City of Brandon

Municipal Servicing Standards

Section 5

Land Drainage

Rev 00 (2025)

TABLE OF CONTENTS

TABLES

APPENDICES

LAND DRAINAGE SERVICING STANDARDS REVISION HISTORY

Municipal Servicing Standards (MSS) Sections may be reviewed, updated or otherwise modified at any time. The Proponent's Engineer shall ensure that the current version of the MSS Section is applied.

Where such alternative solutions, systems, or approaches are being considered, a written proposal outlining the benefits, limitations, and total cost of ownership of the proposed solution shall be submitted to the City of Brandon Engineering Department for formal approval.

The following table summarizes the revision history:

Table 1-1 – Revisions to MSS

Date	Modification or Adjustment
July 2025	Municipal Service Standards – Section 5 – Land Drainage

1.0 OBJECTIVE

This standard describes the design of stormwater collection, conveyance, storage and quality mitigation systems. These systems must be designed to provide safe, cost-effective handling of drainage from rainfall and snowmelt and reduce the potential for flooding damage to infrastructure and property and creating unsafe conditions for people and road traffic.

This standard is a living document that will evolve as design standards develop. Older standards were concerned mainly with stormwater conveyance, but newer and evolving standards will place a greater emphasis on on-site retention to reduce the size of receiving infrastructure and extend the design life of existing infrastructure, the use of natural processes to reduce maintenance requirements, improve stormwater quality and integration of stormwater facilities with parkland to improve quality of life and provide wildlife benefits.

This standard is presented with the following main subject areas:

- Design Parameters and Methods for drainage infrastructure
- Lot Grading and Surface Drainage, including roadway surface drainage and subdrainage
- Design of Major System Components, including channels, culverts and ponds and other storage infrastructure
- Design of Minor System Components, including pipes, catch basins, manholes, outfalls and related works
- Building Services
- (*future addition*) Low Impact Development

A separate guidance document entitled "City of Brandon Naturalized Stormwater Pond Guidelines" dated April 9, 2018, for the design of naturalized stormwater basins is available from the City of Brandon website at:

https://brandon.ca/what-is-happening-in-your-neighbourhood/completed-projects/naturalized-stormwater-retention-basin-s

1.1 Engineering Submissions

For all submissions and approvals required as part of a Proponent's project refer to Section 2 – Engineering Submission Standards.

2.0 DESIGN PARAMETERS AND METHODS

Stormwater systems shall be designed to convey runoff from a design rainfall event through both minor and major system components to an appropriate stormwater outlet. Both major and minor systems may be active during a rainfall event, however each is designed to protect the development for different volumes and intensities of rainfall.

The minor system is designed to convey runoff from lower magnitude, frequent rainfall events, and consists of sewers, catch basins, manholes, culverts, outlet structures and oil and grit separators. These common, or 'minor' rainfall events are expected to occur multiple times a year and the minor system is to convey runoff without impacting the functionality of the service area or development (no major overland flow or ponding).

The major system is designed to attenuate and convey runoff from larger magnitude, less frequent rainfall events that exceed the conveyance capacity of the minor system, and consists of storm drains, ditches, overflow channels, roadway gutters, retention ponds, detention ponds and naturalized stormwater ponds.

- During minor rainfall events (less than or equal to 20% annual probability of occurrence or 5-year return frequency), major system components will function as simple conveyance elements.
- During larger magnitude or 'major' rainfall events (greater than 20% annual probability of occurrence or 5-year year return frequency), major system components are expected to accommodate runoff by allowing water to attenuate in major system components, reducing the amount of conveyance required by minor system components.

2.1 Minor System

The minor system shall be designed to:

- Accommodate the 20% probability (1-in-5 year) design rainfall event.
- The peak hydraulic grade line (HGL) should be at least 300 mm below low ground elevation throughout the system.

2.2 Major System

The major system shall be designed to:

- Accommodate the 1% probability (1-in-100 year) design rainfall event.
- Attenuate runoff in major system elements on site, with ponding depth not exceeding:
  - Surface Storage: refer to Subsection 3.3 for parking lots and Subsection 3.4 for roadways.
  - Open channel systems: Top of sideslope for ditches or swales.
  - Ponds: refer to Subsections 4.3 for wet ponds, 4.4 for dry ponds and 4.5 for naturalized ponds.

- Convey runoff with flow depths not exceeding:
  - Top of curb for roadway gutters, or specified roadway encroachment limits (refer to Table 3-1)
  - Top of sideslope for ditches or swales, or ideally top of subgrade (bottom of granular road structure) for roadside ditches.
  - Public easements are required for major storm drainage overflow routes located on private property. A minimum easement width of 6.0 m is required.

2.3 Stormwater System Discharge Rates

Stormwater systems shall discharge to an appropriate downstream system at a rate:

- Not exceeding pre-development runoff rate for:
  - New closed conduit or open channel systems within City rights-of-way.
  - Existing closed conduit or open channel systems within City rights-of-way, where it can be shown that the existing drainage system has adequate capacity.
- Not exceeding Province of Manitoba requirements for:
  - Existing open channel systems (culverts, drains, coulees, highway and provincial road ditches) within or draining to Provincial rights-of-way. Designer should confirm Province of Manitoba requirements prior to commencing design.

2.4 Design Rainfall

Intensity-Duration-Frequency (IDF) curves based on Environment Canada's published IDF data for Brandon CDA station (Environment Canada Station 5010486, in service from 1960 to 1985) fit to a Gumbel Extreme Value Type 1 probability distribution are shown in Table 2-1 – IDF Equation Parameters. This family of IDF curves has historically been used in Brandon for the design of stormwater infrastructure and was developed as part of the 2003 Southeast Drainage Basin study. The City may consider adjustments to account for future climate change scenarios.

2.4.1 Intensity-Duration-Frequency (IDF)

Intensity-Duration-Frequency (IDF) curves are mathematical functions which represent the probability that a given rainfall intensity will occur within a given time duration based on historical extreme weather data.

The three parameter IDF equation is as follows:

Equation 2-1 – Intensity-Duration-Frequency (IDF) Equation

$$i = \frac{a}{(t+b)^c}$$

Where:

- i is rainfall intensity (mm/hr).
- t is time of concentration (minutes)
- a is a magnitude parameter estimated by curve fit (mm/hr).
- b is a time constant estimated by curve fit (minutes).
- c is an exponent estimated by curve fit (dimensionless).

IDF parameters based on historical Brandon CDA data for various design storms are found in Table 2-1, a summary of rainfall intensities and rainfall amounts for various times of concentration found in Table 2-2 and Table 2-3.

Table 2-1 – IDF Equation Parameters

IDF Parameter	**Units**	**Probability of Event**					
		50% (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
'a'	(mm/hr)	575.9	837.3	1009.6	1236.3	1406.1	1569.5
'b'	(min)	5.6	4.7	4.4	4.2	4.1	4.0
'c'		0.774	0.767	0.764	0.763	0.763	0.762

Table 2-2 – Summary of Design Rainfall Intensities (mm/hr)

Time		**Probability of Event**					
Mins	**Hrs**	**50%** (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
5	**0.083**	92.6	146.6	182.3	227.4	260.8	294.2
10	**0.167**	68.7	106.5	131.6	163.3	186.7	210.1
15	**0.25**	55.4	85.1	104.8	129.7	148.1	166.5
30	**0.50**	36.3	55.1	67.6	83.5	95.2	106.8
60	**1**	22.6	34.2	41.9	51.6	58.8	66.0
120	**2**	13.7	20.7	25.3	31.2	35.5	39.9
180	**3**	10.1	15.3	18.8	23.1	26.3	29.5
360	**6**	6.0	9.1	11.1	13.7	15.6	17.5
720	**12**	3.5	5.4	6.6	8.1	9.2	10.4
1440	**24**	2.1	3.2	3.9	4.8	5.5	6.1

Table 2-3 –Summary of Design Rainfall Depths (mm)

Time		Probability of Event					
Mins	Hrs	50% (1-in-2 year)	20% (1-in-5 year)	10% (1-in-10 year)	4% (1-in-25 year)	2% (1-in-50 year)	1% (1-in-100 year)
5	**0.083**	7.7	12.2	15.2	18.9	21.7	24.5
10	**0.167**	11.4	17.8	21.9	27.2	31.1	35.0
15	**0.25**	13.8	21.3	26.2	32.4	37.0	41.6
30	**0.50**	18.1	27.6	33.8	41.7	47.6	53.4
60	**1**	22.6	34.2	41.9	51.6	58.8	66.0
120	**2**	27.3	41.3	50.7	62.4	71.0	79.7
180	**3**	30.3	45.9	56.3	69.3	78.9	88.5
360	**6**	35.9	54.5	66.9	82.4	93.7	105.3
720	**12**	42.2	64.3	79.1	97.6	111.0	124.7
1440	**24**	49.5	75.8	93.4	115.2	131.1	147.4

2.4.2 Design Rainstorms

Artificial rainfall hyetographs for use in storage design and computer modelling shall use the “Chicago” design rainstorm. Design rainstorms are discretized to a 10-minute timestep and a using an assumed 33% storm advancement factor (meaning that the storm peak occurs at 33% of the storm duration). Design storms are truncated when the intensity drops below a threshold of 2.4 mm per hour. This lower threshold represents a single 0.2 mm tip from a metric rain gauge within a 5-minute duration (e.g. the smallest amount of rain that can be recorded within the smallest duration that is investigated).

10-minute discretized design rainstorms for Brandon are shown in Appendix 5A.

2.5 Calculation Methodology

The calculation methodology to be used by the designer will depend on the overall catchment size, complexity of the proposed drainage system, and the criticality of the downstream system:

Rational Method shall be used for:

- Developments with an overall catchment area of 20 ha or less.
- Developments with an overall catchment area of 20 ha or greater, in the conceptual or feasibility stages of design only.

Dynamic Stormwater Modelling shall be used for:

- Developments with an overall catchment area of 20 ha or greater.
- Developments with a catchment area of 20 ha or less and discharging to a critical existing LDS system as determined by the City.

- Critical existing LDS system refers to an existing LDS system that is known by the City to be at or near capacity and additional verification with a dynamic model is required to demonstrate the proposed development will not negatively impact the receiving system.

- Development that includes storage, including stormwater ponds, underground storage chambers or surface storage (parking lots, roofs).

The duration of the analysis should be sufficient to fully capture the entire rainstorm event and return of the system hydraulic grade line elevation to within 100 mm of the normal water level in retention ponds or detention pond bottom elevation.

2.5.1 Rational Method

The Rational Method calculates instantaneous peak flow for a given average rainfall intensity based on an estimated time of concentration and for a given catchment area and runoff coefficient.

