SECTION 6 – DRAINAGE SYSTEM DESIGN REQUIREMENTS

6.01. General

The purpose of the Town of Prosper's drainage design standards is to provide for the safety and welfare of the general public, protect and improve water quality, protect aquatic resources, and mitigate flood and erosion related damages to private and public property including lakes and streams within the community while allowing economic development and growth within the Town and Extraterritorial Jurisdiction.

This section provides guidelines for design of drainage facilities in the Town of Prosper. The procedures outlined herein shall be followed for all drainage design and review of plans submitted to the Town. It is the responsibility of the Engineer to provide all necessary calculations and designs described herein. The Engineer shall provide the Town the data, calculations, and designs necessary to demonstrate the design does not adversely impact the surrounding or downstream property and meet local, state, and federal rules, regulations, and requirements. Deviation from the requirements of these standards may be granted by the Deputy Director of Engineering Services.

Additional guidelines and requirements for the design of drainage facilities in the Town of Prosper can be found online at www.prospertx.gov or by clicking the hyperlinks below.

- Town of Prosper Subdivision Ordinance No. 17-41
- Doe Branch Ordinance No. 15-51
- Stormwater Management Program
- Stormwater Ordinance No. 16-77

6.02. Estimating Stormwater Runoff

The selection of which method to use for calculating runoff depends upon the size of drainage basin contributing runoff at a most downstream point of a project. Two methods are generally acceptable for use in the Town of Prosper for design of drainage facilities: The Rational Method and the Unit Hydrograph Method. The Rational Method is acceptable for drainage basins less than 200 acres while the Unit Hydrograph Method is required for larger drainage basins.

Runoff computations shall be based upon fully developed watershed conditions in accordance with the Future Land Use Plan from the Town of Prosper's Comprehensive Plan or existing zoning, whichever is greater. If an approved detention/retention facility is in operation, the design engineer may size downstream drainage facilities based on consideration of the detention effects of the facility if approved by the Deputy Director of Engineering Services.

A. Rational Method

The Rational Method should be used to estimate stormwater runoff peak flows from small drainage basins of less than 200 contributing acres. The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required.

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration, T_C (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA$$

where:

Q = maximum rate of runoff (cfs)
C = runoff coefficient (dimensionless)
I = average rainfall intensity for a duration equal to the T_C (In/hr)
A = drainage area contributing to the design location (acres)

B. All runoff calculations shall be based upon a fully developed watershed. Larger coefficients shall be used if considered by the Deputy Director of Engineering Services to be appropriate to the project Runoff Coefficient (C).

Runoff coefficients "C" shall be based on the Future Land Use Plan, which is included in the Town's Comprehensive Plan. Runoff coefficients reflecting various land uses can be found in Table 6.1.

Composite runoff coefficients based on the percentage of different types of surfaces or land uses in the drainage areas can be considered when overall developed land use does not necessarily match up with those shown in Table 6.1. Composites can be made with the values from Table 6.1 by using percentages of the different developed land uses. If a land use is not covered in Table 6.1, appropriate values may be considered from the latest NCTCOG iSWM Manual. A detailed analysis with calculations shall be provided when using composite runoff coefficients.

TABLE 6.1 – Values for Runoff Coefficient

Land Use (Zoning)	Runoff Coefficient "C"	Min. Inlet Time (*minutes*) "T_C"
Agricultural and Undeveloped Areas	0.30	20
Estate Style Residential (> 1 acre)	0.45	15
Low-Density Residential (> 1/2 acre)	0.55	15
Medium-Density Residential (≥ 1/4 acre)	0.60	15
High-Density Residential (< 1/4 acre)	0.65	15
Town Home	0.70	10
Multiple Family	0.85	10
Non-Residential Uses	0.85	10
Neighborhood Park*	0.55	15
Community Park*	0.85	15
Open Spaces and Common Areas	0.40	15
School	0.85	10
Thoroughfares	0.90	10

**coefficients for park areas are dependent on types of improvements including but not limited to parking lots, trails, playgrounds, and various structures.*

C. Time of Concentration

Time of concentration (T_C) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet. The NRCS methodology (formerly SCS method) shall be used to determine time of concentration in the Town of Prosper. This method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path.

$$T_C = T_{t1} + T_{t2} + T_{t3} + ... + T_n$$

where:
T_C = time of concentration (min)
T_{tn} = travel time of a segment n (min)
n = number of segments comprising the total hydraulic length

The segments used in for this method may be of three types: sheet flow, shallow concentrated flow, and open channel flow.

1. Sheet Flow: The maximum allowable length for sheet flow is 100 feet. The T_t in minutes for sheet flow is determined using the following equation:

$$T_t = \frac{0.42(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

T_t = travel time (min)
n = Manning's roughness coefficient (Table 6.2)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall, 3.6 in
S = slope of hydraulic grade line (land slope, ft/ft)

Table 6.2 Sheet Flow 'n' Values

(flow depth generally ≤ 0.1 ft)

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
• Residue cover less than 20%	0.06
• Residue cover greater than 20%	0.17
Grass:	
• Short Prairie Grass	0.15
• Dense grasses[1]	0.24
• Bermuda grass	0.41
Range (natural)	0.13
Woods:	
• Light underbrush	0.40
• Dense underbrush	0.80

1: Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

2. Shallow Concentrated Flow: Begins where sheet flow ends. Travel time is the ratio of flow length to flow velocity and calculated as follows:

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

T_t = travel time (min)
L = flow length (ft)
V = average velocity (fps)

$V_{Unpaved} = 16.1345*(S)^{0.5}$,

$V_{Paved} = 20.3282*(S)^{0.5}$

Average velocities for estimating travel time for shallow concentrated flow should be estimated for paved or unpaved areas using the above equations.

3. Open Channel Flow: Shallow concentrated flow is assumed to occur after sheet flow ends at shallow depths of 0.1 to 0.5 feet. Beyond that channel flow is assumed to occur. Open channels are assumed to begin where surveyed cross-sectional information has been obtained, where channels are visible on aerial photographs, or where bluelines (indicating streams) appear on U.S. Geological Survey

(USGS) quadrangle sheets, or in some cases, closed storm sewer systems. The T_t for open channel flow is determined using the following equations:

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

$$V = \frac{1.49 r^{\frac{2}{3}} S^{\frac{1}{2}}}{n}$$

L = flow length (ft)
T_t = travel time (min)
V = average velocity (ft/s)
r = hydraulic radius (ft), A/P
A = cross sectional flow (ft^2)
P = wetted perimeter (ft)
S = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient

The Engineer shall compare the calculated time to the time listed in Table 6.1. If the calculated T_C differs from the value in Table 6.1, the Engineer shall provide information to justify the T_C calculations.

D. Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Intensity-duration frequency (IDF) relationships were developed using information acquired from the US Geological Survey (USGS) in cooperation with Texas Department of Transportation (TxDOT) for each county in the North Central Texas region. The information below is based on these results as found in the NCTCOG iSWM manual.

The design of all storm drainage facilities within the Town of Prosper shall be based on rainfall information as derived by the equation below (if the Rational Method is used). Storm durations shall be chosen based on the appropriate time of concentration.

$$I = \frac{b}{(T_c + d)^e}$$

Where:
I = the average rainfall intensity (in/hr)
T_C = time of concentration in minutes,
b,d,e = dimensionless coefficients based on the specific rainfall return period (Table 6.3 and 6.4)

TABLE 6.3 – Intensity Coefficient Table - Collin County

	2-Year	5-Year	25-Year	100-year
b	50.523	64.259	76.069	86.709
d	9	11	11	11
e	0.79822	0.78901	0.75875	0.73702

Quick Reference: Tc 10 → I_{100} = 9.20 in/hr
Tc 15 → I_{100} = 7.86 in/hr

TABLE 6.4 – Intensity Coefficient Table - Denton County

	2-Year	5-Year	25-Year	100-year
b	50.455	65.467	78.538	95.776
d	9	11	11	12
e	0.80553	0.79891	0.76912	0.7566

Quick Reference: Tc 10 → I100 = 9.24 in/hr
Tc 15 → I100 = 7.91 in/hr

E. Unit Hydrograph Method

Unit Hydrograph Methods may be used to compute storm water discharges for all watersheds, no matter the size, and shall exclusively be used to compute storm water discharges produced by (1) watersheds where large, regional storm water detention facilities exist or are anticipated upstream of the project, or (2) all watersheds larger than 200 acres.

1. Two unit hydrograph methods are acceptable to use in the Town of Prosper: the NRCS (formerly SCS) Dimensionless Unit Hydrograph Method and the Snyder's Unit Hydrograph Method. Any other method shall be based upon standard and accepted Engineering Principles normally used in the profession, and subject to the approval of the Deputy Director of Engineering Services prior to use.

2. The unit hydrograph method shall be based upon fully developed watershed conditions assuming no effects from small on-site detention facilities for maintaining peak rate of runoff. The detention effects of large regional detention facilities can be taken into account in unit hydrograph methods.

3. Circumstances that may require the use of a unit hydrograph method include open channels, reclaiming floodplains, creating lakes, regional detention/retention facilities or building other types of drainage related facilities on major drainage courses. Design engineers of these types of facilities should be aware that designing for fully developed watershed conditions shall require calculation of fully developed flows instead of flows used from Federal Emergency Management Agency's (FEMA) flood insurance studies for the Town of Prosper. Use of rational method is allowed for design of storm sewer within the project area.

F. Hydrologic Computer Programs

HEC-HMS shall be used in developing all hydrologic models. Other hydrologic models may be used upon approval from the Deputy Director of Engineering Services. The following criteria should be used:

a. 24-hour storm duration using an SCS Type II distribution.
b. Rainfall Intensity values calculated using coefficients provided in Tables 6.3 and 6.4.
c. T_C values shall be calculated as shown in Section 6.02C.

d. The SCS Runoff Curve Number (CN) used shall be based on fully developed watershed conditions. CN values for urban areas are provided in Table 6.5. Reference NRCS TR-55: Urban Hydrology for Small Watersheds for other area types.

TABLE 6.5 – Runoff Curve Numbers for Urban Areas

Cover type and hydrologic condition	*Average percent impervious area* [2]	A	B	C	D
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and I_a=0.2S

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

All studies and reports using this information shall be submitted to the Town of Prosper for review in both a hard copy report and electronic format. Input and output shall be summarized in all reports.

