STORM DRAINAGE DESIGN CRITERIA

Community Development Department
Engineering Division

Town of Windsor, Colorado

Revised July 1, 2020

STORM DRAINAGE DESIGN CRITERIA

TOWN OF WINDSOR, COLORADO

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July 21, 2006 Revisions

1. All references to the Director of Public Works throughout the manual have been changed to the Director of Engineering.
2. Page 2-4, paragraph B(2)(k)(5), the design details and operational data requirement was revised to apply to the 10 year and 100 year depths, rather than the 5 year and 100 year depths.
3. Page 3-2, Table 3-1, the values for gravel streets have been revised to the values used by the Urban Drainage and Flood Control District.
4. Page 3-3, the formula for to for developed watersheds was incorrectly stated, it has been revised to the correct formula.
5. Page 4-5, Section 4.2.3, the minimum width for crosspans on local streets has been revised to 8 feet.
6. The second paragraph of Section 8, Section 8.1, Section 8.2 and Section 8.3 have been revised to reflect the Town's current standard of over-detention which is to detain the 100 year developed flows with a maximum release at the 10 year historic rate.
7. Section 8.3 has been revised to reflect the Town's current standard of providing water quality treatment in detention ponds.

April 2019 Revisions

1. Sections 8 and 9 have been revised to include the Municipal Separate Storm Sewer System (MS4) references and guidance.
2. Bibliography: November 2010 version of the Urban Storm Drainage Criteria Manual is referenced.

July 2020 Revisions

1. Page 2-5 Section 2.2 IV.C(2)(3) has been revised to include direction on the required registration of detention ponds with the State of Colorado, and the requirement to include permanent water quality control measure operation and maintenance plans in drainage reports.
2. Page 8-3 through Page 8-5, Section 8.3.2.1 Water Quality Design Exclusions, has been added.
3. Page 9-2 Section 9 has been revised to disallow the use of crushed concrete as traction material for VTCs or concrete washout areas, or staging area stabilization material.
4. Page 4-3 Section 4.1.3.1 has been revised, changing the depth of water from 18 inches to 12 inches over the gutter flowline.
5. Page 4-5, Table 4-3 has been revised, changing the design runoff for major storm from 18 inches to 12 inches.
6. Cover page updated.

SECTION 1 - DRAINAGE POLICY

1.1 Purpose and Scope

The purpose of this manual is to set forth the technical criteria to be used in the analysis and design of drainage systems within the Town of Windsor, Colorado. All subdivision plats, planned unit developments, or any other proposed construction must include an adequate plan for storm drainage based on thorough analysis using this manual as a guide. Approval shall be made by the Director of Engineering of the Town of Windsor prior to any phase of construction.

A large portion of the criteria and design aids included in this manual originated from the Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual (Volumes 1 & 2). That manual shall be used as a reference for any information not detailed in this Storm Drainage Design Criteria.

In the Town's best interest, the Director reserves the right to issue and enforce more stringent criteria should adverse conditions exist. Also, occasions may arise where the minimum standards are either inappropriate or cannot be justified economically. In these cases, a variance to these criteria may be considered. All designs varying from the criteria shall require the Director's written approval prior to final approval of plans.

The Town of Windsor Storm Drainage Design Criteria shall be considered supplemental material to the Town Code. If a conflict should arise, the Code shall govern. References to standards refer to the latest edition, including amendments in effect and published at the time drainage plans are approved.

1.2 Policy Summary

The policy statements contained in this summary are the basis for all storm water management within the Town of Windsor and are to be used as guidelines in the design and evaluation of all storm drainage facilities. Policy statements are presented for the following topics:

- General
- Master Drainage Plans
- Design Standards
- Major and Minor Drainage Systems
- Natural Drainageways
- Open Channels
- Irrigation Ditches
- Storm Water Detention
- Revisions to the Design Criteria

1.2.1 General

In general these policy statements address five areas of concern: (1) overall storm drainage planning and management; (2) the interface between urban development and irrigation facilities such as dams, reservoirs and canals; (3) the treatment of historic drainageways and natural channels; (4) the requirements and specifications for engineering design of storm drainage facilities; and (5) the quality and extent of urban stormwater runoff.

1.2.2 Design Standards

The policy of the Town will be to design drainage improvements and to evaluate the designs of drainage improvements submitted to the Town for approval based on the standards and specifications set forth in the Town of Windsor Storm Drainage Design Criteria.

1.2.3 Major and Minor Drainage Systems

The policy of the Town shall be to define the major drainage systems and to require drainage design for the minor and major drainage systems. For **all** land uses, design of the facilities within the major and minor drainage systems shall be based on the runoff expected from the 100-year and 2-year storms, respectively, unless the master drainage plan for the area specifies a different level of protection.

1.2.4 Natural Drainageways

The policy of the Town shall be to direct runoff from new urban development into historic and natural drainageways and to promote the recreational use and enhance the aesthetic value of such drainageways wherever possible.

1.2.5 Open Channels

The policy of the Town shall be to seek to maintain stability in major runoff channels in and near developing areas. Developments in and near major runoff channels shall be required to adopt measures to assure that excessive erosion does not occur under peak flood flow. Realignment of natural channels in urban areas shall not be permitted unless the Town approves an engineering design which will control erosion under conditions of peak flow.

1.2.6 Irrigation Ditches

The policy of the Town shall be to avoid discharge of runoff from urban areas into irrigation facilities except as required by water rights or where such discharge is in conformance with approved master drainage plans for the canals. Further, wherever new development will alter patterns of drainage into irrigation ditches by increasing flow rates or volumes, or will change the historic concentration points of runoff, the Town shall require each new development to obtain the written consent of the appropriate ditch company before approving the development.

In addition, the policy of the Town shall be that whenever irrigation canals cross major drainage channels in developing areas, the Town shall require the developer to separate peak stormwater runoff flows from normal canal flows. The developer shall also provide adequate maintenance right of way as required by the owners of the ditch or reservoir.

1.2.7 Storm Water Detention

The policy of the Town shall be to require detention storage of stormwater runoff as directed by individual master drainage plans and a hydrologic routing analysis. In basins where a master drainage plan has not been approved, the Town may require detention storage in accordance with this Design Criteria where such storage is deemed necessary to protect irrigation structures or downstream development.

1.3 General Design Criteria

Except where specified herein, the procedure, criteria, and standards set forth in the Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual (Volumes 1 & 2) shall be used for the analysis of any drainage system.

The runoff analysis for a particular area shall be based on the zoned land use for that area. Contributing runoff from upstream areas shall be determined based on the existing land use and topographic characteristics of those areas. The master drainage plan for the development area shall be used for all runoff calculations if one is available.

Natural topographic features shall be the basis of location for easements and future runoff calculations. Average land slopes may be utilized in runoff calculations in both developed and undeveloped areas; however, existing drainage patterns and slopes shall be used wherever they are defined. The designed drainage facilities must be able to handle the design flows with virtually no erosion damage to the system.

Streets shall not be used as primary floodways for major storm runoff. The amount of runoff in the street shall not exceed the limits established in SECTION 4 - STREETS.

Natural drainageways are to be used whenever feasible. Alteration to natural drainage patterns shall be approved only if thorough investigation and analysis shows no hazard or liability.

The planning and design of the drainage system shall not be such as to simply transfer the problem from one location to another or create a more hazardous condition downstream. Although improvements may not have to be made upstream or downstream of a subdivision, provisions shall be made in every subdivision in the form of an easement or drainageway for the 100-year storm to pass through that subdivision.

All drainage improvements shall be as natural in appearance as possible to be aesthetically pleasing.

Maintenance access shall be provided for all drainage and flood control facilities.

Where a master drainage plan for a given area of the Town is available, proposed drainage systems shall conform to that plan. In areas where a master plan is not available, major drainageways and easements shall be located in order to provide continuity with existing drainage conditions. These drainageways and easements shall be shown on all drainage plans. Master drainage plans are available at the Office of the Director of Engineering.

SECTION 2 - DOCUMENTATION

2.1 General Requirements

All analyses and designs of storm drainage systems within the Town of Windsor shall be submitted to the Public Works Department for review and must obtain the Director of Public Work's written approval prior to any phase of construction.

For a Preliminary Plat submittal, two copies of a preliminary drainage report and plan in accordance with Section 2.2 of this Storm Drainage Design Criteria are required.

For a Final Plat submittal, the following are required:

1) Two copies of a final drainage report and plan containing the information identified in Section 2.2.
2) Six sets of construction drawings containing the information identified in Section 2.3.

2.2 Drainage Reports and Plans

The following is an outline and required information guideline for preliminary and final drainage reports. The items noted with an asterisk (*) are not required for a preliminary report.

I. Title Page

   A. Report type (Preliminary or Final)
   B. Project name
   C. Preparer name, firm, date
   D. P.E. seal and signature of preparer

II. Introduction

   A. Site location
      1. City, county, street grid
      2. Adjacent development
   B. Site description
      1. Existing topography, ground cover, use, etc.
      2. Existing drainage facilities, major channels, flood hazard zones and studies, irrigation ditches
   C. Proposed project description
   D. Flood hazard and drainage studies relevant to site

III. Historic Drainage System (discuss the following)

A. Major basin
   1. Relationship to major basin channel
   2. Major basin drainage characteristics, topography, runoff, use, cover, etc.

B. Sub-Basin and site drainage
   1. Initial and major storms
   2. Off-site flows
   3. Existing drainage patterns: channelized or overland flow, volumes, points of discharge from site
   4. Effect of historic flows upon adjacent properties

IV. Proposed (Developed) Drainage System (discuss the following)

A. Criteria
   1. Basin and subbasin size
   2. Hydrologic method (Rational or CUHP)
   3. Design storm frequencies - initial and major

B. Runoff
   1. Developed flow rates and paths

C. Detention
   1. Volumes required and provided
   2. Release rates and method of release
   3. Excess storm water passage

D. Streets
   *1. Depth and velocity of flow for initial and major storms
   2. Storm drainage system

*E. Open channel flow
   1. Type channel (lining)
   2. Maximum depth and velocity

F. Storm sewers and culverts

G. Other water quality control measure(s)

V. Conclusions

A. Discuss impact of improvement
   1. Benefits - Does improvement reduce existing drainage problems?
   2. Adverse impacts with solutions to mitigate impact

B. State compliance with applicable criteria
   1. Detention ponds (or other water quality control measures)
   *2. Depth and velocity of street flow
   *3. Channel flow depth and velocity
   4. Areas in or adjacent to designated floodplains