Equation 2-2 - Rational Method Equation

$$Q = K_u C i A$$

Where:

- Q is the estimated flow (m^3/s)
- C is the Runoff Coefficient (dimensionless)
- i is Rainfall Intensity (mm/hr), estimated with the IDF equation
- A is Drainage Area (ha)
- K_u is a Unit Conversion Factor equal to 1 / 360 = 0.00278 (or 1.0 for Imperial units),

2.5.1.1 Runoff Coefficient

- Feasibility or Conceptual design stages may use typical Runoff Coefficients for common area types.
- Preliminary and Detailed design stages shall use composite runoff coefficients. Composite runoff coefficients shall:
  - be calculated for at least one representative subcatchment per area type, or for each subcatchment where the subcatchments do not follow a standard configuration.
  - be calculated using an area-weighted method:

Equation 2-3 – Composite Runoff Coefficient Area-Weighted Method Equation

$$C = \frac{\sum\left((C_1A_1) + (C_2A_2) \cdots (C_nA_n)\right)}{\sum(A_1 + A_2 \cdots A_n)}$$

Where:

- C is the Composite Runoff Coefficient (dimensionless)
- $C_1, C_2 \cdots C_n$ are the Runoff Coefficients for each surface type (dimensionless)
- $A_1, A_2 \cdots A_n$ are the Areas for each surface type (square meters or hectares)

Table 2-4 – Runoff Coefficients for Rational Formula

Area Type	**Runoff Coefficient, C**
Single Detached Residential Dwelling	0.50
Low Density Multi-Unit Dwellings (Duplex/Semi-detached)	0.60
Low Density Multi-Unit Dwellings (Row-house, townhouse, walk-up apartments)	0.70
Medium Density Multi-Unit Dwellings (Mixed Use Development)	0.75
High Density Multi-Unit Dwellings (Medium to High Rise Apartments)	0.85
Low / Medium Density Commercial (Service Stations, Convenience Stores, Motels, Medium Sized Hotels, Highway Commercial, etc.)	0.90
High Density Commercial (Shopping malls, Retail Centre's)	0.90
Industrial (Large Graveled lots)	0.70
Industrial (Large Paved Lots)	0.90
Institutional (Schools)	0.50
Parks, Cemeteries	0.20
Playgrounds	0.30
Unimproved Areas	0.20
Farmland	0.20
Surface Type	**Runoff Coefficient, C**
Roofs	0.95
Pavement (concrete or asphalt)	0.95
Pavement (paving stones)	0.85
Gravel	0.65
Grass (boulevards, lawns)	0.15
Grass (lawns, steep >7%)	0.20
Grass (side slopes, >15%)	0.25
Farmland (cultivated)	0.30
Natural prairie (grass)	0.20
Forested and brush areas	0.15

2.5.1.2 Time of Concentration

Time of Concentration is the time it takes runoff to travel from the hydraulically furthest point within the contributing catchment area to reach the design location. Determination of Time of Concentration requires estimation of two components, "Inlet Time" and "System Travel Time".

2.5.1.3 Inlet Time

Inlet Time is the time required for flow from the farthest point of a subcatchment to reach the point of inflow into a defined conveyance system (flow channel or LDS inlet). Inlet time should be calculated as the sum of continuous flow segments, including sheet flow and shallow concentrated flow.

- Sheet flow time of concentration shall be calculated using the Kinematic Wave Equation. This equation is iterative with the Rainfall IDF equation.
- Sheet flow length shall not exceed 100 m, beyond this surface runoff tends to concentrate and form small rills and gullies
- Beyond flow lengths of 100 m, the shallow concentrated flow equation shall be used.
- Minimum inlet time shall be 5 minutes. Computed times of concentration less than 5 minutes shall be set to 5 minutes.

Equation 2-4 – Kinematic Wave Equation

$$T_s = \frac{K_s n^{0.6} L^{0.6}}{i^{0.4} S^{0.3}}$$

Where:

- T_S is the sheet flow travel time (minutes)
- n is the Manning roughness coefficient for overland sheet flow (dimensionless) see
- Table 2-5
- i is the estimated rainfall intensity (mm/hr) computed using the Rainfall IDF equation
- L is the surface flow length (m) typically 100 m maximum before becoming shallow concentrated flow
- S is the ground surface Slope (as a fraction in m/m)
- K_S is a constant equal to 6.988 (or 0.939 for Imperial units)

* Note: This equation is iterative as rainfall intensity depends on time of concentration which is not known. An initial estimate of time of concentration is used and the calculation is repeated until the solution reaches equilibrium within a tolerance of 0.1 minutes or less.

Equation 2-5 – Shallow Concentrated Flow Equation (HEC-22)

$$T_{sc} = \frac{L}{K_{sc} k S^{0.5}}$$

Where:

- T_{SC} is the shallow concentrated flow travel time (minutes)
- k is an intercept coefficient (dimensionless) see
- Table 2-5
- L is the flow path length for concentrated flow (m)
- S is the ground surface Slope (as a fraction in m/m)
- K_{SC} is a constant equal to 1.0 (or 3.28 for Imperial units)

Table 2-5 – Shallow Concentrated Flow Intercept Coefficient

Surface Cover Type	Manning's Roughness Coefficient (n) for Sheet Flow	Intercept Coefficient (k) for Shallow Concentrated Flow
Pavement (concrete or asphalt)	0.015	0.619
Unpaved (gravel)	0.020	0.491
Grass (lawns)	0.250	0.457
Farmland (cultivated)	0.170	0.274
Natural prairie (grass)	0.150	0.213
Natural prairie (brush)	0.400	0.152
Forested areas	0.800	0.076

2.5.1.4 System Travel Time

Travel Time is the time required for flow to travel from the most upstream point of a conveyance to the downstream end or outlet of the conveyance.

- Manning's Equation shall be used to estimate average flow velocities in pipes and open channels.
- Travel time in closed conduits shall be based on full flow velocity for simplicity.

Equation 2-6 – Manning's Equation (Velocity form)

$$V = \left(\frac{k}{n}\right) R^{2/3} S^{1/2}$$

Where:

- V is the Velocity of flow (m/s)
- n is the Manning's roughness coefficient (dimensionless)
- R = A/W_P or the Hydraulic Radius (m)

- A is the Flow area (m^2)

- W_P is the Wetted Perimeter (m)
- S is the Slope of the hydraulic grade line (as a fraction in m/m)
- k is a Unit Conversion Factor equal to 1.0 (or 1.486 for Imperial units)

2.5.2 Computer Modelling

Large or complex drainage systems shall be modeled using EPA SWMM 5, Computational Hydraulics Inc. PCSWMM and/or Autodesk Storm and Sanitary Analysis (SSA) software. All the drainage components including surface runoff, inlets, outlets, conduits, ditches, culverts, storage systems and outfalls shall be modelled for both the minor and major systems. The model input files and the simulation results including background files shall be electronically submitted to the City for review and approval in a .zip archive format along with sketches, system maps, technical memos and other associated files.

2.6 Design Parameters

2.6.1 Gravity Flow: Minor System

Piped systems shall be designed using the Manning Equation, and assuming full pipe flow for all design conditions.

Equation 2-7 - Manning Equation (Flow Form)

$$Q = \frac{k}{n} AR^{2/3}S^{1/2}$$

Where:

- Q is the estimated Flow (m^3/s)
- n is the Manning roughness coefficient, dimensionless
- A is the Cross sectional area of flow (m^2)
- R is the Hydraulic radius (m)
- S is the Slope of hydraulic grade line (as a fraction in m/m)
- k is a Unit Conversion Factor equal to 1.0 (or 1.486 for Imperial units)

2.6.1.1 Sewer Velocity

Flow velocities within piped systems are limited to:

- Minimum Velocity:

  - 0.9 m/s for mainline sewer at full pipe flow, to allow for the resuspension and transport of settled solids

- Maximum Velocity:
  - 4.5 m/s for mainline sewer at full pipe flow for rainfall events up to the minor system design probability of event.
  - If velocities are higher during larger design rainfall events, designers shall provide design rationale and consider mitigation methods for pipe invert erosion and erosion of manhole interior walls.

2.6.1.2 Pipe Roughness

Manning's roughness coefficient for all sewer design and pipe material types shall be 0.013.

2.6.1.3 Sewer Slope

Minimum and maximum sewer slope shall be based on the minimum and maximum full flow velocity limits specified above.

Table 2-6 – Minimum and Maximum Sewer Slopes (n=0.013)

Pipe Diameter (mm)	Minimum Slope (%)	Maximum Slope (%)
300*	1.00	10.90
375	0.32	8.10
450	0.26	6.36
525	0.22	5.18
600	0.18	4,33
750	0.13	3.22
900 and larger	0.10	2.50

* Catch basin leads only, minimum size for mainline sewers is 375 mm.

Gravity flow for major system components (ditches, swales, channels, culvert groups) shall use the Manning equation.

2.6.1.4 Channel Velocity and Flow Depth

Maximum channel velocities shall be limited based on channel type, and channel material type:

Table 2-7 – Maximum Channel Velocities

Channel Lining Type	Maximum allowable velocity (m/s)	Maximum Flow Depth (m)	Governing Criteria
Culverts	Varies	85% of Diameter	Fish Passage, Channel bed erosion
Gravel surface	1.0	0.30	Surface erosion
Minor swales (lawns, grade breaks)	1.0	0.30	Channel bed erosion
Larger ditches and channels	1.5	Top of slope	Channel bed erosion

Channel Roughness

Table 2-8 – Channel Roughness

Channel Material	Manning's roughness coefficient
Culvert (concrete)	0.013
Culvert (corrugated metal)	0.024
Pavements, gutters	0.015
Bare soil	0.020
Vegetated (grass)	0.035
Vegetated (natural channel)	0.040
Vegetated (natural channel, irregular, pools)	0.095
Riprap	0.035

Hydrologic Modelling Parameters

Hydrologic modelling parameters are recommended as follows:

Table 2-9 – Computer Modelling Parameters

Subcatchment Parameters	**Value**
Catchment Width (m)	Typically based on area divided by flow path length
Flow Path Length (m)	Based on longest of governing hydraulic flow path length
Slope (Residential), (%)	2.0%, or based on lot grading
Slope (commercial, industrial)	1.0%, or based on lot grading
Slope (parks, cemeteries)	1.0%, or based on lot grading
Imperviousness	0 – 100%
Manning's Roughness for Impervious Areas	0.015
Manning's Roughness for Pervious Areas	0.250
Depression Storage for Impervious Areas	2.0 mm
Depression Storage for Pervious Areas	5.0 mm
Percent of Impervious Area with No Depression Storage	25.0%
Infiltration Parameters	
Infiltration Calculation Method	Horton
Maximum Infiltration Rate	100 mm/hr
Minimum Infiltration Rate	13 mm/hr
Decay Constant	4.1 hr^{-1}
Drying Time	7 days

Runoff Coefficient may be derived from imperviousness, or vice versa, using the following conversion:

Equation 2-8 – Runoff Coefficient from Imperviousness

$$C = (IMP \times C_i) + \left((1 - IMP) \times C_p\right)$$

Where:

- C is the Equivalent Runoff Coefficient (dimensionless, typically expressed as a fraction)
- C_i is the Runoff Coefficient of Impervious Areas, typically 0.95
- $C_p$is the Runoff Coefficient of Pervious Areas, typically 0.20
- Imp is the Imperviousness (dimensionless, typically expressed as a fraction)

Equation 2-9 – Impervious from Runoff Coefficient

$$IMP = \frac{1}{(C_i - C_p)} C - \left(\frac{C_p}{C_i - C_p}\right)$$

3.0 LOT GRADING AND SURFACE DRAINAGE

3.1 Single Detached Dwelling Residential Lot Grading

Refer to City of Brandon By-Law No. 6626 and the– MSS - Lot Grading Guidelines for lot grading requirements.