6.04. Street Flow/Capacity

A. Street Flow Limitations

Street capacities shall be designed for the 100-year design storm. The depth of flow in the streets shall follow the limitations in Table 6.6, and shall at no time exceed the top of curb. Inserting the slope (S, ft/ft) into the equations shown on Table 6.7, based on the type of thoroughfare and paving section, provide capacity for streets to comply with the above requirement.

TABLE 6.6 – Water Spread Limits for Roadways During 100-year Storm Event

Street Classification	Limits of Water Spread
Major Thoroughfare (6LD)	One 12-foot lane shall remain open (“dry lane”) in each direction
Minor Thoroughfare (4LD)	One 9-foot lane shall remain open (“dry lane”) in each direction
Collector Road (3L, 2LC)	To top of curb
Residential Road (2LN)	To top of curb
Alley	100-yr storm contained within edge of pavement

TABLE 6.7 – Capacity of Parabolic Crown Streets

Type of Thoroughfare	Cross Fall/ Crown Height	Paving Section (Curb Fact to Curb Face)	Street Capacity (ft^3/sec)	Street Capacity to Crown (ft^3/sec)	Gutter Capacity (ft^3/sec)
2LC	6” crown	36’	$Q=151.46*(S^{1/2})$	$Q=151.46*(S^{1/2})$	$Q=75.3*(S^{1/2})$
2LN	5” crown	30’	$Q=203.13*(S^{1/2})$	$Q=93.14*(S^{1/2})$	$Q=46.57*(S^{1/2})$

Inlets shall be placed upstream of all intersections to minimize cross flow. No flow shall be allowed to cross or bypass inlets at an intersection between two thoroughfares. Residential and collector road intersections shall be designed for flow to not exceed two (2) inches in height across a valley gutter.

6.05. Inlet Design

A. Gutter Flow

Curb inlets shall be placed to ensure that the 100-year flow in a roadway does not exceed the water spread limitation requirements and shall generally follow the design guidance for gutter flow hydraulics published in HEC-22, 3rd Edition, Urban Drainage Design Manual. The following form of the Manning’s equation should be used to evaluate gutter flow hydraulics:

$$Q = \left[\frac{0.56}{n}\right] Sx^{1.67} S^{0.5} T^{2.67}$$

Q = Flow rate (cfs)
S_x = Cross slope (ft/ft)
S = Longitudinal slope (ft/ft)
T = Width of flow (ft)
n = Manning’s roughness coefficient

Depth of flow in the gutter can be calculated using the following modified form of the equation above:

$$y_o = Z\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8}$$

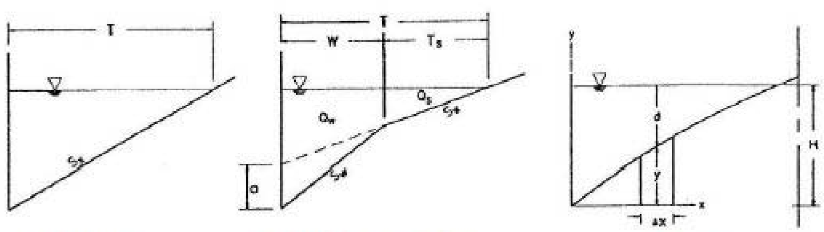
y_o = depth of water in the curb and gutter cross section (ft)
Z = 1.24

Water Spread or ponding width can be calculated as:

$$T = \frac{y}{S_X}$$

Figure 6.1 shows typical gutter cross sections.

FIGURE 6.1 – Typical Gutter Cross Sections



a. Triangular b. Triangular with Gutter c. Parabolic

B. Inlet Capacity Calculations on Grade

Curb opening heights vary in dimension, however, a typical maximum height is approximately 4 to 6 inches. The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by the equation below:

$$L_T = 0.60Q^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6}$$

L_T = Required length of inlet (ft)
Q = Total flow in the roadway (cfs)
S = Roadway longitudinal slope (ft/ft)
S_x = Cross slope (ft/ft)

To determine the capacity of a curb inlet on grade, first determine the ratio of the flow in the locally depressed gutter section to the total flow in the road.

$$E_o = 1 \Big/ \left\{ 1 + \frac{S_w}{S_x}\left[\left(1 + \frac{S_w/S_x}{(T/W) - 1}\right)^{2.67} - 1\right]^{-1} \right\}$$

E_o = Ratio of flow in the depressed gutter to the total flow
S_w = Gutter cross slope (ft/ft)
S_x = Roadway cross slope (ft/ft)

T = Width of flow in roadway (ft)
W = Width of depressed gutter section (ft)

Then calculate the equivalent cross slope at the depressed curb inlet opening.

$$S_e = S_x + \frac{a}{W} E_o$$

S_e = Equivalent cross slope (ft/ft)
S_x = Roadway cross slope (ft/ft)
a = Gutter Depression Depth (ft)
W = Width of depressed gutter section (ft)
E_o = Ratio of flow in the depressed gutter to the total flow

Then calculate the inlet length required to capture 100% of the gutter flow.

The efficiency of a curb inlet opening shorter than L_T is:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$

E = Inlet efficiency
L = Length of the curb inlet opening (ft)
L_T =Required length of inlet to capture 100% of the roadway flow (ft)

The total flow captured by the curb inlet is:

$$Q_i = EQ$$

Q_i = Flow capture by inlet (cfs)
E = Inlet efficiency
Q = Total flow in the roadway (cfs)

Tables 6.8 - 6.12 show the capacity of inlets on straight crown streets, parabolic crown streets and common grate combinations for inlets on grade.

TABLE 6.8 – Capacity of Inlets for Straight Crown Streets

RECESSED AND STANDARD ON GRADE CURB INLET (1/4" per Foot Cross Slope)			RECESSED AND STANDARD ON GRADE CURB INLET (3/8" per Foot Cross Slope)		
Inlet Length	Gutter Slope	Inlet Capacity	Inlet Length	Gutter Slope	Inlet Capacity
8'	6%	3.8 cfs	8'	6%	4.0 cfs
8'	3%	4.0 cfs	8'	3%	4.3 cfs
8'	2%	4.2 cfs	8'	2%	4.5 cfs
8'	1%	4.4 cfs	8'	1%	4.8 cfs
8'	0.6%	4.7 cfs	8'	0.6%	5.2 cfs
10'	6%	4.8 cfs	10'	6%	5.2 cfs
10'	3%	5.1 cfs	10'	3%	5.6 cfs
10'	2%	5.4 cfs	10'	2%	6.0 cfs
10'	1%	5.7 cfs	10'	1%	6.4 cfs
10'	0.6%	6.2 cfs	10'	0.6%	6.9 cfs
12'	6%	6.0 cfs	12'	6%	6.5 cfs
12'	3%	6.4 cfs	12'	3%	7.0 cfs
12'	2%	6.8 cfs	12'	2%	7.5 cfs
12'	1%	7.3 cfs	12'	1%	8.2 cfs
12'	0.6%	7.8 cfs	12'	0.6%	8.7 cfs

Notes:

1. Sage inlets will be designed to accept no more than two (2) cfs per foot of opening.
2. Inlet capacities for other gutter slopes not listed may be interpolated.

TABLE 6.9 – Capacity of Inlets for Parabolic Crown Streets

STANDARD ON GRADE CURB INLET (6" Parabolic Crown)			STANDARD ON GRADE CURB INLET (5" Parabolic Crown)			STANDARD ON GRADE CURB INLET (4" Parabolic Crown)		
Inlet Length	Gutter Slope	Inlet Capacity	Inlet Length	Gutter Slope	Inlet Capacity	Inlet Length	Gutter Slope	Inlet Capacity
8'	6%	4.2 cfs	8'	6%	4.2 cfs	8'	6%	4.1 cfs
8'	3%	4.5 cfs	8'	3%	4.5 cfs	8'	3%	4.4 cfs
8'	2%	4.8 cfs	8'	2%	4.8 cfs	8'	2%	4.6 cfs
8'	1%	5.2 cfs	8'	1%	5.2 cfs	8'	1%	5.0 cfs
8'	0.6%	5.6 cfs	8'	0.6%	5.6 cfs	8'	0.6%	5.3 cfs
10'	6%	5.6 cfs	10'	6%	5.6 cfs	10'	6%	5.3 cfs
10'	3%	6.0 cfs	10'	3%	6.0 cfs	10'	3%	5.7 cfs
10'	2%	6.5 cfs	10'	2%	6.5 cfs	10'	2%	6.1 cfs
10'	1%	7.0 cfs	10'	1%	7.0 cfs	10'	1%	6.6 cfs
10'	0.6%	7.5 cfs	10'	0.6%	7.5 cfs	10'	0.6%	7.1 cfs
12'	6%	7.0 cfs	12'	6%	7.0 cfs	12'	6%	6.6 cfs
12'	3%	7.5 cfs	12'	3%	7.5 cfs	12'	3%	7.2 cfs
12'	2%	8.2 cfs	12'	2%	8.2 cfs	12'	2%	7.5 cfs
12'	1%	9.0 cfs	12'	1%	9.0 cfs	12'	1%	8.4 cfs
12'	0.6%	9.6 cfs	12'	0.6%	9.6 cfs	12'	0.6%	9.0 cfs

Notes:

3. Sage inlets will be designed to accept no more than two (2) cfs per foot of opening.
4. Inlet capacities for other gutter slopes not listed may be interpolated.

TABLE 6.10 – Capacity of Two Grate Combination Inlet on Grade

Flow (Cubic Feet per Second)	Gutter Slope					Flow (Cubic Feet per Second)	Gutter Slope				
	0.6%	1.0%	2.0%	3.0%	6.0%		0.6%	1.0%	2.0%	3.0%	6.0%
1						10	62%	61%	61%	60%	59%
1.5	84%	83%				11	61%	61%	60%	59%	58%
2	78%	77%				12	61%	60%	59%	58%	57%
2.5	75%	74%	73%	73%		13	60%	59%	58%	57%	56%
3	73%	72%	71%	70%	70%	14	59%	58%	57%	56%	56%
3.5	72%	71%	69%	68%	68%	15	58%	58%	57%	56%	56%
4	70%	69%	68%	68%	67%	20	57%	56%	56%	55%	54%
5	68%	67%	67%	66%	65%	25		54%	54%	54%	54%
6	67%	66%	65%	64%	63%	30			54%	54%	54%
7	65%	64%	64%	63%	62%	35				54%	53%
8	64%	63%	62%	62%	61%	40				53%	53%
9	63%	63%	62%	61%	60%	45				52%	52%

Note: Capacity percentages not listed may be interpolated.