VI. Appendixes

A. Hydrologic and hydraulic computations
   1. Runoff (historic)
      a. Historic off-site and site for as many design points as required
         1) Separate time of concentration (T_c) for each design point (Rational Method)
         2) Runoff coefficient or permeability coefficient from Table 3-1
         3) Existing drainage facilities carrying flows - must include flow for <u>entire</u> tributary area for each design point.
         4) Irrigation ditch flows
   2. Runoff (developed)
      a. Off-site and site for as many design points as required.
         1) Separate time of concentration (T_c) for each design point (Rational Method).
         2) Runoff coefficient or permeability coefficient from Table 3-1
         3) Existing drainage facilities carrying flows - must include flow for <u>entire</u> tributary area for each design point.
   3. Detention (or other water quality control measure)
      a. Storage volumes - 10-year and 100-year; release rates - 10-year and 100-year.
      *b. Pond outlet - control structures
         1) Outlet control structure type
         2) Use appropriate outlet discharge calculations - include offsite flows
         3) Consider head at entrance
         4) Provide excess capacity for grates
         5) Use restrictor plate to reduce inlet area when necessary
         6) Compute outlet velocity and provide energy dissipator if velocity exceeds maximum permissible channel velocity
         7) Check excess storm water passage effects
      *c. Size outlet structures for parking areas
      *d. Depths of ponding in parking areas, durations of storage for each storm
   *4. Streets
      a. Compute depths and velocity of flow, for initial & major storm.
      b. Inlet capacities and depths at inlet
   *5. Open Channel Flow
      a. Roughness coefficient
      b. Trickle channel
      c. Depth and velocity for initial and major storms
      d. Channel protection
      e. Minimum freeboard
      f. Hydraulic grade lines
   *6. Storm Sewers and Culverts
      a. Culvert capacity using standard nomographs (Figures 7-2, 7-3, and 7-4 from Section 7)
      b. Storm sewer capacity at each design section
      c. Inlet capacity

d. Flow depth or headwater depth at inlet
e. Drops
f. Weirs
g. Streets, gutters, and crosspans
h. Energy dissipators
i. Hydraulic grade lines
j. Minimum and maximum velocities

B. Drainage Plan Sheets

1. Site Location Map - a portion of U.S.G.S. quad map or similar which shows:
a. Major drainage basin
b. Subbasin boundaries and acreage
c. Floodway and floodplain area
d. Site location

2. Site Drainage Plan - Show the following:
a. Existing and proposed 2-foot contours based on USGS datum (existing contours to extend at least 50' beyond property line.)
b. Location and elevation of USGS benchmarks or benchmarks referenced to USGS.
c. Existing and proposed property lines.
d. Present drainage easements.
e. Street names and grades.
f. Right-of-way and easement requirements.
g. Routing and accumulative flows at the upstream and downstream ends of the site and at various critical points on-site for both initial and major storm run-off.
*h. Finished floor elevations for protection from major storm run-off.
*i. Street cross-sections showing the 100-year flood levels.
j. Existing and proposed flood plains and major channels.
k. Detention pond design and drawings including the following:
1) Location of each detention area
2) Release rates for 10 year and 100 year storms
3) Storage volumes required and provided.
*4) Site plan on 1" = 50' scale or larger with two-foot contour intervals.
*5) Inlet and outlet structure design details and operational data (10 year and 100 year depths).
*6) Emergency excess storm water passage design details.
*7) Side slopes.
*l. Open channel flow in major channels shall be provided with the following information for both storms:
1) Profiles showing existing and proposed channel grades and water surface profiles.
2) Cross-sections on 100-foot stations showing existing and proposed cross sections and required rights-of-way.
3) Locations and size of all existing and proposed structures.
4) Locations and profiles of adjacent utilities.
5) Typical channel section and lining details.
*m. Outlet structure

*n. All hydraulic structure details in conformance with design.

o. Seal by professional engineer licensed to practice in Colorado.

p. Required Notes on Drainage Plan:

1) No building, structure, or fill will be constructed in the detention areas and no changes or alterations affecting the hydraulic characteristics of the detention areas will be made without the approval of the Director of Engineering.
2) Maintenance and operation of the detention areas will remain the responsibility of the property owner. Inspection and maintenance frequencies shall follow the recommendations noted in the Urban Storm Drainage Criteria Manual (USDCM) Vol. 3 Chapter 6, published by Urban Drainage and Flood Control District (aka Mile High Flood District). If the property owner fails in this responsibility, the Town has the right to enter the property, maintain the detention areas, and require reimbursement for the costs that are incurred.
3) Detention volumes and all related drainage appurtenances (including basin boundaries) shall be determined and certified by a registered engineer prior to issuance of the certificate of occupancy for any structure on the site or in the development.

C. Other

1. Substantiating documents relating to discussions or agreements with local jurisdictions, utility companies, irrigation companies or local property owners.
2. Operation and Maintenance Plan for the detention facility, or other permanent water quality control measure, prepared in accordance with USDCM Vol.3, Chapter 6, including (but not limited to) information on access, actions to be taken for maintenance, and inspection frequency.
3. Storm Detention and Infiltration Design Data Sheet. (State regulations require that all detention ponds be registered. The registration is to ensure that all detention ponds are releasing the detained runoff within the standard 72-hours. Please fill out and submit a Stormwater Detention and Infiltration Design Data Sheet, which can be downloaded at the following web site: https://maperture.digitaldataservices.com/gvh/?viewer=cswdif)

   Include the Date Design Sheet with Pond Certification turned into the Town.

2.3 Construction Drawings

Where drainage improvements are to be constructed in accordance with the approved Final Drainage Report, the construction plans (on 24" x 36" mylar) shall be submitted for review and approval prior to construction. A reproducible mylar copy of the approved plans will be submitted to the Town for file. The plans and specifications for the drainage improvements will include:

1. Storm sewers, inlets, outlets and manholes with pertinent elevations, dimensions, type, and horizontal and vertical control indicated.

2. Culverts, end sections, and inlet/outlet protection with dimensions, type, elevations,

and horizontal and vertical control indicated.

3. Channels, ditches, and swales (including side/rear yard swales) with lengths, widths, cross-sections, and erosion control (i.e. riprap, concrete, grout) indicated.

4. Checks, channel drops, and erosion control facilities.

5. Detention pond grading, trickle channels, outlets, and landscaping.

6. Other drainage related structures and facilities (including underdrains and sump pump lines).

7. Maintenance access considerations.

8. Overlot grading and erosion and sedimentation control plan (refer to Chapter 9).

The information required for the drawings and specifications shall be in accordance with sound engineering principles, this Storm Drainage Design Criteria, and the Town requirements for subdivision designs. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the storm drainage facility. The approved Final Drainage Plan shall be included as part of the construction documents for all facilities affected by the drainage plan. Construction drawings shall be signed by a registered professional engineer as being in accordance with the County approved drainage report/drawings.

2.4 "As-Built Drawings"

As constructed plans ("As-Builts") for all public improvements shall be attested to by a professional engineer registered in Colorado and submitted to the Town before the Town will accept the improvements.

The engineer shall include the following statement on the "As-Built" plans:

"I hereby declare that: I have performed a field review of the constructed drainage facilities on this plan, the facilities conform reasonably well to the approved drainage plan, appear to have been constructed in a workmanlike manner, and appear to be adequate for the intended purpose".

____
Registered P.E., State of Colorado
No. __________

The "As-Built" plans must be on file before a Certificate of Occupancy will be issued. The engineer will obtain the Town's original recorded mylar sepia of the approved Drainage Plan Sheet and place the certification and notations required for the "As-Built" drainage plan. The document will then be returned to the Town for filing as the "As-Built" drainage plan.

SECTION 3 - STORM RUNOFF

Two hydrological models can be used to predict storm runoff in the Town of Windsor: the Rational Method and the Colorado Urban Hydrograph Procedure (CUHP). These methods are presented in this section.

3.1 Storm Frequency

Both the initial and major storm shall be taken into consideration in predicting storm runoff for the preliminary and final drainage reports. Historic (existing development) and developed runoffs shall be determined for both storms for the site including the entire basin intercepted by the site.

3.1.1 Initial Storm

The initial storm occurs at fairly frequent intervals. The drainage system for the initial storm is intended to minimize inconvenience, protect against minor damage, and reduce maintenance costs. The design storm frequency interval for the initial storm shall be the 2-year storm.

3.1.2 Major Storm

The major storm occurs less frequently and can have critical impact on land development. The drainage system for the major storm is intended to protect against loss of life or substantial property damage. The design storm frequency interval for the major storm shall be the 100-year storm.

3.2 Rainfall Intensity

Runoff for both the initial and major storm shall be based on the Rainfall Intensity - Duration Curves for Windsor, Colorado shown in Figure 3-1.

3.3 Runoff Computations

Both the initial and major storm runoff quantities shall be computed using the Colorado Urban Hydrograph Procedure for basins 160 acres or larger. The rational method may be used for basins less than 160 acres.

3.3.1 Rational Method

The rational method is based on the rational formula:

$$Q = CIA$$

where Q = discharge, cfs
C = runoff coefficient
I = rainfall intensity, in/hr
A = area, acres

The runoff coefficient, C, used in the Rational Formula shall be based on land use as given in Table 3-1.

Table 3-1
RUNOFF COEFFICIENTS FOR RATIONAL METHOD

		Runoff Coefficients			
	Percent	**Storm Frequency**			
Surface Characteristics	**Impervious**	**2**	**5**	**10**	**100**
Business:					
Commercial Areas	95	.87	.88	.90	.93
Neighborhood Areas	65	.60	.65	.70	.80
Residential:					
Single-Family	40	.40	.45	.50	.70
Multi-Unit (detached)	50	.50	.55	.60	.75
Multi-Unit (attached)	70	.65	.70	.70	.80
1/2 Acre Lots or Larger	30	.30	.40	.45	.65
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.75	.80	.80	.85
Heavy Areas	90	.80	.80	.85	.90
Parks, Cemeteries	7	.15	.25	.35	.60
Playgrounds	13	.20	.30	.40	.70
Schools	50	.50	.55	.60	.75
Railroad Yard Areas	40	.40	.45	.50	.70
Undeveloped Areas:					
Historic Flow Analysis, Greenbelts, Agriculture, Natural Vegetation, Clayey Soils, Sandy Soils	2	.10	.20	.30	.60
Streets:					
Paved	100	.87	.88	.90	.93
Gravel	40	.40	.45	.50	.60
Drives and Walks	96	.85	.87	.90	.92
Roofs	90	.80	.85	.90	.90
Lawns, Sandy Soil	0	.00	.10	.20	.50
Lawns, Clayey Soil	0	.10	.20	.30	.60

The rainfall intensity, I, is determined using the Rainfall Intensity-Duration Curve in Figure 3-1. The time of concentration must first be determined to use this curve. The time of concentration is also known as "inlet time" and represents the time for a unit of water to travel from the most remote portion of the basin to the design point. A separate time of concentration is necessary for the overall basin and each sub-basin or design point. The time of concentration consists of the overland flow time and the channel or conduit flow time:

$$T_c = t_o + t_t$$

where T_c = time of concentration, minutes (five minutes minimum)
t_o = overland flow time, minutes
t_t = channel or conduit flow time, minutes

The overland flow time, t_o, is computed using the formula shown below. This formula, using the 5 year coefficient factor, shall be used to obtain the rainfall intensity for <u>all</u> frequencies of storms. The minimum time of concentration shall be assumed to be five minutes.

$$t_o = \frac{1.8(1.1 - C_5)L^{1/2}}{(S)^{1/3}}$$

where: t_o = time of overland flow, minutes
L = distance of overland flow, feet (not to exceed 400')
S = slope of basin, percent
C_5 = runoff coefficient for five year storm

or, for developed watersheds only,

$$t_o = (L/180) + 10$$

whichever is less.