3.2 Gratings in Pedestrian Pavements

Gratings in sidewalks or pedestrian pavements should comply with industry standards or good practice guides (e.g. US “Americans with Disabilities Act” Accessibility Standards, Clause 302), namely:

- Gratings should have a maximum opening size of 13 mm. High heel friendly openings of not more than 6 mm may be appropriate in certain situations.
- Elongated openings should be placed so that the long dimension is perpendicular to the direction of travel.

3.3 Parking Lot Grading (Multi-Unit Residential, Commercial, Industrial, Institutional)

Parking lots shall be graded to provide adequate drainage. The following are recommended grading limits:

- Minimum Slope Asphalt surface = 1.0%
- Minimum Slope Concrete surface = 0.5%
- Maximum Slope = 5.0%
- Maximum Ponding Depth = 0.3 m
- All low points must include a spill over or overflow location such that the Maximum Ponding Depth is not exceeded. In the event a parking lot inlet grate is clogged, water ponding above the clogged inlet will spill over the overflow location and drain to another parking lot inlet.

3.4 Roadway Drainage

Pavement drainage design for urban cross-section roadways and bridges (to mitigate the encroachment of flow in gutters and ponding at sag low points into the roadway) must be designed to meet the following requirements.

Note that while freeways/expressways (such as the Trans-Canada Highway) do exist within the City, these roadways fall under Provincial jurisdiction and are not part of the design guidance of this document.

- Maximum gutter flow or ponding encroachment into roadway lanes under minor and major design rainfall shall not exceed the performance standards listed in Table 3-1.

- Gutter depth and flow shall not exceed the maximum values listed in Table 3-2.
- It is always preferable to grade roadways away from intersections than to situate inlets near or within intersections or permit cross-flow. This reduces the likelihood of ponding or icing at intersections and potential for splashing onto pedestrian crosswalks.
- Notwithstanding the above statement, it may be necessary especially in reconstruction situations to provide inlets near intersections to control localized ponding or cross-flow. Inlets should be placed no closer to the intersection than 6.0 m from the ends of curb returns to reduce the impacts of ponding or icing at the intersection
- Longitudinal gutter grading around intersection curb shall be a minimum 1.0% to reduce the potential for nuisance ponding.
- The first catch basin of any urban roadway to be located a maximum distance of 200 m from the nearest high point.
- For continuous grades longer than 200 m, on-grade catch basins must be used. The location and number of on-grade catch basins will depend on the spread limit for the roadway classification type.
- Sag low inlet points shall have a maximum spacing of 120 m.
- Catch Basin capacity for on-grade situations shall be computed using the methods outlined in Appendix 5B, based on HEC-22 Chapter 4 methodology.
- Catch Basin capacity for sag low inlet locations shall be computed based on rating curves for the standard City inlet grates. These rating curves are shown in Appendix 5B. Note that multiple inlets may be required to maintain ponding depth or flow encroachment to within the specified limits.
- The effectiveness of inlets in terms of increased depth over the grates may be improved by locating the inlets outside of the roadway travelway in small turnout pockets.

Table 3-1 – Roadway Drainage Requirements for Traffic Safety

Roadway Classification	Local	Collector	Arterial
Design (Minor) Storm	20% Probability (1-in-5 year)	20% Probability (1-in-5 year)	10% Probability (1-in-10 year) 4% Probability (1in-25 year) for depressed roadways
Concept	Ponding and Street Storage permitted	Ponding permitted. Road must remain passable.	Ponding permitted. Road must remain passable.
Spread limit	Water may spread to crown	One travel lane open.	One travel lane open in each direction.
Maximum Depth at Crown	0.00	0.00	0.00
Maximum Depth at Gutter	Curb height	Curb height	Curb height
Cross-flow (through intersections)	Controlled cross-flow permitted.	Controlled cross-flow permitted.	Not permitted.
Design (Major) Storm	4% Probability (1-in-25 year) or greater	4% Probability (1-in-25 year) or greater	1% Probability (1-in-100 year)
Concept	Ponding permitted	Ponding permitted. Road must remain passable.	Ponding permitted. Road must remain passable.
Spread limit	Water may spread across crown.	Water may spread across crown.	Water may spread to crown.
Maximum Depth at Crown	0.15 m maximum	0.15 m maximum	0.00
Maximum Depth at Gutter	0.30 m maximum. Curb overtopping permitted, not to encroach on private property	0.30 m maximum. Curb overtopping permitted, not to encroach on private property	0.40 m maximum. Curb overtopping permitted, not to encroach on private property
Cross-flow through intersections	0.30 m maximum	0.30 m maximum	0.15 m maximum over crown or centreline.

Table 3-2 – Maximum Gutter Flow Depth and Velocity for Pedestrian Safety

Flow Velocity (m/s)	Maximum Permissible Depth (m)
1.00	0.32
2.00	0.21
3.00	0.09

4.0 DESIGN OF MAJOR SYSTEM COMPONENTS

4.1 Channels

Channels (ditches, swales, grade breaks) shall be designed for stormwater conveyance of smaller storms and stormwater attenuation during larger rainstorms.

- Channels should be vegetated with native or boulevard grasses, to be specified by the City depending upon the channel location.
- Where flow velocities exceed erosion resistance of vegetation (or maximum channel velocities noted in Table 2-7), riprap armouring shall be used. Erosion resistance of vegetation should be calculated using HEC-15 "Design of Roadside Channels with Flexible Linings".
- Concrete lined channels may be considered in isolated cases instead of riprap.
- Private easements shall be provided for all swales serving more than two lots or where one or more lots may drain through adjacent private property. A minimum easement width of 3.0 m is required for private lot drainage.
- Channels shall be constructed with maximum 4:1 side slopes, and to the following dimensions:

Table 4-1 – Channel Dimensions

Channel Type	Channel Use	Minimum Bottom Width (m)
V-bottom Swale	Lot grading	N/A
Trapezoidal Channel (Ditch)	Lot grading	1.0
Trapezoidal Channel (Ditch)	Roadside ditch, local streets, collector	1.8
Trapezoidal Channel (Ditch)	Roadside ditch, arterial, highway	3.0

- Deep or wide channel side slopes should be flattened to improve slope stability and to facilitate mowing or haying with larger equipment. Deep channel side slopes shall require a geotechnical assessment for slope stability.
- Roadway gutters that could receive major flow shall be constructed to standard City pavement crossfall and curb or curb and gutter profiles.

4.1.1.1 Channel Slope

Channels should be constructed with the following minimum longitudinal slopes:

Table 4-2 – Minimum Channel Slopes

Channel Type	Minimum Longitudinal Slope (%)
Hard Surfaces	
Roadway Gutters	0.5%
Roadway Gutters around curb return	0.7%
Vegetated Surfaces	
V-Bottom Swale	0.50%
Ditch – 1.0m or less bottom width	0.15%
Ditch – 2.0m or less bottom width	0.10%
Ditch – 3.0m or greater bottom width	0.05%

4.2 Culverts

Culverts may be used for short land crossings (roadway, rail, etc.) of overland flow channels and shall meet the following criteria:

- Minimum diameter of 300 mm, although the City may specify larger minimum sizes for higher capacity ditches.
- Minimum longitudinal slope of 0.5%, with steeper slope of 2.0% being preferable.
- Culverts shall be designed with adequate capacity as follows:
  - Maximum upstream headwater depth of 85% of pipe depth during culvert design flow.
  - Check performance during 1% probability (1-in-100 year) flow to avoid overtopping of the crossing roadway.
- Erosion control measures (e.g. riprap) should be considered at the inlet and outlet for all culverts. Riprap shall be required for all culverts with a diameter of 600 mm or larger.
- Minimum cover shall meet the pipe structure requirements and manufacturer's specifications for protection from live and impact loading and prevent floatation.
- Parallel culverts should be installed with a minimum of one nominal pipe diameter or 1.0 m between pipes, whichever is smaller, to facilitate compaction during construction,
- Culverts through dikes will require shut off valves and an external anti-seepage collar, baffling, high plasticity clay or bentonite cutoff trench or other measures to minimize seepage.
- Culverts should be designed and ballasted to mitigate uplift.
- Culverts larger than 600 mm shall have step bevel mitered or flared end sections to improve hydraulics, better blend to the slope and lessen the amount of exposed pipe in the clear zone or that may be damaged by maintenance equipment.
- Culverts larger than 300 mm entering a closed conduit land drainage system shall include a safety and debris grate. Inlet grates for culverts 600 mm and larger shall be inclined to aid in the floatation of trapped persons or debris.

- Culverts larger than 1050 mm, culverts in fish habitat or any culvert designated as a small animal crossing must include a naturalized stone embedment of 30% of the culvert height and designed to withstand 1% probability (1-in-100 year) flow velocity.
- Corrugated Steel Pipe culverts must be designed with a minimum 75-year service life.
- Acceptable pipe materials include Reinforced Concrete Pipe and Corrugated Steel Pipe.
- Culverts with a diameter of 900 mm or greater or located within major channels require special consideration and may require structural and geotechnical measures to ensure pipes stability. Design calculations including hydraulic performance, erosion protection, fish passage and factor of safety against uplift must be submitted to the City.

4.3 Retention Ponds (Wet Ponds)

Retention ponds, or wet ponds, are stormwater structures featuring a permanent wet basin, designed to attenuate runoff during and following a rainstorm and reduce peak flows to downstream systems. The ponds are designed to slowly drain based on the capacity available in the downstream system. Naturalized design and vegetation are preferred and encouraged as outlined in the City of Brandon Naturalized Stormwater Pond Guidelines (2018).

