2013

CITY OF KERRVILLE

KERR COUNTY

TEXAS

DRAINAGE DESIGN MANUAL

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1.0 GENERAL

This manual contains the minimum storm drainage design criteria to be followed in the design of storm drainage facilities in the City of Kerrville (City). If an item is not covered in this manual other criteria as approved by the City Engineer may be applied.

2.0 DESIGN STORM FREQUENCY

The 1% storm frequency (100–year storm) for fully developed watershed conditions shall be used in all storm sewer designs in the City, unless specified otherwise in this manual. Alternative approaches are only permitted with the approval of the City Engineer or designee.

3.0 DETERMINATION OF DESIGN DISCHARGE

The Rational Method for computing storm water runoff is to be used for hydraulic design of facilities serving a drainage area of less than 150 acres. For drainage areas greater than 150 acres, a Unit Hydrograph method shall be utilized to compute the storm water runoff (i.e., Snyder's Unit Hydrograph, Soil Conservation Service Unit Hydrograph (SCS), or Clark's Unit Hydrograph). If a hydrologic modeling system computer program is utilized to compute the storm water runoff, the model must be compatible with the Army Corps of Engineers HEC-HMS software. A copy of the digital model must be submitted to the Engineering Department with the plan review submittal. In all cases, the detailed calculations utilized to determine the storm water design discharges and a summary of the results must be included within the civil construction plans.

3.1 RATIONAL METHOD

The Rational Method can be used to estimate storm water runoff peak flows for the design of gutter flows, drainage inlets, storm sewer pipe, culverts and small ditches. It is most applicable to small, highly impervious areas. The maximum drainage area that is allowed to be used with the Rational Method is 150-acres.

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area (A), runoff coefficient (C), and the mean rainfall intensity (I) for a duration equal to the time of concentration (T_c).

The Rational Formula is expressed as follows:

$$Q = C \times I \times A \tag{3.1}$$

where: Q = maximum rate of runoff (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall (unitless)
I = average rainfall intensity for a duration equal to the T_c (in/hr)
A = drainage area contributing to the design location (acres)

3.1.1 Time of Concentration

The time of concentration (T_c) can be defined as the time required for water to flow from the most hydraulically remote point in a drainage basin to the point being analyzed. The most hydraulically remote drainage point refers to the route requiring the longest drainage travel time and not necessarily the greatest linear distance. Use of the Rational Formula requires the time of concentration for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). Overland (sheet) flow, shallow concentrated flow and channel flows are components that need to be considered in the calculation of time of concentration. The following methods are recommended for time of concentration calculation.

3.1.1.1 Overland flow – flow over plane surfaces: For each drainage area, the distance is determined from the design point to the most hydraulically remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter. Overland flow distance should not exceed 300 feet. The overland flow time can be determined by the following formula (Equation 3.2) or by the Seelye Chart for Overland Flow Time (Figure A, Appendix A). Note that for overland sheet flow the minimum time is 5 minutes and the maximum overland flow time shall be 20 minutes.

$$T_{overland} = \frac{1.8(1.1 - C)L^{1/2}}{S^{1/3}} \tag{3.2}$$

where: C = runoff coefficient determined from Table 3.2
L = over land flow length in feet (ft)
S = average overland slope in percent (%)

3.1.1.2. Shallow concentrated flow – overland flow usually becomes shallow concentrated flow after a maximum of 300 feet, and before the flow enters a defined channel or drainage system, the flow is considered shallow concentrated flow. Travel time for shallow concentrated flow is calculated as follows:

$$T_{shallow} = \frac{L}{V_{shallow}(60)} \tag{3.3}$$

where: T = time (minutes)
L = shallow concentrated flow length in feet (ft)
$V_{shallow}$ = shallow concentrated flow velocity in feet per second (fps)
$S_{decimal}$ = average water course slope in decimal

$$V_{shallow} = 16.1345\sqrt{S_{decimal}} \quad \text{[for unpaved areas]} \tag{3.4}$$

$$V_{shallow} = 20.3282\sqrt{S_{decimal}} \quad \text{[for paved areas]} \tag{3.5}$$

3.1.1.3 Channel Flow – Velocity in channels should be calculated from the Manning's equation. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions. When actual cross section information is not available, non-floodplain channel velocities for ultimate watershed development should not be less than 6 fps for estimating time of concentration.

$$T_{channel} = \frac{L}{V_{channel}(60)} \tag{3.6}$$

where: T= time (minutes)
$V_{channel}$ = channel flow average velocity (fps)
L = Length of reach along the flow path (ft)

The Channel Velocity is calculated using Manning's Formula as follows:

$$V_{channel} = \frac{1.486\left(R^{2/3}\right)\left(S^{1/2}\right)}{n} \tag{3.7}$$

$$R = \frac{A}{P_w} \tag{3.8}$$

To obtain the total time of concentration, the overland, shallow concentrated, and channel flow times must be added together. For example, if the flow time in a channel is 15 minutes and the overland flow time from a ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes.

3.1.2 Rainfall Intensity (I)

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. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in Table 3.1. Note that the data represented in this table were derived from the Dodson Method as follows:

$$I = \frac{b}{(T+d)^e} \tag{3.9}$$

where: I = rainfall intensity (in/hr)
T = rainfall duration (minutes)
b, d, e = coefficients based upon precipitation data

3.1.3 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3.2 gives the recommended runoff coefficients for the Rational Method.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method may be used where hydrographs can be generated and routed through the drainage system. If a hydrograph is used, the results should be compared to the Rational Method and the more conservative results utilized in the design.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. **This should be checked** particularly in areas where the overland portion is grassy (yielding a long T_c) to avoid underestimating peak runoff.

Table 3.1 Rain-Intensity-Duration Data for Kerr County

		Return Period (Years)					
Coefficients		2	5	10	25	50	100
e		0.789	0.765	0.764	0.763	0.766	0.768
b		49	58	69	80	91	104
d		8.4	8	8	8	8	8.4
Hours	Minutes	Rainfall Intensity (inches per hour)					
0.083	5	6.32	8.15	9.72	11.30	12.76	14.17
	6	5.97	7.70	9.19	10.68	12.05	13.41
	7	5.67	7.31	8.72	10.13	11.43	12.74
	8	5.39	6.95	8.30	9.65	10.88	12.13
	9	5.15	6.64	7.92	9.21	10.39	11.60
	10	4.92	6.36	7.58	8.82	9.94	11.11
	11	4.72	6.10	7.28	8.46	9.54	10.67
	12	4.54	5.86	7.00	8.14	9.17	10.26
	13	4.37	5.65	6.74	7.84	8.84	9.89
	14	4.22	5.45	6.50	7.57	8.53	9.55
0.25	15	4.07	5.27	6.29	7.31	8.24	9.24
	16	3.94	5.10	6.09	7.08	7.98	8.94
	17	3.82	4.94	5.90	6.86	7.73	8.67
	18	3.70	4.80	5.73	6.66	7.50	8.42
	19	3.60	4.66	5.56	6.47	7.29	8.18
	20	3.50	4.53	5.41	6.29	7.09	7.96
	21	3.40	4.41	5.27	6.13	6.90	7.75
	22	3.31	4.30	5.13	5.97	6.72	7.55
	23	3.23	4.19	5.01	5.82	6.56	7.37
	24	3.15	4.09	4.89	5.68	6.40	7.19
	25	3.08	4.00	4.77	5.55	6.25	7.03
	26	3.01	3.91	4.66	5.43	6.11	6.87
	27	2.94	3.82	4.56	5.31	5.97	6.72
	28	2.87	3.74	4.47	5.20	5.85	6.58
	29	2.81	3.66	4.37	5.09	5.73	6.44
0.5	30	2.76	3.59	4.28	4.99	5.61	6.31
	31	2.70	3.52	4.20	4.89	5.50	6.19
	32	2.65	3.45	4.12	4.79	5.39	6.07
	33	2.60	3.39	4.04	4.70	5.29	5.96
	34	2.55	3.32	3.97	4.62	5.20	5.85
	35	2.50	3.26	3.90	4.54	5.10	5.75
	36	2.46	3.21	3.83	4.46	5.01	5.65
	37	2.41	3.15	3.77	4.38	4.93	5.55
	38	2.37	3.10	3.70	4.31	4.85	5.46
	39	2.33	3.05	3.64	4.24	4.77	5.37
	40	2.30	3.00	3.58	4.17	4.69	5.29
	41	2.26	2.95	3.53	4.11	4.62	5.20
	42	2.22	2.91	3.47	4.04	4.55	5.12
	43	2.19	2.87	3.42	3.98	4.48	5.05
	44	2.16	2.82	3.37	3.92	4.41	4.97
0.75	45	2.12	2.78	3.32	3.87	4.35	4.90
	46	2.09	2.74	3.28	3.81	4.29	4.83
	47	2.06	2.70	3.23	3.76	4.23	4.76
	48	2.03	2.67	3.19	3.71	4.17	4.70
	49	2.01	2.63	3.14	3.66	4.11	4.64
	50	1.98	2.60	3.10	3.61	4.06	4.58
	51	1.95	2.56	3.06	3.56	4.00	4.52
	52	1.93	2.53	3.02	3.52	3.95	4.46
	53	1.90	2.50	2.98	3.47	3.90	4.40
	54	1.88	2.47	2.95	3.43	3.86	4.35
	55	1.86	2.44	2.91	3.39	3.81	4.30
	56	1.83	2.41	2.88	3.35	3.76	4.24
	57	1.81	2.38	2.84	3.31	3.72	4.19
	58	1.79	2.35	2.81	3.27	3.68	4.15
	59	1.77	2.33	2.78	3.23	3.63	4.10
1	60	1.75	2.30	2.75	3.20	3.59	4.05
2	120	1.06	1.42	1.69	1.97	2.21	2.50
3	180	0.79	1.06	1.26	1.47	1.65	1.86
6	360	0.46	0.63	0.76	0.88	0.99	1.11
12	720	0.27	0.37	0.45	0.52	0.58	0.66
24	1440	0.16	0.22	0.27	0.31	0.35	0.39

Table 3.2 Rational Method Runoff Coefficients	
Description of Area	**Runoff Coefficient (C)**
Developed:	
Asphalt	0.95
Concrete	0.97
Grass Areas (Lawns, Parks, etc.):	
Poor Condition (Grass Cover < 50% of Area)	
Flat, 0-2%	0.47
Average, 2-7%	0.53
Steep, over 7%	0.55
Fair Condition (Grass Cover between 50% & 75% of Area)	
Flat, 0-2%	0.41
Average, 2-7%	0.49
Steep, over 7%	0.53
Good Condition (Grass Cover > 75% of Area)	
Flat, 0-2%	0.36
Average, 2-7%	0.46
Steep, over 7%	0.51
Undeveloped:	
Cultivated	
Flat, 0-2%	0.47
Average, 2-7%	0.51
Steep, over 7%	0.54
Pasture/Range	
Flat, 0-2%	0.41
Average, 2-7%	0.49
Steep, over 7%	0.53
Forest/Woodlands	
Flat, 0-2%	0.39
Average, 2-7%	0.47
Steep, over 7%	0.52
Land Use	
Single Family Residential (40% Impervious Cover)	
Flat, 0-2%	0.60
Average, 2-7%	0.66
Steep, over 7%	0.69
Multifamily Residential (65% Impervious Cover)	
Flat, 0-2%	0.76
Average, 2-7%	0.79
Steep, over 7%	0.81
Retail/Office/Light Commercial (80% Impervious Cover)	
Flat, 0-2%	0.85
Average, 2-7%	0.87
Steep, over 7%	0.88
Regional Commercial/Industrial (95% Impervious Cover)	
Flat, 0-2%	0.94
Average, 2-7%	0.95
Steep, over 7%	0.96

Adapted from: 1. iSWM Design Manual for Development/Redevelopment, 2006
2. City of Austin, Drainage Criteria Manual, 2007

3.2 UNIT HYDROGRAPHS

For drainage areas greater than 150 acres, a Unit Hydrograph method shall be utilized to compute the storm water runoff (i.e., Snyder's Unit Hydrograph, Soil Conservation Service Unit Hydrograph (SCS), or Clark's Unit Hydrograph). If the Army Corps of Engineers HEC-HMS software is utilized to compute the storm water runoff, a copy of the digital model must be submitted to the Engineering Department with the plan review submittal. Additionally, the detailed calculations utilized to determine the storm water design discharges must be included in the civil construction plans.

The methodologies specified and approved by the US Army Corps of Engineers manual for the Snyder's Unity Hydrograph, Soil Conservation Service Unit Hydrograph (SCS) and the Clark's Unit Hydrograph are hereby adopted by this manual and included by reference.

4.0 STREET DRAINAGE

The design flow of water in a street is related to its interference with traffic, public safety, parking, & pedestrian access and the chance of flood damage to surrounding properties. Interference with traffic is regulated by design limits of the spread of water into or through traffic lanes. Flooding of surrounding properties is regulated by limiting the depth of flow at the curb and by containment of the 100-year design storm flow within the street right of way. The following subsections specify the capacity limitations allowed in the City of Kerrville streets.

4.1 PERFORMANCE STANDARDS & LIMITATIONS

4.1.1 Flow Velocity

The maximum velocity of street flow shall not exceed 10 feet/second. At street "T" intersections, the flow velocity must be checked on the stem of the "T" to ensure that flow will not traverse the crown and opposing curb of the crossing street and enter onto private property.

4.1.2 Flow Depth

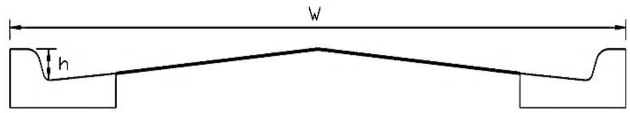
The flow depth shall be limited to the top of curb for the 4% chance (25-year) storm event.

The 1% (100-year) storm event shall be confined to be within the limits of the street rights-of-ways.

Once capacity has been reached, flows shall be conveyed via a public drainage system. In general, the flows listed in Table 4.1 shall not be exceeded without substantiating computations satisfactorily demonstrating that adverse impacts will be eliminated.