The channel or conduit flow time (t_t) is to be determined from the velocity of flow computed for the hydraulic properties of the channel, ditch, gutter, pipe or sewer. Manning's equation for channel flow is useful for these calculations. Figure 3-2 may be used to determine the velocity in the following formula for t_t:

$$t_t = \frac{L}{60V}$$

Where: L = distance of flow in hydraulic structure, feet
V = velocity of flow, fps

3.3.2 Colorado Urban Hydrograph Procedure

Runoff for basins 160 acres or larger must be determined using the Colorado Urban Hydrograph Procedure. This procedure is detailed in the Urban Flood Control Drainage Design Criteria Manual. The incremental design rainfall for use with the CUHP method is given in Table 3-2.

Table 3-2
INCREMENTAL RAINFALL DEPTH FOR WINDSOR, COLORADO (INCHES)

	Design Storm				
Time (min)	**2-Year** 1-hr depth = 0.95	**5-Year** 1-hr depth = 1.35	**10-Year** 1-hr depth = 1.63	**50-Year** 1-hr depth = 2.28	**100-Year** 1-hr depth = 2.58
5	0.019	0.027	0.033	0.030	0.026
10	0.038	0.050	0.060	0.080	0.078
15	0.080	0.118	0.134	0.114	0.119
20	0.152	0.207	0.245	0.182	0.207
25	0.238	0.338	0.408	0.342	0.362
30	0.133	0.176	0.196	0.570	0.646
35	0.060	0.078	0.091	0.274	0.362
40	0.048	0.059	0.070	0.182	0.207
45	0.029	0.049	0.062	0.114	0.160
50	0.029	0.049	0.052	0.114	0.129
55	0.029	0.041	0.052	0.073	0.103
60	0.029	0.041	0.052	0.073	0.103
65	0.029	0.041	0.052	0.073	0.103
70	0.019	0.041	0.052	0.054	0.052
75	0.019	0.034	0.052	0.054	0.052
80	0.019	0.030	0.041	0.041	0.031
85	0.019	0.030	0.031	0.041	0.031
90	0.019	0.030	0.031	0.032	0.031
95	0.019	0.030	0.031	0.032	0.031
100	0.019	0.020	0.031	0.032	0.031
105	0.019	0.020	0.031	0.032	0.031
110	0.019	0.020	0.031	0.032	0.031
115	0.010	0.020	0.028	0.032	0.031
120	0.010	0.018	0.021	0.032	0.031
TOTALS	1.104	1.886	1.886	2.636	2.988

3.4 Offsite Flows

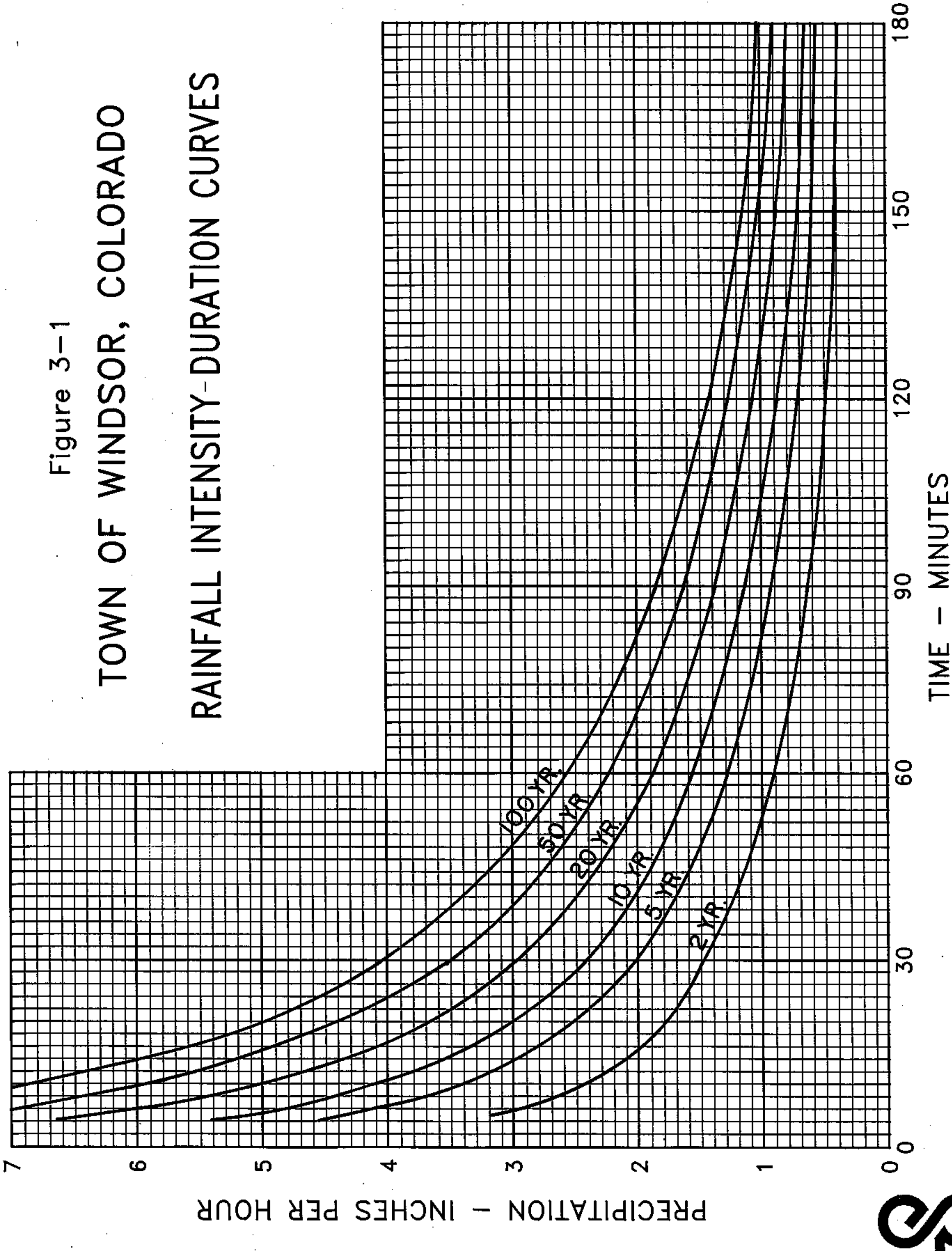
Flows entering the proposed development from outside the property are offsite flows. The offsite storm runoff shall be determined and included in the drainage system design.

Available drainage reports for offsite developed areas effecting the property shall be reviewed and considered in the drainage system planning and design.

Runoff entering the site from offsite shall be computed using the runoff coefficients based upon existing development or, for undeveloped land, based on values in Table 3-1 for offsite flow analysis, whichever is greater.

3.5 Irrigation Ditches

Irrigation ditches frequently intercept natural drainage. However, they may or may not be able to adequately convey runoff because their capacity is dependent on the operation of control structures. Considering an irrigation ditch as part of the drainage system may result in redirection of storm waters possibly resulting in damage to downstream property owners. For the purposes of routing the initial and major storms, irrigation ditches shall be assumed to be flowing full at all sections and to not intercept any storm drainage.



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SECTION 4 - STREETS

This section should be used to evaluate the allowable drainage encroachment on public streets. Planning submittals will be reviewed by the Town of Windsor for compliance with the criteria herein.

This criteria is based on the idea that although streets are an integral part of the urban drainage system, their primary function is to carry traffic. The efficiency and safety of traffic flow and the serviceability of streets is influenced by storm drainage. Sheet flow, gutter runoff, ponding, lateral flow across lanes, storm duration, and physical condition of the pavement can all have an impact on how well traffic functions. These factors should be controlled to within acceptable limits so that the traffic flow is minimally impacted.

4.1 Design Criteria

Grade, initial storms, and major storms are the factors which set forth the minimum design criteria for the runoff in urban streets as follows.

4.1.1 Grade

The minimum gutter grade shall be 0.4%. The maximum gutter grade shall be such that the average flow velocity does not exceed 10 feet per second unless site conditions govern otherwise.

The cross-slope on all streets shall be a minimum of 2.0% and may vary from 2.0% to 4.0%. The street and gutter section having the most restrictive capacity shall be used for design. Gutter flow design capacities shall utilize the reduction factor in Figure 4-2.

At the intersection of two streets, any variation in grade shall be governed by the characteristics of the drainage, the traffic patterns for that intersection, and the design requirements for streets as set forth by the Town of Windsor.

4.1.2 Allowable Capacity - Initial Storms

The determination of the street runoff carrying capacity shall be based on the following procedure:

1. Compute the theoretical flow conditions for pavement encroachment.
2. Apply a reduction factor to the theoretical flowrate to allow for field conditions (see Section 4.1.2.3 Allowable Gutter Flow).

4.1.2.1 Street Encroachment

The following criteria should be used regarding the allowable street encroachment from the initial storm for streets in the Town of Windsor. A storm drainage system shall begin where the encroachment reaches these limits:

Local:
1. No curb topping - where no curbing exists, encroachment shall not extend over property lines.
2. Flow may spread to crown of street.

Collector:
1. No curb topping - where no curbing exists, encroachment shall not extend over property lines.
2. Flow spread must leave at least one lane width free of water.

Arterial:
1. No curb topping - where no curbing exists, encroachment shall not extend over property lines.
2. Flow spread must leave at least one-half of roadway width free of water in each direction.

4.1.2.2 Theoretical Capacity

Once the allowable pavement encroachment has been established, theoretical gutter capacity for the initial storm shall be computed using the following revised Manning's equation for flow in shallow triangular channels:

$$Q = \frac{0.56\, Z\, S^{1/2}\, y^{8/3}}{n}$$

where
- Q = theoretical gutter capacity, cfs
- y = depth of flow at face of gutter, feet
- n = roughness coefficient
- S = channel slope, ft/ft
- Z = reciprocal of cross slope, ft/ft

A nomograph based on the previous equation has been developed and is included in Figure 4-1. The graph is applicable for all gutter configurations. An "n" value of 0.016 should be used for all calculations involving street runoff.

4.1.2.3 Allowable Gutter Flow

In order to calculate the actual allowable flow rate for the initial storm, the theoretical capacity shall be multiplied by a reduction factor. These factors are determined by the curve in Figure 4-2 entitled "Reduction Factors for Allowable Gutter Capacity", which relates the reduction factor to the slope of the gutter.

4.1.2.4 Cross Street Flow

The allowable cross street flow for the initial storm is shown in Table 4-1. Both the theoretical and allowable cross street flow shall be determined by the methods described in the preceding sections. However, the gutter slope variable should be replaced with the cross street water surface slope.