TABLE 6.11 – Capacity of Three Grate Combination Inlet on Grade

Flow (Cubic Feet per Second)	A. Gutter Slope					Flow (Cubic Feet per Second)	B. Gutter Slope				
	0.6%	1.0%	2.0%	3.0%	6.0%		0.6%	1.0%	2.0%	3.0%	6.0%
1	100%	100%				10	69%	67%	66%	65%	64%
1.5	100%	100%				11	68%	67%	65%	64%	63%
2	93%	90%	87%	86%		12	67%	66%	64%	63%	62%
2.5	90%	87%	84%	81%		13	66%	65%	64%	63%	61%
3	86%	83%	79%	77%	75%	14	66%	64%	63%	62%	60%
3.5	83%	80%	76%	75%	73%	15	65%	64%	62%	61%	59%
4	81%	78%	75%	73%	71%	20	62%	61%	59%	59%	58%
5	77%	75%	73%	71%	70%	25	59%	58%	57%	57%	56%
6	75%	73%	71%	69%	68%	30		57%	57%	56%	56%
7	73%	71%	69%	68%	67%	35			56%	56%	54%
8	71%	69%	68%	67%	66%	40			55%	55%	54%
9	70%	68%	67%	66%	64%	45				54%	53%

Note: Capacity percentages not listed may be interpolated.

TABLE 6.12 – Capacity of Four Grate Inlet on Grade

Flow (Cubic Feet per Second)	C. Gutter Slope					Flow (Cubic Feet per Second)	D. Gutter Slope				
	0.6%	1.0%	2.0%	3.0%	6.0%		0.6%	1.0%	2.0%	3.0%	6.0%
1	100%					10	75%	73%	70%	68%	67%
1.5	100%	100%				11	74%	72%	69%	68%	67%
2	100%	100%	97%	96%		12	74%	70%	67%	66%	66%
2.5	98%	97%	95%	87%		13	73%	70%	67%	66%	65%
3	96%	95%	85%	83%	80%	14	71%	69%	66%	65%	64%
3.5	95%	89%	83%	81%	78%	15	70%	68%	66%	65%	64%
4	90%	85%	81%	79%	77%	20	67%	66%	63%	62%	59%
5	84%	81%	78%	76%	75%	25		64%	61%	60%	58%
6	82%	79%	76%	74%	73%	30		62%	60%	58%	57%
7	80%	77%	74%	73%	71%	35		61%	59%	57%	56%
8	80%	76%	73%	71%	69%	40			58%	57%	56%
9	76%	74%	71%	70%	68%	45				56%	55%

Note: Capacity percentages not listed may be interpolated.

C. Inlet Capacity Calculations in Sag

The capacity of a curb inlet in sag depends on the water depth at the curb opening and the height of the curb opening. The inlet operates as a weir to a depth equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage and the capacity should be based on the lesser of the computed weir and orifice capacities.

1. If the depth of flow in the gutter (d) is less than or equal to 1.4 times the inlet opening height (h), (d≤1.4H), determine the length of inlet required considering weir control. Calculate the capacity of the inlet when operating under weir conditions with the following equation:

$$Q = C_w(L + 1.8W)d^{1.5}$$

2. Rearrange the above equation to produce the following relation for curb inlet length required:

$$L = \left(\frac{Q}{C_w {d_o}^{1.5}}\right) - 1.8W$$

Q = total flow reaching inlet (cfs)
C_w = weir coefficient (3.0)
d_o = head at inlet opening (ft)
L = length of curb inlet opening (ft)
W = lateral width of depression (ft)

$$d_o = z\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8}$$

d_o = depth of water in the curb and gutter cross section (ft)
Q = gutter flow rate (cfs)
n = Manning's roughness coefficient

S = longitudinal slope (ft/ft)
S_x = pavement cross slope (ft/ft)
Z = 1.24

3. If the depth of flow in the gutter is greater than the inlet opening height (d>H), determine the length of inlet required considering orifice control. The equation for interception capacity of a curb opening as an orifice follows:

$$Q = C_o h L \sqrt{2 g d_e}$$

Q = total flow reaching inlet (cfs)
C_o = orifice coefficient = 0.70
h = depth of opening (ft) (this depth will vary slightly with the inlet detail used)
L = length of curb opening inlet (ft)
g = acceleration due to gravity = 32.2 ft/s^2
d_e = effective head at the centroid of the orifice (ft) d_e=d – h/2

Rearranging the equation allows a direct solution for required length:

$$L = \frac{Q}{C_o h \sqrt{2 g d_e}}$$

4. If both steps 1 and 2 were performed (i.e., h<d≤1.4h), choose the larger of the two computed lengths as being the required length.

5. Select a standard inlet length that is greater than the required length, a minimum ten foot (10') opening, and shall have a minimum twenty-one inch (21") lateral.

6. Table 6.13 shows the capacity of common grate combination sizes for inlets at a sag.

TABLE 6.13 – Capacity of Grate Combination Inlet at Sag

Grate Combination Inlet Size	Max. Depth of Flow (Feet)	Inlet Cap.	Grate Combination Inlet Size	Max. Depth of Flow (Feet)	Inlet Cap.	Grate Combination Inlet Size	Max. Depth of Flow (Feet)	Inlet Cap.
2	0.6	15 cfs	3	0.6	22 cfs	4	0.6	29 cfs
2	0.5	13 cfs	3	0.5	18 cfs	4	0.5	24 cfs
2	0.4	10 cfs	3	0.4	15 cfs	4	0.4	20 cfs
2	0.3	8 cfs	3	0.3	12 cfs	4	0.3	16 cfs
2	0.2	6 cfs	3	0.2	9 cfs	4	0.2	11 cfs

D. Wye Inlet Capacity Calculations (weir)

$$\frac{Q}{P} = 3.1d^{3/2}$$

Q = flow (cfs)
P = perimeter of opening (ft)
d = depth (ft)

Wye (drop) inlets shall be located to collect water on non-paved areas where it is not practical to use a headwall. Table 6.14 shows the capacity for common drop inlet sizes of wye inlets.

TABLE 6.14 – Capacity of Wye Inlets

Standard Drop Inlet Size	Max. Depth of Flow (Feet)	Inlet Capacity	Standard Drop Inlet Size	Max. Depth of Flow (Feet)	Inlet Capacity	Standard Drop Inlet Size	Max. Depth of Flow (Feet)	Inlet Capacity
2' X 2'	1.0	22 cfs	3' X 3'	1.0	33 cfs	4' X 4'	1.0	44 cfs
2' X 2'	0.9	19 cfs	3' X 3'	0.9	28 cfs	4' X 4'	0.9	37 cfs
2' X 2'	0.8	16 cfs	3' X 3'	0.8	23 cfs	4' X 4'	0.8	32 cfs
2' X 2'	0.7	14 cfs	3' X 3'	0.7	19 cfs	4' X 4'	0.7	26 cfs
2' X 2'	0.6	10 cfs	3' X 3'	0.6	15 cfs	4' X 4'	0.6	20 cfs
2' X 2'	0.5	8 cfs	3' X 3'	0.5	12 cfs	4' X 4'	0.5	16 cfs
2' X 2'	0.4	6 cfs	3' X 3'	0.4	8 cfs	4' X 4'	0.4	11 cfs
2' X 2'	0.3	4 cfs	3' X 3'	0.3	5 cfs	4' X 4'	0.3	7 cfs
2' X 2'	0.2	2 cfs	3' X 3'	0.2	3 cfs	4' X 4'	0.2	4 cfs

E. Curb Inlet Placement

1. Placing several curb inlets at a single location is only permitted in areas with steep grades (4% or greater) to prevent flooding and avoid exceeding street capacity in flatter reaches downstream.

2. No more than twenty feet (20') of inlet shall be constructed at one location along one curb line.

3. Curb inlets shall be placed upstream from street intersections.

4. An emergency overflow path shall be provided on the plans for sag locations. An emergency overflow path is the path the storm water will take if the drainage facility becomes clogged or ceases to function as designed. The emergency overflow path must be located within public right-of-way or within a drainage easement on developed property and shall provide relief prior to water exceeding typical right-of-way elevation.

   a. An emergency overflow path shall include a minimum 3-foot concrete flume. See section 6.14.2 for easement and placement requirements.
   b. Concrete flume can be eliminated at discretion of Deputy Director of Engineering Services if evidence provided to assure Town that property owner would not alter grades. This can be done with CC&R language having HOA police and enforce overflow path design or similar.

5. Inlets are required at the beginning and/or end of a superelevation to prevent flow across the roadway.

F. The Inlet Spreadsheet provided in Figure 6.2 shall be provided with the construction plans for review by the Town. The spreadsheet provided is a minimum, more information may be provided in the Inlet Spreadsheet. A description of each of the columns shown is provided below:

FIGURE 6.2 – Inlet Calculations

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
INLET CALCULATIONS																				
INLET LOCATION		Design Storm Frequency	AREA RUNOFF - Q=CIA				Runoff (CFS)	Carry Over From Upstream Inlet	Total Gutter Flow	Gutter Slope	Gutter Capacity to Crown	Pavement Capacity to TC	Crown Type	SELECTED INLETS			Intercepted Flow	Carry Over to Downstream	Downstream Inlet No.	Comments
			Time of "C"	Intensity (in/hr)	Runoff Coeifficient	Area in Acres								Length "LI"	Type	Inlet Capacity				
No.	Station Number	(years)	(min)	"I"	"C"	"A"	"Q"	(cfs)	(cfs)	(ft/ft)	(cfs)	(cfs)		(feet)		(cfs)	(cfs)	(cfs)	No.	