Table 4.1 – Street Capacity Table for Standard Parabolic Curb & Gutter Asphalt Streets

Manning's n = 0.018
h=0.54



Street Slope (%)	City Standard Curbed & Guttered Street Width (feet)					
	30		36		42	
	Q (cfs)	V(fps)	Q (cfs)	V(fps)	Q (cfs)	V(fps)
0.40	17.2	2.1	20.8	2.1	24.3	2.1
0.45	18.3	2.3	22.0	2.3	25.8	2.3
0.50	19.3	2.4	23.2	2.4	27.2	2.4
0.55	20.2	2.5	24.4	2.5	28.5	2.5
0.60	21.1	2.6	25.5	2.6	29.8	2.6
0.65	22.0	2.7	26.5	2.7	31.0	2.7
0.70	22.8	2.8	27.5	2.8	32.2	2.8
0.75	23.6	2.9	28.5	2.9	33.3	2.9
0.80	24.4	3.0	29.4	3.0	34.4	3.0
0.85	25.1	3.1	30.3	3.1	35.4	3.1
0.90	25.9	3.2	31.2	3.2	36.5	3.2
0.95	26.6	3.3	32.0	3.3	37.5	3.3
1.00	27.3	3.4	32.9	3.4	38.4	3.4
1.50	33.4	4.1	40.2	4.1	47.1	4.2
2.00	38.6	4.8	46.5	4.8	54.4	4.8
2.50	43.1	5.3	52.0	5.3	60.8	5.4
3.00	47.2	5.8	56.9	5.9	66.6	5.9
3.50	51.0	6.3	61.5	6.3	71.9	6.3
4.00	54.5	6.7	65.7	6.8	76.9	6.8
4.50	57.9	7.1	69.7	7.2	81.6	7.2
5.00	61.0	7.5	73.5	7.6	86.0	7.6
5.50	64.0	7.9	77.1	7.9	90.2	8.0
6.00	66.8	8.2	80.5	8.3	94.2	8.3
6.50	69.5	8.6	83.8	8.6	98.0	8.6
7.00	72.2	8.9	86.9	8.9	101.7	9.0
7.50	74.7	9.2	90.0	9.3	105.3	9.3
8.00	77.1	9.5	92.9	9.6	108.7	9.6
8.50	79.5	9.8	95.8	9.9	112.1	9.9
9.00	79.7	10.0	95.1	10.0	110.5	10.0
9.50	76.5	10.0	91.2	10.0	106.0	10.0
10.00	73.5	10.0	87.7	10.0	101.9	10.0
10.50	70.8	10.0	84.5	10.0	98.2	10.0
11.00	68.3	10.0	81.5	10.0	94.7	10.0
11.50	66.0	10.0	78.7	10.0	91.6	10.0
12.00	63.8	10.0	76.2	10.0	88.6	10.0

4.2 ALLOWABLE FLOW SPREAD

4.2.1 Residential Streets

Runoff in a residential street from the 4% design frequency flows shall not exceed a depth of the lowest top of curb. Stormwater shall be removed from the streets by inlets or openings into adjacent public drainage systems. They shall generally be placed at low points and as frequently as necessary to avoid exceeding water spread & depth criteria.

4.2.2 Collector Streets

Based upon the 4% storm event, Flow Spread shall be designed to provide at least one (1) open 12-foot traffic lane at the center of the street. For divided collectors, the flow spread shall be designed to provide one (1) open travel lane in each direction. Wherever possible, a collector street shall not be crossed with surface drainage unless approved by the City Engineer.

4.2.3 Major and Minor Arterials

Based upon the 1% design frequency flows, Flow Spread shall be designed to not exceed one (1) travel lane in each direction. Bypass from upstream inlets in excess of 5-cfs is not allowed into major or minor arterial intersections.

4.2.4 Alleys

The 1% design frequency flows shall not exceed the capacity of the paved alley section. Alley capacities must be checked at all alley turns and intersections to determine if curbing is needed or if grades should be flattened. Curbing must be required for at least 10-feet on either side of an inlet in an alley and on the other side of the alley so that the top of the inlet is even with the high edge of the alley pavement.

In residential areas where the standard 10-foot wide alley section capacity is exceeded, a wider alley may be used to provide more drainage capacity. Curbs shall not be added to alleys to increase the capacity. Where a particular width alley is required, such as a 12-foot width, a wider alley, such as a 16-foot width, may be required for greater capacity. Approximate increases in right-of-way widths will be necessary.

4.3 INTERSECTIONS

Inlet placement and storm sewer size shall ensure that design storm flows are intercepted along street legs entering the intersection in advance of the curb returns connecting the streets based on the criteria provided in this manual. In no case shall inlets be placed in the curved portion of curbs connecting intersecting streets. Where storm flow is allowed to pass through an intersection, valley gutter design must provide for smooth, uninterrupted traffic flow.

Intersection Pair	Intercept	Valley Gutter Criteria
Arterial - Arterial	All legs	No valley gutters
Arterial - Collector	All legs	No valley gutters
Arterial - Residential	All legs	No valley gutters
Collector - Collector	All legs	No valley gutters
Collector - Residential	Residential legs	Valley gutters can parallel Collector
Residential - Residential	Two legs preferred	Valley gutters acceptable

5.0 ROADWAY DITCH REQUIREMENTS

When roadway ditches are used in-lieu of city standard curb & gutter, the following standards shall apply. If any of the below requirements cannot be achieved, an alternative to mitigate the deficiency shall be submitted for review and approval by the City.

1. The ditch shall not be less than 24 inches in depth.
2. The side slopes shall not be steeper than 3H:1V.
3. Provisions for armoring and/or vegetation for erosion control on the side slopes and bottom shall be shown on the plans.
4. The ditch shall convey the flows generated by the 1% storm event.
5. The flow velocity in an unarmed ditch shall generally not exceed 6 feet per second. Reference Table 6.1a for further velocity control information.

6.0 <u>OPEN CHANNELS, CULVERTS, AND BRIDGES</u>

6.1 GENERAL DESIGN CONSIDERATIONS

- Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.

- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater (minor) systems.

- It is important to ensure that the combined on-site flood control system and major stormwater system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.

- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

6.2 OPEN CHANNELS

Natural or lined open channels shall be designed to convey the flood peak flows while at the same time be designed in such a way to minimize erosion and maintain the stability of the stream banks. Concrete lined channels are generally discouraged by the City. **Bioengineering** techniques may be used in natural channels with side slopes no steeper than 3H:1V. Construction of a low-flow channel, where possible, is another recommended option. Low-flow channels should be sized using the channel forming discharge or the 2-year storm. The design engineer is reminded that it may be extremely difficult to obtain the proper permits from the State and Federal authorities for concrete channel designs. In addition, developers are responsible for acquisition of all regulatory agency permits.

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.

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

- 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 superelevation.

- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 6.1 illustrates a compound section that carries the 50% design frequency flows (2-year storm event) and 1% design frequency 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 1V:12H slope to ensure drainage.

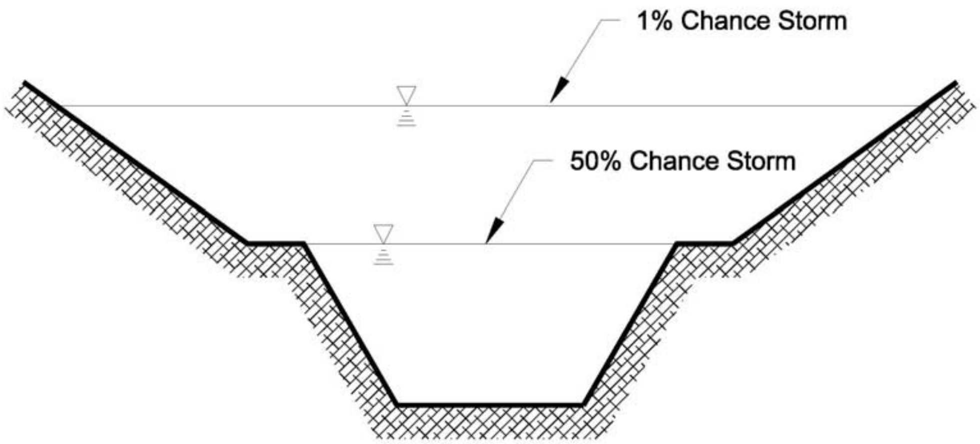


Figure 6.1 Compound Channel Section

6.2.1 Open Channel Lining Types

The three main classifications of open channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod, and bioengineering techniques. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see Appendix B-References, *iSWM Technical Manual* for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation only linings may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation.

If low flows are prevalent, a hard lined pilot channel may be needed, and it should be wide enough to accommodate maintenance equipment.

Flexible Linings – Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

6.2.2 Uniform Flow Calculations

Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = \frac{1.486}{n} R^{\frac{2}{3}} \sqrt{S} \tag{6.1}$$

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} \sqrt{S} \tag{6.2}$$

$$S = \left(\frac{Q\,n}{1.486\,A\ R^{\frac{2}{3}}}\right)^2 \tag{6.3}$$

where: v = average channel velocity (ft/s)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = wetted cross sectional area or cross sectional area of flow (ft^2)
R = hydraulic radius A/P (ft) [see equation 3.8]
S = slope of the channel or channel bed (ft/ft)

Note that when solving for *S* in Equation 6.3, *S* represents the energy gradient, which is the head loss per length of flow path. When *S* is less than 0.1%, the energy gradient is approximately the bed slope.

Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for natural channels are given in Table 6.1 For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. Recommended Manning’s n values for various artificial channels are provided in Table 6.2.

Table 6.1 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels		
Channel Description	**Manning's n**	**Maximum Permissible Channel Velocity (ft/s)**
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	***6**
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	***6**
3. Some weeds, light brush on banks	0.035	***6**
4. Some weeds, heavy brush on banks	0.050	***6**
5. Some weeds, dense willows on banks	0.060	***6**
• For trees within channels with branches submerged at high stage, increase above values by	+0.010	***6**
• Irregular section with pools, slight channel meander, increase above values by	+0.010	***6**
Floodplain – Pasture		***6**
1. Short grass	0.030	***6**
2. Tall grass	0.035	***6**
Floodplain – Cultivated Areas		***6**
1. No crop	0.030	***6**
2. Mature row crops	0.035	***6**
3. Mature field crops	0.040	***6**
Floodplain – Uncleared		***6**
1. Heavy weeds scattered brush	0.050	***6**
2. Wooded	0.120	***6**
MAJOR NATURAL STREAMS		***6**
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush	Range from 0.028 to 0.060	***6**
UNLINED VEGETATED CHANNELS		***6**
Clays (Bermuda Grass)	0.030	***6**
Sandy and Silty Soils (Bermuda Grass)	0.030	***6**
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	2.5
Silts	0.030	1.5
Sandy Silts	0.030	3
Clays	0.030	5
Coarse Gravels	0.030	6
Shale	0.030	8
Rock	0.025	15

(Adapted from: iSWM Technical Manual, 2010)

***Reference Table 6.1a Velocity Control**

Table 6.1a Velocity Control				
Velocity (fps)	Type of Facility Required	Hydraulic Radius	Correction Factor	Maximum Permissible Velocity (fps)
1 to 6 (maximum average velocity = 6 fps)	Vegetated Earthen Channel	0-1 1-3 3-5 5-8 8-10 Over 10	0.83 0.92 1.05 1.15 1.225 1.25	5 5.5 6.3 6.9 7.35 7.5
6 to 8	Concrete Retards	NA	NA	NA
>8	Concrete Lining or Drop Structures	NA	NA	NA

Table 6.2 Manning's Roughness Coefficients (n) for Artificial Channels				
		Depth Range		
Category	***Lining Type***	**0-0.5 ft**	**0.5-2.0 ft**	**>2.0 ft**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D50	0.044	0.033	0.030
	2-inch D50	0.066	0.041	0.034
Rock Riprap	6-inch D50	0.104	0.069	0.035
	12-inch D50	–	0.078	0.040
Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth. *Some "temporary" linings become permanent when buried.				

(Source: HEC-15, 1988; iSWM TM, 2010)

6.2.3 Critical Flow Calculations

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A_c^3}{T} \tag{6.4}$$

where: Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)
A_c = critical depth cross-sectional area (ft^2)
T = top width of water surface (ft)

Note: A trial and error procedure is needed to solve Equation 6.4. The cross-sectional area is a function of the critical depth and can be factored out depending upon the geometry of the channel section. For a rectangular channel:

$$d_c = \frac{A_c}{T} \quad \text{[rectangular]} \tag{6.5}$$

where: d_c = critical depth

Therefore, Equation 6.4 can be rewritten as:

$$d_c^3 = \frac{Q^2}{g} \text{ [rectangular]} \tag{6.6}$$

6.2.4 Semi-Empirical Calculations

Semi-empirical equations (as presented in Table 6.3) or section factors (as presented in Figure 6.2) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$Z = \frac{Q}{\sqrt{g}} \tag{6.7}$$

where: Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec2)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

- A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
- If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
- If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
- If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane, or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant (V^2/2g).

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = \frac{v}{\sqrt{\frac{gA}{T}}} \tag{6.8}$$

where: Fr = Froude number (dimensionless)
v = velocity of flow (ft/s)
g = acceleration of gravity (32.2 ft/sec^2)
A = cross-sectional area of flow (ft^2)
T = top width of flow (ft)
Q= discharge rate for design conditions (cfs)
d = depth corresponding to velocity v (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 6.3 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

where:
d_c = critical depth (ft)
Q = design discharge (cfs)
g = acceleration due to gravity (32.2 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1] See Figure 6.2 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

(Source: iSWM TM, 2010)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location of the hydraulic jump and its sequent depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profile and specific force (momentum principle) and specific energy relationships are used. For computational methods refer to Chow, 1959, TxDOT, 2002, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
$\left.\right\}$ Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Figure 6.2 Open Channel Geometric Relationships for Various Cross Sections
(Source: USDA, SCS, NEH-5 1956; iSWM TM, 2010)

6.2.5 Flow Considerations

- Channel capacity shall be determined to accommodate the discharge from a 4% design storm event assuming build-out conditions for all of the contributing drainage area. In addition, the channel shall be designed to convey the 1% storm event flows generated from the developed on-site conditions and the existing off-site conditions.
- Where supercritical flow is encountered, allowances shall be made in the design for the proper handling of the water.
- Velocity of flow shall not be less than 2.5 fps for the 4% storm event.
- Maximum velocities for the design flow shall be less than the values given in Table 6.1 for the type of surface treatment(s) specified.
- Where the minimum velocities cannot be maintained or when low flows are expected on a regular basis, a concrete pilot channel or approved equal shall be constructed to convey the 50% (2-year) storm event.
- Channels shall be designed to convey the 1% storm without overtopping the channel and shall be designed with a minimum freeboard equal to one foot above the 4% chance storm design depth or 20% of the design depth, whichever is less.