Table 4-1
ALLOWABLE CROSS STREET FLOW FOR THE INITIAL STORM

Street Classification	Design Runoff for Initial Storm
Local and Collector	Where crosspans are allowed, depth of flow shall not exceed six inches.
Arterial	None

4.1.3 Allowable Capacity - Major Storms

The determination of the allowable street flow due to the major storm shall be based on the following criteria:

1. Compute the theoretical capacity based on allowable depth and inundated area.
2. Apply a reduction factor to the theoretical flowrate to allow for velocity conditions.

4.1.3.1 Street Encroachment

Specific criteria regarding the allowable street encroachment from a major storm for streets in the Town of Windsor are as follows:

Local and Collector:

1. Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line unless buildings are flood proofed.
2. The depth of water over the crown shall not exceed six inches.

Arterial:

1. Residential dwellings, public, commercial, and industrial buildings shall not be inundated at the ground line unless buildings are flood proofed. Depth of water at the street crown shall not exceed six inches to allow operation of emergency vehicles. The depth of water over the gutter flowline shall not exceed 12 inches. In some cases, the 12 inch depth over the gutter flowline is more restrictive than the six inch depth over the street crown. For these conditions, the most restrictive of the two criteria shall govern.

These guidelines should also be used in order to determine allowable street ponding of storm water from a major storm.

4.1.3.2 Theoretical Capacity

Once the allowable street encroachment has been established for the major storm, theoretical runoff capacity shall be computed using Manning's equation as follows:

$$Q = \frac{1.486\, R^{2/3} S^{1/2} A}{n}$$

where
- Q = capacity, cfs
- n = roughness coefficient
- R = hydraulic radius, A/P, feet
- S = slope, ft/ft
- A = area, ft^2

Appropriate "n" values are located in Table 4-2.

Table 4-2
MANNING'S ROUGHNESS COEFFICIENTS FOR DRAINAGE SURFACES

Surface	Roughness Coefficient
Gutter and Street	0.016
Dry Rubble	0.035
Mowed Kentucky Bluegrass	0.035
Rough Stony Field with Weed	0.040
Sidewalk and Driveway	0.016

4.1.3.3 Allowable Gutter Flow

The theoretical capacity for the major storm must be reduced by a reduction factor in order to obtain the actual allowable flow rate. The procedures and criteria are the same as those found in Section 4.1.2.3 "Allowable Gutter Flow".

4.1.3.4 Cross Street Flow

The criteria in Table 4-3 should be used to determine the allowable cross street flow for the major storm. Both the theoretical and allowable cross street flow shall be determined by the methods described in the preceding sections. However, the gutter slope variable should be replaced with the cross street water surface slope.

Table 4-3
ALLOWABLE CROSS STREET FLOW FOR THE MAJOR STORM

Street Classification	Design Runoff for Major Storm
Local (includes places, alleys, marginal access and collector)	12 inch depth above gutter flow line
Arterial	6 inches or less over crown

4.2 Drainage Facilities at Intersections

4.2.1 Storm Sewer Inlets

Any storm sewer inlet located in an intersection shall be designed so that the encroachment of gutter flow on the intersection does not exceed the specified encroachment for that street and design storm as described in Sections 4.1.2.1 and 4.1.3.1, "Street Encroachment" for initial and major storms, respectively.

4.2.2 Drop Inlet Culverts

When sufficient grade is available, a drop inlet culvert may be used to transport runoff under a street when a storm sewer system is not justified. The culvert must be designed to handle sufficient runoff so that the encroachment of runoff on the intersection is limited to that allowed for the street.

4.2.3 Crosspans

Crosspans may be installed to transport runoff across local streets where storm sewer systems are not justified. Crosspans shall be a minimum of eight feet wide. Larger widths may be required by the Director of Engineering. The minimum grade on crosspans shall be 0.5% at the flowline of the pan. No crosspans are allowed on arterial or collector streets except in extreme cases when approved by the Director of Engineering. When used on arterial or collector streets, minimum width shall be 10 feet. Crosspan approach design must be reviewed and approved by the Town of Windsor. Covered crosspans, notched crosspans and bubblers will not be allowed.

SECTION 5 - STORM SEWERS

The criteria presented in this section shall be used in the design and evaluation of the storm sewer systems for the Town of Windsor as well as the basis for the review of all planning submittals.

The term storm sewer refers to the underground system of inlets, pipes, manholes or junctions, outlets and other appurtenant structures designed to carry storm runoff to major drainageways. A storm sewer system is required where the allowable street capacities to carry initial and major storm runoff are exceeded (see SECTION 4 - STREETS).

The storm sewer system demands attention from the engineer and the public because its primary function is to collect and convey regularly recurring storm runoff with minimal inconvenience and damage to the public. The storm sewer system must be designed to minimize the nuisances of regularly recurring storms.

Capacities of storm sewers shall be computed using Manning's equation unless the storm sewer is designed for pressure flow. Storm sewers with pressure flows shall be designed to withstand the forces of such pressure in accordance with the appropriate standards. The hydraulic gradient shall be calculated for each storm sewer system.

The placement of storm inlets shall be determined by a thorough analysis of the drainage area and streets involved. These inlets shall be located where sump (low-spot) conditions exist or where allowable street encroachments are exceeded.

5.1 Design Criteria

5.1.1 Design Storm Frequency

The storm sewer system is required when the allowable street capacity for storm runoff is exceeded and shall be designed for the 2-year storm interval.

5.2 Storm Sewer Pipe

5.2.1 Pipe Materials

Pipe materials shall be determined based on abrasion, strength requirements, other appropriate field conditions, and soil and water conditions. The selected materials will be reviewed by the Director of Engineering.

Concrete pipe shall be used underneath street pavement sections within the right-of-way. Other approved pipe materials may be used under driveways, through other drainageways, and open space areas. Corrugated metal pipe shall not be used when seasonal or continuous flows of groundwater are anticipated.

In some cases, the Director of Engineering may require a soils report documenting the minimum resistivity, pH, sulfate content, and chloride content of the soil. If there is reason to believe the stormwater flowing through the pipe is corrosive, an analysis of the stormwater should also be submitted.

5.2.2 Pipe Size and Strength

The minimum allowable pipe diameter is 12 inches for main trunks and laterals. The minimum inside dimension is 12 inches for elliptical and arch pipe. The conduit shall be of sufficient structural strength to withstand the AASHTO HS-20-44 loading. The structural design shall conform to those methods and criteria recommended by the manufacturer of the pipe material being used and for the conditions found at the installation site.

5.2.3 Roughness Coefficient

The following roughness coefficients, "n", should be used for pipes in Manning's equation:

Reinforced Concrete:	n = .013
Corrugated Metal:	n = .024
PVC:	n = .010

The average flow velocity in any conduit should not be less than 2.0 feet per second for the initial storm.

5.2.4 Alignment Conflicts with Other Utilities and Ditches

5.2.4.1 Water Mains

If alignment conflicts arise between storm sewers and water mains, the water mains are usually relocated. If relocation does not occur, the minimum clearance between storm sewers and water mains shall be 18 inches whenever possible.

If storm sewers are above or less than 18 inches below water mains or within a horizontal distance of 10 feet, they shall consist of a 20 foot section of structural sewer pipe, centered over or under the water main. The connecting joints shall be encased. If structural pipe is not used, storm sewers 12 inches in diameter shall be fully encased and those greater than 12 inches shall have all joints encased for the 20 foot section. Crossings shall meet all Colorado State Department of Health criteria. Corrugated metal pipe shall be fully encased for all sizes for the conditions described above.

Encasements shall consist of a six inch thick, reinforced concrete collar which extends 12 inches on either side of the joint. The minimum reinforcement shall be #4 bars, continuous, placed at each corner of the section tied with #3 bars at three foot centers.

5.2.4.2 Other Utilities

The minimum clearance between storm sewers and other utilities shall be 12 inches unless one of the pipes is encased, in which case the minimum clearance shall be six inches.

5.2.4.3 Ditch Crossings

Prior to crossing any irrigation ditch, written approval must be obtained from the ditch company. Ditch crossings shall be constructed in accordance with Ditch Crossing Detail included in the Appendix.

5.2.4.4 Backfill and Other Protection

To prohibit settling or failure of either pipe system, suitable backfill materials, backfill compaction, and other protection deemed necessary by the Director of Engineering shall be provided. Typical bedding requirements are shown in the Bedding Requirements Details in the Appendix.

5.2.5 Requirements for Grates for Pipes

Grates may be required over pipe inlets and outlets where danger exists because of a drop in elevation along a sidewalk or road, a siphon, or open pipes at playgrounds, parks, or residential areas. Grates will generally not be required for most culverts through embankments and crossing streets.

Grates shall meet the following requirements:

1. Grates shall be constructed with minimum 5/8" diameter steel bars. Reinforcing bars shall not be used.
2. Bar spacing shall be six inches unless site conditions are prohibitive.
3. All exposed steel shall be galvanized in accordance with AASHTO M111.
4. Welded joints shall be galvanized with a rust preventive paint.
5. Grates shall be secured to the headwall or end section by removable devices such as bolts or hinges to allow maintenance access, prevent vandalism, and prohibit entrance by children.

5.3 Rational Method for Sizing Storm Sewer System

The method presented in this section is from the Urban Storm Drainage Criteria Manual. The following step by step procedure should be used with Figure 5-1. This procedure is for the average situation and variations will often be necessary to fit the actual field conditions.

Column 1: Determine and enter design point location

Column 2: List basins contributing to this point which have not previously been analyzed.

Column 3: Enter length of flow path between previous design point under consideration.

Column 4: Determine the inlet time for the particular design point. For the first design point on a system the inlet time will be equal to the time of concentration. For subsequent design points, inlet time should also be tabulated to determine if it may be of greater magnitude than the accumulated time of concentration from upstream basins. If the inlet time exceeds the time of concentration from the upstream basin, and the area tributary to the inlet is of sufficient magnitude, the inlet time should be substituted for time of concentration and used for this and subsequent basins. See the runoff part of this criteria for methods of determining inlet time.

Column 5: Enter the appropriate flow time between the previous design point and the design point under consideration. The flow time of the street should be used if a significant portion of the flow from the basin is carried in the street.

Column 6: Pipe flow time should generally be used unless there is significant carryover from above basins in the street.

Column 7: The time of concentration is the summation of the previous design point time of concentration and the intervening flow time.

Column 8: Rational Method Runoff Coefficient, "C", for the basins listed in Column 2 should be determined and listed. The "C" value should be weighted if the basins contain areas with different "C" values.

Column 9: The intensity to be applied to the basins under consideration is obtained from the Rainfall Intensity-Duration Curves. The intensity is determined from the time of concentration and the frequency of design for this particular design point.

Column 10: The area in acres of the basins listed in Column 2 is tabulated here. Subtract ponding areas which do not contribute to direct runoff such as rooftop and parking lot ponding area.

Column 11: Direct runoff from the tributary basins listed in Column 2 is calculated and tabulated here by multiplying Columns 8, 9, and 10 together.

Column 12: Runoff from other sources, such as controlled releases from rooftops, parking lots, base flows from groundwater, and any other source, are listed here.

