
14	B-1	4.1	12.5	26.2
15	C-2	13.5	38.0	79.8
16	D-1	2.4	7.7	16.2
17	E-1	10.4	35.0	72.6
18	E-2	1.6	6.3	13.0

DRAINAGE LEGEND:

GENERAL FLOW DIRECTION FLOW

BASIN LIMITS

A = BASIN DESIGNATION
B = AREA IN ACRES
C = PERCENT IMPERVIOUSNESS

DESIGN POINT

CITY AND COUNTY OF DENVER
Development Services
Site Engineering

DS Project NO. PR-2011-065 (2011-0177)

PROJECT NAME: SAFEWAY DISTRIBUTION CENTER

DESIGNED BY DAP	DATE 10-24-10	DATE ISSUED: 09-28-11	DRAWING NO.
DRAWN BY DAP	DATE 10-24-10		D1.0
CHECKED BY AEH	DATE 09-26-11	SHEET 1 OF 2 SHEETS	

PREPARED UNDER THE SUPERVISION OF
TAIT & ASSOCIATES, INC.

ALEX HOIME, PE 41703 DATE SEAL

CALL UTILITY NOTIFICATION CENTER OF COLORADO

1-800-922-1987

CALL 2 BUSINESS DAYS IN ADVANCE BEFORE YOU DIG, GRADE, OR EXCAVATE FOR THE MARKING OF UNDERGROUND MEMBER UTILITIES

ENGINEERS NOTE TO CONTRACTOR:

THE EXISTENCE AND LOCATION OF ANY UNDERGROUND UTILITIES, PIPES, AND/OR STRUCTURES SHOWN ON THESE PLANS WERE OBTAINED BY A SEARCH OF AVAILABLE RECORDS. TO THE BEST OF OUR KNOWLEDGE, THERE ARE NO EXISTING UTILITIES EXCEPT AS SHOWN ON THESE PLANS. THE CONTRACTOR SHALL ASCERTAIN THE TRUE VERTICAL AND HORIZONTAL LOCATION OF THOSE UNDERGROUND UTILITIES TO BE USED AND SHALL BE RESPONSIBLE FOR ANY DAMAGE TO ANY PUBLIC OR PRIVATE UTILITIES, SHOWN OR NOT SHOWN HEREON.

BASIS OF BEARINGS:

BEARINGS BASED ON THE NORTH LINE OF LOT 3 AS NORTH 88°03'20" EAST BETWEEN MONUMENTS SHOWN ON THE TOPOGRAPHIC SURVEY.

BENCH MARK

MARTIN/MARTIN ENGINEERING PROJECT BENCHMARK, USGS DISK STAMPED Y259 (LAT 33°46'33", LONG 104°55'49") ELEVATION 5257.26 (NGVD 29 5257.37 RECORD)