6.2.6 Physical Considerations

- The maximum side slope for a non-armored or reinforced open channel shall be 3H:1V unless proposed erosion control data and slope stability calculations are submitted and approved by the City Engineer.
- The minimum longitudinal slope shall be 1% (100H:1V) for earthen or vegetative lined channels to prevent formation of standing water. The slope may be reduced to 0.5% if a concrete pilot channel or city approved alterative is provided to convey the 5-year storm event.
- Special channel linings and energy dissipation features must be used to compensate for high velocities and hydraulic jumps associated with supercritical flow. The channel must contain the hydraulic jump throughout the extent of the supercritical profile.
- The maximum allowable deflection angle for bends in designed channels shall be 30 degrees. The outside of horizontal curves shall provide additional channel bank height and surface treatment as necessary to fully contain the design flow and prevent erosion and overtopping. Allowance for extra freeboard shall be made when the centerline radius of the channel is less than three (3) times the bottom width. Where sharp bends or high velocities are involved, the designer shall account for extra freeboard requirements using the following formula as a minimum:

$$d_2 - d_1 = \frac{V^2 (T + b)}{2gR} \qquad (6.9)$$

where:
d_1 = depth of flow at the inside of the bend (ft)
d_2 = depth of flow at the outside of the bend (ft)
b = bottom width of channel (ft)
V = average approach velocity in the channel (ft/sec)
T = width of flow at the water surface (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R = center line radius of the turn or bend (ft)

6.3 CULVERTS & BRIDGES

6.3.1 General Overview

A culvert is a hydraulically short conduit, open on both ends, generally used to convey stormwater runoff through a roadway or an embankment and typically constructed without manholes, inlets, or catch basins. For economy and hydraulic efficiency, culverts are typically designed to operate with the inlet submerged during the design storm event. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. According to FHWA standards, a culvert with a clear opening of more than 20-feet, measured along the center of the roadway between inside of end walls, is considered a bridge.

6.3.2 Design Considerations

The design engineer shall keep head losses and velocities within the guidelines specified in this manual and where not included shall be within generally acceptable engineering practices. This normally requires selecting a structure which creates a slight headwater (1.2 times the culvert height) and has a flow velocity at or below the allowed maximum. Velocities in culverts are normally limited to the maximum allowed in the downstream channel unless there is some form of energy dissipation at the outfall.

6.3.3 Flow Control

In the hydraulic design of culverts, an investigation must be made into the type of flow condition through the culvert. The flow will be controlled, or limited, either at the culvert entrance or the outlet, and is designated either inlet or outlet control, respectively.

> **Inlet Control** – Inlet control occurs when the barrel capacity exceeds the culvert inlet capacity and the tailwater elevations is too low to control. In other words, the headwater depth entrance geometry at the inlet will control the amount of water entering the barrel. The roughness, length of culvert barrel, and outlet conditions do not affect capacity for culverts with inlet control.
>
> **Outlet Control** – Outlet control occurs when the culvert inlet capacity exceeds the barrel capacity or the tailwater elevation causes backwater effect through the culvert. In this case, the tailwater elevation, slope, length and roughness of the culvert barrel will determine the hydraulic capacity of the culvert even though the entrance conditions are such that a larger flow could be conveyed.
>
> Proper culvert design should include an analysis to determine whether the inlet is outlet or inlet controlled. For more information on inlet and outlet control, see TxDOT's Hydraulic Design Manual, 2011 or latest edition.

6.3.4 Freeboard

Freeboard, the vertical clearance between the design water surface and the lowest point of the roadway at the culvert, is included as a safety factor in the event of clogging of the culvert. One foot (1') of freeboard above the 1% chance water surface elevation is required. Bridges shall be designed to pass the 1% storm event, fully developed watershed conditions, peak flow with two feet (2') of clearance below the lowest part of the open span of the bridge, commonly called the low chord.

6.3.5 Headwalls & Entrance Conditions

1. Headwalls are structural appurtenances located at the ends of a culvert that are typically formed of cast-in-place concrete. The purpose of these structures are:

   a. To retain the fill material and reduce erosion of embankment slopes.
   b. To improve hydraulic efficiency.
   c. To provide structural stability to the culvert ends and serve as a counterweight to offset buoyant or uplift forces.

2. Headwalls shall be designed to fit the conditions of the site, and constructed according to the City of Kerrville Standard Details, or the Texas Department of Highways and Public Transportation Details, unless approved otherwise by the City Engineer.

6.3.6 Outlet Velocity

The velocity in the culvert is likely to be higher than that in the channel because the culvert usually constricts the available channel area. This increased velocity can cause streambed scour and bank erosion in the vicinity of the culvert outlet. There may also be eddies resulting from flow expansion. It is important to control the amount of scour at the culvert outlet because of the possibility of undermining of the headwall and loss of support of the culvert itself. Bank erosion may threaten nearby structures and may also disrupt the stability of the channel itself.

At many locations, use of a simple outlet treatment (e.g., cutoff walls, concrete aprons, rock rubble rip-rap, other) may provide adequate protection against scour. At other locations, adjustment of the barrel slope may be sufficient to prevent damage from scour.

When the outlet velocity exceeds the erosive velocity in the downstream channel, considerations should be given to energy dissipation devices (e.g., dissipation blocks, stilling basins, rip-rap basins, etc).

7.0 INLET DESIGN

All storm sewer inlets shall be designed to capture the fully developed flows and located to comply with Section 4.0 of this manual. Figures A through O may be used to determine the capacity of specific inlets under various conditions.

The following is a list of guidelines for inlet placement:

1. The maximum length of inlets at one location along a street shall not exceed 20 feet.
2. Placing several inlets at a single location is permitted in areas with steep grades in order to reduce bypass and avoid exceeding street capacities in flatter reaches downstream.
3. To minimize water draining through an intersection, inlets should be placed upgrade from an intersection.
4. Inlets should also be located in alleys upgrade of intersections and where necessary to prevent water from entering intersections in amounts exceeding the allowed street capacity.
5. Inlets should be placed upstream from right angle turns.
6. Any discharge of concentrated flow into streets and alleys requires a hydraulic analysis of street and alley capacities.
7. Inlet boxes designed more than 4.5’ deep require a special design.
8. All “Y’ inlets and inlets 10-feet or greater shall have a minimum 21-inch lateral. All smaller inlets shall have a minimum lateral of 18-inches.
9. Inlets at a sag point require a minimum of 10-feet of opening, unless approved otherwise by

the City Engineer.
10. The end of recessed inlet boxes shall be at least 10-feet from a curb return for an intersection or driveway. The inlet shall be located to minimize interference with the use of adjacent property. Inlets shall not be located across from median openings where a future drive approach may be added.
11. Data shown at each inlet shall include storm drain stationing, size of inlet, type of inlet, top of curb elevation, throat of inlet opening, and flowline of inlet.
12. Inlet box depth shall not be less than 4-feet.
13. Interconnecting inlets on lateral shall be avoided.
14. Grate type inlets, except for combination inlets, shall be avoided.

7.1 POSITIVE OVERFLOW

The approved storm sewer system shall provide positive overflow at all Low Points. The term "Positive Overflow" means that when inlets do not function properly due to clogging or when the design capacity of the conduit is exceeded, the excess flow can be conveyed overland along an improved/armored course.

8.0 CLOSED CONDUIT SYSTEMS

All enclosed drainage systems shall be hydraulically designed using Manning's Equation:

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} \sqrt{S} \qquad (8.1)$$

where: Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = inside cross-sectional area of conduit (ft2)
R = hydraulic radius A/P (ft) [see equation 3.8]
S = slope of the energy grade line (ft/ft)

Table 8.1 provides recommended Manning's n values for different types of closed conduit materials.

Alignments of proposed storm sewer systems shall utilize existing easements and rights-of-ways where possible. No other utility parallel with the storm sewer system shall be located within 5-feet horizontally. Storm drainage systems shall be designed so that the necessary trenching will not undermine existing surface structures, utilities or trees. The minimum bury depth for storm drain systems shall be three feet (3'). Storm sewer junction structures with manhole access shall be provided as follows:

- For underground systems consisting of pipe diameters less than 48-inches shall be spaced a maximum of 500-feet apart.
- For underground systems consisting of pipe diameters 48-inches and larger shall be spaced a maximum of 1000-feet apart.

Horizontal and vertical curve design for storm sewer systems shall take into account joint closure. Half tongue exposure is the maximum opening permitted with tongue and groove pipe. Where vertical and/or horizontal alignment require greater deflection, radius pipe on curved alignment shall be used.

The minimum pipe size allowed in the City of Kerrville is 18-inches in diameter.

Table 8.1 Manning's n Values for Closed Conduits		
Type of Conduit	**Wall & Joint Description**	**Manning's n**
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001; iSWM TM, 2010

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

8.1 HYDRAULIC GRADIENT OF CONDUITS

After computing the runoff rate to each inlet, the size and gradient of pipe required to carry the design storm must be determined. The City of Kerrville requires that all hydraulic gradient calculations begin at the outfall of the system. The following are criteria for the starting elevation of the hydraulic gradient:

1. Starting hydraulic grade at an outfall into a creek, channel or pond shall be the 1% chance storm event water surface elevation.
2. In lieu of a known or calculated 1% chance storm event water surface elevation, the starting hydraulic gradient shall not be below the top of pipe.

Calculations of the 1% storm event hydraulic grade line shall be provided on all storm sewer profiles and begin from the downstream starting hydraulic grade line elevation and progress upstream using Manning's formula. Adjustments are made in the hydraulic grade line whenever the velocity in the line changes due to conduit size changes or discharge changes.

Hydraulic grade line "losses" or "gains" for connections, pipe size changes, and other velocity changes must be accounted for and can be calculated by the following formulas:

VELOCITY DIFFERENCE	
$V_1 < V_2$	$V_1 > V_2$
$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$ (8.2)	$h_j = \frac{V_2^2}{4g} - \frac{V_1^2}{4g}$ (8.3)

where: h_j = Head loss (Hydraulic Jump) in feet
V_1 = Upstream Velocity in fps
V_2 = Downstream Velocity in fps
g = the acceleration of gravity (32.2 ft/sec^2)

In determining the hydraulic gradient for a lateral, begin with the hydraulic grade of the trunk line at the junction plus the h_j 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 (see Equation 6.3).

Head losses at structures, such as manholes, wye branches, bends, junction boxes and inlets, shall be calculated as shown in Figures 8.1 & 8.2. The minimum head loss used at any structure shall be 0.1 feet.

The basic equation takes the form as set forth below with the various conditions of the coefficient "K_j" shown in Table 8.2.

$$h_j = \frac{V_2^2}{2g} - K_j \frac{V_1^2}{2g} \quad (8.4)$$

where: h_j = Junction or structure head loss in feet
v_1 = Velocity in upstream pipe in fps
v_2 = Velocity in downstream pipe in fps
K_j = Junction or structure coefficient of loss.

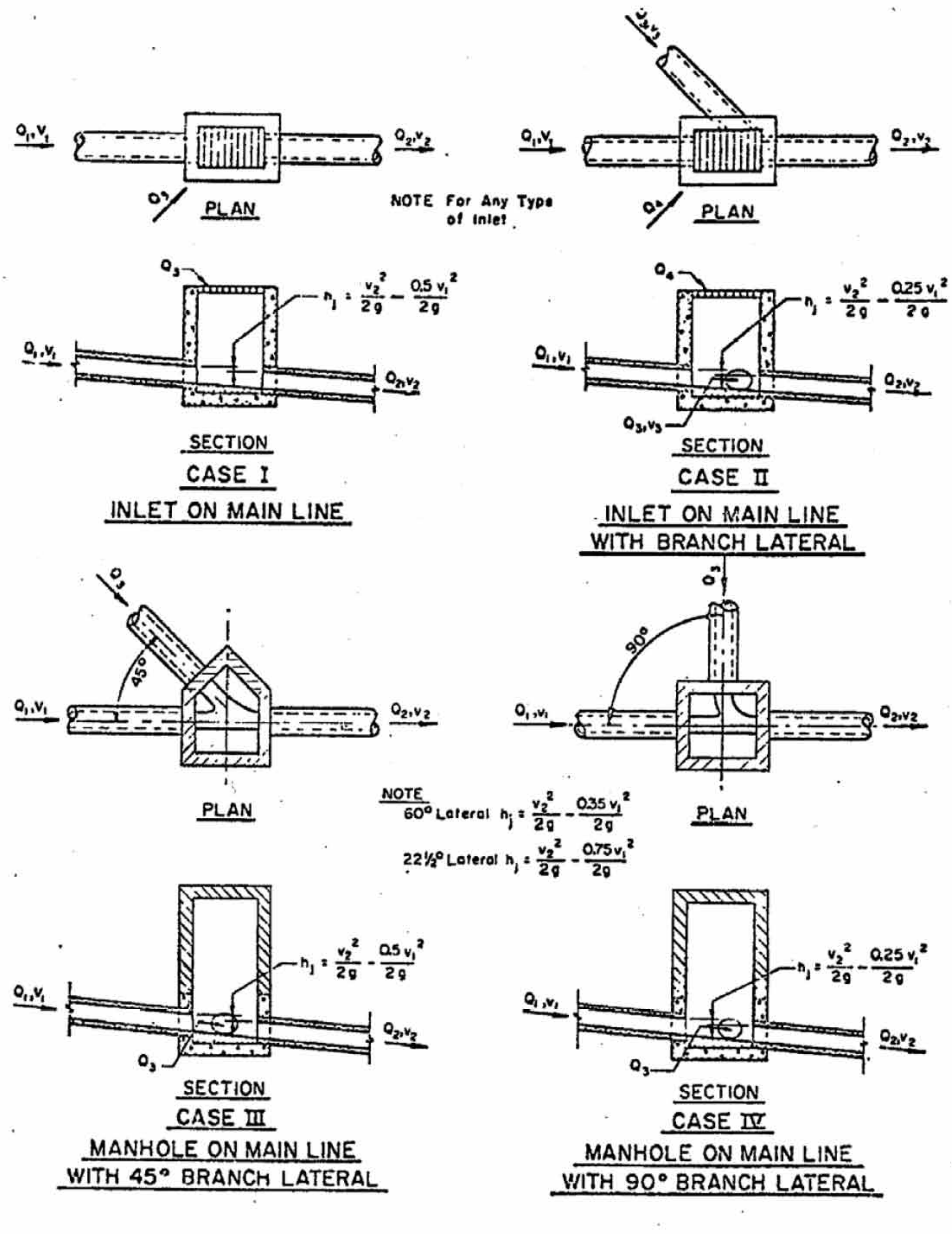
In the case where the manhole is at the very beginning of a line or the line is laid with bends or on a curve, the equation becomes the following without any velocity of approach.