Column 13: The total of runoff from the previous design point summation plus the incremental runoff listed in Columns 11 and 12 is listed here.

Column 14: The proposed street slope is listed in this column.

Column 15: The allowable capacity for the street is listed in this column. Allowable capacities should be calculated in accordance with procedures set forth in Section 4, "Streets".

Column 16: List the proposed pipe grade.

Column 17: List the required pipe size to convey the quantity of flow necessary in the pipe.

Column 18: List the capacity of the pipe flowing full with the slope expressed in Column 16.

Column 19: Tabulate the quantity of the pipe flowing full with the slope expressed in

Column 16.

Column 20: List the actual velocity of flow for the volume of runoff to be carried in the street.

Column 21: List the quantity of flow determined to be carried in the pipe.

Column 22: Tabulate the actual velocity of flow in the pipe for the design Q.

Column 23: Include any remarks or comments which may effect or explain the design. The allowable quantity of carryover across the street intersections should often be listed for the initial design storm. When routing the major storm through the system, required elevations for adjacent buildings can often be listed in this column.

5.4 Storm Inlets

Storm inlets shall be installed where sump (low spot) conditions exist or where allowable street runoff capacities are exceeded. Typical details showing curb, gutter, sidewalk, and various storm inlets are included in the Appendix.

Inlets are classified as being either "continuous grade" or "sump". "Continuous grade" refers to an inlet located where the grade of the street has a continuous slope past the inlet, and adjacent ponding does not occur. "Sump" refers to the inlet where water has been restricted in a low point, which can occur at a change in grade of a street or at an intersection due to crown slope of a cross street.

All curb openings shall be constructed with the opening at least two inches below the flow line elevation. The minimum transition length shall be 3'-6".

Reduction factors must be applied to the theoretical inlet capacity, because collection of debris, pavement overlays, parked vehicles, and other factors decrease inlet capacity.

The outlet pipe of the storm inlet shall be sized on the basis of the theoretical capacity of the inlet, with a minimum diameter of 12 inches.

5.4.1 Theoretical Inlet Capacity

The theoretical capacity of inlets in a low point or sump shall be determined from Figures 5-2 and 5-3.

The theoretical capacity of curb openings on a continuous grade shall be determined from Figures 5-4, 5-5, and 5-6.

The standard curb-opening is illustrated by Figure 5-4 and is defined as having a gutter downstream from the opening, has a depression depth (a) equal to W/12 feet at the curb face, and a curb opening height (h) of at least 0.5 feet. The graph as presented by Figure 5-5 is based on a depression apron width (W) equal to 2 feet and depression width (a) equal to 2 inches. The pavement cross-section is straight to the curb face; however, a street section with gutters cross-sloped steeper than the street can also be analyzed using Figure 5-6. Since the figures are based on an inlet opening free of obstructions, the reduction factors discussed previously shall be utilized.

5.4.2 Inlet Capacity Reduction Factors

Table 5-1 should be used to determine the reduction factor to apply to the theoretical inlet capacity. The modification results in a more realistic inlet capacity considering debris clogging, parked vehicles, pavement overlays, etc.

Table 5-1
INLET CAPACITY REDUCTION FACTORS

Drainage Condition	Inlet Type	Percentage of Theoretical Capacity
Sump or Continuous Grade	CDOH Type R-Curb Opening:	
	5'	80%
	10'	85%
	15'	90%
Parking Lots, Medians	Area inlet	80%

5.5 Manholes

Manholes shall be placed wherever there is a change in size, elevation, or slope; abrupt change in direction; where there is a junction of two or more systems or laterals; or when the maximum allowable spacing has been reached. Typical manhole details are included in the Appendix. Table 5-2 lists the maximum allowable manhole spacing for storm sewers.

Table 5-2
MAXIMUM ALLOWABLE MANHOLE SPACING

Vertical Pipe Dimension (inches)	Maximum Allowable Distance Between Manholes and/or Cleanouts
15 to 36	400 feet
36 to 60	500 feet
60 and larger	750 feet

5.5.1 Barrel Size

The interior diameter of all "straight through" storm sewer manholes is specified in Table 5-3.

Table 5-3
MANHOLE BARREL DIAMETER

Horizontal Pipe Dimension (inches)	Minimum Barrel Diameter (feet)
15-24	4
27-42	5
Greater than 42	6

SECTION 6 - OPEN CHANNELS

This section presents the technical criteria for the hydraulic evaluation and design of open channels. The hydraulics of an open channel can be complex, encompassing many different flow conditions from steady state uniform flow to unsteady, rapidly varied flow. Channels should be designed so that critical depth flows or supercritical flows are avoided.

Channels can be natural or artificial (man-made) and can be lined or unlined. Natural open channels are those that have been created as a result of the natural erosion process. Artificial open channels are constructed to direct the flow of water and include irrigation and drainage ditches and canals. Natural channels may require modification if carrying storm runoff from an urbanized area to assure stabilization against erosion. Artificial channels must also be designed for stability against erosion. Unlined channels are technically lined with vegetation. Lined channels are lined with concrete, rock, or synthetic fabric.

All open channels shall be designed to carry the major storm runoff (100-year recurrence interval). Lined channels shall be utilized when flow hydraulics, topography, or right of way limitations control.

6.1 Unlined Channels

Unlined channels shall be used when hydraulics, topography, and right of way limitations so permit.

6.1.1 Flow Computation

Flow computations shall assume uniform flow and utilize the following Manning's equation:

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

where
- Q = flow quantity, cfs
- n = roughness coefficient
- R = hydraulic radius, feet
- S = channel bottom slope, ft/ft
- A = cross-sectional area, ft^2

The minimum roughness coefficients to be used for unlined channels are as follows:

Table 6-1
MINIMUM ROUGHNESS COEFFICIENTS FOR UNLINED CHANNELS

Type of Channel	Coefficient
Earthen, straight and uniform, no brush or debris	
a. Grasses	
Depth of Flow $\leq$ 2.0 ft.	0.060
Depth of Flow > 2.0 ft.	0.035
b. Earth Bottom with Riprap Sides	0.040

6.1.2 Side Slopes

Unlined channel side slopes shall be a maximum of 4:1 on all improvements subject to future maintenance by the Town. Any slopes steeper than this are not permissible unless stabilization is used, which is subject to approval by the Director of Engineering.

6.1.3 Depth of Flow

The maximum depth of flow for an unlined channel shall be 4.0 feet. The critical depth shall be determined for both the major and initial storms in order to assure that super critical flow does not occur. The minimum amount of freeboard shall be 1.0 foot or additional capacity for 1/3 of the design flow.

6.1.4 Horizontal Curves

Curved, unlined channels shall not have a centerline radius of less than 100 feet or twice the top width at the design flow. Slope protection shall be provided if necessitated by the hydraulics of the channel.

6.1.5 Channel Slopes and Velocities

Unlined channel slopes shall be constructed so that flow velocities do not exceed 7.0 feet per second during the major storm nor less than 2.0 feet per second for the initial storm. Drop structures may be used to control the grade in order to meet these limits. For naturally lined channels (grass cover), flow velocities shall not exceed 7.0 feet per second in the presence of erosion resistant soils and 5.0 feet per second in the presence of easily eroded soils. Velocities exceeding 5.0 feet per second should only be used where good covers and proper maintenance can be obtained.

6.1.6 Trickle Channels or Underdrain Piping

Unlined channels shall be designed with trickle channels or underdrain piping to carry low flows. The capacity of such devices shall be sufficient to carry 0.5 - 1.0%

of the major storm runoff. Trickle channels shall be stabilized to prevent erosion damage and should have a natural, meandering appearance. Grass should not be used as a lining for trickle channels that are continuously flowing. When concrete trickle channels are used, they shall be shaped as shown in the Open Channel Detail in the Appendix.

6.2 Lined Channels

Where conditions for unlined channels cannot be met, open channels shall be lined.

6.2.1 Flow Computation

Flow computations shall assume uniform flow and utilize the following Manning's equation:

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

where Q = flow quantity, cfs
n = roughness coefficient
R = hydraulic radius, feet
S = channel bottom slope, ft/ft
A = cross-sectional area, ft^2

Figure 6-2 lists the minimum roughness coefficients to be used for lined open channels.

Table 6-2
MINIMUM ROUGHNESS COEFFICIENTS FOR LINED OPEN CHANNELS

Lining	Roughness Coefficient
Riprap*	$0.0395(d_{50})^{1/6}$
Grouted riprap	0.023 - 0.030
Wire enclosed rock	0.035
Concrete	0.013
Float finish	0.015
Unfinished	0.017
Concrete bottom with sides of:	
Grouted riprap	0.020
Riprap	0.030

* Equation from the Urban Storm Drainage Criteria Manual. Does not apply to very shallow flow (hydraulic radius less than or equal to two times the maximum rock size) where the coefficient will be greater than indicated by the formula. d_{50} = the mean stone size in feet.

6.2.2 Freeboard and Water Surface Rise in Curved Channels

A minimum of one foot of freeboard shall be incorporated into major channels. Small channels shall have either one foot of freeboard or additional capacity of one-third of the design flow.

The design of lined channels on bends or curves shall take into consideration the centrifugal and gravitational forces on the flow within the channel section. The following equation from the Army Corps of Engineers' publication "Hydraulic Design of Flood Control Channels", July, 1970, should be used to determine the water surface rise:

$$y = \frac{CV^2 W}{gr}$$

where
- y = rise in water surface between a theoretical level water surface at the center line and outside water surface elevation (superelevation), feet
- C = coefficient (see following tabulation)
- V = mean channel velocity, ft/sec
- W = channel width at elevation of centerline water surface, feet
- g = acceleration due to gravity, ft/sec^2
- r = radius of channel centerline curvature, feet

The total rise in water surface due to superelevation and standing waves can be computed using the values for "C" listed below:

Table 6-3
SUPERELEVATION FORMULA COEFFICIENTS

Flow Type	Channel Cross Section	Type of Curve	Value of C
Tranquil	Rectangular	Simple Circular	0.5
Tranquil	Trapezoidal	Simple Circular	0.5
Rapid	Rectangular	Simple Circular	1.0
Rapid	Trapezoidal	Simple Circular	1.0
Rapid	Rectangular	Spiral Transitions	0.5
Rapid	Trapezoidal	Spiral Transitions	1.0
Rapid	Rectangular	Spiral Banked	0.5

6.2.3 Stability of Lined Channels

All lined channels shall be protected from hydrostatic uplift forces by the use of either drain piping, weep holes, or appropriate footings.

Flow at the Froude number near 1.0 is unstable and should be avoided.

If supercritical flow is unavoidable, all concrete channel sections shall be continuously reinforced, both longitudinally and laterally.

When lined channels with high velocity flows enter unlined channels with subcritical flow, a structure for the purpose of dissipating energy shall be required.

A combination of channel stabilization measures may be utilized if acceptable hydraulic conditions exist, subject to approval by the Director of Engineering. Concrete, gabions, slope mattresses, riprap, and other approved measures can be used. Gabions, slope mattresses, and riprap smaller than 12 inches shall either be buried on maintainable slopes (4:1) or grouted to prevent vandalism.