- Column A: Inlet number.
- Column B: Inlet station number.
- Column C: Design storm.
- Column D: Time of concentration (minutes).
- Column E: 100-year Intensity (in/hr).
- Column F: Runoff Coefficient (C).
- Column G: Drainage area size (acres).
- Column H: 100-year runoff, Q=CIA (cfs).
- Column I: 100-year flow reaching the inlet from the upstream inlet (cfs).
- Column J: 100-year total gutter flow (Column 8 + Column 9) (cfs).
- Column K: Longitudinal slope of the approach gutter (ft/ft).
- Column L: Street capacity based on Manning's equation. For sag inlets calculate the street capacity for both the lower and higher station sides of the inlet and use the greater of the two.
- Column M: Total ROW capacity as a function of the cross-sectional area of the ROW at the inlet. For sag inlets, the total ROW capacity on the lower station side of the inlet.
- Column N: Street crown section type (straight crown or parabolic).
- Column O: Length of the inlet (feet).
- Column P: Inlet type.
- Column Q: Total capacity of the inlet (cfs).
- Column R: Flow from upstream (cfs).
- Column S: Flow continuing to downstream inlet (cfs).
- Column T: Downstream inlet number.
- Column U: Important comments relating to the inlet.

6.06. Closed Conduit Systems (Storm Sewers)

A. Design flow

A storm sewer's size, shape, slope, and friction resistance controls its hydraulic capacity. All enclosed systems shall be hydraulically designed and all required calculations shall be provided in the civil construction plans. The hydraulic gradient shall be calculated using the design flow, appropriate pipe size, and Manning's equation:

$$Q = \left(\frac{1.486}{n}\right) A \left(R^{2/3}\right) \left(S^{1/2}\right)$$

Q = runoff rate (cfs)
A = cross sectional area of the conduit (ft^2)
n = Manning's roughness coefficient (0.013 for concrete)

R = hydraulic radius (ft) (Area of conduit divided by wetted perimeter, R=A/P)
S = Slope of the hydraulic gradient (ft/ft)

B. Hydraulic Gradient

Conduits must be sized and slopes must be set such that runoff flows smoothly down the drainage system. To insure this smooth passage, the hydraulic gradient must be at the proper elevations. The proper starting elevation of the hydraulic gradient shall be set according to the applicable criteria listed below:

1. When a proposed conduit is to connect to an existing storm sewer, the hydraulic gradient of the proposed storm sewer should start at the elevation of the hydraulic gradient of the existing storm sewer based on an evaluation of the existing storm sewer with respect to the requirements found in this section. This criterion will be used for existing systems whether or not they are designed in accordance with this article.

2. When a proposed conduit enters an open channel, creek or flood control sumps, the 100-year hydraulic gradient of the proposed conduit should start at the 25-year water surface elevation of the receiving channel or creek when the ratio of the drainage area of the receiving creek (at the development) to the development area is 15 or greater. For ratios of less than 15, the 100-year water surface will be used on the receiving creek. Not only is it important to use the proper starting elevation for the hydraulic gradient, but proper hydraulic gradient elevations must be maintained for the length of the conduit. The inside top of the conduit should be at or below the hydraulic gradient. However, effort should be made to keep the top of the pipe as close to the hydraulic gradient as possible so that deep excavations to lay pipe are not required.

   The hydraulic gradient shall be kept one foot below the top of curb. If this cannot be obtained, the hydraulic gradient shall be at least 1.5 $V_1^2/2g$ feet below the gutter line, where V_1 is the velocity in the lateral.

C. Hydraulic Design

1. The hydraulic grade line (HGL) must be calculated for all storm sewer mains and laterals using appropriate head loss equations. In all cases, the storm sewer HGL must be at least one foot (1') below top of curb at each inlet.

2. In partial flow conditions, the HGL represents the actual water surface within the pipe. The velocity of the flow should be calculated based on actual area of flow, not the full flow area of the pipe or box.

3. Unless partial flow conditions exist, the beginning hydraulic gradient shall begin at either the inside top of the pipe or at the hydraulic gradient of the receiving stream at the coincident frequency, whichever is higher.

D. Lateral Design

1. The HGL shall be calculated for all proposed laterals and inlets, and for the existing laterals being connected into a proposed drainage system.

2. Laterals shall intersect the storm sewer at 60 degree angles. Connecting more than one lateral into a storm drain at the same joint localizes head losses; however, a manhole or junction structure must be provided. An exception to this rule may be considered when the diameter of the main line is more than twice as great as the diameter of the largest adjoining lateral.

3. Laterals shall not connect into downstream inlets.

4. All wye inlets and curb inlets ten feet (10') or larger shall have twenty-one inch (21") laterals as a minimum. All other curb inlets shall have eighteen inch (18") laterals as a minimum. Laterals shall be designed with future developed conditions in mind to facilitate extensions and increased flows.

E. Velocity Head Losses (H_L)

1. Adjustments are made in the HGL whenever the velocity in the main changes due to conduit size changes or discharge changes.

2. In determining the HGL for the lateral, begin with the hydraulic grade of the trunk line at the junction plus the H_L due to the velocity change. Where the lateral is in full flow, the hydraulic grade is projected along the friction slope calculated using Manning's Equation.

3. H_L losses or gains for pipe size changes, and other velocity changes will be calculated by the following formula:

$$H_L = \left[\frac{(V_2)^2}{2g}\right] - \left[\frac{(V_1)^2}{2g}\right]$$

H_L = Head loss or gain (ft)
V_1 = Upstream velocity (fps)
V_2 = Downstream velocity (fps)
g = Gravity constant (32.2 ft/s^2)

4. H_L for pipe in full flow at manholes, bends, and inlets, where the flow quantity remains the same, shall be calculated as follows:

$$H_L = K_j\left[\frac{V^2}{2g}\right]$$

H_L = Head loss or gain (ft)
V = Velocity in the lateral (fps)
g = Gravity constant (32.2 ft/s^2)
K_j = Coefficient of loss per Table 6.15

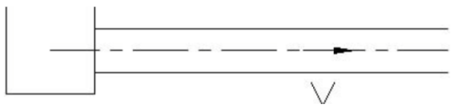
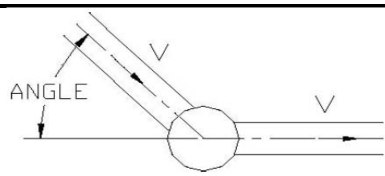
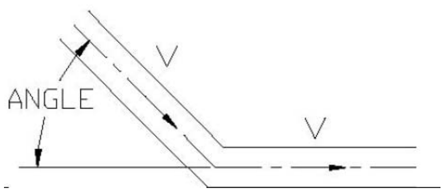
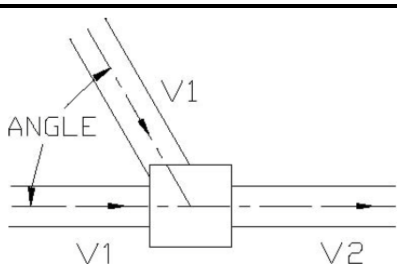
5. Head losses or gains at wyes, manholes and junction boxes where there is an increase in flow quantity shall be calculated as follows:

$$H_L = \left[\frac{(V_2)^2}{2g}\right] - K_j\left[\frac{(V_1)^2}{2g}\right]$$

H_L = Head loss or gain (ft)
V_1 = Upstream velocity (fps)
V_2 = Downstream velocity (fps)
g = Gravity constant (32.2 ft/s^2)
K_j = Coefficient of loss per Table 6.15

Pipe size changes associated with a wye connection shall be omitted from HGL computations.

Table 6.15 Velocity Head Loss in Closed Conduits

Inlet			
Schematic		K_j	
		1.25	
Manhole at Change in Pipe Direction			
Schematic	Angle	K_j	
	90°	0.55	
	60°	0.48	
	45°	0.42	
	30°	0.30	
	0°	0.05	
Bend in Pipe			
Schematic	Angle	K_j	
	60°	0.45	
	45°	0.35	
	30°	0.20	
Pipe Wyes			
Schematic	Angle	K_j	
		with MH	Without MH
	22 ½°	0.75	0.95
	45°	0.50	0.75
	60°	0.35	0.60
	90°	0.25	0.50

F. Storm Sewer Calculation Spreadsheet

The Engineer shall include a completed Storm Sewer Calculations Spreadsheet in the construction plans. An example spreadsheet can be found in Figure 6.3.

FIGURE 6.3 – Storm Sewer Calculation

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
RUNOFF COLLECTION POINT (Inlet or Manhole)		Distance Between Collection Points	INCREMENTAL DRAINAGE AREA				Accumulated "CA"	Time at Upstream Station (min)	Design Storm Frequency (years)	Intensity "I" (in/hr)	Storm Water Runoff "Q" (cfs)	Slope of Hydraulic Gradient "S" (ft/ft)	Selected Storm Sewer Size	Velocity in Sewer Between Connection Points "V" (fps)	Velocity Head Loss at Upstream Station	Head Loss Coeff. "Kj"	Inlet, Manhole, Bends or Junction Box Losses	Flow time in Sewer (min	Time at Downstream Station (min)	Remarks
Upstream Station	Downstream Station		Area No.	Drainage Area "A" (acres)	Runoff Coeff. "C"	Incremental "CA"														

G. Storm Sewer

1. Alignments of proposed storm sewer systems shall use existing easements and right-of-way. If located within an easement, the storm sewer shall be centered within the easement. If located within right-of-way, the centerline of the storm sewer shall be located under paving seven feet (7') from the back of curb. No part of the storm sewer is to be designed within the lime treated subgrade of a proposed pavement.

2. At points of change in storm sewer size, pipe crowns (soffits) shall be set at the same elevation.

3. Horizontal and vertical curve design for storm sewers shall take into account joint closure.

4. A minimum full flow velocity of two and a half feet per second (2.5 fps) and a minimum slope of one half percent (0.3%) shall be maintained in the pipe unless otherwise approved by the Deputy Director of Engineering Services.

5. Only standard sizes shall be used. The minimum allowable pipe size is eighteen inches (18") for public storm sewer. Pipe sizes shall not be decreased in the downstream direction, unless otherwise approved by the Deputy Director of Engineering Services.

6. The minimum grades shown in Table 6.16 will be maintained in the pipe.

TABLE 6.16 – Minimum Grades for Concrete Pipes

Pipe Diameter (Inches)	Slope (Feet/100 Feet)	Pipe Diameter (Inches)	Slope (Feet/100 Feet)
18	0.180	51	0.045
21	0.150	54	0.041
24	0.120	60	0.036
27	0.110	66	0.032
30	0.090	72	0.028
33	0.080	78	0.025
36	0.070	84	0.023
39	0.062	90	0.021
42	0.056	96	0.019
45	0.052	102	0.018
48	0.048	108	0.016

7. Laterals shall be connected to trunk lines using manholes or manufactured wye connections. Special situations may require laterals to be connected to trunk lines by a cut-in (punch-in). However, such cut-ins must be approved by the Deputy Director of Engineering Services.