$$h_j = K_j \frac{V_2^2}{2g} \quad (8.5)$$

TABLE 8.2 Junction or Structure Coefficient of Loss

Case No.	Reference Figure	Description of Condition	Coefficient K_j
I	13-1	Inlet on Main Line	0.50
II	13-1	Inlet on Main Line with Branch Lateral	0.25
III	13-1	Manhole on Main Line with 45° Branch Lateral	0.50
IV	13-1	Manhole on Main Line with 90° Branch Lateral	0.25
V	13-2	45° Wye Connection or cut-in	0.75
VI	13-2	Inlet or Manhole at Beginning of Line	1.25

VII	13-2	Conduit on Curves for 90° * Curve radius = diameter Curve radius = 2 to 8 x diameter Curve radius = 8 to 20 x diameter	 0.50 0.25 0.10
VIII	13-2	Bends where radius is equal to diameter 90° Bend 60° Bend 45° Bend 22.5° Bend Manhole on line with 60° Lateral Manhole on line with 22.5° Lateral	 0.50 0.43 0.35 0.20 0.35 0.75
* Where bends or other than 90° bend coefficient can be used with the following percentage factor applied: 60° Bend = 85%, 45° Bend = 70%, 22.5° Bend = 40%			



MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

FIGURE 8.1

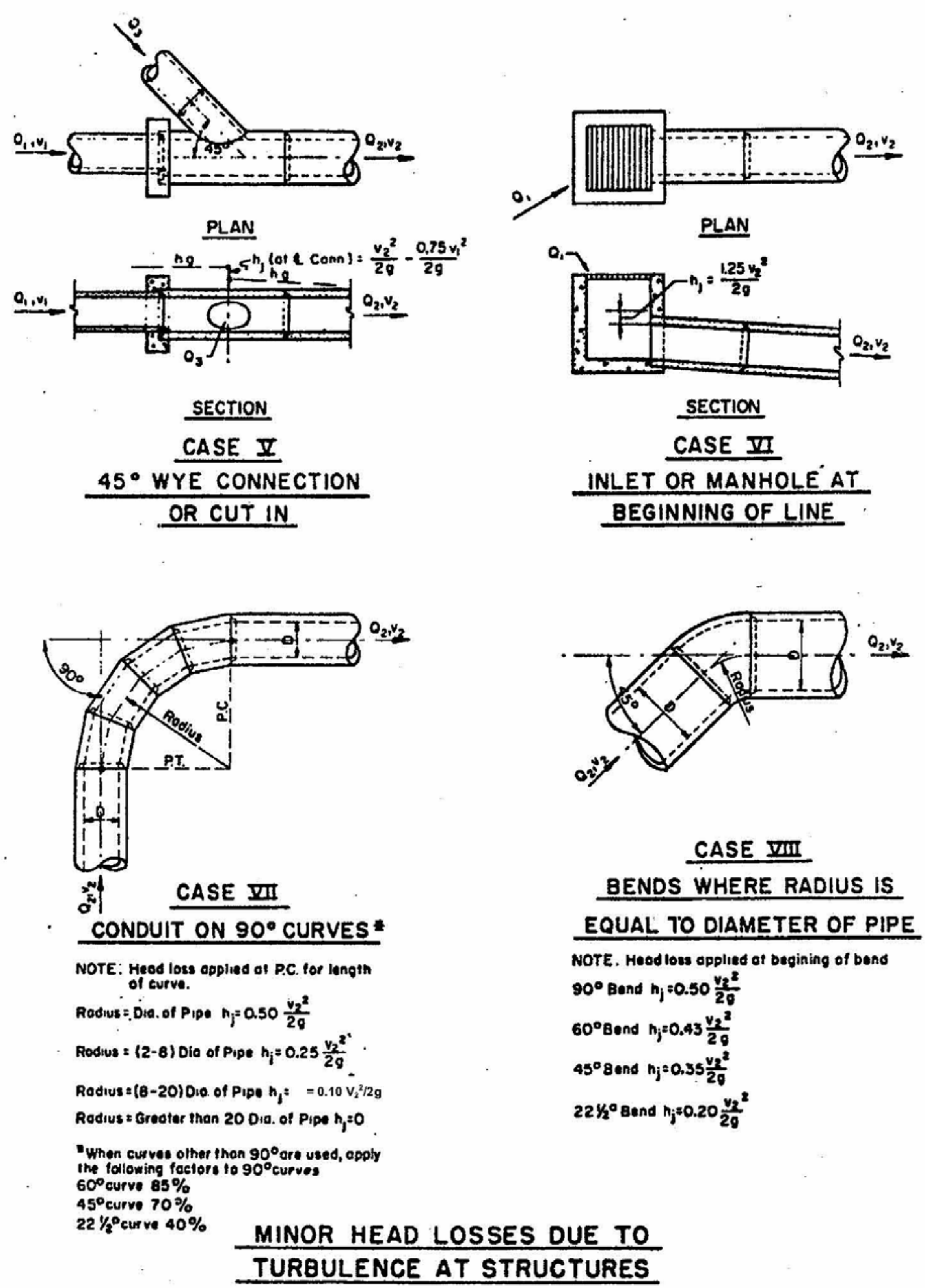

PLAN
SECTION
CASE V
45° WYE CONNECTION OR CUT IN
h_j (at ℄ Conn) = $\frac{v_2^2}{2g} - \frac{0.75 v_1^2}{2g}$
PLAN
SECTION
CASE VI
INLET OR MANHOLE AT BEGINNING OF LINE
$h_j = \frac{1.25 v_2^2}{2g}$
CASE VII
CONDUIT ON 90° CURVES*
NOTE: Head loss applied at P.C. for length of curve.
Radius = Dia. of Pipe $h_j = 0.50 \frac{v_2^2}{2g}$
Radius = (2-8) Dia of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$
Radius = (8-20) Dia. of Pipe h_j = 0.10 $V_2^2/2g$
Radius = Greater than 20 Dia. of Pipe h_j = 0
*When curves other than 90° are used, apply the following factors to 90° curves
60° curve 85%
45° curve 70%
22 ½° curve 40%
CASE VIII
BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE
NOTE. Head loss applied at begining of bend
90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$
60° Bend $h_j = 0.43 \frac{v_2^2}{2g}$
45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$
22½° Bend $h_j = 0.20 \frac{v_2^2}{2g}$
MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

FIGURE 8.2

8.2 VELOCITIES AND GRADES

Storm sewer systems should operate with velocities of flow sufficient to prevent clogging. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps.

The maximum velocities in storm sewer systems are important mainly due to the possibility of excessive erosion on the pipe material. To reduce this erosive potential, the City of Kerrville requires that the maximum velocity in a storm sewer system be 15 fps.

Storm sewer system discharging into an open channel or ditch shall not exceed a velocity of 6 fps without armoring and/or dissipation devices installed at the outfall.

8.3 MATERIALS

Reinforced concrete pipe is the preferred pipe material for public storm sewer systems in the City of Kerrville; however, alternatives may be acceptable on a case-by case basis if approved by the City Engineer.

9.0 STORM WATER DETENTION

Storm water detention shall be provided to mitigate increased peak flows in the City of Kerrville. The purpose of the mitigation is to minimize downstream flooding impacts from upstream development. Storm water detention basins shall be categorized as (On-Site" or "Regional", where On-Site basins are those which are located off-channel and provide stormwater management for a particular project or development, and Regional basjns are designed to provide stormwater management in conjunction with other improvements on a watershed-wide basis. These categories are further subdivided into "Small" and "Large", depending on tributary area impounded volume. Small On-Site basins have drainage areas less than 25 acres, and Large On-Site basins have drainage areas between 25 and 64 acres. Small Regional basins impound up to 150 ac-ft, and Large Regional basins impound more than 150 ac-ft, with any Regional basin having an embankment over 15' categorized as large. The following criteria shall be applied in the design of storm water detention facilities:

1. A fee may be assessed by the City of Kerrville in-lieu of constructing on-site detention if there are existing facilities in place or that are proposed in the near future that would account for the increase runoff from the proposed development improvements.
2. On-site storm water detention shall be provided to control post-development runoff down to pre-development conditions. The proposed cumulative storm water discharges from a development site shall not exceed the calculated discharges under existing conditions.
3. An existing conditions drainage area map shall be provided with the civil construction plans and include the detailed calculations used to determine the existing conditions flow rate. In calculating the existing conditions flow rate, the designer shall also calculate the existing conditions travel time and plot the drainage path on the map. Reference Section 3.1 in this manual for information on calculating time of concentration.
4. Storm water detention facilities for watersheds of up to 150 acres in size shall be designed using the "Modified Rational Method" (see example below).
5. Storm water detention facilities for watersheds over 150 acres shall be designed using a detailed Unit Hydrograph method (i.e., Snyder's or SCS).
6. A summary of the detailed detention calculations shall be provided on the construction plans. If a unit hydrograph is used to size the detention for watersheds over 150 acres, a separate report summarizing the detailed calculations shall be provided to the City for review and referenced on the construction plans. Additionally, if a HEC-HMS unit hydrograph computer model is utilized, a digital copy in HEC-HMS format shall also be provided.

7. Stage-storage-discharge values shall be tabulated and flow calculations for discharge structures shall also be shown on the construction plans. The stage-storage-discharge values shall be provided in a table format and include stages at a maximum of 1 foot increments.
8. Storm water detention facilities shall be designed for the 50% (2-year), 20% (5-year), 4% (25-year), and 1% (100-year) storm events.
9. Off-site areas draining through the proposed development site shall not be allowed to pass through the proposed on-site storm water detention facility unless the off-site area is released through the proposed detention facility at pre-development conditions and the actual travel time is considered. Otherwise, the off-site flows shall be conveyed via a separate drainage system and bypass the proposed detention facility.
10. Large On-Site and Regional Storm water detention basins shall be designed with a maintenance access wide enough for a 10' wide tracked backhoe to maneuver. This generally requires a minimum of a 12' wide maintenance access be provided in all detention basins. The maximum cross slope shall not exceed 2.0% and the longitudinal slope shall not exceed 6H:1V. Basins with permanent storage (retention basins) must include dewatering facilities to provide for maintenance.
11. When an earthen embankment is proposed for detention, a typical embankment section and specifications for fill shall be included in the construction plans. No earthen embankment shall exceed a slope greater than 3H:1V.
12. An armored emergency spillway shall be provided above the 1% storm water surface elevation and have sufficient capacity to convey the 1% storm with the following minimum freeboard to top of embankment. The spillway design calculations shall be included in the construction plans.

DETENTION BASIN CLASS	MINIMUM FREEBOARD
On-Site Small	0.5'
On-Site Large	1.0'
Regional: Small	2.0'
Regional: Large	*

*Design storm event and required freeboard for Large Regional ponds shall be determined by a dam breach analysis based on the principles outlined in Chapter 299 of the Texas Administrative Code. The dam breach analysis shall be submitted to the City Engineer for approval.

13. Minimum crest widths for earthen embankments shall be as follows:

EMBANKMENT HEIGHT	MINIMUM CREST WIDTH
Up to 4'	3'
>4' to 6'	4'
>6'	As recommended by geotechnical engineer

14. Storm runoff may be detained within parking lots. However, the engineer should be aware of the inconvenience to both pedestrians and traffic. The location of ponding areas in a parking lot should be planned so that this condition is minimized. Stormwater ponding depths (for the 100-year storm) in parking lots are limited to an average height of eight (8) inches with a maximum of twelve (12) inches.
15. All detention basins shall be stabilized to prevent erosion. For earthen detention basins, stabilization shall be defined as the uniform establishment of perennial vegetative cover with a density of at least 70% of the native background for all unpaved areas and areas not covered by permanent structures, or equivalent permanent stabilization measures (such as the use of riprap, gabions or geotextiles) have been employed.

16. State rules and regulations regarding impoundments and dams shall be observed in the design and maintenance of storm water detention facilities.
17. Outflow structures for storm water detention facilities shall be designed so that discharge flows at a non-erosive rate.
18. An outlet control structure such as an orifice or weir placed at the inlet end of the outfall pipe is to provide an integrated stage-discharge such that a wide range of storms can be effectively controlled. Emergency overflow structures and paved positive overflow channels shall be included with the design of detention systems.
19. Whenever possible, detention basins shall fit in the natural contour of the land, be aesthetically pleasing and be free draining. A grading plan with 1-foot intervals shall be placed on the construction plans. Maintenance access shall be provided for each basin. The bottom slope shall be a minimum of 1% towards the outfall structure. Detention basins shall be designed with short and long term erosion control.
20. A detention basin maintenance plan must be submitted to the City Engineer prior to final acceptance. A sample detention basin maintenance plan is included on the following page of this manual.
21. Detention basins shall be enclosed within a detention easement and the filed easement document shall be provided to the City Engineer prior to final acceptance

DETENTION BASIN MAINTENANCE PLAN
City of Kerrville Project No.
(**Project Name**)

The following are guidelines for the overall maintenance of the detention basin.

- *Inspections*. The detention system should be inspected to assure proper operation at least 4 times annually. One of these inspections should occur during or immediately following wet weather.

- *Sediment Removal*. Remove sediment from outlet weir structure, and downstream of the outlet at least 2 times annually, or when depth reaches 3 inches. When sediment accumulation in other areas of the basin, fills the basin by 10% of the basin volume, all sediment should be removed and disposed of properly.

- *Mowing*. The side slopes, and embankment of a detention basin must be mowed regularly to discourage woody growth and control weeds. Grass areas in and around basins must be mowed at least four times annually to limit vegetation height to 12 inches. More frequent mowing to maintain aesthetic appeal may be necessary in landscaped areas. When mowing is performed, a mulching mower should be used, or grass clippings should be caught and removed.

- *Debris and Litter Removal*. Debris and litter will accumulate near the outfall weir and should be removed during regular mowing operations and inspections. Particular attention should be paid to floating debris that can eventually clog the outfall weir.

- *Erosion Control*. The pond side slopes and embankment may periodically suffer from slumping and erosion, although this should not occur often if the soils are properly compacted during construction. Regrading and revegetation may be required to correct the problems.

- *Nuisance Control*. Standing water or soggy conditions in the detention basin can create nuisance conditions for nearby residents. Odors, mosquitoes, weeds, and litter are all occasionally perceived to be problems. Most of these problems are generally a sign that regular inspections and maintenance are not being performed (e.g., mowing and debris removal).