6.2.3 Miscellaneous Lined Channel Details

Concrete lining shall be finished, as close as possible, to the degree of roughness used in the design channel.

Lined channels must have the bottom sloped so that the flow is channelized towards the centerline of the channel.

6.3 Irrigation Ditches

Irrigation ditches shall not be used as outfall points for initial or major drainage systems unless such use is shown to be without unreasonable hazard or excessive future maintenance costs substantiated by thorough hydraulic engineering analysis. Neither the concept of the irrigation ditch's acceptance of "historical runoff" or the approval by the particular ditch company or authority of a drainage outfall point into the irrigation system shall be considered sufficient justification to waive the requirement of a thorough hydraulic analysis.

Required ditch flows shall be determined by existing water rights below the design points and documentation of the water rights flowing across and below the property shall be submitted with the hydraulic analysis for approval.

Such hydraulic analyses must obtain written acceptance by both the Director of Engineering and the appropriate ditch authority prior to the approval of utility plans incorporating an irrigation ditch into the storm drainage system.

Any information that is part of the permanent records in the Director's office pertaining to previous analysis or studies of the irrigation ditches may be obtained upon request by any interested party.

SECTION 7 - CULVERTS

The design of culverts shall follow the procedures presented in this section. The review of all planning submittals will be based thereon.

Culverts are closed conduits that allow for the passage of water under an embankment such as a road, railroad, or canal. Culverts differ from sewers in length (culverts are generally much shorter) and in the way they collect water. Flows enter culverts via an open channel located at a similar elevation, and flows enter sewers via storm inlets from above. The geometry of the inlet plays a major role in determining the capacity of the culvert.

7.1 Design Criteria

7.1.1 Sizing Culverts

Factors to be considered in the sizing of a culvert include:

- minimum allowable culvert diameter
- calculated design flow quantities
- allowable cross street flow for the street under which it passes

The minimum allowable culvert diameter shall be 15 inches. When an elliptical or arch pipe is used, the minimum inside dimension shall be 12 inches.

The appropriate storm should be used in computing the flow quantities that will be routed through the culvert. Culverts passing under arterial streets shall have sufficient capacity to carry the 100 year storm. Culverts passing under collector and local streets shall be designed to carry the 10 year storm. Refer to Section 7.2 for procedures for determining culvert capacity.

When the flow in the open channel exceeds the capacity of the culvert and overtops the street, it must not exceed the allowable cross street flow for the major storm which is defined in SECTION 4 - STREETS and in Table 4-3, Allowable Cross Street Flow for the Major Storm. If the flow quantities exceed the capacity of the culvert and the street, then the size of the culvert must be increased until the criteria is met.

7.1.2 Structural Requirements

The structural design of culverts shall conform to those methods and criteria recommended by the manufacturer for that culvert type and for the conditions found at the installation site. The minimum standards set forth in the current American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges shall also be adhered to.

For large culverts or where groundwater is a problem, the design shall include necessary provisions to resist hydrostatic uplift forces that could result in failure of the structure.

7.1.3 Velocity

In the design of culverts, both the minimum and maximum velocities must be considered.

7.1.3.1 Minimum Velocity

A minimum velocity is required to provide self-cleansing in the culvert. A minimum velocity of two feet per second is recommended.

7.1.3.2 Maximum Velocity

Maximum velocity also effects the design of culverts; it is controlled by the depth of the maximum allowable headwater, and it is evaluated in selecting adequate channel protection at the outlet.

The velocity of the water passing through the culvert increases with an increase in headwater. Headwater can cause excessive velocities if ponding occurs above the entrance to the culvert or if the culvert is relatively deep compared to the street above. To assure safety and stability of the channel, limits are set on the headwater (HW) as defined in Table 7-1. These limits control the maximum velocity that occurs in the culvert.

Table 7-1
MAXIMUM HEADWATER:DIAMETER RATIOS (HW/D)

Storm Frequency	HW/D
10 year	$\leq$1.0
100 year	$\leq$1.5

Ponding above the entrance to a culvert can cause property or roadway damage, culvert clogging, saturation of fills, detrimental upstream deposits of debris, or inundation of existing or future facilities. Such ponding shall not be allowed. In some cases, the limitations defined in the above table will help control nuisance ponding. This chart should not be used in evaluating the outlet from a designated detention facility.

At the culvert outlet, channel protection is required because of the effects of the eddy currents. The amount of protection depends on the maximum outlet velocity. The higher the outlet velocity, the more the protection. The selection of adequate channel protection is presented in Section 8, Erosion and Sediment Control.

7.1.4 Inlet and Outlet Structures

For public roads and driveway culverts larger than 15 inches, culverts must be designed with protection at the inlet and outlet areas.

All inlet structures shall be designed to minimize entrance losses and should be fitted with a headwall with wingwalls or a flared end section.

The outlet area shall also include a headwall with wingwalls or an end section in addition to the necessary channel protection.

7.2 Culvert Hydraulics

This section presents the procedures for determining the headwater and culvert capacity for inlet and outlet conditions. It must be determined which condition controls for both major and initial storm runoff. The condition which results in the greatest headwater depth governs. The use of Figure 7-1 will facilitate the evaluation of culvert capacity.

7.2.1 Inlet Control Conditions

Under inlet control conditions, the cross-sectional area of the culvert, the inlet geometry, and the headwater are the factors that effect culvert capacity. The slope of the culvert is assumed to be steep enough so that the culvert does not flow full and tailwater does not effect the flow.

There are two types of inlet control conditions:

1. The unsubmerged condition in which the headwater is not sufficient to submerge the top of the culvert, the culvert inlet slope is supercritical, and the culvert acts like a weir.

2. The submerged condition in which the headwater submerges the top of the culvert, but the pipe does not flow full, and the culvert acts like an orifice.

The capacity of a culvert in either the unsubmerged (HD<1) or submerged condition shall be determined using Figure 7-2. Figure 7-2 includes two nomographs which were empirically developed by the Bureau of Public Roads and the Federal Highway Administration for circular concrete and corrugated metal pipe.

7.2.2 Outlet Control Conditions

Outlet control will govern if the tailwater is deep enough, the culvert slope sufficiently flat, and the culvert sufficiently long. The factors that effect the capacity of the culvert in outlet control include the inlet geometry and associated losses, the culvert material with friction losses, and the tailwater condition.

There are three types of outlet control culvert flow conditions:

1. The culvert top and outlet are submerged by the headwater and tailwater, and the culvert flows full.

2. The top of the culvert is submerged by the headwater, but the outlet is unsubmerged by the tailwater.

3. The headwater is insufficient to submerge the top of the culvert, the culvert slope is subcritical, and the tailwater depth is lower than the pipe critical depth.

By combining Bernoulli's Equation (conservation of energy) with equations for inlet losses and friction losses, the following equation results for determining the capacity of a culvert for outflow control conditions:

$$H = \frac{(K_e + 1 + 29n^2L/R^{1.33})V^2}{2g}$$

where
- H = total energy head, feet
- K_e = entrance loss coefficient (see Table 7-5)
- n = Manning's coefficient (see Tables 7-2,3,4)
- L = length of culvert, feet
- R = hydraulic radius, feet
- V = velocity of flow, fps
- g = gravitational constant, ft/sec^2

This equation can be used when the culvert is flowing full under either condition #1 or #2 above. If the culvert is not flowing full at the outlet under conditions #2 or #3 above, the hydraulic grade line at the outlet is approximated by averaging the critical depth and the culvert diameter, which is used if the value is greater than the tailwater depth (TW) to compute headwater depth (HW). Figures 7-3 and 7-4 can be used to determine head for circular concrete and corrugated steel pipe.

The following Tables 7-2 through 7-5 shall be used with the above equation.

Table 7-2
MANNING'S "n" VALUES FOR CORRUGATED STEEL PIPE

Corrugations	Annular 2 2/3" x 1/2"	Helical 1 1/2" x 1/4"		Helical 2 2/3" x 1/2"				
	All Diam.	8"	10"	12"	18"	24"	36"	48"
Unpaved	.024	.012	.014	.011	.014	.016	.019	.024
25% Paved	.021					.015	.017	.020
Fully Paved	.012					.012	.012	.012

Corrugations	Annular 3" x 1" All Diam.	Helical - 3" x 1" 36"	48"	54"	60"	66"	72"
Unpaved	.027	.021	.023	.023	.024	.025	.026
25% Paved	.023	.019	.020	.020	.021	.022	.022
Fully Paved	.012	.012	.012	.012	.012	.012	.012

Table 7-3
MANNING'S "n" VALUES FOR STRUCTURAL PLATE METAL PIPE

Corrugations 6" x 2"	Diameters 5 ft	7 ft	10 ft	15 ft
Plain - unpaved	.033	.032	.030	.028
25% Paved	.028	.027	.026	.024

Table 7-4
MANNINGS "n" VALUES FOR PIPE/CULVERT MATERIALS

Material	Capacity Calculation	Velocity Calculation
Concrete (newer pipe)	.013	.011
Concrete (older pipe)	.015	.012
Plastic (smooth)	.011	.009
CSP (annular 3"x1" corrugations)	.027	.024

Table 7-5
ENTRANCE COEFFICIENTS, K_e, FOR PIPE CULVERTS
FOR USE WITH OUTLET CONTROL HEAD EQUATION

Type of Entrance	Entrance Coefficient, K_e
Headwall	
Grooved edge	0.20
Rounded edge, (0.15D radius)	0.15
Rounded edge, (0.25D radius)	0.10
Square edge, (cut concrete and CMP)	0.40
Headwall and 45^{o} singwall	
Grooved edge	0.20
Square edge	0.35
Headwall with parallel wingwalls spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting entrance	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping entrance	
Mitered to conform to slope	0.70
Flared end section	0.50

7.3 Box Culverts

Box culverts may be utilized where their use can be economically justified. A detailed hydraulic analysis shall be performed for box culverts using similar procedures as for pipe culverts, but with the appropriate box culvert nomographs ("Hydraulic Charts for the Selection of Highway Culverts", HEC-5, December, 1965, available from the Federal Highway Administration) and coefficients (Table 7-6). The following criteria is unique to box culverts and should be followed in their design. Prior to construction of any box culvert, all hydraulic computations shall be submitted for approval.

Table 7-6
ENTRANCE COEFFICIENTS, K_e, FOR CONCRETE BOX CULVERTS FOR USE WITH OUTLET CONTROL HEAD EQUATION

Type of Entrance	Entrance Coefficient, K_e
Headwall parallel to embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square edged at crown	0.70

7.3.1 Free Flowing Conditions

When box culverts are to be considered free flowing conduits, they shall be designed so that the flow does not reach the box ceiling.

All box culverts that are considered free flowing shall be designed with a minimum of one foot of freeboard.

Box culverts under free flow conditions shall be designed and analyzed the same as open channels by using Manning's equation. Entrances shall be designed to assure that proper design flows are maintained.