Retention pond design criteria are as follows:

- Storage volume to be based on the 1% probability (1-in-100 year) design rainstorm event.
- Consideration of the movement of local groundwater into and out of the pond. Impermeable soil liners with a hydraulic conductivity of 1 x 10-6 cm/s or less may be used reduce the potential groundwater infiltration or exfiltration.
- Minimum normal water surface area of 1.0 ha (2.47 ac)
- Preferred length to width ratio of 2:1 to 3:1.
- Avoid narrow or dead areas where floating debris may accumulate.
- Maximum allowable side slopes:
  - 4:1 from pond bottom to nominally 0.5 m below NWL.
  - 7:1 from 0.5 m below NWL to HWL. Side slopes of less than 7:1 may be used if geotechnical slope stability analysis demonstrates that the side slope will have a minimum factor of safety of 1.5 against all types of slope failure, if the pond is fenced above freeboard level, and if the pond is located in an industrial area. Flatter side slopes may be warranted due to soil and groundwater conditions. Steeper side slopes will not be permitted in residential areas.
  - 10:1 above HWL to freeboard level.
- Allowable NWL depths:
  - Minimum NWL depth of 2.0 m.
  - Maximum NWL depth of 3.5 m.

- Active storage depth between NWL and HWL of 1.2 m. Increasing the active depth to 1.5 m may be permitted in certain situations such as commercial or industrial park area and if the designer can provide a valid reason to do so.
  - Freeboard above HWL of 0.6 m. Reducing the freeboard to 0.3 m may be permitted in certain situations if the designer can provide a valid reason to do so.
  - All buildings within 30 m of the pond HWL extents must be constructed with main or basement floor level not less than the freeboard level.
  - Ponds in proximity to the Assiniboine River shall also consider the 100-year return river flood level as the HWL. Such ponds will have two HWLs, one from rainfall and another from river stage.
- Submerged inlet and outlet pipes:
  - Preferred versus unsubmerged inlets and outlets to minimize obstruction from floating debris and reduce ice damage.
  - Crown of inlet and outlet pipes must be a minimum of 0.6 m below NWL, to situate them below the depth where a person might wade onto the pipe.
  - Submerged inlets or outfalls do not require grates unless they are exposed during low flow periods. Shall be located to mitigate short circuiting of flow through the pond, to promote settling of sediment.
- Unsubmerged inlet and outlet pipes:
  - Must include headwalls and grates.
  - The outlet must be set at or nominally below NWL. Outlet pipe inverts will not be considered to be an elevation control structure, a control weir is required.
- Weir control structures are required for all retention ponds. These structures must be a closed manhole, and must be designed to provide minor adjustment for levelling of the weir (e.g. a weir plate with minimum 100 mm of vertical adjustment for levelling).
- Pond maximum drawdown time:
  - 48 hours (2 days) or less following the end of 20% probability (1-in-5 year) rainfall.
  - 120 hours (5 day) or less following 1% probability (1-in-100 year) rainfall.
- Minimum 3.0 m buffer strip required between pond HWL and property lines or park dedications.
- Shoreline erosion protection shall be provided for 1.5 horizontal metres below NWL and a minimum of 0.6m above HWL.
- Pond must include signage for public education and safety.

4.4 Detention Ponds (Dry Ponds)

Detention ponds, or dry ponds, function the same as wet bottom retention ponds, however instead of a permanent water body, a basin with a dry bottom is used. Retention ponds are preferred over detention ponds; however, detention ponds will be considered where construction of retention ponds is not feasible.

Detention ponds design criteria are as follows:

- Storage volume to be based on the 1% probability (1-in-100 year) design rainstorm event.
- Detention ponds shall not be used to retain runoff for design rainstorms of 50% probability (1-in-2 year) or smaller.
- Preferred length to width ratio of 2:1 to 3:1.
- Maximum side slopes of 7:1.
    - Slopes steeper than 7:1 may be used if geotechnical slope stability analysis demonstrates that the side slope will have a minimum factor of safety of 1.5 against all types of slope failure, if the pond is fenced above freeboard level, and if the pond is located in an industrial area.
- If the detention pond also acts as a conveyance with flow-through, include a low flow channel designed to accommodate 50% probability (1-in-2-year) flow. This channel shall have a minimum depth of 0.3 m.
    - Where low flow channels are not included or not appropriate because the pond bottom will be used for sporting or recreation when dry, a subdrainage system should be used to improved drainage and aid in drying the pond bottom following a rainstorm.
- Pond bottom to include both longitudinal slope and crossfall of minimum 1.0% to promote drainage of the pond following rainstorms. Steeper 2% crossfall is preferred to aid in drying of the pond bottom and reducing the potential for nuisance ponding and mosquitoes.
- Active storage depth between pond bottom and HWL of 1.2 m maximum. Freeboard above HWL of 0.6 m. Reducing the freeboard to 0.3 m may be permitted in certain situations if the designer can provide a valid reason to do so.
    - All buildings within 30 m of the pond HWL extents must be constructed with main or basement floor level not less than the freeboard level.
- Increasing the active depth to 1.5 m may be permitted in certain situations if the designer can provide valid rationale to do so.
- Inlets, outlets or combined inlet / outlet pipes whether a culvert or connection to a closed conduit land drainage system must include inclined grates to force persons and floating debris up towards the water surface.
- The entire pond footprint to be vegetated with upland grass seed, or a combination of upland grass seed on the slopes and specialized wet meadow grass seed in the pond bottom area. The City may request the use of boulevard seed for specific park areas for compatibility with existing or proposed landscaping. Proposed seed mixes are to be submitted to the City for approval.

- Ponds must include signage for public education and safety.

4.5 Naturalized Stormwater Ponds (NSP)

Naturalized Stormwater Ponds shall meet criteria provided in the City of Brandon Naturalized Stormwater Pond Guidelines (2018).

4.6 Underground Stormwater Storage

- Underground stormwater storage (tanks, oversized pipes, grid structures) may be used on private sites to attenuate flow and manage discharge from the site but will not be acceptable for use in public property or rights-of-way where the City is expected to take ownership.

5.0 DESIGN OF MINOR SYSTEM COMPONENTS

5.1 Storm Water Pipes

5.1.1 Sizing

Storm sewer pipes shall be sized to accommodate design flows and shall have a minimum diameter of:

- 375 mm for mainline land drainage sewers.
- 300 mm for catch basin leads.

5.1.2 Depth of Cover

The minimum depth of cover to crown of pipe shall be:

- 1.8 m for mainline land drainage sewers mains.
- 1.2 m for catch basin leads and must be at least 0.1 m below bottom of granular road structure.

A maximum preferred depth of 6.0 m to pipe invert for all pipes, except under certain situations such as inverted siphons undercrossing other infrastructure. Local topography may require deeper sewers and these special locations should be reviewed with the City prior to finalizing design.

Pipes must be designed to prevent floatation. In high groundwater areas, a subdrain system may be installed within the mainline sewer trench to promote localized lowering of groundwater. These subdrains should be connected to both upstream and downstream manholes. The subdrains need not be placed near or below the mainline sewer invert, it is usually sufficient to situate them at an elevation matching mainline sewer crown.

5.1.3 Clearances

Table 5-1 – Minimum Storm Sewer Main Separations

Sewer Main in Proximity to:	Minimum Clearance (m)	
	Horizontal[1]	Vertical[2]
Domestic Sewer Mains	3.0	0.5
Watermains	3.0	0.5
Water Services	3.0	0.3[3]
Existing or Proposed Shallow/Above Ground Utilities	3.0	
Domestic Sewer Manholes	3.0[3]	
Edge of Right of way and/or Easements	3.0	

[1] Separation is defined as centreline to centreline.

[2] Separation is defined as actual clearance from outside of pipe to outside of other pipe.

[3] Where separation from domestic sewer manholes of water services cannot be met, manhole sidewall or pipe crossing shall be insulated.

5.1.4 Alignment

Land Drainage Sewer mains must be designed as follows:

- Straight alignment between manholes, at an offset parallel to the adjacent property line.
- Located within the street section of the road right of way. Where sewer mains are not located within a road right of way, manholes and other appurtenances shall be accessible by a route suitable for travel by a heavy maintenance vehicle.

5.1.5 Pipe Material

Refer to SCS for allowable pipe materials

5.1.6 Pipe Structure Design

Refer to Appendix 5B for pipe structure design.

5.2 Catch Basins

5.2.1 Catch Basin Locations

Catch basins shall be located:

- At all low points within pavement.
- As required on continuous roadway slopes to manage flow encroachment into travel lanes.
- Outside of:
  - Intersections, minimum 5m away from of stop lines or ends of intersection curb returns
  - Cross walks
  - Driveways
- Rear yard catch basins within residential neighbourhoods will not be permitted.

5.2.2 Catch Basin Capacity

Catch basin inlet capacity shall ensure that ponding limits in Subsection 0 are not exceeded, and multiple catch basins may be required.

5.2.3 Catch Basin Barrels

- Barrel diameter: 750 mm or 900 mm (see 5.3.4 for catch-basin manholes)
- Sump depth: 600 mm

5.2.4 Catch Basin Leads

All catch basin leads that discharge to LDS systems shall tie in at a manhole. Where catch basin leads discharge to an overland system, an outlet grate must be used on the downstream end.

- Maximum lead length shall be 30 m, if a longer catch basin lead is required then a catch basin-manhole should be substituted
- Minimum lead size shall be 300 mm, with a minimum slope of 1.0%.
- Minimum lead size shall be 375 mm when two or more catch basins are connected together. Hydraulic calculations shall be provided to confirm the sizing of catch basin leads serving more than two catch basins.

5.2.5 Catch Basin Restriction

Where restriction is required to limit the amount of flow into the catch basin lead (to provide increased surface storage or ponding), restriction shall be achieved by a permanent device (such as grouting a smaller restriction pipe into the catch basin lead invert at the manhole). Orifice plates, or any removable restriction device will not be permitted as they may easily be removed intentionally or unintentionally by persons who do not understand their intended purpose.

Catch basin restriction undertaken as a retrofit on a public catch basin must consider the potential encroachment of ponding onto private property and mitigate the potential for private property damage.

5.3 Manholes

5.3.1 Manhole Locations

Manholes shall be located at:

- the upstream end of each land drainage sewer main.
- changes in pipe size.
- changes in alignment
- All junctions.

Manhole spacing shall not exceed 120 m.

5.3.2 Manhole Barrels

All manhole barrels shall:

- have a minimum diameter of 1200 mm when connected pipe diameters are 525 mm or less.
- have a minimum diameter of 1500 mm when pipe diameters exceed 525 mm.

- Manholes serving larger sewers shall maintain a minimum of 300 mm of intact barrel between pipe penetration breakouts.

Refer to the manufacturer's recommendations for minimum manhole diameters to suit various connecting pipe sizes and connection angles. Suggested minimum manhole sizes are shown in Table 5-2.