8. Vertical curves in the storm pipe will not be permitted, and horizontal curves must meet manufacturer's requirements for offsetting of the joints.

9. The cover over the crown of circular pipe and box culverts shall be at least two feet from proposed top of pavement, or from ground surface when no pavement is anticipated. As a general rule and being the responsibility of the design engineer, the pipe cover should be based on the type of pipe used, the expected loads and the supporting strength of the pipe. Direct traffic box sections or less than required cover may be allowed in special situations with the approval of the Deputy Director of Engineering Services.

10. Corrugated metal and plastic pipe will not be allowed beneath pavement in public drainage easements and rights-of-way. Plastic pipe can be used in other locations only if authorized by the Deputy Director of Engineering Services.

11. In situations where only the downstream portion of an enclosed storm sewer system is being built, stub-outs for future connections must be included.

12. The required storm sewer and inlet capacity to meet existing and future needs, if applicable, shall be provided.

H. Manhole Placement

The following is a list of guidelines governing the placement of storm sewer manholes to ensure adequate accessibility of storm drainage system:

1. Storm sewer lines shall have points of access no more than five hundred feet (500') apart. For pipes forty-eight inches and larger access points shall be provided every one thousand feet (1,000'). A manhole shall be provided where this condition is not met.

2. A manhole shall be required where two or more pipes connect into a main at the same joint.

3. Trunk line size changes for lines with a diameter difference greater than 24 inches.

4. Vertical alignment changes where the algebraic slope difference ≥ 5%. No vertical bends allowed.

5. Future collection points as determined by the Deputy Director of Engineering.

I. Outfall Design

The Engineer shall demonstrate the drainage from the site is conveyed to an adequate outfall. An adequate outfall is a structure or location that is adequately designed as to not cause adverse flooding conditions, erosion, or any other adverse impacts. An adequate outfall shall also have capacity to convey the increased runoff.

1. All outfalls shall be designed with appropriate headwalls or outfall structures.

2. A stage-discharge curve shall be developed for the full range of flows that the structure would experience.

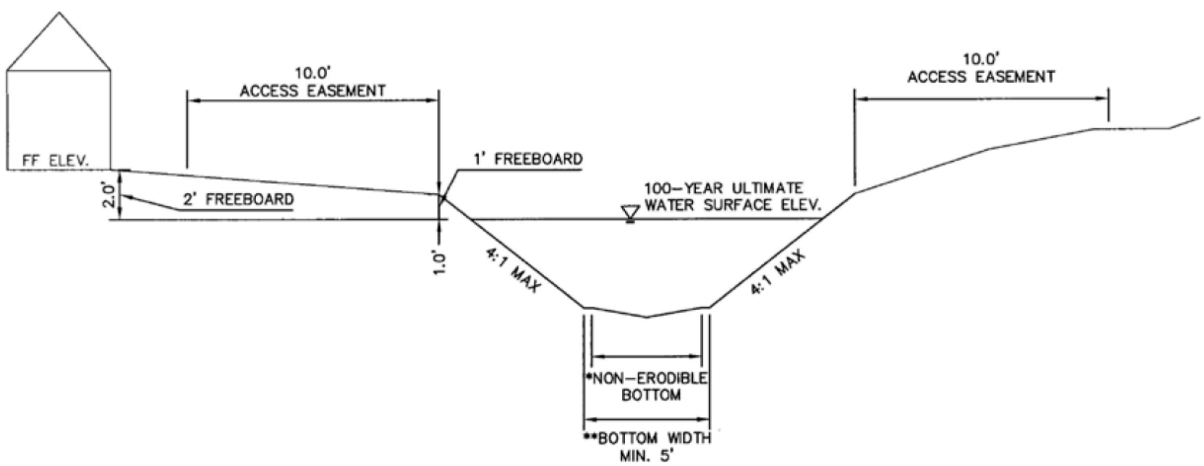
3. The maximum discharge velocities of the storm pipe shall not exceed the permitted velocity of the receiving channel or conduit at the outfall to prevent erosive conditions. The maximum outfall velocity of a conduit in partial flow shall be computed for partial depth and shall not exceed the maximum permissible velocity of the receiving channel unless controlled by an appropriate energy dissipater (e.g. stilling basins, impact basins, riprap protection).

6.07. Open Channel Design

Excavated open channels may be used to convey storm waters where the construction costs and/or long-term maintenance cost involved with a closed storm sewer system is not justified economically. Open channels shall be designed to convey the full design discharge.

The allowable excavated channel cross section is shown in Figure 6.4. the roughness coefficients allowed for various types of excavated channel cover are shown in Table 6.17. These maximum coefficients do not apply for drainage facilities discharging off-site. The Engineer shall determine maximum discharge velocities.

Figure 6.4 - Open Channels – Excavated



SCALE: 1:5

*NON-ERODIBLE BOTTOM SHALL BE DESIGNED BY THE ENGINEER AND DOCUMENTATION AND CALCULATIONS SHALL BE PROVIDED TO CITY STAFF FOR REVIEW. GRADES SHALL ENSURE POSITIVE DRAINAGE THROUGHOUT THE CHANNEL.

**MINIMUM BOTTOM WIDTH SHALL BE BASED UPON PROJECT SPECIFIC CHANNEL MAINTENANCE NEEDS. BOTTOM WIDTHS SMALLER THAN WHAT IS SHOWN SHALL BE APPROVED BY THE DIRECTOR OF ENGINEERING SERVICES.

THE DIRECTOR OF ENGINEERING SERVICES MAY REQUIRE HYDRAULIC MODELING OF THE CONSTRUCTED CHANNEL TO CONSIDER A MANNINGS VALUE THAT REFLECTS A "MAINTAINED CHANNEL (0.25–0.35)" AND A "NON-MAINTAINED CHANNEL (0.35–0.055)".

A. Unlined, unvegetated excavated channels are not allowed. Construction of excavated channels will not be considered complete until the channel banks are stabilized. Vegetation selected for channel cover must conform with allowable vegetation from the Landscaping Approved Materials List.

B. Supercritical flow shall not be allowed in channels except at drop structures and other energy dissipaters.

C. At transitions in channel characteristics maximum velocities must be analyzed. Velocities should be reduced to prevent erosion using either energy dissipaters and/or wider less steep channel.

D. Channel armoring for erosion control shall be provided where deemed necessary by the Deputy Director of Engineering Services.

E. Minimum channel bottom widths are recommended to be equal to twice the depth of flow of the channel. Compound channels are encouraged (See 6.07.L) and channel bottoms of each section should reflect corresponding storm event flows. Any permanent open channel shall have a minimum bottom width of five feet (5').

F. If the channel cannot be maintained from the top of the bank, a maintenance access ramp shall be provided and included within the drainage easement.

G. All open channels require a minimum freeboard of one foot (1') above the one-hundred (100) year water surface elevation or below top of bank, whichever is greater.

H. The minimum slope for an excavated improved channel is one percent (1%) unless a pilot channel is constructed, or otherwise approved by the Deputy Director of Engineering Services.

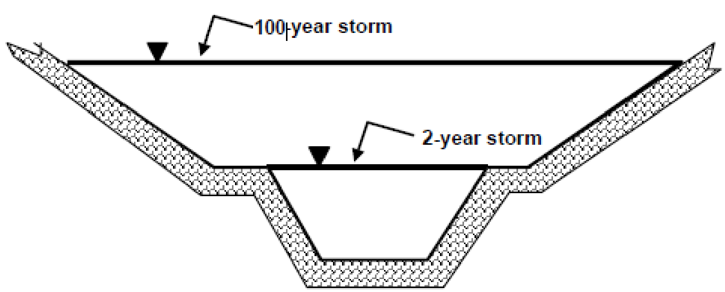
I. Earthen sides above the lined section or totally earthen channels shall be on at least a four horizontal to one vertical (4:1) slopes and shall have approved ground cover to prevent erosion.

J. Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.

K. Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress and super elevation.

L. Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 6.5 illustrates a compound section that carries the 2-year and 100-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 12:1 slope to ensure drainage.

FIGURE 6.5 - Compound Channel



The 2-year channel section shall mimic natural stream sinuosity meander within the full channel section. Landscaping in accordance with landscape design section shall be allowed along the compound channel shelf.

M. Table 6.17 provides allowable ranges for roughness coefficients of open channels.

Table 6.17 - Channel Roughness Coefficients

Channel Description	Roughness Coefficient		
	Minimum	Normal	Maximum
Minor Natural Streams			
Moderately Well Defined Channel			
• Grass and weeds, little brush	0.025	0.030	0.033
• Dense wees, little brush	0.030	0.035	0.040
• Weeds, light brush on banks	0.030	0.035	0.040
• Weeds, heavy brush on banks	0.035	0.050	0.060
• Weeds, dense willows on banks	0.040	0.060	0.080
Irregular Channel with Pools and Meanders			
• Grass and weeds, little brush	0.030	0.036	0.042
• Dense weeds, little brush	0.036	0.042	0.048
• Weeds, light brush on banks	0.036	0.042	0.048
• Weeds, heavy brush on banks	0.042	0.060	0.072
• Weeds, dense willows on banks	0.048	0.072	0.096
Flood Plain, Pasture			
• Short grass, no brush	0.025	0.030	0.035
• Tall grass, no brush	0.030	0.035	0.050
Flood Plain, Cultivated			
• No crops	0.025	0.030	0.035
• Mature crops	0.030	0.040	0.050
Flood Plain, Uncleared			
• Heavy weeds, light brush	0.035	0.050	0.070
• Medium to dense brush	0.070	0.100	0.160
• Trees with flood stage below branches	0.080	0.100	0.120
Major Natural Streams			
• Moderately well-defined channel	0.025	---	0.060
• Irregular channel	0.035	---	0.100
Unlined Vegetated Channels			
• Mowed grass, Clay soil	0.025	0.030	0.035
• Mowed grass, Sandy soil	0.025	0.030	0.035
Unlined Unvegetated Channels			
• Clean gravel section	0.022	0.025	0.030
• Shale	0.025	0.030	0.035
• Smooth rock	0.025	0.030	0.035
Lined Channels			
• Smooth finished concrete	0.013	0.015	0.020
• Riprap (rubble)	0.300	0.400	0.500

N. Water surface elevations and flow velocities in channels are impacted by the maintenance condition in the channel. Calculations shall be performed assuming maintained and unmaintained vegetative conditions. Lower (maintained) Manning’s values shall be used to determine maximum velocities, while higher (unmaintained) Manning’s values shall be used to determine water surface elevations per Table 6.17.