NO.	DESCRIPTION	BY	DATE
	REVISIONS		

TAIT & ASSOCIATES, INC.
6163 EAST COUNTY ROAD 16
LOVELAND, CO 80537
(970) 613-1447 TEL
(970) 613-1897 FAX
www.tait.com

TAIT

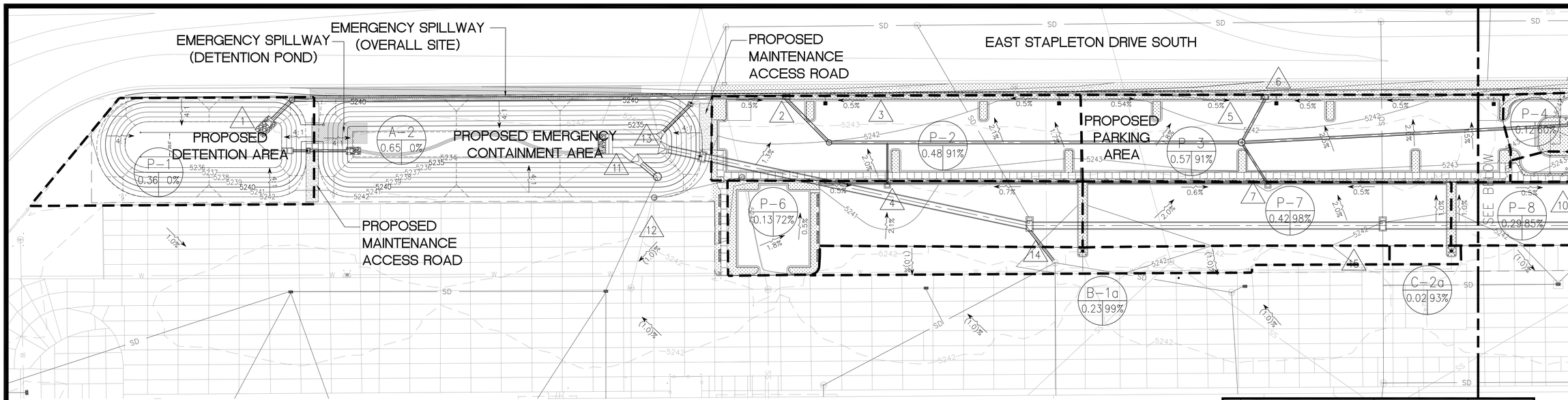
DRAINAGE PLAN

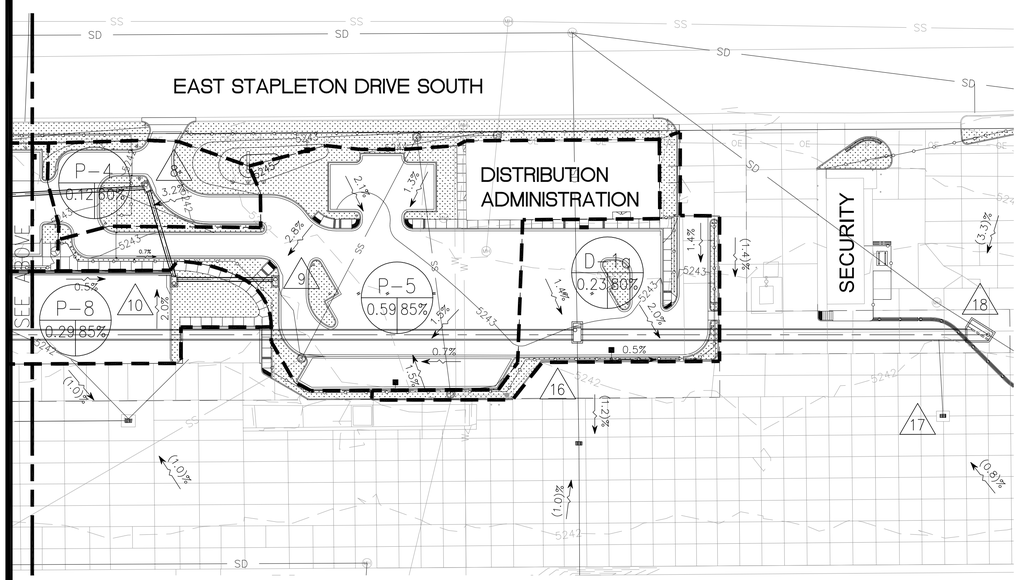
NORTH EMPLOYEE AND VISITOR PARKING AREA
SWC I-70 AND DAHLIA STREET ,DENVER, CO
SAFEWAY INC.

DRAWN: DAP
DATE: 03-16-11
CHECKED: AEH
DATE: 08-17-11
REVISION #: 2
DATE: 08-16-11
JOB NO: CO1176F

D1.0

CITY OF DENVER APPROVAL SET - 09/28/11





DRAINAGE LEGEND:

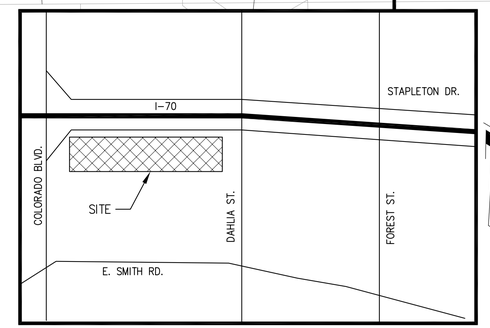
GENERAL FLOW DIRECTION — FLOW

BASIN LIMITS

A = BASIN DESIGNATION
B = AREA IN ACRES
C = PERCENT IMPERVIOUSNESS

DESIGN POINT

0 40 60 80
(IN FEET)
1 INCH = 40 FEET



VICINITY MAP
1"=1000'

DETENTION BASIN SUMMARY TABLE

	WATER SURFACE ELEVATION (FEET)	VOLUME (ACRE-FT)	RELEASE RATE (CFS)
WATER QUALITY CAPTURE VOLUME	5236.67	0.08	---
10-YR + WATER QUALITY CAPTURE VOLUME	5238.35	0.34	0.91
100-YR + 1/2 WATER QUALITY CAPTURE VOLUME	5239.34	0.54	3.04

NOTE: SITE IS LOCATED IN A ZONE X FLOODPLAIN

LEGEND

- EXISTING MAJOR CONTOUR (5245)
- EXISTING MINOR CONTOUR (5244)
- EXISTING CHAIN LINK FENCE
- EXISTING SANITARY SEWER (SS)
- EXISTING WATERLINE (W)
- EXISTING STORM SEWER (SD)
- EXISTING OVERHEAD ELECTRIC (OE)
- EXISTING STORM MANHOLE
- EXISTING UTILITY POLE
- EXISTING BOLLARD
- EXISTING LIGHT POLE
- EXISTING FIRE HYDRANT
- PROPOSED MAJOR CONTOUR (5245)
- PROPOSED MINOR CONTOUR (5244)
- PROPOSED CHAIN LINK FENCE
- PROPOSED SANITARY SEWER (SS)
- PROPOSED STORM DRAIN
- PROPOSED MANHOLE
- PROPOSED TURNSTILE
- PROPOSED CONCRETE BARRIER
- PROPOSED BOLLARD
- PROPOSED FIRE HYDRANT
- PROPOSED LIGHT POLE

PROPOSED BASIN TABLE:

DESIGN POINT	CONTRIBUTING BASIN	BASIN SIZE (acres)	5-YR RUNOFF (cfs)	100-YR RUNOFF (cfs)
1	P-1	0.36	0.19	1.19
2	P-2	0.48	1.73	3.66
6	P-3	0.57	2.12	4.49
8	P-4	0.12	0.31	0.78
9	P-5	0.58	1.81	3.95
4	P-6	0.40	1.50	3.18
7	P-7	0.36	1.46	3.01
10	P-8	0.17	0.59	1.30
11	A-2	0.65	0.40	2.59
14	B-1b	0.23	0.93	1.91
15	C-2a	0.02	0.08	0.17
16	D-1a	0.23	0.72	1.59

CITY AND COUNTY OF DENVER
Development Services
Site Engineering

DS Project NO. PR-2011-065 (2011-0177)

PROJECT NAME: SAFEWAY DISTRIBUTION CENTER

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CITY OF DENVER APPROVAL SET - 09/28/11