Table 5-2 – Suggested Minimum Manhole Diameters

Sewer Diameter (mm)	Manhole Diameter (mm) Straight Through Installation	Manhole Diameter (mm) Right Angle Installation
450 and smaller	1200	1200
525	1200	1350
600	1350	1500
750	1500	1800
900	1800	2100
1050	1800	2400
1200	2100	2700

Manholes shall be designed to resist floatation.

5.3.3 Hydraulic Losses

Hydraulic losses at manholes shall be managed by:

- Providing benching to the spring lines of connecting pipes.
- Dropping downstream inverts by 30 mm for deflection angles between inlet and outlet greater than 45° but not exceeding 95°.
- The preferred vertical alignment for pipes is to match pipe crowns at pipe diameter changes, however matching inverts may be permitted if it can be demonstrated that preserving pipe depth of cover is required.

5.3.4 Catch Basin Manholes

Catch basin manholes, or manholes located at low points with surface water inlet covers may be used at the discretion of the City. Catch basin manholes shall include:

- Minimum barrel diameter: 1200 mm.
- Sump depth: minimum 300 mm.
- Manholes on the upstream and downstream side of inverted siphons shall include a minimum 300 mm sump.

5.4 Outfalls

The following design criteria shall apply to stormwater outfalls to a receiving water body such as a pond or the Assiniboine River:

- Outfalls shall be submerged where possible, with a minimum depth of from NWL to invert of 1.5 m, or minimum water depth above the pipe crown of 1.0 m, whichever provides greater depth between pipe crown and NWL. This situates the pipe outside of the zone where it could be subject to damage from ice or floating debris
- Where submergence is not possible:
  - The outfall invert shall be located minimum 300 mm above NWL.
  - If discharging to a river or stream, the outfall shall be protected from ice or waterborne debris damage by insetting the end of the pipe into the riverbank by a minimum horizontal distance of 3.0 m.
  - Outlet erosion protection including riprap with separation geotextile underlay placed downstream of the outfall to the bottom of the receiving water body or extend a minimum of 3.0 m into the water body. The riprap shall be inset into the riverbank such that to top surface of the riprap matches the existing riverbank.
  - The outfall pipe trench shall include a 300 mm thick granular drainage layer below the granular pipe surround to aid in the equalization of local groundwater and receiving water level. This granular drainage layer shall extend 15 m upstream of the end of pipe or to the top of pond freeboard level or the top of natural riverbank, whichever is greater.
  - If the outfall pipe penetrates a dike, the granular drainage layer must terminate at the dike and an anti-seepage collar, diaphragm, or high plasticity clay or bentonite-enriched cutoff trench installed to minimize the potential for seepage.
  - Include an outlet safety grate to prevent access. The grate should be constructed of galvanized steel with nominal horizontal bar spacing of 150mm.

Stormwater outfalls to a receiving open channel:

- Include riprap, complete with geotextile placed downstream of the outfall to the opposite side of the receiving ditch and up the opposite ditch side slope to prevent erosion. The riprap shall be inset into the channel or riverbank such that to top surface of the riprap matches the channel or riverbank surface.
- Include a removable safety grate with nominal horizontal bar spacing of 150 mm to prevent access.

5.5 Oil and Grit Separators

Oil and grit separators (OGS) may be required at the City's discretion for locations where potential discharge of hydrocarbons or contaminated sediments may be present.

Oil and grit separators are required for:

- Industrial, Commercial, Institutional or Multi-Unit Residential sites with 1500 m^2 or greater paved areas.
- All commercial or industrial sites which handle hydrocarbons such as gas stations, lube and oil change facilities, vehicle maintenance and mechanical shops, and sites with on-site fuel storage.

Oil and grit separators design criteria is as follows:

- Sized to capture a 1-hour, 50% probability (1-in-2-year) post development flow.
- Remove at least 85% Total Suspended Solids (TSS) during 2-year flow, based on the target particle size greater than or equal to 75 microns. Assumed sediment composition for design is shown in Table 5-3.
- Include a high flow bypass for larger storms to avoid resuspension of settled solids.
- Located upstream of stormwater ponds, underground storage tanks, or pumping stations to prevent sediment and oil from entering these facilities.

The following sediment profile should be used for sediment capture estimates, based on water at 4° C:

Table 5-3 – Sediment Composition (based on City of Calgary 2011)

Size Range (µm)	Target Particle (µm)	Particle Density (kg/m3)	Settling Velocity (m/s)	Fraction of Total Load (%)
<10	10	1500	1.738 E-05	23%
10 - 20	20	2000	1.389 E-04	9%
20 - 50	50	2500	1.304 E-03	13%
50 - 75	75	2650	3.221 E-03	11%
75 - 150	150	2650	1.240 E-02	12%
>150	300	2650	3.229 E-02	32%

Appendix A Design Rainstorms

Table 5A-6- 1 - Design Rainstorms with 10-minute timestep

Time Step Ending (minutes)	Intensity (mm/hr) for Storms of Different Annual Probability of Event (return frequency in years)					
	50% (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
10	2.70	2.57	2.42	2.45	2.47	2.43
20	3.16	2.80	2.57	2.57	2.57	2.51
30	3.85	3.08	2.74	2.69	2.67	2.60
40	5.01	3.45	2.94	2.84	2.79	2.69
50	7.45	3.93	3.18	3.00	2.92	2.79
60	17.14	4.60	3.46	3.18	3.06	2.90
70	73.28	5.59	3.82	3.39	3.22	3.02
80	22.40	7.28	4.26	3.63	3.41	3.15
90	12.10	10.83	4.85	3.92	3.61	3.29
100	8.47	24.91	5.67	4.28	3.85	3.45
110	6.60	106.55	6.88	4.71	4.13	3.63
120	5.45	32.56	8.93	5.26	4.46	3.84
130	4.67	17.59	13.24	5.98	4.86	4.07
140	4.11	12.31	30.23	6.98	5.35	4.34
150	3.68	9.60	131.57	8.47	5.97	4.65
160	3.34	7.93	39.46	10.98	6.79	5.02
170	3.06	6.80	21.38	16.23	7.93	5.47
180	2.83	5.97	15.02	36.95	9.62	6.02
190	2.64	5.35	11.74	163.28	12.46	6.72
200	2.47	4.85	9.72	48.22	18.41	7.64
210		4.45	8.35	26.16	41.87	8.91
220		4.12	7.35	18.41	186.71	10.81
230		3.83	6.58	14.41	54.64	13.99
240		3.59	5.98	11.94	29.65	20.64
250		3.39	5.49	10.26	20.88	46.82
260		3.20	5.08	9.04	16.34	210.09
270		3.04	4.73	8.10	13.55	61.07
280		2.90	4.44	7.36	11.65	33.18
290		2.77	4.18	6.76	10.26	23.39
300		2.65	3.96	6.26	9.20	18.33
310		2.54	3.76	5.84	8.36	15.21
320		2.45	3.58	5.47	7.68	13.08
330			3.43	5.16	7.11	11.53
340			3.28	4.88	6.63	10.34
350			3.15	4.64	6.22	9.40

Time Step Ending (minutes)	**Intensity (mm/hr) for Storms of Different Annual Probability of Event (return frequency in years)**					
	50% (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
360			3.03	4.42	5.86	8.63
370			2.92	4.23	5.55	8.00
380			2.82	4.05	5.27	7.46
390			2.73	3.89	5.03	7.00
400			2.64	3.74	4.80	6.60
410			2.56	3.61	4.60	6.25
420			2.49	3.49	4.42	5.94
430			2.42	3.37	4.25	5.66
440				3.26	4.10	5.41
450				3.17	3.96	5.18
460				3.07	3.83	4.98
470				2.99	3.71	4.79
480				2.91	3.60	4.62
490				2.83	3.49	4.46
500				2.76	3.40	4.32
510				2.69	3.30	4.18
520				2.63	3.22	4.05
530				2.57	3.14	3.94
540				2.51	3.06	3.83
550				2.46	2.99	3.72
560					2.92	3.63
570					2.85	3.54
580					2.79	3.45
590					2.73	3.37
600					2.68	3.29
610					2.62	3.22
620					2.57	3.15
630					2.52	3.08
640					2.48	3.02
650					2.43	2.96
660						2.90
670						2.85
680						2.79
690						2.74
700						2.69
710						2.65
720						2.60
730						2.56

Time Step Ending (minutes)	**Intensity (mm/hr) for Storms of Different Annual Probability of Event (return frequency in years)**					
	50% (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
740						2.52
750						2.48
760						2.44

Table 5A-6-2 - Design Storm Parameters

Parameter	**Annual Probability of Event**					
	50% (1-in-2 year)	**20%** (1-in-5 year)	**10%** (1-in-10 year)	**4%** (1-in-25 year)	**2%** (1-in-50 year)	**1%** (1-in-100 year)
Total Depth (mm)	32.4	59.9	69.8	91.4	108.3	126.3
Total Duration (min)	200	320	430	550	650	760
Peak Intensity (mm/hr)	73.28	106.55	131.57	163.28	186.71	210.09

Appendix B Pavement Drainage Design Methodology

5B PAVEMENT DRAINAGE

5B.1 Design Criteria

Pavement drainage design criteria are as follows:

- Pavement Drainage analysis is required for higher capacity, urban cross-section roadways, including major collectors, arterials and urban cross-section highways. It is not required for local or minor collector roadways with lower traffic volume and speed.
- Refer to Section 3.4 for roadway drainage requirements for traffic safety.
- Design rainstorms to be based on IDF Curves and Chicago type synthetic hyetographs presented earlier in this manual.
- The minimum time of concentration for Rational Method design runoff estimation for small pavement catchments was assumed to be 5.0 minutes.
- The design catchment is the right-of-way itself, plus a nominal allowance for external runoff from grassed boulevard areas or contributions from adjoining properties as required. A nominal contribution of 6 m beyond the right of way limits is recommended.
- The velocity of shallow gutter flow (not cross-flow) should not exceed the velocity vs. maximum depth listed in Table 3-2.
- Inlet grate capacity on-grade configuration was assumed to be in accordance with FHWA Hec-12 / HEC-22 procedures, based on the dimensions of City standard inlet grates and assuming the grate performance approximately matches FHWA Grate P-50x100. Other specialty or project-specific grates including bridge deck drains may be used as required. Inlet grates on-grade should assume 30% derating due to debris or clogging.
- Inlet grate capacity in sag locations (no velocity) was assumed to be in accordance with rating curves developed from first principles from weir and orifice equations and shown on Figure 5B.. The 30% derated performance curves should be used for design to include consideration for debris or clogging.
- Turnout pockets for on-grade or sag inlets may be used to improve capacity by increasing head on the grates and situate catch basins outside of the travel lanes.