I agree to perform the above maintenance items on the Detention Basin.

______________________________ ______________________________

OWNER (Please print name) DATE

____

SIGNATURE

MODIFIED RATIONAL METHOD
DETENTION BASIN DESIGN (EXAMPLE)

Given: A 10-acre site, currently pasture land with on an average slope of 5% percent, is to be developed into a single family residential subdivision (typical lot will have 60-70% impervious cover). The entire area is proposed to drain into a proposed detention basin. The existing time of concentration (Tc) has been determined to be 21 minutes and the proposed 15 minutes.

Determine: Maximum release rate and required detention storage for the 1% storm event.

Solution:

1. Determine 1% storm event's peak runoff rate prior to site development. This is the maximum allowable release rate from the site after development.
2. Determine the inflow hydrograph for storms of various durations in order to determine maximum volume required with the release rate determined in Step 1 below.
   Note: Incrementally increase durations (1-minute normally & 5-minutes maximum) to determine maximum required storage volume. The duration with a peak inflow less than maximum release rate, or where required storage is less than storage for the prior duration, is the last increment.

Step 1:
<u>Present Conditions</u> $Q = C \times I \times A$

C = 0.51
Tc = 21 min
I = 7.75 in/hr
Q = (0.51) (7.75) (10.0) = 39.5 cfs (Max allowable release rate)

Step 2:
<u>Future Conditions</u> (Single family Residential 65% Impervious Cover)

C = 0.79
Tc = 15 min
I = 9.24 in/hr
Q = (0.79) (9.24) (10.0) = 73.0 cfs

Step 3:
Check various duration storms

10 min	I = 11.11	Q= 0.79 x 11.11 x 10 = 87.8
15 min	I = 9.24	Q= 0.79 x 9.24 x 10 = 73.0
20 min	I = 7.96	Q= 0.79 x 7.96 x 10 = 62.9
25 min	I = 7.03	Q= 0.79 x 7.03 x 10 = 55.5
30 min	I = 6.31	Q= 0.79 x 6.31 x 10 = 49.9
35 min	I = 5.57	Q= 0.79 x 5.57 x 10 = 45.4
40 min	I = 5.29	Q= 0.79 x 5.29 x 10 = 41.8

Maximun Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow for each duration.

Inflow = (Storm Duration) X (Respective Peak Discharge) X (60 sec/min)
Outflow = (Half of the respective inflow duration) X (control release discharge) X (60 sec/min)

10 min. Storm	Inflow 10 x 87.8 x 60 sec/min	=	52,654 cf
	Outflow 0.5 x 25 x 39.5 x 60 sec. /min	=	29,644 cf
	Storage	=	**23,010 cf**
15 min. Storm	Inflow 15 x 73.0 x 60 sec /min.	=	65,667 cf
	Outflow 0.5 x 30 x 39.5 x 60 sec. /min	=	35,573 cf
	Storage	=	**30,094 cf**
20 min. Storm	Inflow 20 x 62.9 x 60 sec /min.	=	75,456 cf
	Outflow 0.5 x 35 x 39.5 x 60 sec. /min	=	41,501 cf
	Storage	=	**33,955 cf**
25 min. Storm	Inflow 25 x 55.5 x 60 sec /min.	=	83,275 cf
	Outflow 0.5 x 40 x 39.5 x 60 sec. /min	=	47,430 cf
	Storage	=	**35,845 cf**
30 min. Storm	Inflow 30 x 49.9 x 60 sec /min.	=	89,777 cf
	Outflow 0.5 x 45 x 39.5 x 60 sec. /min	=	53,359 cf
	Storage	=	**36,419 cf**
35 min. Storm	Inflow 35 x 45.4 x 60 sec /min.	=	95,343 cf
	Outflow 0.5 x 50 x 39.5 x 60 sec. /min	=	59,288 cf
	Storage	=	**36,055 cf**
40 min. Storm	Inflow 40 x 41.8 x 60 sec /min.	=	100,210 cf
	Outflow 0.5 x 55 x 39.5 x 60 sec. /min	=	65,216 cf
	Storage	=	**34,994 cf**

Maximun Volume required is 36,419 cf at 30 min. storm duration.

10.0 MINIMUM LOT AND FLOOR ELEVATIONS

Minimum lot and floor elevations shall be established as follows:

(1) Lots abutting a natural or excavated channel shall have a minimum elevation for the buildable area of the lot at least one-foot higher than the top of channel bank or 1% storm event water surface elevation, whichever is higher.

(2) Any habitable structure on property in or abutting a floodplain shall conform to the City's Floodplain Management Ordinance. All structures must be located at least one (1) foot above the 1% storm floodplain.

(3) Where lots do not join a natural or excavated channel, minimum floor elevations shall be a minimum of one (1) foot above the street curb or edge of alley, whichever is higher. Where the intent of the development is to preserve the natural condition of the site (Tree Preservation), the finished floor elevation may be lower if approved by the City Engineer. Such approval will require special design parameters to ensure runoff from the street or alley does not flow into or across the lot.

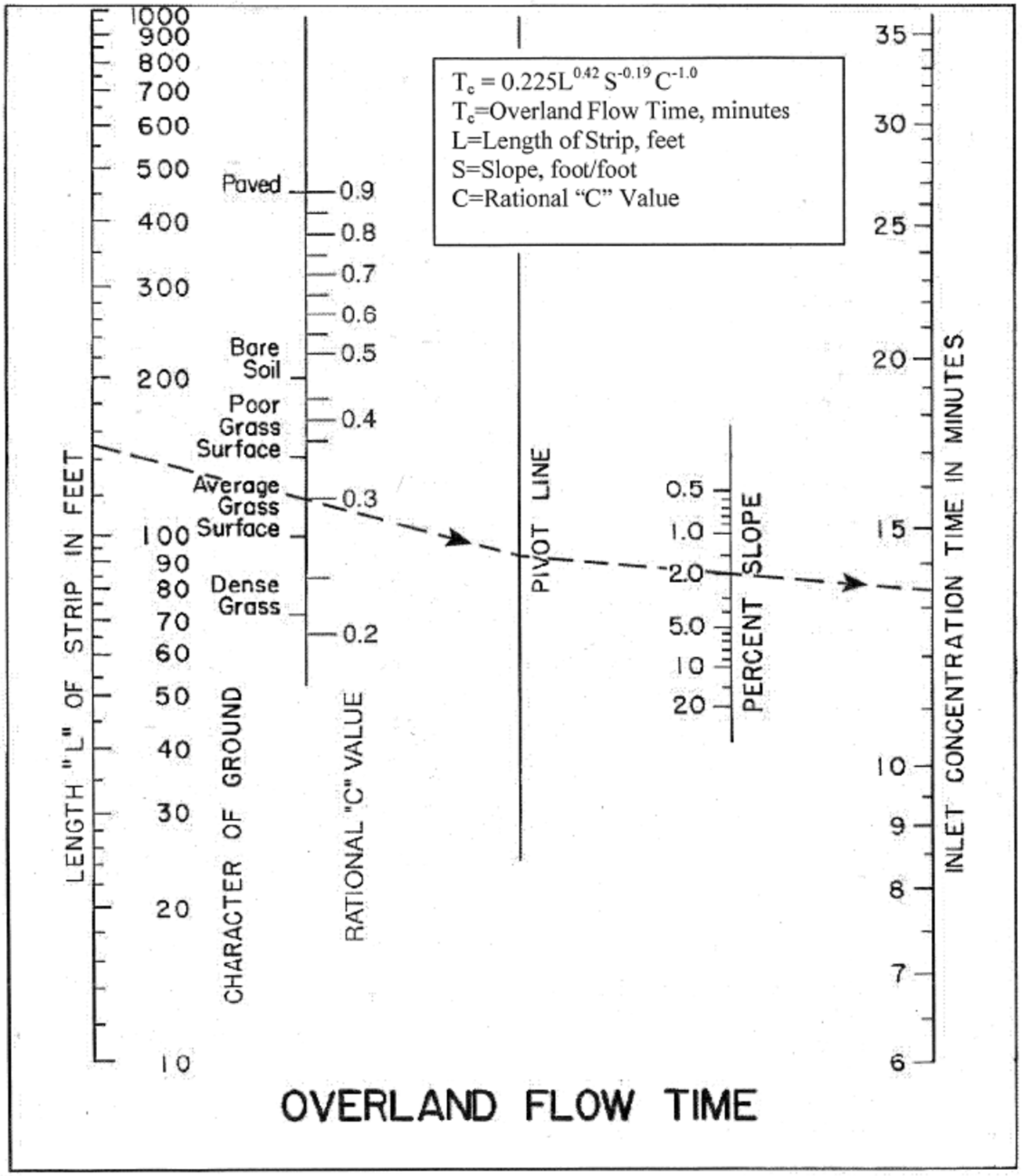
11.0 DRAINAGE EASEMENTS

Drainage Easements shall be provided for all storm sewer systems conveying runoff from one property to another. Drainage Easements for storm sewer pipe shall not be less than 15 feet, and easement widths for open channels shall be at least 20 feet wider than the top of the channel, 15 feet of which shall be on one side to serve as an access for maintenance purposes. Where easements are proposed parallel with property lot lines, the easements shall not be allowed to straddle lot lines; instead, the easement must be located on one side of each lot.

APPENDIX A - FIGURES

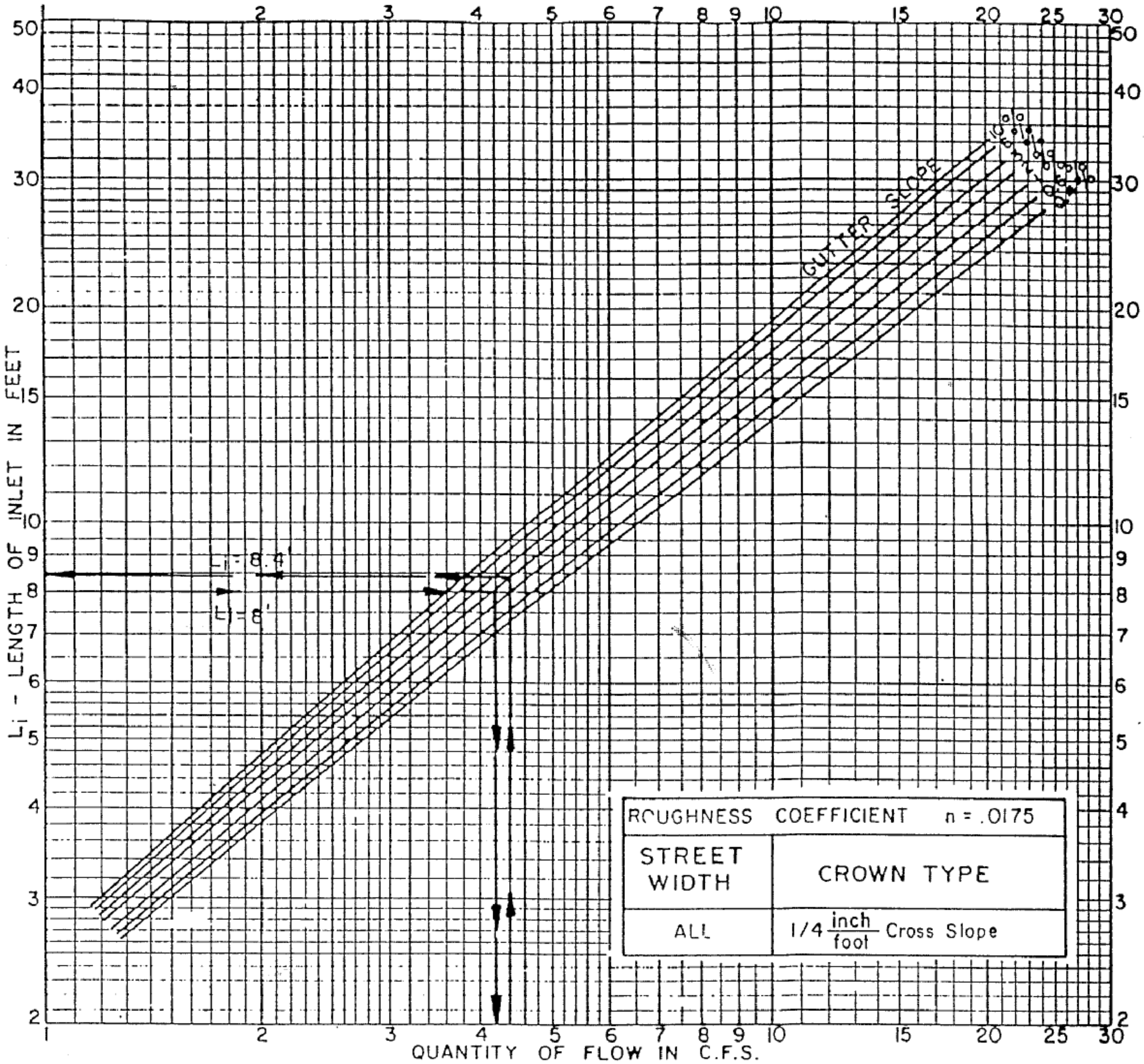
DRAINAGE DESIGN MANUAL

CITY OF KERRVILLE
KERR COUNTY
TEXAS



(Source: Data Book for Civil Engineers, Vol. I – Design, 1951)

Figure A



EXAMPLE

Known:

Pavement Width = 24'
Gutter Slope = 2.0 %
Pavement Cross Slope = 1/4"/1'
Gutter Flow = 4.4 cfs

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 4.4 cfs
Intersect Slope = 2.0 %
Read L_i = 8.4'