7.3.2 Encompassing Flows

Box culverts may be designed so flows may encompass the entire culvert section. For such a case, the culvert capacity shall be determined using either inlet or outlet control where applicable. Measures shall be taken to prevent structural failure due to surcharging or pressure head.

7.3.3 Other Details

Minimum velocities should not be less than two feet per second. The maximum velocity shall not be restricted provided that adequate erosion control and energy dissipation is provided.

Backwater should be considered for all box culvert conditions.

All curves, bends, and transitions shall be analyzed in detail to insure proper flow. Access manholes and/or inlets shall be installed to prevent air entrapment as well as allowing surface drainage to enter the culvert. The characteristics of the individual system shall govern the spacing of the box culverts; however, spacing shall be such to allow no more than a three foot difference in hydraulic grade. Where velocities are in excess of 20 feet per second, a volume swell of 20% due to air entrainment shall be included in the analysis.

All box culverts shall be protected from hydraulic uplift by the use of drain piping or weep holes. Access must be provided for maintenance, including access for vehicles for large box culverts.

SECTION 8 – DETENTION

The criteria presented in this section shall be used in the planning and design of detention facilities in the Town of Windsor. The review of all planning submittals will be based on these criteria, along with any additional criteria as required by the Water Quality Control Division, Colorado Department of Public Health and Environment (CDPHE).

Detention facilities are intended to store the excess storm water associated with increased imperviousness of a basin due to development activities, and discharge it at a reduced rate. Designing these facilities to meet water quality standards will also reduce stormwater pollution.

There are two types of detention: local and regional. Local detention serves the onsite area within the boundaries of the development. Local detention typically occurs in parking lots, rooftops, or small grassy areas (although the area may be large for a large development). Regional detention serves both onsite and offsite areas in and around the development. It often occurs as part of a park system or greenbelt area. All detention areas that collect runoff from offsite areas are considered to be regional detention.

8.1 General Storage Requirements

Requirements for detention of storm water runoff shall be based on the development's location within its major drainage basin as determined by hydrologic routing analysis.

If the development is in an area that has a master drainage plan, the stormwater shall be released in accordance with it. In cases where the master drainage plan has been approved, but improvements to provide adequate outfall facilities to protect downstream property have not been constructed, additional detention may be required until the outfall has been constructed.

In basins where a master drainage plan has not been approved, the Town may require detention to protect irrigation structures or downstream facilities. The detention should meet the criteria of this section.

Should a release rate greater than that specified above be desired, the design engineer must carefully analyze the downstream conditions and show that no adverse effects will occur. This analysis shall include any and all information required by the Town. All pertinent calculations (such as volume and peak discharge) shall be submitted for review. Approval by the Director of Engineering is necessary before the detention facility for the development site is designed.

Examples of when detention requirements may be varied include: new developments which will decrease the percentage of impervious area, sites which are adjacent to major outfalls and their peak runoff will not increase the peak runoff of the outfalls, and situations where detention requirements have already been met through previous phases of the development. Any variances shall be approved by the Director of Engineering, and these areas must be thoroughly analyzed to show that no hazards will be created downstream.

Any embankment constructed that will result in a surface area, volume, and/or embankment height as specified in Colorado Revised Statutes 37-87-105, shall require the approval of the State Engineer's Office. All detention storage areas shall be designed and constructed in compliance with current state statutes and/or the criteria presented in this manual.

8.2 Design Frequency

All detention facilities and outlet structures shall be designed for the 2-year and 100-year storm frequencies. A typical subdivision drainage system with onsite detention requirements would consist of flow in the storm sewer and allowable flow in the gutter, which combined would carry the flows from the "minor" storm. These flows would be detained to release at the historic 2-year flow rate. Flows from the larger storms would be conveyed by street surface flow, larger storm sewer systems or an open channel with capacity for the "major" flood. As the storm intensity increases (i.e. 10-year and 100-year storms), the onsite detention would reduce the developed flood peaks to undeveloped levels or less, with the 100-year developed flows being reduced to the 10-year historic flow. During calculation of the major storm runoff, the benefits of upstream onsite detention can be accounted for during the routing of flood peaks through the development.

8.3 Design Criteria for Sizing Local and Regional Detention Ponds

8.3.1 Over-detention Requirement

The Town has adopted a Master Drainage Plan for all of the Urban Growth Management Area. The Master Drainage Plan requires that over-detention be implemented as a part of any new development. The detention shall be designed such that the maximum release rate for the 100-year storm in the developed condition shall not exceed the flow rate for the 10-year storm in the historic condition. The allowable discharge rate may be computed by using the formula:

$$Q_{max} = .24A$$

Where Q_{max} = the maximum allowable detained release rate
A = tributary area in acres

However, in any areas identified by the Town Stormwater Master Plan as having required release rates that are less than 10-year historic, the more restricted rate will apply.

Detention volume requirements shall be determined by the FAA method, SWMM modeling, or other routing methods approved by the Town.

In cases where downstream facilities are not capable of conveying discharges at the 10-

year historic flow rate, additional restrictions on allowable release rates may be necessary. These will be reviewed on a case-by-case basis and allowable release rates will be set as necessary to insure safe conveyance of the design discharges.

8.3.2. Water Quality Design

Water quality treatment shall be included in the design of all new detention facilities in accordance with the criteria provided in the Urban Storm Drainage Criteria Manual Vol III. Water quality capture volume shall be included in addition to the volume required for detention.

All new development and redevelopment projects in the Town of Windsor are required to meet requirements outlined in the CDPHE General Permit COR090000 associated with Municipal Separate Storm Sewer Systems. Specifically, projects that will result in land disturbance greater than or equal to one acre, or are less than one acre of disturbance but part of a larger common plan of development will require water quality treatment.

The Water Quality Capture Volume (WQCV) standard and the regional WQCV are the standard water quality control measures to be used on development projects in Windsor. The designer or applicant shall use criteria from Urban Storm Drainage Criteria Manual Vol III, Chapter 3 for calculation of the WQCV using a 40 hour drain time. Urban Drainage and Flood Control District spreadsheets may be used for calculation of the WQCV as well as associated orifice plate design. If site constraints do not facilitate the use of WQCV standards, one of the other approved control measures in the CDPS General Permit COR090000 (MS4 permit) may be used.

8.3.2.1 Water Quality Design Exclusions

In rare cases, and as allowed by the MS4 permit, a site may be excluded from the requirement to provide water quality treatment for stormwater run-off. To qualify for any exclusion, the designer or applicant will note the specific exclusion as detailed below, provide the specifics for any exclusion claimed, and indicate the impervious acreage on the drawing sets submitted for review to the Town of Windsor.

(A) Pavement Management Sites: Sites, or portions of sites, for the rehabilitation, maintenance, and reconstruction of roadway pavement, which includes roadway resurfacing, mill and overlay, white topping, black topping, curb and gutter replacement, concrete panel replacement, and pothole repair. The purpose of the site must be to provide additional years of service life and optimize service and safety. The site also must be limited to the repair and replacement of pavement in a manner that does not result in an increased impervious area and the infrastructure must not substantially change. The types of sites covered under this exclusion include day-to-day maintenance activities, rehabilitation, and reconstruction of pavement. “Roadways” include roads and bridges that are improved, designed or ordinarily used for vehicular travel and contiguous areas improved, designed or ordinarily used for pedestrian or bicycle traffic, drainage for the roadway, and/or parking along the roadway. Areas primarily used for parking or access to parking are not roadways.

(B) Excluded Roadway Redevelopment: Redevelopment sites for existing roadways, when one of the following criteria is met:

1) The site adds less than 1 acre of paved area per mile of roadway to an existing roadway, or

2) The site does not add more than 8.25 feet of paved width at any location to the existing roadway.

(C) Excluded Existing Roadway Areas: For redevelopment sites for existing roadways, only the area of the existing roadway is excluded from the requirements of an applicable development site when the site does not increase the width by two times or more, on average, of the original roadway area. The entire site is not excluded from being considered an applicable development site for this exclusion. The area of the site that is part of the added new roadway area is still an applicable development site.

(D) Aboveground and Underground Utilities: Activities for installation or maintenance of underground utilities or infrastructure that does not permanently alter the terrain, ground cover, or drainage patterns from those present prior to the construction activity. This exclusion includes, but is not limited to, activities to install, replace, or maintain utilities under roadways or other paved areas that return the surface to the same condition.

(E) Large Lot Single Family Sites: A single-family residential lot, or agricultural zoned lands, greater than or equal to 2.5 acres in size per dwelling and having a total lot impervious area of less than 10 percent. A total lot imperviousness greater than 10 percent is allowed when a study specific to the watershed and/or MS4 shows that expected soil and vegetation conditions are suitable for infiltration/filtration of the WQCV for a typical site, and the permittee accepts such study as applicable within its MS4 boundaries. The maximum total lot impervious covered under this exclusion shall be 20 percent.

(F) Non-Residential and Non-Commercial Infiltration Conditions: This exclusion does not apply to residential or commercial sites for buildings. This exclusion applies to applicable development sites for which post-development surface conditions do not result in concentrated stormwater flow during the 80th percentile stormwater runoff event. In addition, post-development surface conditions must not be projected to result in a surface water discharge from the 80th percentile stormwater runoff events. Specifically, the 80th percentile event must be infiltrated and not discharged as concentrated flow. For this exclusion to apply, a study specific to the site, watershed and/or MS4 must be conducted. The study must show rainfall and soil conditions present within the permitted area; must include allowable slopes, surface conditions, and ratios of impervious area to pervious area; and the permittee must accept such study as applicable within its MS4 boundaries.

(G) Sites with Land Disturbance to Undeveloped Land that will Remain Undeveloped: Permittees may exclude sites with land disturbance to undeveloped land (land with no human-made structures such as buildings or pavement) that will remain undeveloped after the site.

(H) Stream Stabilization Sites: Permittees may exclude stream stabilization sites.

(I) Trails: Permittees may exclude bike and pedestrian trails. Bike lanes for roadways are not included in this exclusion, unless attached to a roadway that qualifies under another exclusion in this section.

(J) Oil and Gas Exploration: Permittees may exclude facilities associated with oil and gas exploration, production, processing, or treatment operations, or transmission facilities, including activities necessary to prepare a site for drilling and for the movement and placement of drilling equipment, whether or not such field activities or operations may be considered to be an applicable construction activity.

8.4 Grading, Depth, Freeboard, and Trickle Flow Requirements

The banks of any detention facility shall not be steeper than 4:1 (horizontal: vertical) to allow for access for maintenance vehicles. Measures shall be taken to control standing water in the pond site and to control nuisance flows. Minimum slopes of 1.0% for grassed and pavement surfaces and 0.4% for curb and gutter surfaces are recommended in detention ponds.

The maximum depth from water surface to outlet invert for parking lot detention is six inches for the 10-year design and 18 inches for the 100-year design. The maximum depth from water surface to outlet invert for rooftop detention is three inches. There is no maximum depth for grassed detention facilities as governed by the Colorado Dam Safety Program.