5B.2 Flow in Gutters

Flow in gutters for inlet design was computed using the Modified Manning equation. Modification of the Manning equation is necessary because the channel hydraulic radius in the standard Manning equation is not well represented by the extremely flat pavement crossfall (i.e. approximately 85% of the flow conveyance occurs in the deeper half of the channel width). For design purposes, the channel cross-section is represented by a simple triangle, with one side slope being the (simplified) vertical face of curb and the other side slope being the pavement crossfall. The Modified Manning equation is as follows, with parameters shown on the sketch below:

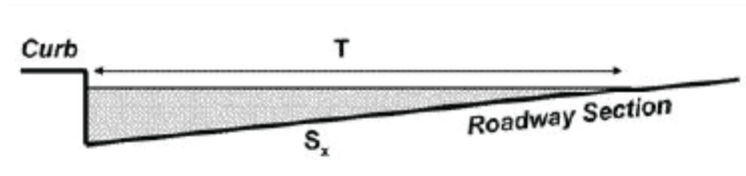

Figure 5B.1 - Flow in a Gutter

Equation 5B.1 - Modified Manning Formula for Gutter Flow

$$Q = \left(\frac{k}{n}\right) S_X^{1.67} S_L^{0.5} T^{2.67}$$

Where:

- Q is the estimated flow (m3/s)
- k is a constant (equal to 0.376 for metric units)
- n is the Manning roughness (assumed to be 0.016 for pavements, and neglected for the curb face)
- SX is the pavement crossfall (m/m)
- SL is the longitudinal grade (m/m)
- T is the top width of flow, also known as the spread (m)

The standard City curb and gutter sections from standard drawings STD1112S (Barrier Curb and Gutter) and STD1112T (Rolled Curb) form the City of Brandon Standard Construction Specifications – Section 02514 “Concrete Construction” both have gutter pan crossfall that is steeper than the typical roadway crossfall, resulting in a ‘composite’ section for flow as shown in the following figure (from FHWA HEC-22, 3ed).

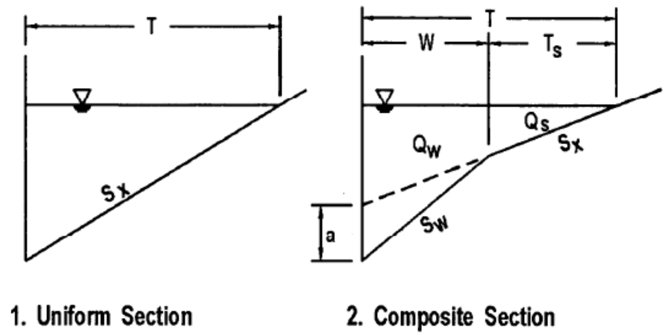

Figure 5B.2 - Uniform and Composite (Depressed) Gutters

The gutter depression is 15 mm deep (at the curb face) and 300 mm wide, with a crossfall of 5.0% vs. the typical 2.0% pavement crossfall. The process for estimating flow and spread using the Modified Manning Equation is slightly different for a composite gutter, and requires the flow to be estimated separately for the depressed section and non-depressed section.

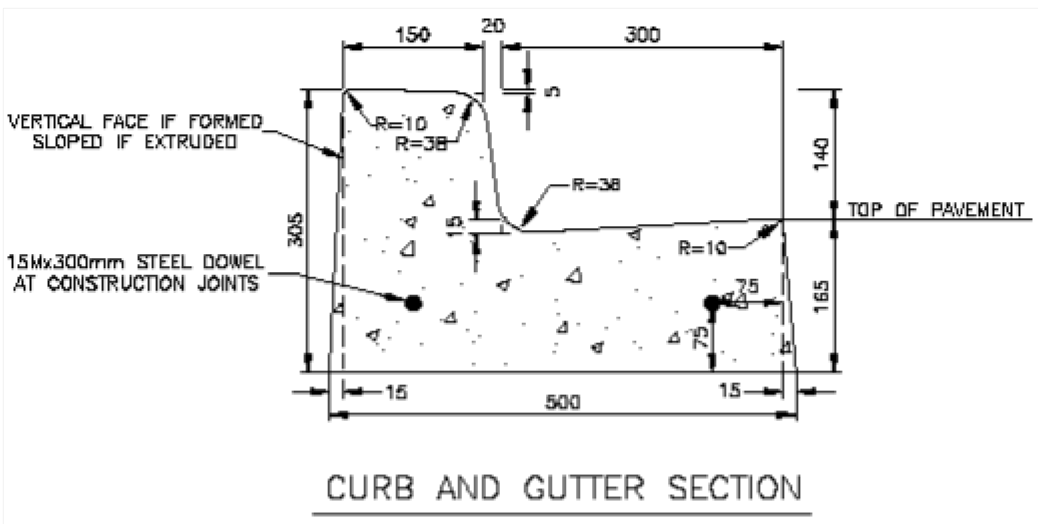

Figure 5B.3 - Curb and Gutter Section as Composite Gutter

5B.3 Inlet Capacity

Inlet capacity is designed and analyzed using different methods for on-grade and sag locations.

5B.3.1 Inlets On Grade

Inlet capture for on-grade inlets subject to flow velocity was based on the US FHWA HEC-22 “Urban Drainage Design Manual” Third Edition theoretical methodology. The assumed on-grade inlet grate was the City Barrier Curb and Gutter Inlet (Titan model TF-103 or WD Valve Box WD-59)) or the flat rectangular surface grate (Titan model TF-100) used for inlets in approaches. Information for the Rolled Curb and Gutter Inlet (Titan model TF-102 or WD Valve Box model WD-55) is also provided, although this inlet will not typically be used on Collector or Arterial roads where inlet capacity analysis is typically undertaken.

The velocity where splash-over first occurs for these grates was assumed to be similar to the FHWA P-50x100 grate since no splash-over testing of these grates is known. The curb opening inlet does not come into play in this simplified calculation.

The dimensions and parameters of typical grates are shown on Table 5B-1. The parameters are defined as follows:

- Effective Width – the width of grate from the face of curb measured transversely into the roadway and extending to the limit of grate openings (e,g. not the width of the removable grate cover).
- Effective Length – the length of the grate openings measured longitudinally along the roadway (e.g. not the length of the removable grate cover).
- Splash-over Velocity – the assumed velocity where flow leaps across the grate, estimated using FHWA HEC-22 Chart 5A and based on a FHWA P-50x100 grate. Local grates constructed of cast iron are not similar to the FHWA grates fabricated from steel plate, but no splash-over velocity testing has been done for the local grates. The splash-over velocity 'V' for this grate can be expressed as a quadratic function of effective grate length 'L':

Equation 5B.2 - Assumed Inlet Splash-over Velocity Equation

$$V = -0.4019L^2 + 2.1091L + 0.2945$$

Table 5B-1 - Grates for On-grade Inlets on Roads and Bridges

Inlet Type	**Effective Width** (m)	**Effective Length** (m)	**Splash-over Velocity** (m/s)
Barrier Curb & Gutter Inlet (Titan TF-103)	0.404	0.472	1.20
Rolled Curb & Gutter Inlet (Titan TF-102)	0.450	0.497	1.24
Rectangular Surface Inlet (Titan TF-100)	0.574	0.528	1.30

Flow in a gutter bounded by a curb or barrier is separated into two distinct parts for inlet design, described below and as shown on the following sketch:

- Frontal flow (Qw), the portion of the gutter flow that directly encounters and runs over top of the grate.

- Side flow (Qs), the portion of gutter flow spread further out into the pavement that does not pass directly over top of the grate, but could enter the grate from the side as it flows past.
- Any flow not intercepted by the grate is known as bypassed flow.

Figure 5B.4 - Frontal Flow and Side Flow at an Inlet

Equation 5B.3 - Flow Characteristics near an On-Grade Inlet

$$Qt = Qw + Qs$$

Where:

- Q_t is total gutter flow
- Q_w is frontal flow (based on the width of the inlet compared to the spread of flow into the gutter.
- Q_s is side flow (computed from total flow minus frontal flow)

The HEC-22 methodology involves computing the efficiency of flow capture for an inlet for the separate components of frontal flow and side flow. Frontal flow capture efficiency is dependent upon the velocity of gutter flow compared to the splash-over velocity of the grate. Once the velocity of flow exceeds the splash over velocity of the grate, a portion of the flow will splash over the grate with only a fraction of the flow intercepted. The splash-over velocity is specific to each grate type and depends upon the geometry of the grate bars and the length of the grate (measured longitudinally along the gutter). The side flow capture efficiency depends upon the gutter flow velocity, pavement crossfall and the length of the grate.

For a simple gutter, the ratio of frontal flow to total gutter flow is estimated as follows:

Equation 5B.4 - On-Grade Inlet Capture Efficiency Equation

$$E_0 = \frac{Q_W}{Q_t} = 1 - \left(1 - \frac{W}{T}\right)^{8/3}$$

Where:

- E_0 is the ratio of frontal flow to total gutter flow
- Q_t is total gutter flow (m^3/s)
- Q_w is frontal flow in width W (m^3/s)

- W is the width of the grate (m)
- T is the top width of flow or flow spread (m)

The remainder of the flow bypassing the grate is known as side flow:

Equation 5B.4 - Side Flow Equation

$$Qs = Qt - Qw$$

The ratio of frontal flow intercepted by the grate is estimated as follows:

Equation 5B.5 - Frontal Flow Efficiency Equation

$$R_f = 1 - K_u(V - V_0)$$

Where:

- R_f is the ratio of frontal flow intercepted (frontal interception efficiency)
- K_u is a constant equal to 0.295 (metric units)
- V is the velocity of flow in gutter (m/s)
- V_0 is the splash over velocity of grate, a function of grate bar type and length (m/s)

The ratio of side flow efficiency is estimated as follows:

Equation 5B.6 - Side Flow Efficiency Equation

$$R_s = \frac{1}{\left(1 + \frac{K_u V^{1.8}}{S_x L^{2.3}}\right)}$$

Where:

- R_s is the ratio of side flow intercepted (side interception efficiency)
- K_u is a constant equal to 0.0828 (metric units)
- V is the velocity of flow in gutter (m/s)
- S_x is the pavement crossfall (fraction, not percent slope)
- L is the Length of grate (m)

The total flow intercepted by the grate is computed based on the ratio of frontal flow intercepted and the ratio of side flow intercepted:

Equation 5B.7 - Intercepted Flow by On-Grade Inlet

$$Q_{int} = Q_t\left(R_f E_0 + R_s(1 - E_0)\right)$$

Where:

- Q_{int} is the estimated intercepted flow (m^3/s)
- Q_t is total gutter flow (m^3/s)
- E_0 is the ratio of frontal flow to total gutter flow
- R_f is the efficiency of frontal flow capture
- R_s = Efficiency of side flow capture

Note that flow capture by a curb opening grate is typically not considered for a combination curb and gutter inlet, in accordance with HEC-22 procedures.