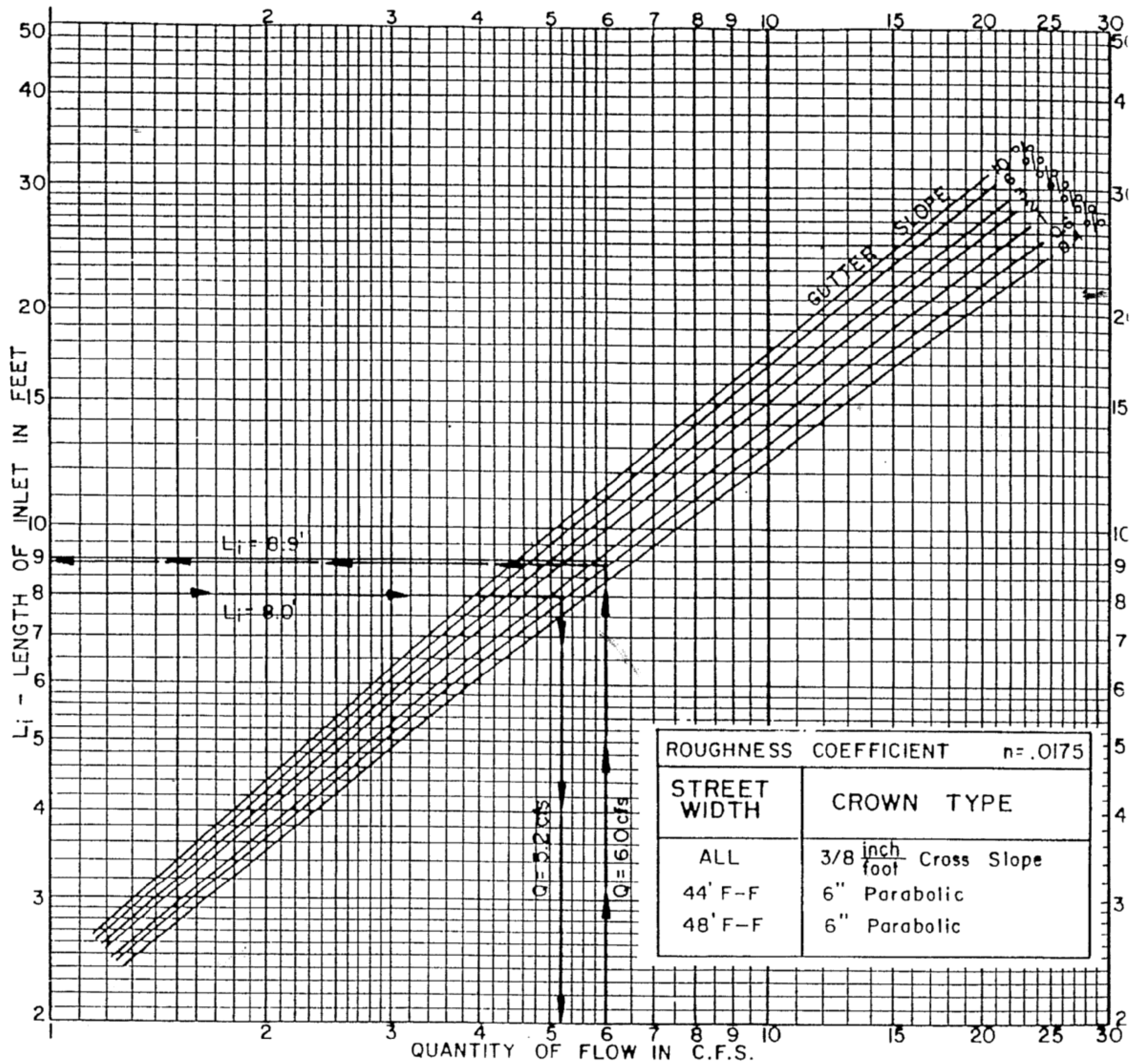
Decision:

1 Use 10' Inlet
No Flow Remains in Gutter
2. Use 8' Inlet
Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2.0%
Read Q = 4.2 cfs
Remaining Gutter Flow =
4.4 cfs — 4.2 cfs = 0.2 cfs

RECESSED AND STANDARD
CURB OPENING INLET
CAPACITY CURVES
ON GRADE

Figure B



EXAMPLE

Known:
- Pavement Width = 44'
- Gutter Slope = 0.6 %
- 6" Parabolic Crown
- Gutter Flow = 6.0 cfs

Find:
- Length of Inlet Required (L_i)

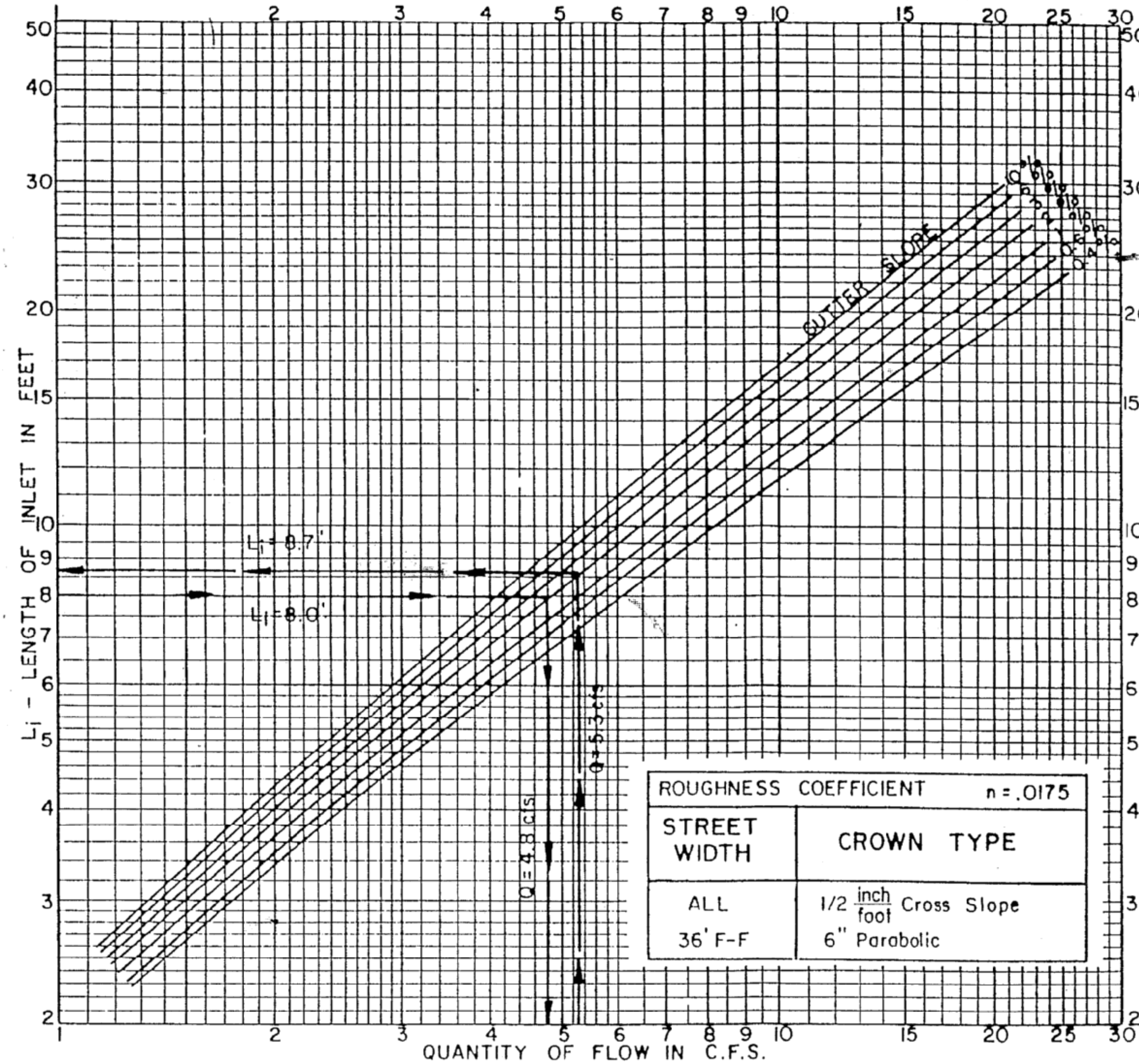
Solution:
- Enter Graph at 6.0 cfs
- Intersect Slope = 0.6 %
- Read L_i = 8.9'

Decision:
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 0.6 %
Read Q = 5.2 cfs
Remaining Gutter Flow =
6.0 cfs — 5.2 cfs = 0.8 cfs

RECESSED AND STANDARD
CURB OPENING INLET
CAPACITY CURVES
ON GRADE

Figure C



EXAMPLE

Known:
- Pavement Width = 36'
- Gutter Slope = 2%
- 6" Parabolic Crown
- Gutter Flow = 5.3 c f s

Find:
- Length of Inlet Required (L_i)

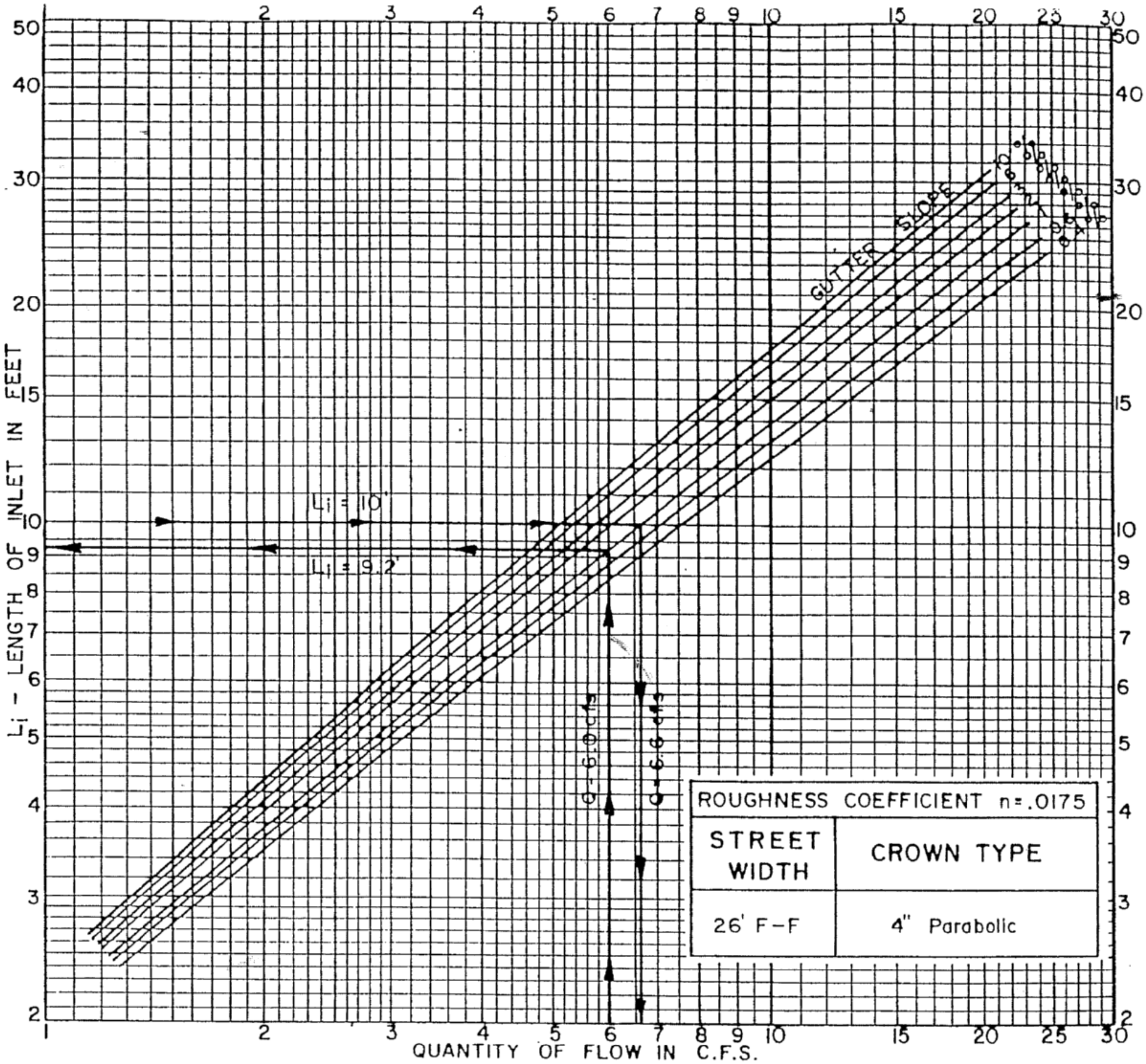
Solution:
- Enter Graph at 5.3 c f s
- Intersect Slope = 2%
- Read L_i = 8.7'

Decision:
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2%
Read Q = 4.8 c f s
Remaining Gutter Flow =
5.3 c f s – 4.8 c f s = 0.5 cfs

RECESSED AND STANDARD
CURB OPENING INLET
CAPACITY CURVES
ON GRADE

Figure D



ROUGHNESS COEFFICIENT n=.0175	
STREET WIDTH	CROWN TYPE
26' F–F	4" Parabolic

EXAMPLE

Known:
Pavement Width = 26'
Gutter Slope = 1%
4" Parabolic Crown
Gutter Flow = 6.0 cfs

Find:
Length of Inlet Required (L_i)