O. Any channel modification must meet the applicable requirements of all Local, State and Federal regulatory agencies.

P. *Erosion prevention.* All channel sections must consider and account for channel stabilization in their design. This requirement pertains to all sections whether they are left in their natural condition or are modified in any manner. Three sets of requirements are provided depending upon the relationship of the existing channel to the limits of the developer/owner's property boundaries. The Deputy Director of Engineering Services shall have the discretion to require the implementation of the portion of these requirements as deemed necessary, depending on the specifics of the property being developed or improved or to allow the escrow of funds sufficient to provide for the construction of a proportionate amount of channel improvements in lieu of actual construction. This discretion may be exercised when a small section of improvements is not deemed by the Deputy Director of Engineering Services to be economically practicable.

a. In cases where the entire channel section is contained within the limits of the developer/owner property boundaries, the developer/owner shall:
   i. Provide for an improved stabilized channel cross-section which reduces all velocities to six fps or below for vegetated channels. The channel improvements must meet all requirements of this article.
   ii. For vegetated channel sections with channel velocities ranging from six to eight fps, construct grade control structures within the channel and overbank areas to prevent erosion. Grade control structures shall have a minimum effective depth of three feet below existing or proposed grades with an adequate number of structures to prevent less than one foot of degradation.

b. In cases where the property boundary follows the centerline of the channel or incorporates only a portion of the channel cross-section, the developer/owner shall:
   i. Determine the design section required to provide for an improved stabilized channel cross-section which reduces all velocities to six fps or below for vegetated channels. The design channel section must meet all requirements of this article.
   ii. The design section may include vegetated channel sections with channel velocities ranging from six to eight fps, provided that grade control structures are included within the channel and overbank areas to prevent erosion. Grade control structures shall have a minimum effective depth of three feet below existing or proposed grades with an adequate number of structures to prevent less than one foot of degradation.
   iii. The developer/owner shall construct or escrow funds for construction of the portion of the design improvements required on their property for the ultimate channel design. The Deputy Director of Engineering Services shall have the discretion to determine the portion of the design improvements to be constructed/escrowed by the developer/owner. In most instances, the developer/owner shall construct one-half of the improvements on their property.
   iv. If grade control structures are incorporated into the design, the developer/owner shall coordinate with adjacent owners in order to construct these features in their entirety at the time of the initial portion of the channel improvements.
   v. The developer/owner shall provide for a drainage easement and access/ maintenance easement consistent with the portion of the improvements provided.

c. In cases where the developer/owner owns property adjacent to channel or floodplain areas but does not own a portion of the channel or floodplain area, the developer/owner shall (at the discretion of the Deputy Director of Engineering Services):
   i. Determine the channel improvement configuration necessary to meet the requirements of subsection (b)(i) above, and
   ii. Shall provide a dedicated easement to the Town for the portion of this future improvement configuration, including necessary maintenance and access easement, which will include the developer/owner property.

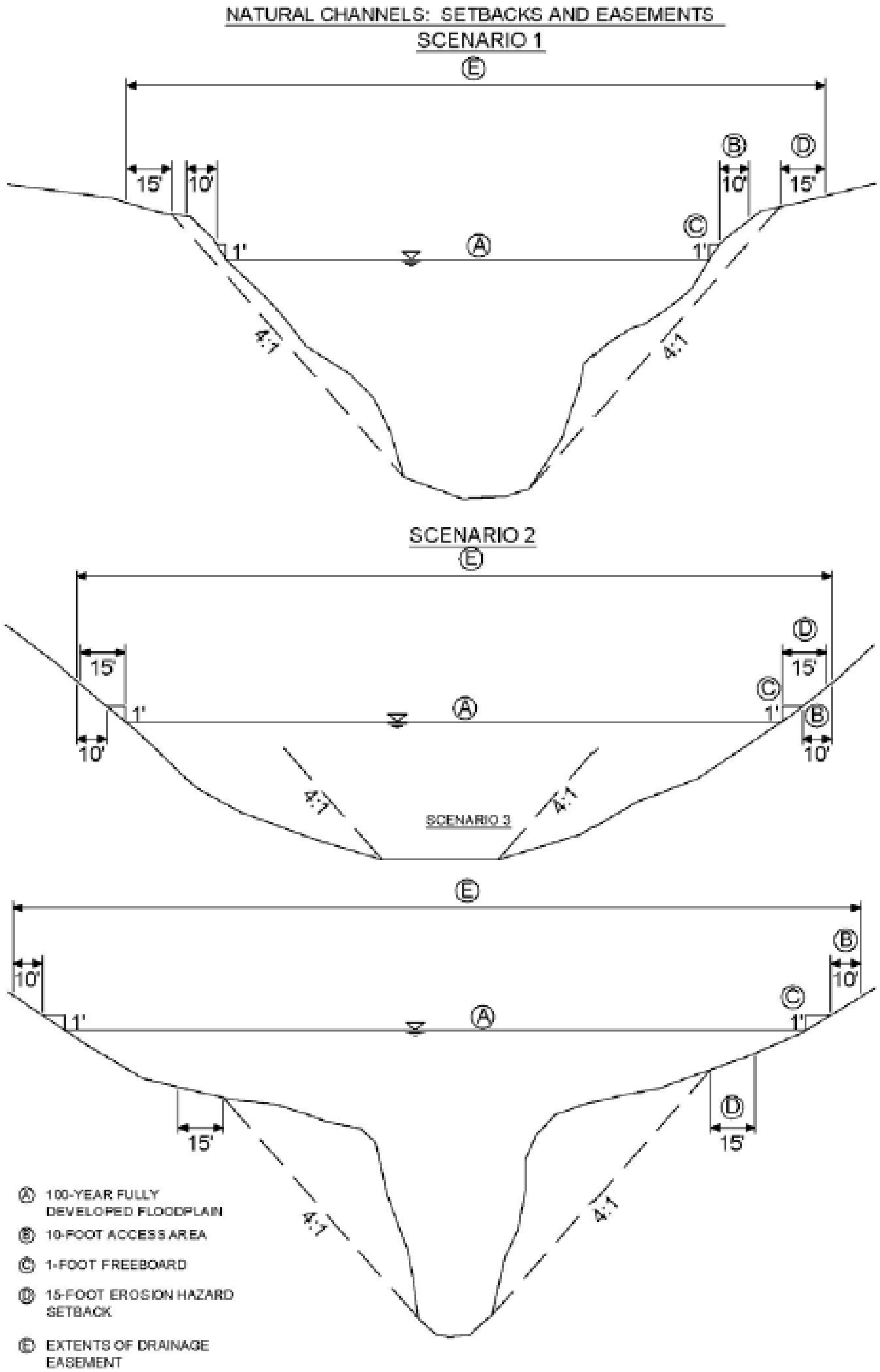
Q. An erosion hazard setback shall be contained within the Drainage Easement for all existing and proposed channels. The purpose of this set back is to reduce the potential for any damage to a private lot or street right-of-way caused by the erosion of the bank. The erosion hazard setback shall be determined as follows, and is provided in Figure 6.6.

1. For stream banks composed of material other than rock, locate the toe of the natural stream bank. Project a 4:1 line sloping away from the toe until it intersects finished grade. From this intersection add fifteen feet (15') away from the bank. This shall be the limit of the erosion hazard setback.

2. Figure 6.6 is intended to illustrate various scenarios under which the erosion hazard setback can be applied and how it interacts with the floodplain access easement. Scenario 1 shows a situation where the setback may be located outside the 100-year floodplain and access easement boundaries. Scenarios 2 and 3 show locations where the erosion hazard set back will be located inside the 100-year floodplain and access easement boundaries.

3. Any modifications within the area designated as erosion hazard setback, will require a geotechnical and geomorphological stability analysis, and a grading permit (two separate items).

Figure 6.6 - Natural Open Channels



6.08. Culvert Design

A. All culverts, headwalls, wingwalls, and aprons shall be designed in conformance with the Town Standard Details. The Engineer is responsible for selecting the applicable detail and culvert size.

B. Culvert calculations shall be provided to the Town for review. Calculations may include, but are not limited to, headwall, tailwater, and flowline elevations, lowest adjacent grade and structure elevations, inlet and outlet control calculations and velocity calculations.

C. There is no minimum freeboard requirement for culvert crossings; however, an emergency overflow path shall be identified and provided on the construction plans. An emergency overflow path is the path the storm water will take when the drainage facilities becomes clogged or does not function in the manner as to which it was designed. The emergency overflow path shall be limited to public right-of-way or drainage easements.

D. Culverts should always be aligned to follow the natural stream channel. The engineer shall provide sufficient information to analyze the upstream and downstream impacts of the culvert and illustrate the interaction of the channel and culvert alignment.