On-grade inlets are generally not very efficient. 100% efficiency would mean that a grate captures 100% of the flow. On-grade inlets using standard City TF-103 cast iron grates typically capture only 25 to 40% of total gutter flow, so multiple inlets are often required to reduce puddle spread effectively.

If additional on-grade inlet capacity is required and cannot be achieved by installing multiple catch basin units, then there are several other options including:

- Install custom oversized inlets.
- Install curb openings to drain to an off-pavement collection system.
- Install inlets in a turnout pocket, described later in this Section.

- Change the pavement grading design to include a shallow sag inlet, especially on the upstream side of intersection to mitigate cross-flow.

5B.3.2 Inlets in Sag

Inlets in sag locations are not subject to velocity, so simple hydraulic rating curves for weirs and orifices are valid to simulate the performance of the grates. The gutter grates and curb openings are complex castings or built up from steel bar stock and the dimensions and geometry varies from jurisdiction to jurisdiction, so published rating curves obtained by hydraulic testing are the best source of grate performance information.

No published rating curves are available for typical City of Brandon inlet grates, so inlet rating curves for sag inlets were developed from first principles assuming weir and orifice flow based on the dimensions of standard local grates and assumed weir and orifice coefficients. These estimates should be corrected if laboratory or improved calculation methods are available.

The Barrier Curb and Gutter inlet including a surface grate and curb opening is represented by the Titan Foundary TF-103 Barrier Curb and Gutter Inlet surface grate and 'side' opening. Note that the similar WD Valve Box WD-59) has a larger surface grate (but identical curb box), so the assumed conservative rating curve is based on the smaller TF-103.

During low flow, the grate perimeter (except for the curb section) performs like a weir. The standard Barrier Curb and Gutter Inlet has an effective width (measured transversely) of 0.401 m and length (measured longitudinally along the roadway gutter) of 0.472 m, and a total weir length of 1.274 m. Note that the effective width and length of the grate are the outermost to outermost dimensions of the slot openings and not the dimensions of the removable grate cover.

Weir flow for the grate was estimated using the weir equation:

Equation 5B.8 - Weir Equation

$$Q_{gw} = C_w * L * H^{1.5}$$

Where:

- Q_{gw} is the estimated weir flow from the grate perimeter (m^3/sec)
- Cw is the Weir Flow Coefficient, assumed to be 1.80 ($m^{0.5}$/sec).
- L is the weir length, equal to grate perimeter excluding the curb section (m), equal to 1.27 m for the standard Barrier Curb and Gutter inlet grate. The weir is assumed to have no end contractions at the curb line, since flow along the curb line will cross the weir perpendicularly.
- H is the average depth of flow on the weir (m)

As the depth increases, the grate is flooded and the slots function as a group of orifices with total area 0.076 m^2. Since the grate has crossfall, the effective depth for weir and orifice calculations was assumed to be the average depth over the grade, rather than analyzing each orifice separately.

Orifice flow for the grate was estimated using the orifice equation:

Equation 5B.9 - Orifice Equation

$$Q_{go} = C_o * A * (2gH)^{0.5}$$

Where:

- Q_{go} is the estimated orifice flow through the grate (m3/s)
- C_o is the Orifice Flow Coefficient, assumed to be 0.61.
- A is the Orifice Area, equal to the sum of areas of each opening in the grate (m^2), equal to 0.076 m^2 for the standard barrier curb and gutter inlet grate. Low flow performance where only some of the grate openings are admitting flow is not analyzed since during low flow the weir operation will govern grate performance.
- g is the acceleration due to gravity, 9.81 m/s^2.
- H is the average depth of flow on the grate (m), estimated as the depth at the middle of the grate.

The following sketch shows how weir flow governs the lower depth performance of the grate and orifice flow governs the higher depth performance. The sketch shows the two curves intersecting, but sometimes it is necessary to connect the curves with a short, interpolated segment.

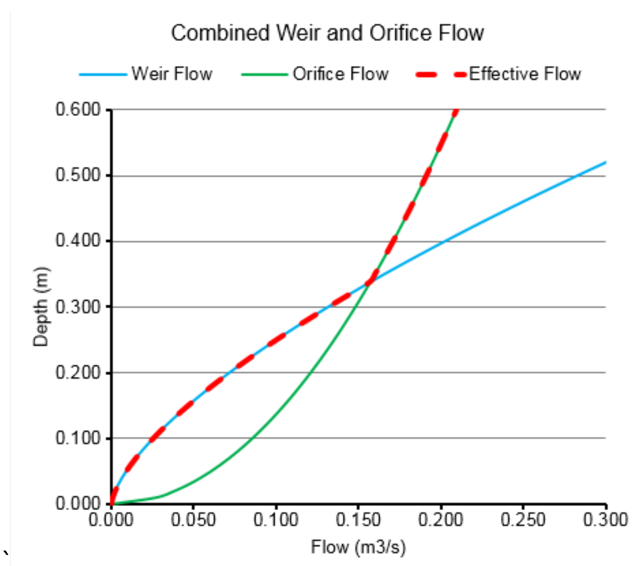

Figure 5B.5 - Combined Weir-Orifice Performance

Likewise, under low flow conditions, the double row of 56 mm high by 162 mm wide vertical openings in the curb inlet act like a series of weirs, or more simply as a single 0.486 mm wide weir but only after the grate begins to flood and flow can reach the curb inlet. This weir was assumed to have two end contractions at the curb, but not at the narrow interior dividers. The effective weir length was assumed to be the total width of all three openings, excluding the vertical divider bars. A weir flow coefficient of 1.80 was assumed.

As the depth increases, the 6 openings (3 wide x 2 high) in the curb inlet functions as a group of orifices with total area 0.054 m^2 and assumed 0.61 orifice flow coefficient. For the curb inlet, the transition between weir flow and orifice flow is interpolated as a straight line. The performance of the lower and upper weir-orifice slots was computed separately, the combined. The transition between weir flow and orifice flow was assumed to be a straight line between the weir rating curve and orifice rating curve, between the depths equal to the full height of the slot opening up to an arbitrary 140% of the slot height.

The rating curve for the Barrier Curb and Gutter Inlet was estimated by combining the surface grate weir-orifice and curb opening weir orifice. The typical assumption of adding both rating curves has been reported to overestimate the capacity, so a reduced capacity equation based on the work of Guo, McKenzie and Mohammadi “Design of Street Sump Inlet”, ASCE Journal of Hydraulic Engineering, Vol 135, No. 11 (November 2009) was assumed:

Equation 5B.10 - Combination Inlet in Sag Equation

$$Q = Q_g + Q_c - K_c\sqrt{Q_g Q_c}$$

Where:

- Q is the estimated combined inlet flow in sag (m^3/s)
- Q_g is the surface grate flow (m^3/s)
- Q_c is the curb opening flow (m^3/s)
- K_c is a reduction factor, assumed to be 0.40 for cast iron grates

The Titan Foundary TF-102 and WD Valve Box WD-55 Rolled Curb inlet was handled somewhat differently. Since the inlet slots on the rolled curb section are more horizontal than verical, the inlet was assumed to act like surface grate only and not as separate surface grate and curb opening. As the depth over the grate increases to include the curb slots, the weir perimeter was increased to include the additional opening width. The uppermost slot along the curb-side of the grate was assumed to contribute fully as a weir along the back side of the grate at a depth of 120% over the maximum depth (e.g. 120% of the 95 mm height between depressed gutter and top of curb). The weir length along the zigzag pattern of the bicycle-friendly slots was simplified as a straight line rather than follow the zipzag.

Once the depth increased to where the performance as a orifice would govern, the slots were assumed to act like an orifice. Rating curves were developed for both the older 1981 bicycle grate with 9 zigzag slots and the newer 2014 ‘grader friendly’ version with 7 zig zag slots on the gutter pan and 3 straight slots on the base of curb and rolled curb.

The Titan Foundary TF-100 Rectangular Surface Inlet was calculated similarly as a weir-orifice. Since the weir and orifice rating curves intersect, no transition zone between the curves was assumed to be required. Two versions of the rating curve were developed, one representing the grate in an open area and a second representing the grate installed near a curb.

The standard Round Grate represented by a Titan Foundary TF-101 or WD Valve Box WD-50G was computed similarly as a single weir-orifice.

The rating curves for six typical pavement inlet types are shown on **Figure 5B.6**, including both tabular depth versus derated flow and as a graph. The assumed discharge coefficients were 1.80 for all weir types and 0.61 for all orifice types.

For design, sag inlets were assumed to operate at 30% reduced capacity due to partial clogging. This was calculated by simply decreasing the estimated flow by 30% (e.g. multiplying by 1 minus 0.30 = 0.70) for the rating curves, and not by assuming a shorter weir perimeter or decreased orifice area due to clogging. The derating can be removed from the tabular rating curve values by dividing the flow values by 0.70 or multiplying by 1.429.

Derating Factor 30%

Flow (cms)

Depth (m)	Round Grate in Open	Square Grate in Open	Square Grate near Curb	Rolled Curb (newer)	Rolled Curb (older)	Barrier Curb & Gutter
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.010	0.002	0.003	0.002	0.002	0.001	0.002
0.020	0.006	0.007	0.006	0.005	0.004	0.005
0.030	0.011	0.013	0.011	0.009	0.008	0.009
0.040	0.016	0.020	0.017	0.015	0.012	0.014
0.050	0.023	0.028	0.024	0.021	0.017	0.020
0.060	0.030	0.037	0.031	0.027	0.022	0.026
0.070	0.038	0.047	0.039	0.035	0.028	0.032
0.080	0.046	0.058	0.048	0.044	0.034	0.039
0.090	0.051	0.069	0.057	0.055	0.041	0.046
0.100	0.054	0.072	0.067	0.072	0.048	0.051
0.110	0.056	0.076	0.076	0.077	0.055	0.055
0.120	0.059	0.079	0.079	0.080	0.071	0.059
0.130	0.061	0.083	0.083	0.083	0.074	0.062
0.140	0.064	0.086	0.086	0.086	0.076	0.066
0.150	0.066	0.089	0.089	0.089	0.079	0.069
0.160	0.068	0.092	0.092	0.092	0.082	0.072
0.170	0.070	0.094	0.094	0.095	0.084	0.075
0.180	0.072	0.097	0.097	0.098	0.087	0.077
0.190	0.074	0.100	0.100	0.101	0.089	0.080
0.200	0.076	0.102	0.102	0.103	0.091	0.082
0.220	0.080	0.107	0.107	0.108	0.096	0.087
0.240	0.083	0.112	0.112	0.113	0.100	0.092
0.260	0.087	0.117	0.117	0.118	0.104	0.096
0.280	0.090	0.121	0.121	0.122	0.108	0.100
0.300	0.093	0.125	0.125	0.126	0.112	0.104
0.320	0.096	0.129	0.129	0.131	0.116	0.108
0.340	0.099	0.133	0.133	0.135	0.119	0.111
0.360	0.102	0.137	0.137	0.138	0.123	0.115
0.380	0.105	0.141	0.141	0.142	0.126	0.118
0.400	0.108	0.145	0.145	0.146	0.129	0.121
0.420	0.110	0.148	0.148	0.150	0.132	0.125
0.440	0.113	0.152	0.152	0.153	0.135	0.128
0.460	0.115	0.155	0.155	0.157	0.139	0.131
0.480	0.118	0.159	0.159	0.160	0.142	0.134
0.500	0.120	0.162	0.162	0.163	0.144	0.137