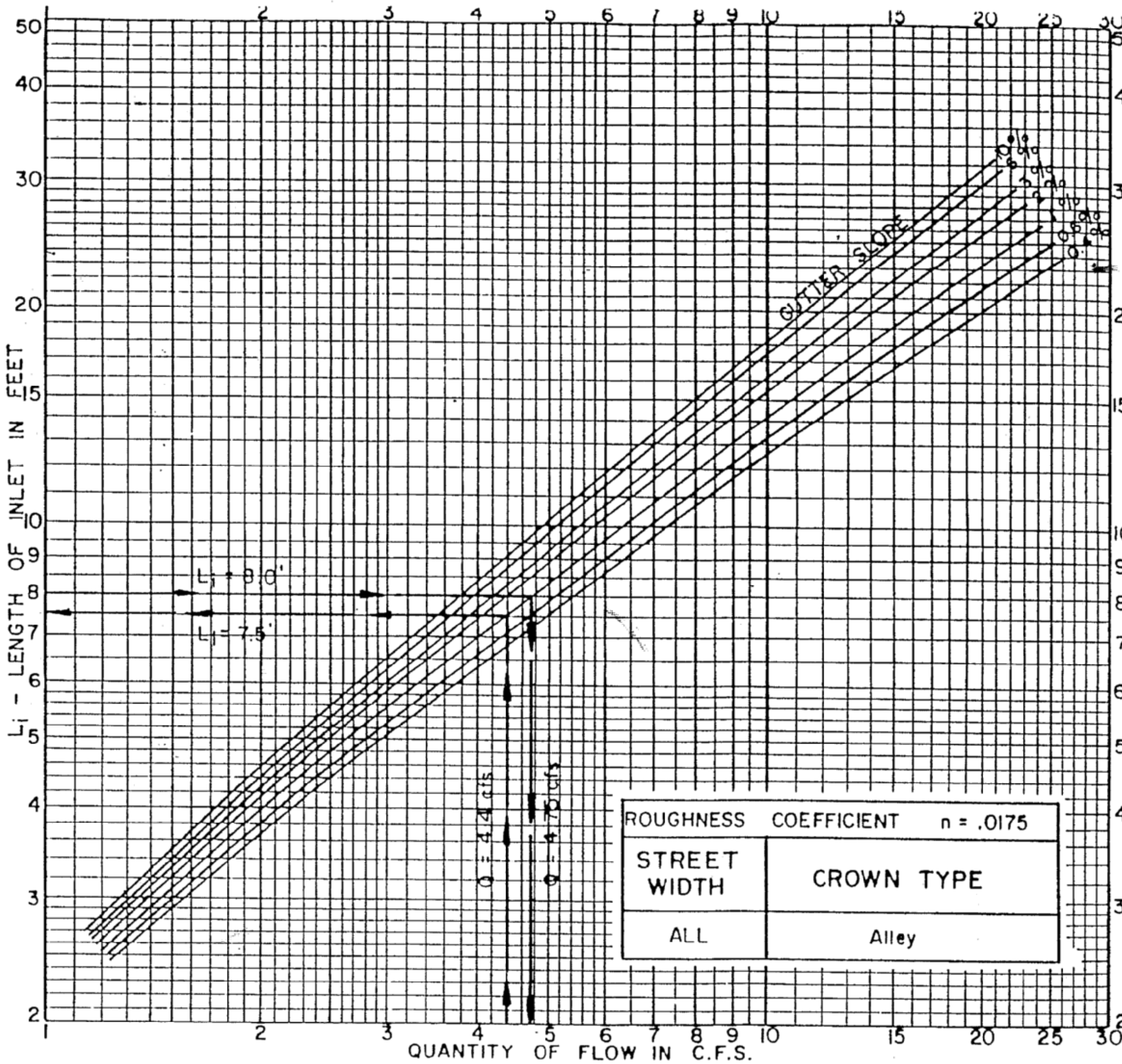
Solution:
Enter Graph at 6.0 cfs
Intersect Slope = 1%
Read L_i = 9.2'

Decision:
1. Use 10' Inlet
No Flow Remains in Gutter
2. Use 8' Inlet
Intercept Only Part of Flow

Use 10' Inlet
Enter Graph at L_i = 10'
Intersect Slope = 1%
Read Q = 6.6 cfs
No Flow Remains in Gutter

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

Figure E



EXAMPLE

Known:
- Pavement Width = 16'
- Gutter Slope = 1%
- Pavement Cross Slope = 1/4"/1'
- Gutter Flow = 4.4 cfs

Find:
- Length of Inlet Required (L_i)

Solution:
- Enter Graph at 4.4 cfs
- Intersect Slope = 1%
- Read L_i = 7.5'

Decision:
1. Use 8' Inlet
   No Flow Remains in Gutter
2. Use 6' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 1%
Read Q = 4.75 cfs
No Flow Remains in Gutter

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE F

EXAMPLE

Known:

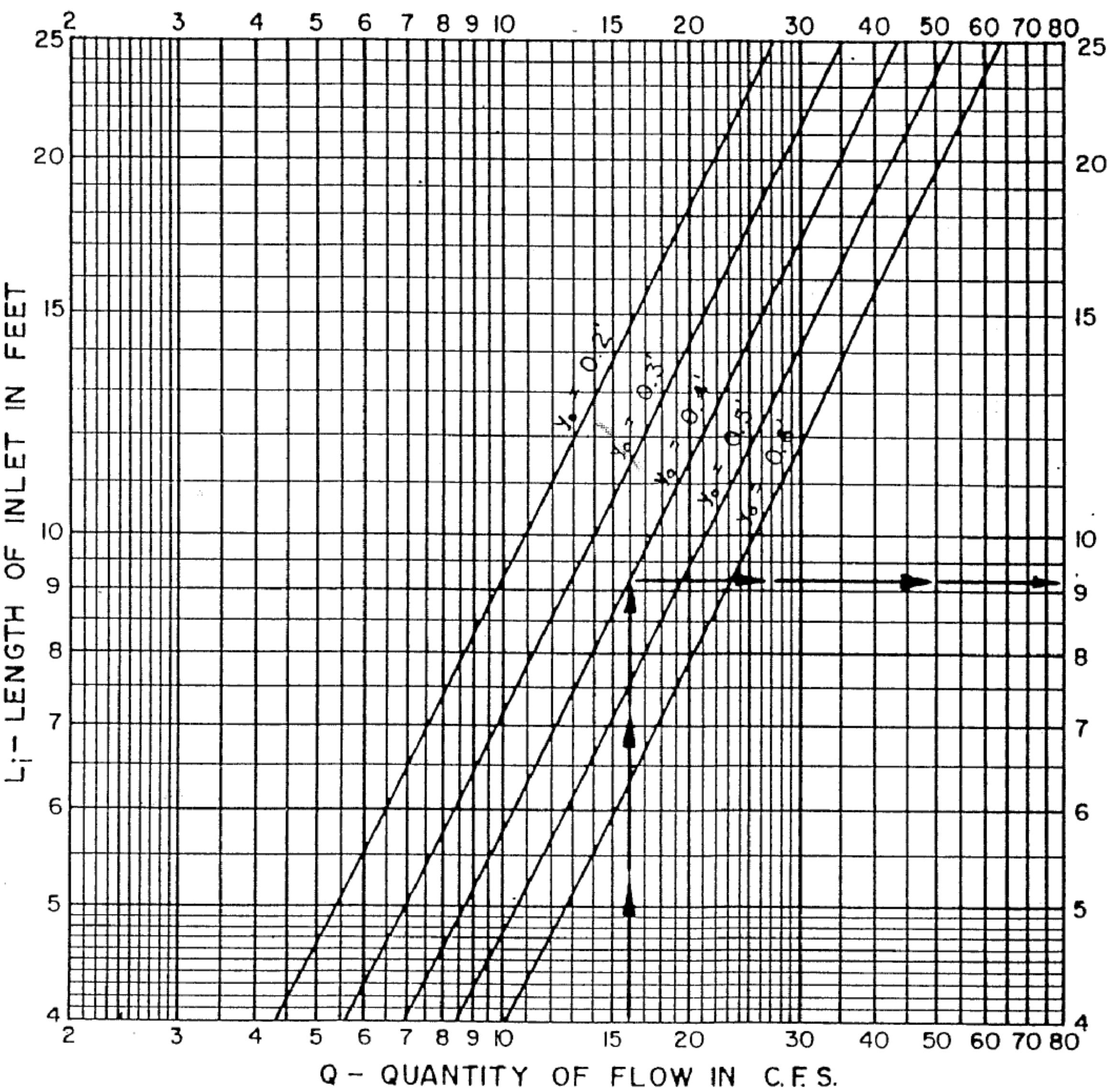
Quantity of Flow = 16.0 c.f.s.
Maximum Depth of Flow Desired in Gutter At Low Point (y_o) = 0.4'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 16.0 c.f.s.
Intersect y_o = 0.4'
Read L_i = 9.2'
Use 10' Inlet



ROUGHNESS COEFFICIENT n = .0175	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES AT LOW POINT

FIGURE G

EXAMPLE

Known:

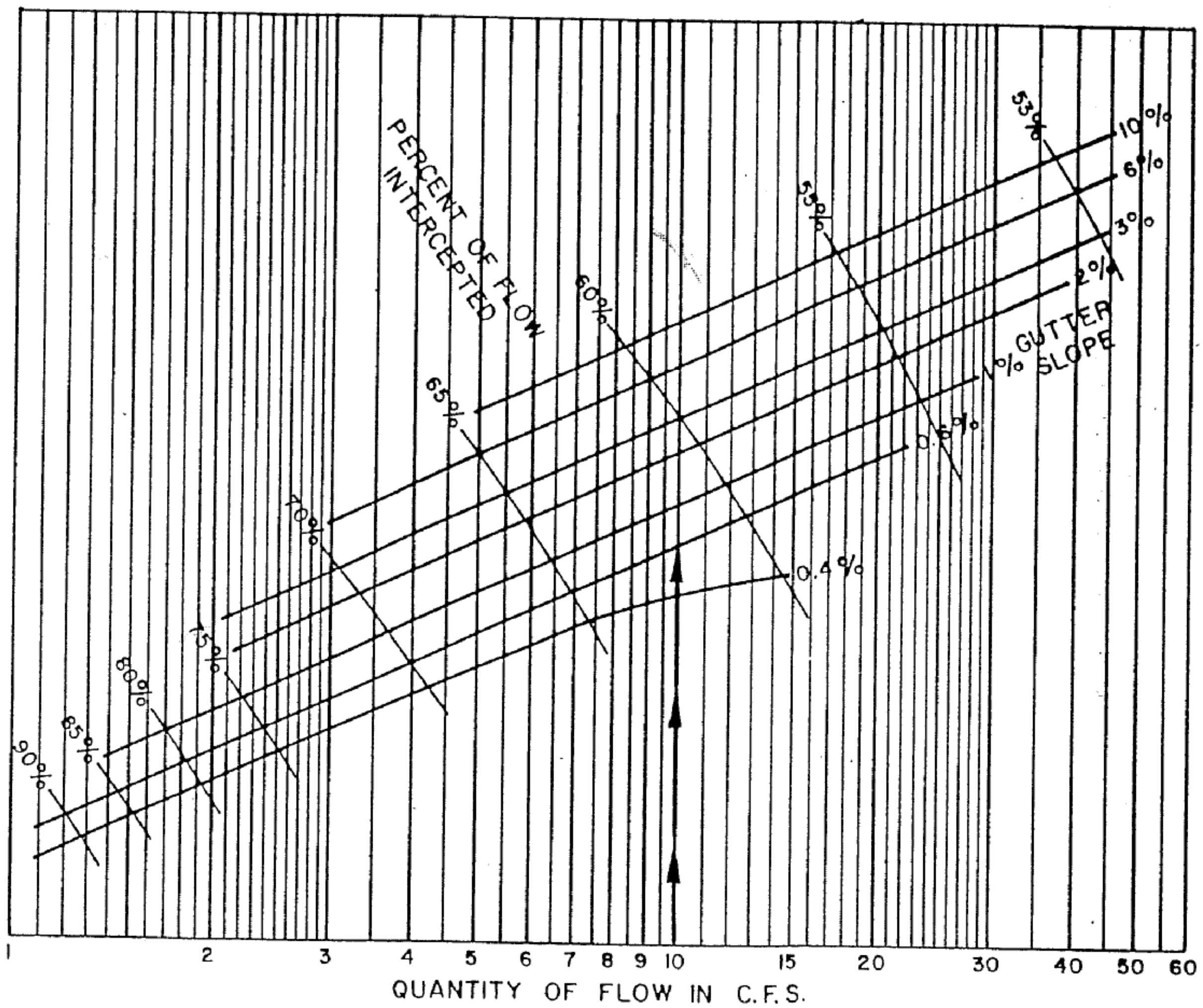
Quantity of Flow = 10.0 c.f.s.
Gutter Slope = 0.6 %

Find:

Capacity of Two Grate Combination Inlet

Solution:

Enter Graph at 10.0 c.f.s.
Intersect Slope = 0.6 %
Read Percent of Flow Intercepted = 62 %
62 % of 10.0 c.f.s. = 6.2 c.f.s. as Capacity of Two Grate Combination Inlet
Remaining Gutter Flow = 10.0 c.f.s − 6.2 c.f.s. = 3.8 c.f.s.



TWO GRATE COMBINATION INLET
CAPACITY CURVES
ON GRADE

FIGURE H

EXAMPLE

Known:

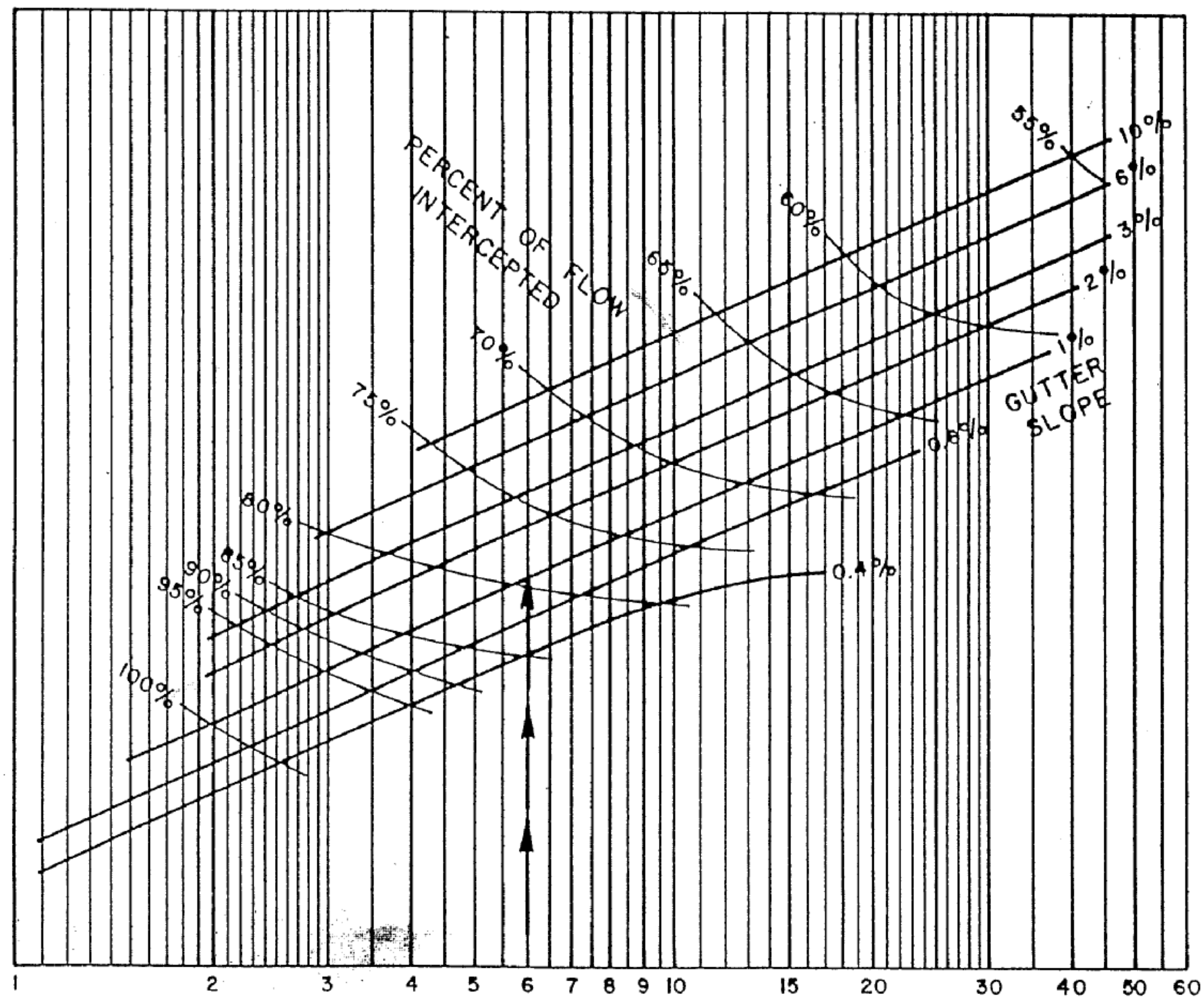
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0 %

Find:

Capacity of Four Grate Combination Inlet

Solution:

Enter Graph at 6.0 c.f.s.
Intersect Slope = 1.0 %
Read Percent of Flow Intercepted = 79 %
79 % of 6.0 c.f.s. = 4.7 c.f.s.
as Capacity of Four Grate Combination Inlet
Remaining Gutter Flow =
6.0 c.f.s. – 4.7 c.f.s. = 1.3 c.f.s.