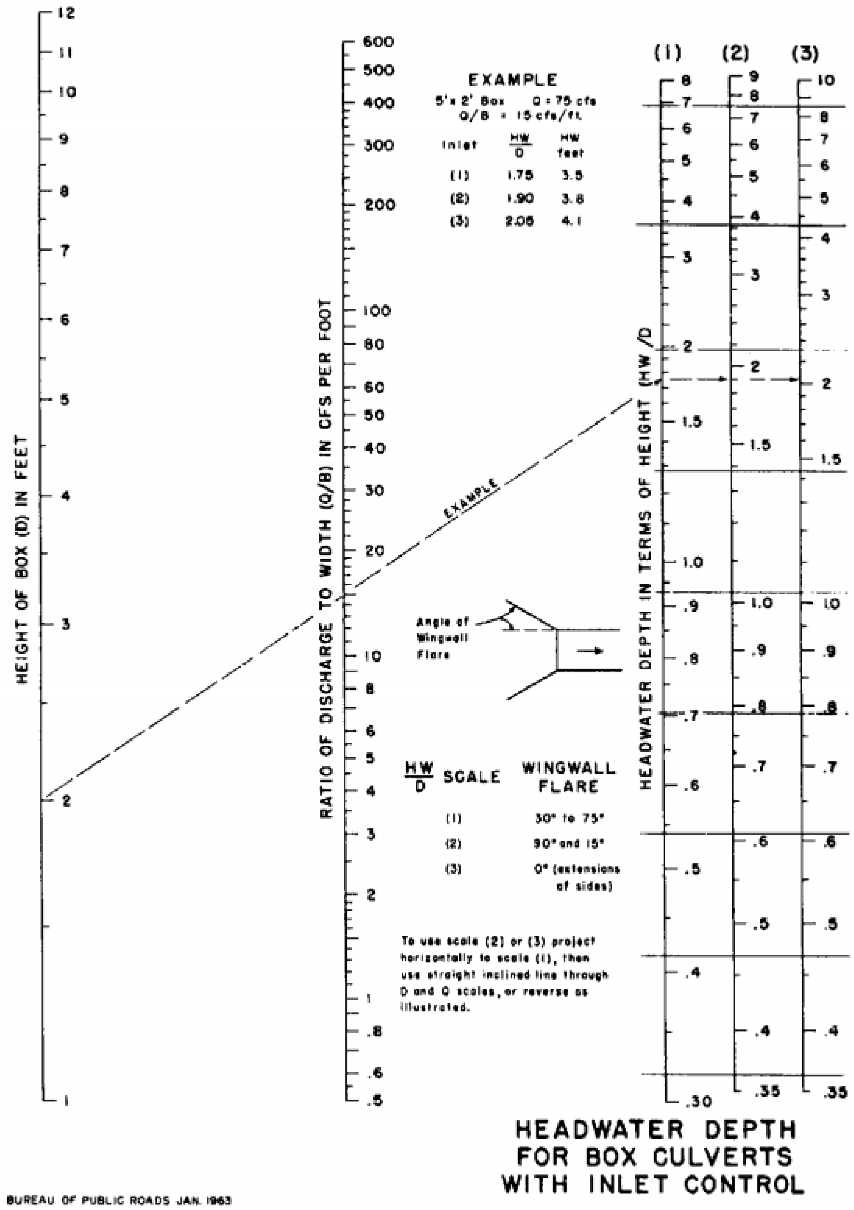
E. The hydraulic design of culverts shall be based upon design guidelines set forth by TxDOT, the U.S. Department of Transportation, or other suitable material.

F. Headwalls and Entrance Conditions:

1. The Engineer shall be responsible for the headwall and wingwall designs. Headwalls and endwalls refer to the entrances and exits of structures, respectively, and are usually formed of cast-in-place concrete and located at either end of the drainage system. Wingwalls are vertical walls, which project out from the sides of a headwall or endwall.

2. The culvert headwater depth should be determined using the nomograph shown in Figure 6.7 below.

FIGURE 6.7 – Headwater Depth for Box Culverts with Inlet Control



3. Concrete culvert headwalls and wingwalls shall use form-liner surface finish unless otherwise approved by the Deputy Director of Engineering Services.

6.09. Detention/Retention Basin Design

A. Town of Prosper Subdivision Ordinance and as amended requires detention within the town limits and its ETJ.

1. On September 8, 2015, Water Detention in Doe Branch Ordinance No. 15-51 was passed waiving detention requirements in the Doe Branch basin provided that creek stabilization is addressed and "pinch points" are analyzed. "Pinch points" include, but are not limited to culverts under BNSF railroad and under DNT service roads.

2. Where regional detention is not provided or locations in Doe Branch basin where infrastructure or other conditions create restrictions that would not allow release for fully developed flow, detention meeting the requirements of this section (6.09) shall be required.

B. Detention Storage Calculations

1. Detention shall be provided for the 2, 5, 25, and 100-year design storms. Storage and outlet structures shall accommodate all four-frequency storm events.

2. Detention facilities without upstream detention areas and with drainage areas of twenty (20) acres or less can be designed using the Modified Rational Method otherwise the Unit Hydrograph Method shall be used.

3. If the Unit Hydrograph Method is used, the model shall extend through the downstream point where the proposed development creates no adverse impacts. Include existing detention facilities within the influenced watershed.

C. Wet/Retention Pond Criteria

1. Wet ponds, aka Retention Ponds, are required in the Town of Prosper to satisfy detention requirements. Wet ponds provide amenities and increased water quality functions as compared to dry ponds.

2. Wet ponds should be designed to maintain a permanent pool of water throughout the year.

3. Wet pond design should consider water quality, and include a means of aeration such as fountains. Ponds shall be a minimum of 6 ft deep, but should be designed to minimize potential of algae growth. Reduced pond depths may be allowed with approval by the Deputy Director of Engineering Services.

4. All areas above the normal pool water surface shall be irrigated or shall have an approved vegetative ground cover to prevent erosion.

5. Per Zoning Ord Ch 4, Sec 8.9.C.7 for non-residential development, detention ponds that are located between the building and street and contain a constant water level, are landscaped, or otherwise treated as an amenity for the development, as determined by the Director of Development Services or his/her designee, may be calculated toward the required open space

6. “Retention” is used as the nomenclature in this subsection and any other portion of Section 6.09 to reference <u>wet</u> detention ponds. Detention is used throughout to as general language for the controlled release of storm water runoff due to development.

D. Underground Detention

1. Underground detention is allowed and encouraged on small development sites where regional detention is not provided.

2. Underground detention shall be located in a drainage and detention easement with the perimeter of easement being a minimum of 5 ft from edge of facility (or 1.5x the depth, whichever is greater).

3. All underground detention facilities shall have at least one access point for maintenance purposes.

   a. Periodic removal of sediment or other debris is required for proper function of detention facility. Access points shall be placed in manner for easy access and proper maintenance.

b. Access shall be provided at point of restriction if less than 18" in diameter to allow ease of removal of trapped debris.

4. Underground detention shall have a minimum slope of 0.30% for all portions of invert/bottom of facility.

5. Material for underground detention shall be reinforced concrete, HDPE pipe or aluminized corrugated metal pipe. However, alternative material may be allowed with the approval of the Deputy Director of Engineering Services. Long-term stability and maintenance shall be considered with approval of alternative materials.

6. Underground facilities (including easements for detention) shall not be located under fire lanes without approval by Fire Marshal. If approved, only a single run of pipe shall be allowed to cross perpendicularly to fire lane. No pipe or other facility for underground detention shall be allowed to run longitudinally within a fire lane. All material under fire lanes shall be reinforced concrete.

7. No other public utility shall be allowed to cross an underground detention facility with exception of a single run of pipe.

8. All components of underground detention shall be privately maintained in accordance with standard detention easement language.

9. Upstream flow from public storm sewer shall not be permitted to flow through an underground facility without approval by the Deputy Director of Engineering Services. If allowed, measures shall be incorporated into underground detention design to provide an emergency outlet in case the detention outlet structure becomes blocked. This emergency outlet shall be placed in manner to prevent damage to structures on upstream development in likelihood underground detention facility becomes blocked.

E. Detention facilities, when required, shall be designed based upon the following minimum criteria:

1. Detention structures shall have a minimum of one foot (1') of freeboard above the 100-year water surface elevation. If one foot (1') of freeboard increases the capacity of the pond by a factor greater than 125% required for the 100-year water surface elevation, then the pond shall be designed to have a freeboard up to the 125% capacity.

2. The steepest side slope permitted for a vegetated embankment is 4:1.

3. Earth embankments used to temporarily or permanently impound surface water must be constructed according to specifications a required based on geotechnical investigations of the site and all regulatory requirements.

4. Detention facilities shall be designed with an emergency spillway in case the primary outfall ceases to function as designed. The spillway shall be designed to pass a minimum of the 100-year flood event.

5. It shall be the engineer of record's responsibility to determine if a stability analysis is necessary based on global overturning and rapid drawdown. The stability analysis shall be performed by a licensed geotechnical engineer. Global overturning shall be based on full hydrostatic loading (at 100-year flood stage). The stability analysis from rapid drawdown conditions shall consider saturated soil conditions without the hydrostatic loading. A minimum factor of safety of 1.25 shall be required.

6. All non-residential development over 10 acres are encouraged to provide regional retention pond placed on a common area lot (or easement) to be maintained by a Property Owners Association.

7. The Applicant shall provide a maintenance plan for the detention facility as part of the design. The maintenance plan shall indicate the ingress and egress locations to enter and maintain the pond, maintenance roles and responsibilities, contact information for the party responsible for the maintenance, and a maintenance schedule.

8. Access shall be provided to the banks and bottom of a retention facility for maintenance.

   a. Access ramps shall have width no less than 10 ft wide on a maximum longitudinal slope of 10:1.
   b. The Applicant shall provide an operations and maintenance plan that will detail access.
   c. Facilities with permanent pools shall address dewatering procedures.

9. It is the responsibility of the Engineer to consider pedestrian and vehicular safety in the design of detention facilities. Perimeter rails or fencing may be required.

10. Criteria established by the State of Texas for dam safety (TAC Title 30, Part 1, Chapter 299) and impoundment of state waters (Texas Water Code Chapter 11) shall apply where required by the state, and where, in the Engineer's judgement, the potential hazard requires these more stringent criteria.

F. Process for Waiver from Wet or Underground Detention

If a development is unable to utilize either retention or underground detention due to physical constraints, hydraulic limitations or other hardships, the use of a dry detention pond will require Town Council approval. The Property Owner shall submit written notice to the Engineering Services Department to request a Public Hearing no later than twenty-one (21) days before the next Town Council meeting.

6.10. Bridge Design Hydraulics

A. The Town requires that head losses and depth of flow through bridges to be determined with a HEC-RAS program or other approved program. The following guidelines pertain to the hydraulic design of bridges:

1. All bridge and culvert designs shall contain the peak flow of the 100-year frequency storm based on fully developed watershed conditions within the right-of-way or drainage easement limits.

2. Fully developed 100-year water surface must not be increased upstream of bridges and culverts based on comparison to revised effective model, unless otherwise approved by Deputy Director of Engineering Services.

3. Excavation of the natural pilot channel is not allowed as compensation for loss of conveyance.

4. Channelization upstream or downstream of the proposed bridge will normally only be permitted when necessary to realign the flow to a more efficient angle of approach.

5. Side swales may be used to provide additional conveyance downstream of and through bridges.

6. Bridges are to be designed with the lowest point (low beam) low chord at least two feet (2') above the water surface elevation of the design storm.

B. A scour analysis shall be submitted with bridge design plans.

1. Projected changes in channel stability upstream and downstream of the structure shall be evaluated when establishing the structure type, channel grades and crossing geometry. Appropriate stabilization measures are required.