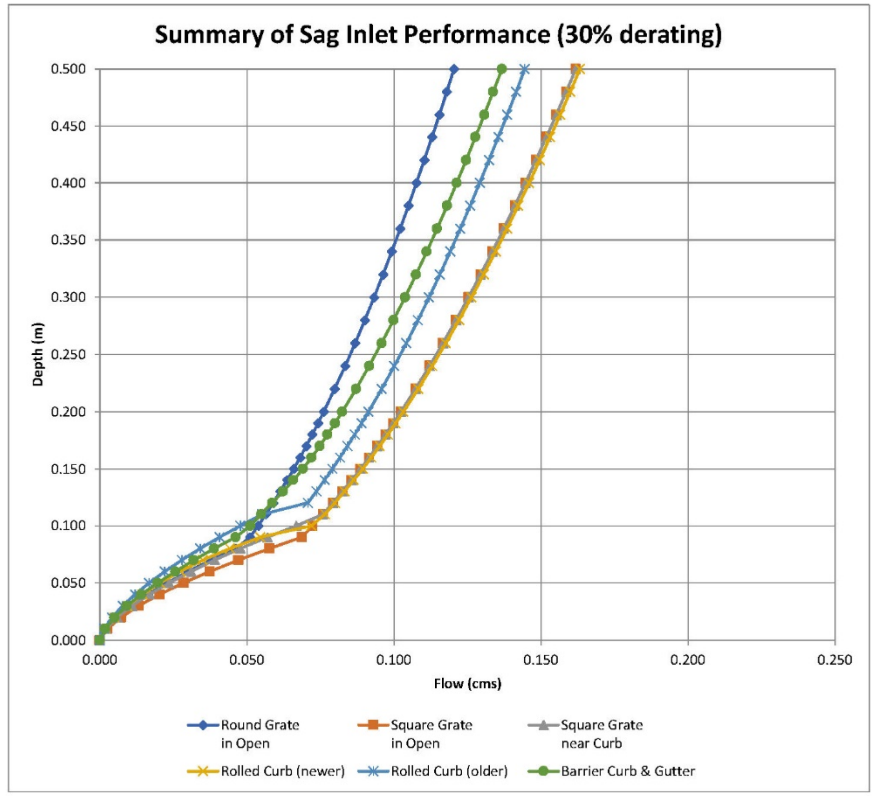

Figure 5B.6 – Summary of Sag Inlet Performance (30% derating)

5B.3.3 Inlets in Turnout Pockets

Where additional on-grade inlet capacity is required but the creation of a sag low inlet point is not desired, then a higher capacity on-grade inlet can be achieved by relocating on-grade inlets into a small turnout pocket. The turnout is a widened gutter section, widen an additional 1.0 to 1.5m into the boulevard, refer to Standard Construction Specifications for turnout pocket detail.

The pocket has the same or greater crossfall as the roadway and creates a mini-sag situation with more head on the inlets and allowing the curb inlet portion to come into play. Since the pocket is outside of the normal driving lane, there is greater opportunity to construct localized grading to achieved greater head on the inlet grates.

The turnout pocket is analyzed for both the curb opening capacity, using FHWA HEC-22 methodology, and the sag inlet capacity described in the previous Section, and the lowest capacity governs the estimated overall pocket inlet capacity. Typically, the curb opening length governs the overall capacity during lower gutter flow depth, and the sag inlet capacity governs the overall capacity during higher depth of gutter flow.

The required length of curb opening for complete flow capture is estimated as follows:

Equation 5B.10 - Length of Curb Opening

$$L_T = K * Q^{0.42} * S_L^{0.30} * \left(\frac{1}{n * S_X}\right)^{0.60}$$

Where:

- L_T is the estimated total length of curb opening for complete flow capture (m).
- K is a constant, equal to 0.817 for metric units.
- Q is gutter flow (m^3/s).
- S_L is the longitudinal slope (fraction, not percent slope).
- n is the Manning roughness of pavement, typically 0.016 for new broom-finished concrete gutter pan.
- S_X is the pavement crossfall (fraction, not percent slope).

The efficiency of a curb opening shorter that L_T is estimated as follows:

Equation 5B.11 - Flow Capture Efficiency of a Curb Opening

$$E_c = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \text{ for } L < L_T$$

Where:

- E_c is the capture efficiency of curb opening.
- L is the actual length of curb opening, shorter than L_T (m).

Standard properties for turnout pockets were assumed to be as follows:

Table 5B.2 - Turnout Pocket Inlet Properties

Parameter	Single	Double	Triple
Number of Inlets	1	2	3
Pocket width (m)	1.10	1.10	1.10
Effective Curb Opening Length (m)	6.5	8.5	10.5
Pocket crossfall (%)	2.0%	2.0%	2.0%
Sag Performance	Std Barrier C&G Inlet (in sag)	Std Barrier C&G Inlet (in sag)	Std Barrier C&G Inlet (in sag)

In addition to higher capacity, the installation of catch basins within turnout pockets also improves roadway ride and extends catch basin grate life since vehicles are not driving over the catch basin units. However, the pockets are susceptible to collecting debris and litter and the pocket itself encroaches into the boulevard area so it not suitable where it would conflict the pedestrian zone.

Appendix C Pipe Structure Design

5C PIPE STRUCTURE DESIGN

Pipe structure design shall be completed using industry standard methods for rigid and flexible pipe types. Design calculations shall be submitted for all pipes deeper than 4.0 m cover and 525 mm and larger diameter.

5C.1 Design Loading

Design loading for pipe structure design is as follows:

- Dead Loading – Assumed backfill unit weight of 2,100 kg/m^3 (approx. 130 pcf) or using the results of soil unit weight analysis from site-specific geotechnical testing.
- Live Loading - AASHTO HS20 or CHBDC CL-600 design vehicle.
- Groundwater / Buoyancy – 1.0m below ground surface or using the results of recent site-specific piezometer testing.

5C.2 Rigid Pipe External Load Design

The use of ACPA Indirect Design is required for rigid pipes deeper than 4.0 m of cover.

The use of ACPA / OCPA / CCPPA online PipePac Three Edge Bearing analysis design tool is recommended, available at:

https://www.concretepipe.org/pipe-box-resources/software/pipepac/

Design Parameters are as follows:

- Design Standard – ASTM C76 / C76M
- Wall thickness, B or C wall matching expected pipe supplier
- Soil Type - Gravelly Sand 2,120 kg/m^3 (or site-specific)
- Pipe Fluid Load to be included
- Trench Width – reasonable estimate based on excavation, working room and trench box, typically 0.45 to 0.60 m outside of pipe walls on either side of trench
- Ku’ = 0.1924 (Granular soil without cohesion)
- Live Load – AASHTO HS 20 or CL-600 design vehicle plus impact loading, and check versus worst case construction equipment loading.
- Pipe Bedding – Assumed Type 2 (VAF = 1.40) or Type C

5C.3 Flexible Pipe External Load Design

External Load Design for flexible buried pipes is required for flexible pipes deeper than 4.0 m of cover. This analysis should use the Modified Iowa Method to estimate pipe deflection. The maximum permissible deflection for sewers is 5%.

Equation 5C.1 - Modified Iowa Formula

$$\Delta x = \frac{Constant * Load}{Pipe\ Stiffness + Soil\ Stiffness} = \frac{K(D_L P_d + P_l)}{0.149PS + 0.061E'_{design}}$$

Where:

- Δx is the estimated horizontal deflection (as a percentage of diameter)
- K is the empirical flexible pipe bedding factor (not to be confused with the rigid pipe bedding factor), assumed to be equal to 0.100 for design and representing bedding angles between 60 and 70 degrees which are achievable in practice,
- D_L is the empirical deflection lag factor (or time lag factor), a measure of the increase in load with time as the backfill (or soil overtop of the pipe installed by trenchless methods) settles. A value of 1.5 is recommended for design,
- P_d is the dead load pressure, equal to the soil prism load (kPa or psi),
- P_l is the live load pressure (kPa or psi),
- PS is the pipe stiffness (kN/m/m equating to units of kPa or lbs/in/in equating to units of psi),
- E'_{design} is the Composite Modulus of Soil Reaction (kPa or psi), a function of the Moduli of Soil Reaction of both the pipe backfill material and the surrounding native soil material, and the relative width of the backfill material.

Vertical deflection can be assumed to be approximately equal to horizontal deflection or Masada's correction used to estimated vertical deflection:

Equation 5C.2 - Masada's Vertical Deflection Formula

$$\left|\frac{\Delta y}{\Delta x}\right| = 1 + \frac{0.0094E'_{design}}{PS}$$

Where

- Δy is the estimated vertical deflection,
- Δx is the estimated horizontal deflection computed with the Modified Iowa formula.

5C.3.1 PVC Pipe

For PVC pipe, the use of the Uni-Bell PVC Pipe Association online external load design calculator is recommended, and is available at:

https://www.uni-bell.org/External-Load-Design-Calculator

Recommended design parameters are as follows:

- Modulus of Elasticity E (PVC to ASTM D1784 cell class 12454) 2,758,000 kPa (400,000 psi)
- Deflection Lag Factor 1.5
- Bedding Constant K 0.100
- Trench Width - – reasonable estimate based on excavation, working room and trench box, typically 0.45 to 0.60 m outside of pipe walls on either side of trench.
- Modulus of Soil Reaction – 6,900 kPa (1,000 psi) assuming coarse grained soils and moderate compaction to AWWA 605 Type 4 embedment, or estimated using detailed soil properties.
- Maximum permissible vertical deflection for design = 5.0%

5C.3.2 Corrugated Steel Pipe

CSPI "Handbook of Steel Drainage and Highway Construction Products" Chapter 6 methodology for pipe structural design is recommended, and available at:

https://cspi.ca/technical-information/