The minimum required freeboard for grassed and parking lot detention facilities is one foot above the computed 100-year water surface elevation. The minimum required freeboard for rooftop detention is three inches above the computed 100-year water surface elevation.

To carry low flows, detention ponds shall be designed with trickle channels or underdrain piping. Trickle channels shall be stabilized to prevent erosion damage. Grass should not be used as a lining for trickle channels that are continuously running water. If concrete trickle channels are used, they shall be at least four feet wide.

8.5 Embankment Protection

Whenever an embankment is used to contain water, it shall be protected from catastrophic failure due to overtopping. Overtopping can result when outlets are obstructed or when a greater than 100-year storm occurs. Embankment protection shall be provided in the form of erosion protection on the downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. All spillways shall have a concrete weir. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway should be set equal to or above the 100-year water surface elevation.

SECTION 9 – EROSION AND SEDIMENT CONTROL

All applicable development involving soil disturbance submitted to the Town's Planning Department for review shall include adequate erosion and sediment controls. The Phase II Municipal Separate Storm Sewer (MS4) permit issued to the Town by the Colorado Department of Public Health and Environment (CDPHE), Water Quality Control Division (Division) requires the Town to implement a program for Construction Sites activities within the MS4 permit area. Adequate erosion and sediment control measures are necessary to reduce the negative impact development can have on land water resources_during and after construction. All grading, excavations, open cuts, and other land disturbances shall be adequately protected from wind and stormwater erosion. NOTE: "Best Management Practices" or "BMPs" have traditionally been used in reference to the structures and practices used for erosion and sediment control. CDPHE is now using "Control Measures" or "CMs" in place of this terminology as it has a broader application and is inclusive of all of the structures and practices used for erosion and sediment control. These terms will be used interchangeably in this section.

9.1 Standard Practices for Construction

The following practices, as described in the Urban Drainage Flood Control District's "Urban Storm Drainage Criteria Manual, Vol. III, Stormwater Quality" are required for controlling erosion. Please note that this is not an all-inclusive list. Consult the Grading, Erosion and Sediment Control Plan specific to the project for any site specific details:

1. **It is better to minimize erosion than to rely solely on sedimentation removal from construction site runoff.** Erosion control BMPs limit the amount and rate of erosion occurring on disturbed areas. Sediment control BMPs attempt to capture the soil that has been eroded before it leaves the construction site. Despite the use of both erosion control and sediment control BMPs, some amount of sediment will remain in runoff leaving a construction site, but the use of a "treatment train" of practices can help to minimize offsite transport of sediment. The last line of treatment such as inlet protection and sediment basins should be viewed as "polishing" BMPs, as opposed to the only treatment on the site.

2. Install initial erosion and sediment control practices before construction begins. Promptly install additional BMPs for inlet protection, stabilization, etc., as construction activities are completed. Limit the amount of disturbed area at any given time on a site to the extent practical.

3. Protection of existing vegetation on a construction site can be accomplished through installation of a construction fence around the area requiring protection. In cases where upgradient areas are disturbed, it may also be necessary to install perimeter controls to minimize sediment loading to sensitive areas such as wetlands.

Existing vegetation may be designated for protection to maintain a stable surface cover as part of construction phasing, or vegetation may be protected in areas designated to remain in natural condition under post-development conditions (e.g., wetlands, mature trees, riparian areas, open space).

4. If trees are to be protected as part of post-development landscaping, care must be taken to avoid several types of damage, some of which may not be apparent at the time of injury. Potential sources of injury include soil compaction during grading or due to construction traffic, direct equipment-related injury such as bark removal, branch breakage, surface grading and trenching, and soil cut and fill. In order to minimize injuries that may lead to immediate or later death of the tree, tree protection zones should be developed during site design, implemented at the beginning of a construction project, as well as continued during active construction.

5. Temporary seeding can be used to stabilize disturbed areas that will be inactive for an extended period. Permanent seeding should be used to stabilize areas at final grade that will not be otherwise stabilized. Effective seeding includes preparation of a seedbed, selection of an appropriate seed mixture, proper planting techniques, and protection of the seeded area with mulch, geotextiles, or other appropriate measures.

6. Final stabilization practices for obtaining a vegetative cover should include, as appropriate: seed mix selection and application methods; soil preparation and amendments; soil stabilization practices (e.g., crimped straw, hydro mulch or rolled erosion control products); and appropriate sediment control BMPs as needed until final stabilization is achieved.

This is not an all-inclusive list of control measures. Applicable details, practices and specifications listed in Urban Drainage Flood Control District's "Urban Storm Drainage Criteria Manual, Vol. III, Stormwater Quality", Chapter 7, Sections 4-9 are adopted by reference and must be used in the preparation of construction plans. BMP details from Chapter 7 are to be included on construction drawings as applicable, with the following exceptions:

- EC-5 Filter Berms (CB): do not use
- SC-3 Straw Bale Barriers (SBB): do not use
- SC-4 Brush Barrier (BB): do not use
- SC-5 Rock Socks (RS): do not butt ends together; use the alternative installation method – overlap ends 12"
- SC-6 Straw Bale Inlet Protection (IP-6): do not use
- SC-10 Chemical Treatment (CT): do not use
- SM-4 Vehicle Tracking Control (VTC): proprietary devices not listed in this specification will be considered by the Town

Please note that crushed concrete is not allowed as VTC material, staging area stabilization material, or CWO traction material.

New methods and control measure devices are occasionally introduced to the market. Use of a new control measure not listed in the Urban Storm Drainage Criteria Manual will be considered with the presentation of specifications, history of use and any other technical details requested by the Stormwater Program Coordinator.

9.2 Riprap

Riprap (angular rock) or other materials shall be used to reduce erosion along channel banks, in channel beds, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosion is likely to occur.

This section outlines one method for sizing riprap to be used for channel linings. This criteria can be used as a guide, however, alternate design methods may be used, if approved by the Town Engineer. More detailed information about the design method for channel linings as well as other structures such as erosion control for culverts, can be found in the Urban Storm Drainage Criteria Manual, Vol. I (Revised August 2018). All riprap shall adequately protect stream banks, roads, utilities, and other structures against collapse due to soil erosion.

Channel linings constructed with riprap, grouted riprap, or gabions should only be used for subcritical flow conditions where the Froude number is 0.8 or less. If the Froude number is greater than 0.8, the channel shall be lined with concrete. The Froude number for channels is defined as:

$$N_f = V/(gD_m)^{0.5}$$

where N_f = Froude number
V = average channel velocity, fps
G = acceleration due to gravity, ft/sec^2)
D_m = mean depth of flow, feet

Several gradations of riprap are listed in Table 9-1. The minimum average size designation for loose riprap shall be 12 inches. Smaller sizes of riprap shall be either buried on side slopes which can be easily maintained (4:1 maximum slope) or grouted where slopes are steeper. Grouted riprap should meet all the requirements for regular riprap except that the smallest rock fraction (smaller than the 10 percent size) should be eliminated from the gradation. A reduction of riprap size by one size designation (such as from 18 inches to 12 inches) is permitted for grouted riprap.

Table 9-1
CLASSIFICATION AND GRADATION OF ORDINARY RIPRAP

Riprap Designation	Percent of Total Weight Smaller than the Given Size	Stone Size (in pounds)	d_{50}* (inches)
Class 6**	70-100	85	6
	50-70	35	
	35-50	10	
	2-10	<1	
Class 12	70-100	440	12
	50-70	275	
	35-50	85	
	2-10	3	
Class 18	100	1275	18
	50-70	655	
	35-50	275	
	2-10	10	
Class 24	100	3500	24
	50-70	1700	
	35-50	655	
	2-10	35	

* d_{50} = Mean particle size (at least 50 percent of the mass shall be stones equal or larger than this dimension).
** Bury on 4:1 side slopes or grout rock if slopes are steeper.

Table 9-2 lists riprap requirements for a stable channel lining based on the following relationship:

$$VS^{.017}/((d_{50})^{0.5}(S_s-1)^{0.66}) = 5.8$$

where V = mean channel velocity, fps
S = longitudinal channel slope, ft/ft
S_s = specific gravity of rock (minimum S_s = 2.50)
D_{50} = rock size for which 50 percent of the riprap is smaller by weight, feet

The rock sizing requirements in Table 9-2 are based on the rock having a specific gravity of 2.5 or more. Also, the rock size does not need to be increased for steeper channel side slopes, provided the side slopes are no steeper than 2:1. Rock lined side slopes steeper than 2:1 are not recommended.

Table 9-2
RIPRAP REQUIREMENTS FOR CHANNEL LININGS**

$VS^{.017}/(S_s-1)^{0.66}$*	Rock Type**
0 to 1.4	No riprap required
1.5 to 4.0	Class 6 riprap
4.1 to 5.8	Class 12 riprap
5.9 to 7.1	Class 18 riprap
7.2 to 8.2	Class 24 riprap

* Use S_s = 2.5 unless the source of rock and its densities are known at the time of design.
** Table valid only for Froude number of 0.8 or less and side slopes no steeper than 2:1.

The thickness of the rip rap layer should be at least 1.75 times d_{50} (at least 2.0 times d_{50} in sandy soils) and should extend up the side slopes at least one foot above the design water surface. At the upstream and downstream termination of the riprap lining, the thickness should be increased 50 percent for at least three feet to prevent undercutting. Where only the channel sides are to be lined, the riprap blanket should extend at least three feet below the existing channel bed and the thickness of the riprap layer underneath the channel bed increased to at least three times d_{50} to prevent undercutting.

Filter materials should be used underneath the riprap such as gravel bedding, a filter cloth, or a combination of both to protect channel embankment materials from washing out. Generalized filter material specifications are listed in Tables 9-3 and 9-4. Either a two layer filter (Type I topped by Type II) or a single 12-inch layer of Type II is required to protect fine grained soils. For coarse sand and gravel (50% or more retained on the #40 sieve), only the Type II filter is required. The Type I filter in Table 9-3 is similar to the Colorado Division of Highways concrete sand specification AASHTO M 6. The Type II filter is equivalent to the Colorado Division of Highways Class A filter material except that it permits a slightly larger maximum rock fraction.

Table 9-3
GRADATION FOR FILTER MATERIAL

	Percent Passing Square Mesh Sieves (by wt.)	
Sieve Size	Type I	Type II
3"	-----	90-100
1 1/2"	-----	-----
3/4"	-----	20-90
3/8"	100	-----
#4	95-100	0-20
#16	45-80	-----
#50	10-30	-----
#100	2-10	-----
#200	0-2	0-3

Table 9-4
THICKNESS REQUIREMENTS FOR FILTER MATERIAL

	Minimum Thickness (inches)		
	Fine Grained Soils*		CoarseGrained Soils**
Riprap Designation	**Type I**	**Type II**	**Type II**
Gabions, slope mattresses, Class 6 and Class 12 riprap	4	4	6
Class 18 and Class 24 riprap	4	6	8

* May substitute one 12 inch layer of Type II bedding.

** Fifty Percent or more by weight retained on the #40 sieve.

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