6.11. Energy Dissipators

A. The Engineer shall be responsible for all energy dissipation designs.

B. Rock rip-rap or gabion baskets or mattresses are required for energy dissipation. Other designs may be considered.

C. All energy dissipation designs shall include supporting calculations showing the design is adequate. The Town may require the Engineer to provide a hydraulic model as supporting documentation.

D. All energy dissipators should be designed to facilitate future maintenance. The design of outlet structures in or near parks or residential areas shall give special consideration to appearance and shall be approved by the Deputy Director of Engineering Services.

6.12. Floodplain Development Criteria

A. No new construction is allowed in floodplain areas, but construction is allowed in those areas that have been reclaimed from the floodplain. No new construction will be allowed in floodways.

B. Floodplain alteration shall be allowed only if all the following criteria are met:

1. Flood studies shall include flows generated for existing conditions and fully-developed conditions for the 2, 5, 10, 25, and 100-year storm events.

2. Alterations of the floodplain shall not increase the water surface elevation of the design flood of the creek on other properties.

3. Alterations shall be in compliance with FEMA guidelines.

4. Alterations shall result in no loss of valley storage for a Major Creek, as defined by the Subdivision Ordinance, and a fifteen percent (15%) maximum loss of valley storage for any other tributary for any reach, except at bridge and culvert crossings where it can be proven that there are no detrimental effects downstream.

5. Any alteration of floodplain areas shall not cause any additional expense in any current or projected public improvements, including maintenance.

    a. Alterations are prohibited if done in manner to restrict ability to build safe, efficient hike and bike trail system as determined by the Town (for areas indicated on master hike and bike trail master plan) or where it substantially increases the cost of such trail construction.

6. The floodplain shall be altered only to the extent permitted by equal conveyance on both sides of the natural channel, as defined by the United States Army Corps of Engineers (USACE) in a HEC-RAS analysis. The right of equal conveyance applies to all owners and uses. Including greenbelt, park areas, and recreational areas. Owners may relinquish their right to equal conveyance by providing a written agreement to the Deputy Director of Engineering Services.

7. A grading permit shall be required to perform any grading activities on site.

8. The toe of any fill shall parallel the natural direction of flow.

9. Grading activities in the floodplain shall incorporate and consider other Town planning documents and ordinances.

C. The above criteria shall be met before any floodplain alteration may occur. Typical projects requiring a floodplain alteration including placing fill (whether or not it actually raises the property out of the floodplain) constructing a dam, straightening channel sections, making improvements (substantial or otherwise), to existing structures in a floodplain in which the existing outside dimensions of the structure are increased, and temporary storage of fill materials, supplies equipment.

D. In general, the information needed for the application shall be performing by running a backwater model, such as HEC-2 or HEC-RAS, and a flood routing model, such as TR-20, HEC-1, or HEC-HMS. Unless a pre-existing model is in place, HEC-HMS and HEC-RAS shall be used. The back-water information shall be used to determine that upstream water surface elevations and erosive velocities have not increased. Flood routing information shall be used to ensure that the cumulative effects of the reduction in floodplain storage of floodwater will not cause downstream increases in water surface elevations and erosive velocities.

E. The Engineer is responsible for providing documentation of the relevant USACE approved permits prior to beginning modification to the floodplain, or for providing a signed and sealed statement detailing why such permits are unnecessary.

F. Verification of Floodplain Alterations:

1. Prior to final acceptance by the Town for a certified statement shall be prepared by a Licensed Professional Engineer showing that all lot elevations, as developed within the subject project, meet the required minimum finished floor elevations shown on the construction plans. This certification shall be filed with the Deputy Director of Engineering Services.

2. In addition, at any time in the future when a building permit is desired for existing platted property which is subject to flooding or carries a specified or recorded minimum finished floor elevation, a Registered Professional Land Surveyor shall survey the property prior to obtaining a building permit if not previously certified as required per 6.12.F.1. The certified survey data showing the property to be at or above the specified elevation shall be furnished to the Deputy Director of Engineering Services for approval. Certification of compliance with the provisions of this ordinance pertaining to specified finished floor elevations shall be required.

3. The owner/developer shall furnish, at his expense, to the Deputy Director of Engineering Services sufficient engineering information to confirm that the minimum finished floor elevations proposed are as required by this ordinance. Construction permits will not be issued until a Conditional Letter of Map Revision (CLOMR) or amendment has been accepted by the Deputy Director of Engineering Services for submittal to FEMA. Letters of Map Revision (LOMR) shall be submitted to the Deputy Director of Engineering Services for submittal to FEMA prior to final acceptance of the project. The contractor shall supply to the Deputy Director of Engineering Services all necessary documentation and fees to be forwarded to FEMA for application for a Letter of Map Amendment (LOMA) if the LOMR has not yet become effective.

4. All submittals to FEMA shall be routed through the Engineering Department for review prior to FEMA submittal. Upon approval, the owner/developer will provide the Town with a copy of the CLOMR or LOMR submittal for Town records.

5. All response to FEMA comments shall be submitted to the Town prior to submittal to FEMA. The Town will review the response to comments in a timely manner.

6.13. Drainage Easements

A. The following minimum width exclusive drainage easements are required when facilities are not located within public right-of-way or easements:

1. Storm sewers are to be located within the center of a fifteen-foot (15') drainage easement or one and a half (1.5) times the depth plus the width of the structure rounded up to the nearest five feet (5'), whichever is greater.

   a. Drainage easements shall not be split longitudinally by property lines and shall fall entirely on one lot in residential developments.
   b. Drainage easements shall not be split longitudinally by property lines and shall fall entirely on one lot in non-residential developments unless under pavement and incorporated into a "Fire lane, Access, Drainage, and Utility Easement".

2. Flumes for overflow paths are to be located with the edge being a minimum of one foot (1') off the property line within a minimum ten-foot (10') drainage easement entirely located on one lot.

   a. Overflow flumes may be located in same drainage easement for underground storm pipe where applicable.
   b. Where upstream basin requiring overflow flume is less than 1 acre, drainage easement may be centered down property line of two residential lots. Fences shall be allowed to be anchored in center of flume but done in manner to not restrict overall flow.

B. Storm sewer lines are considered public if they cross property lines and collect runoff from adjacent properties. Drainage easements shall be dedicated to the Town when a drainage system crosses a property line. For single-family residential developments, storm sewer lines shall not cross residential lots unless approved by the Deputy Director of Engineering Services.

C. Drainage Easements shall be dedicated for all floodplains and shall include an erosion hazard setback to reduce the potential for damage due to erosion of the bank.

D. Drainage Easements shall be dedicated for all detention facilities.

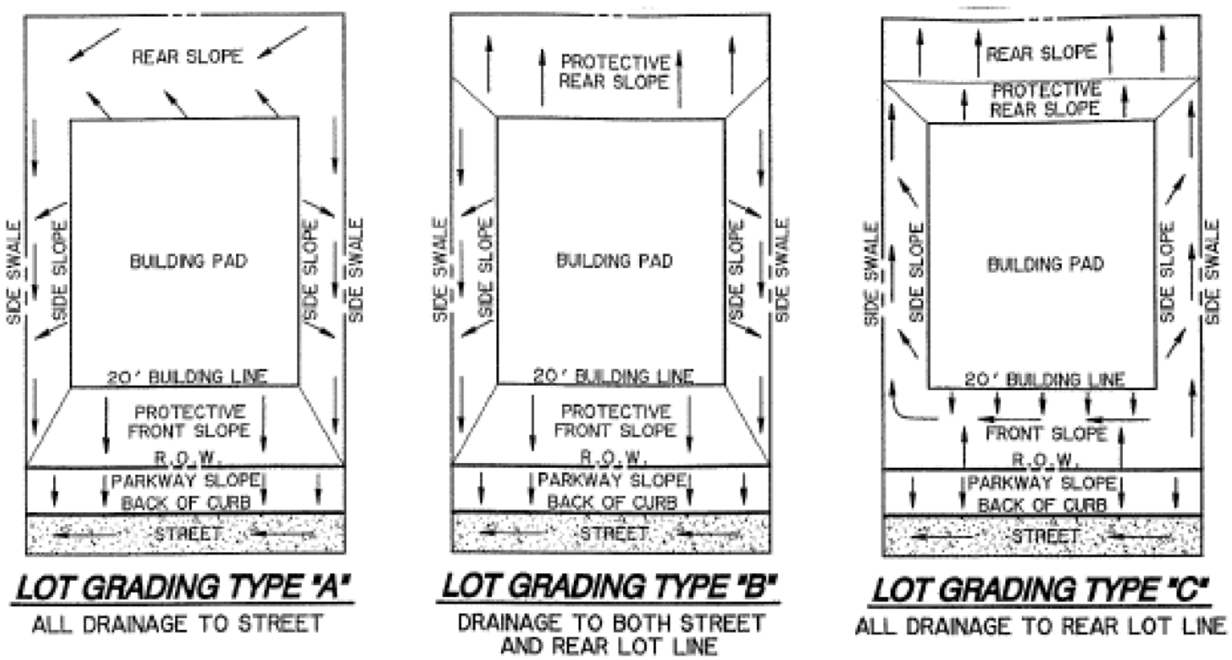
6.14. Miscellaneous Drainage Requirements

A. Lot-to-Lot Drainage

Residential lot-to-lot surface drainage is strictly prohibited. Exceptions include areas needed for positive overflow located within drainage easements and as approved by the Deputy Director of Engineering Services. Figure 6.8 is provided below for reference when performing lot grading designs. Lot grading type and finished floor elevations shall be shown on the construction plans. Type B and Type C Lot Grading must back to alleys, open space, or a drainage easement. Type C Lot Grading may only be used with approval from the Deputy Director of Engineering Services. Refer to the International Residential Building Code (IRC) Section 401.3 for additional requirements.

Commercial lot-to-lot surface drainage is prohibited unless easements have been put in place and is approved by the Deputy Director of Engineering Services.

FIGURE 6.8 Typical Lot Grading Patterns



B. The minimum finished floor elevation for any lot adjacent to a drainage feature shall be two feet (2') above the adjacent 100-year fully developed water surface elevation and shall be shown on the final plat.

C. Should mitigation be required under Section 404 of the Clean Water Act, the areas shall be identified on the engineering construction plans.