FOUR GRATE COMBINATION INLET
CAPACITY CURVES
ON GRADE

FIGURE I

EXAMPLE

Known:

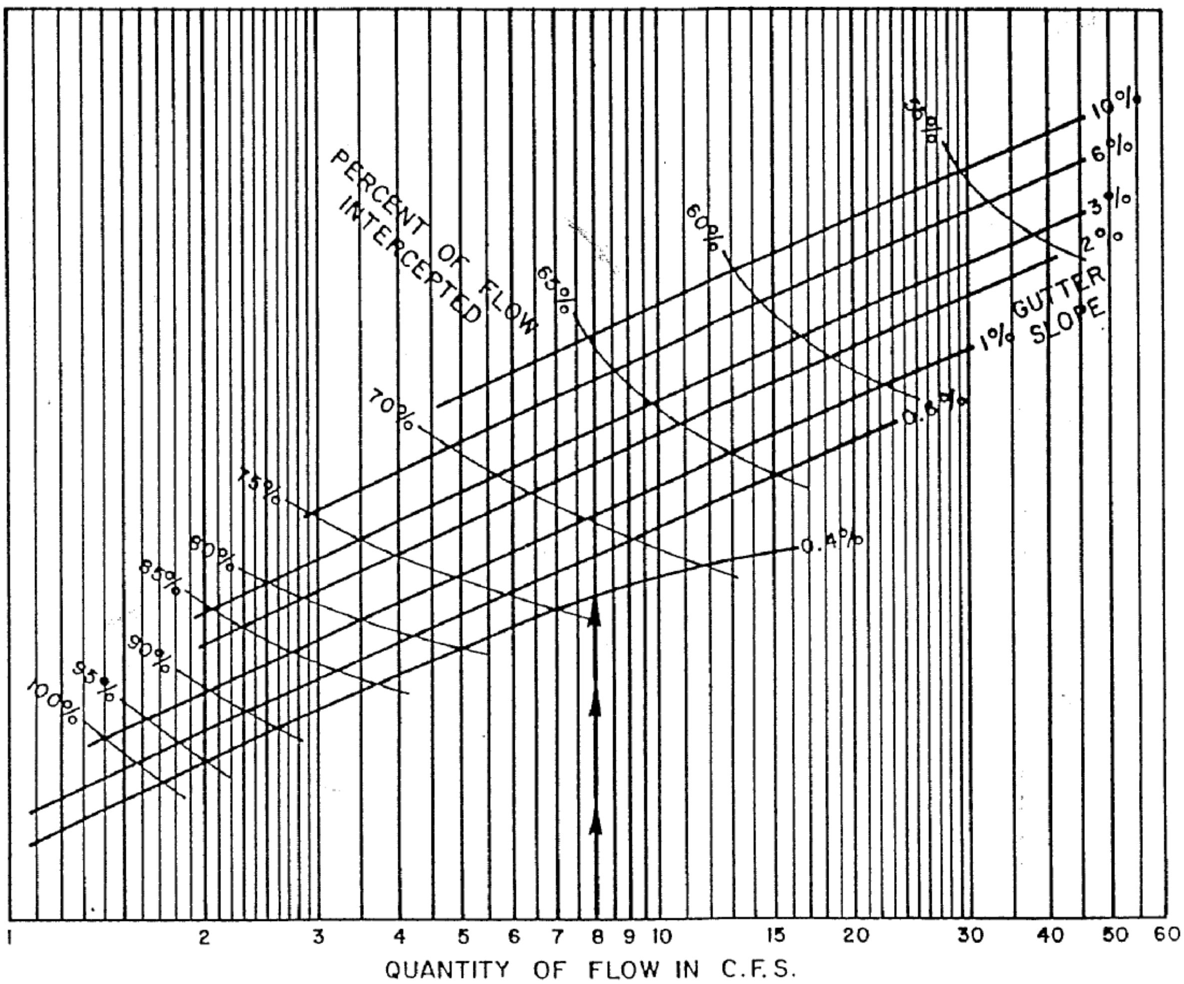
Quantity of Flow = 8.0 c.f.s.
Gutter Slope = 0.4%

Find:

Capacity of Three Grate Inlet

Solution:

Enter Graph at 8.0 c.f.s.
Intersect Slope = 0.4%
Read Percent of Flow
Intercepted = 74%
74% of 8.0 c.f.s. = 5.9 c.f.s.
as Capacity of Three Grate Inlet
Remaining Gutter Flow =
8.0 c.f.s. – 5.9 c.f.s. = 2.1 c.f.s.



THREE GRATE INLET AND
THREE GRATE COMBINATION INLET
CAPACITY CURVES
ON GRADE

FIGURE J

EXAMPLE

Known:

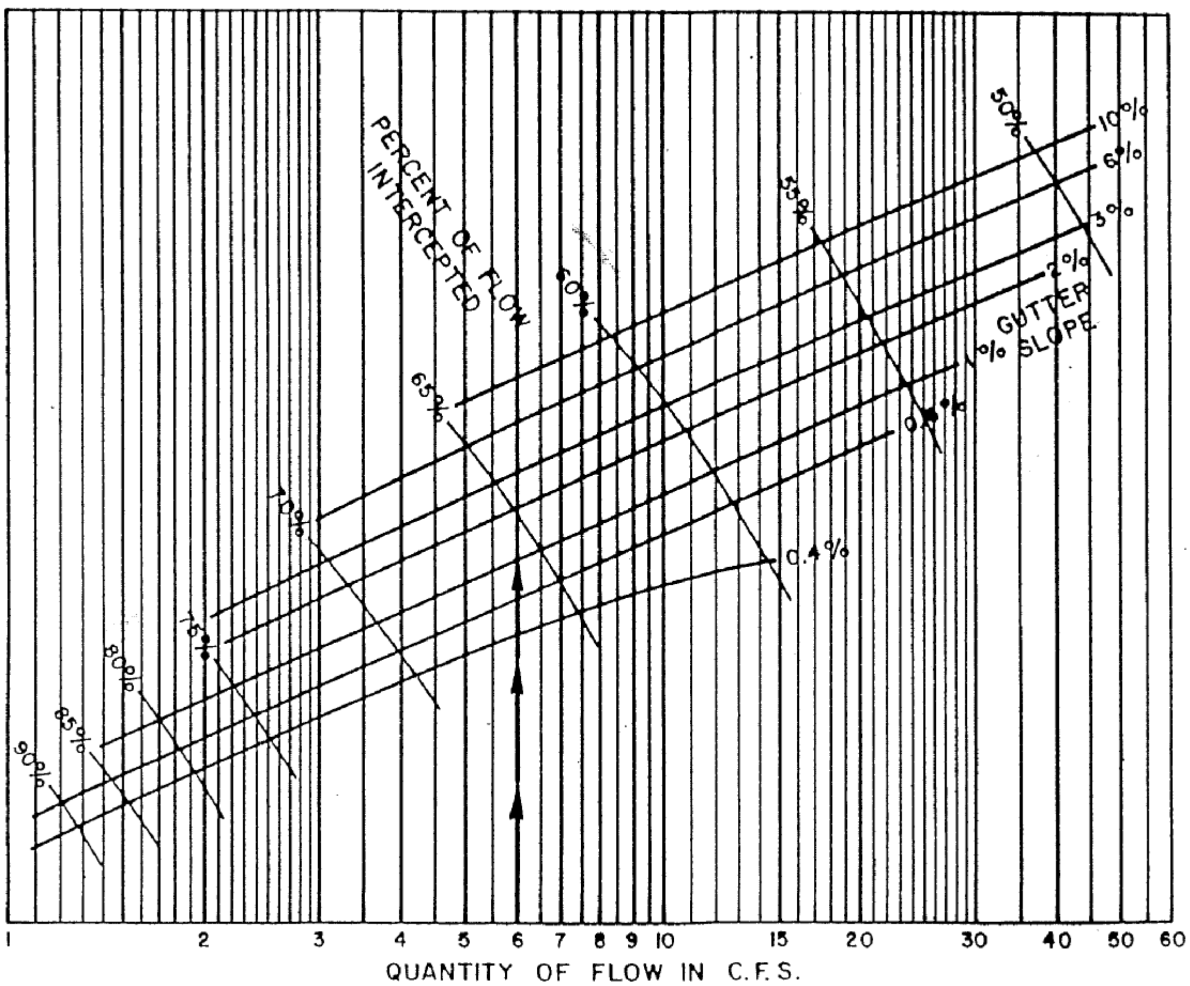
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0%

Find:

Capacity of Two Grate Inlet

Solution:

Enter Graph at 6.0 c.f.s.
Intersect Slope = 1.0%
Read Percent of Flow Intercepted = 66%
66% of 6.0 c.f.s. = 4.0 c.f.s. as Capacity of Two Grate Inlet.
Remaining Gutter Flow = 6.0 c.f.s. – 4.0 c.f.s. = 2.0 c.f.s.



TWO GRATE INLET CAPACITY CURVES ON GRADE

FIGURE K

EXAMPLE

Known:

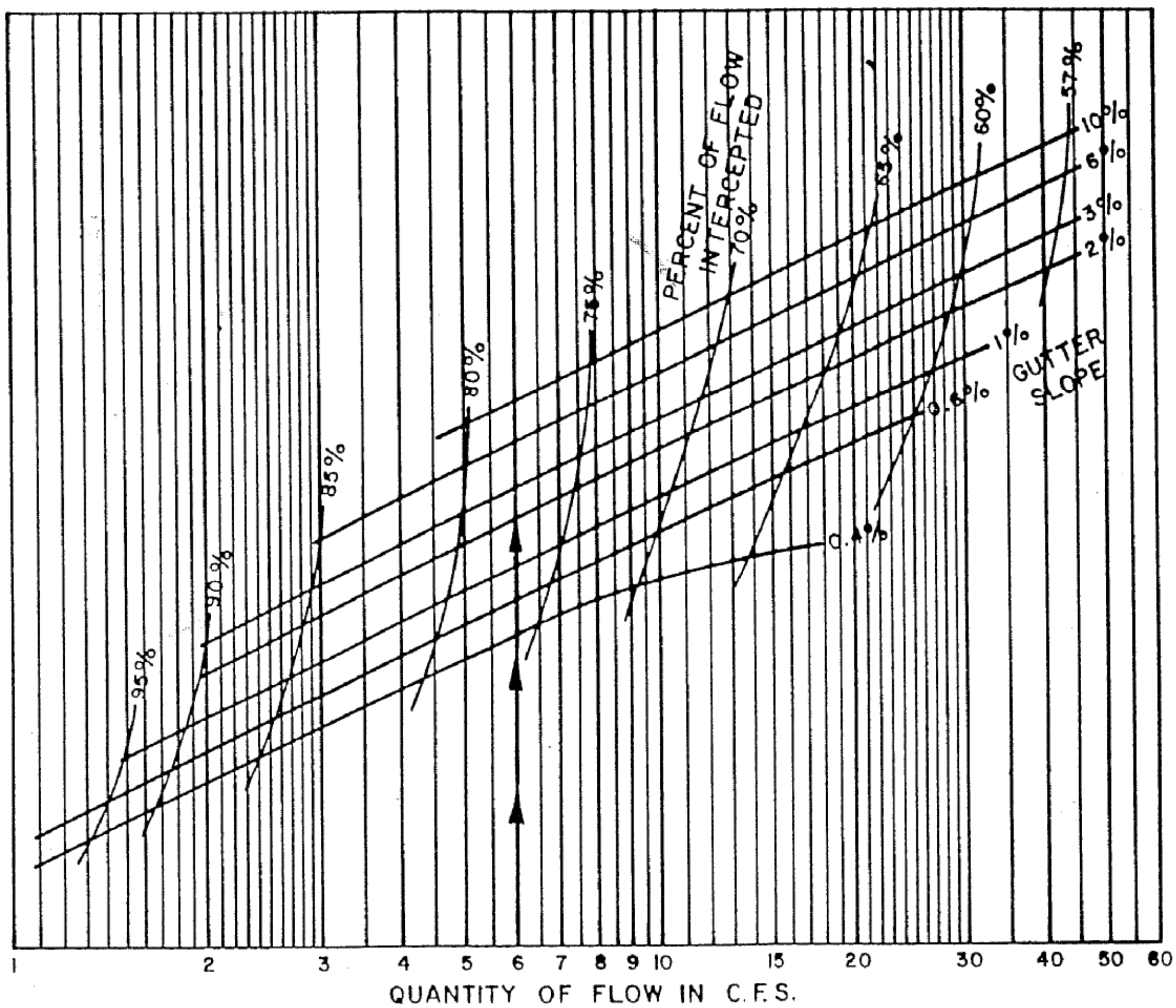
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0 %

Find:

Capacity of Four Grate Inlet

Solution:

Enter Graph at 6.0 c.f.s.
Intersect Slope = 1.0 %
Read Percent of Flow
Intercepted = 77 %
77 % of 6.0 c.f.s. = 4.6 c.f.s.
as Capacity of Four Grate Inlet
Remaining Gutter Flow =
6.0 c.f.s. – 4.6 c.f.s. = 1.4 c.f.s.



FOUR GRATE INLET
CAPACITY CURVES
ON GRADE

FIGURE L

EXAMPLE

Known:

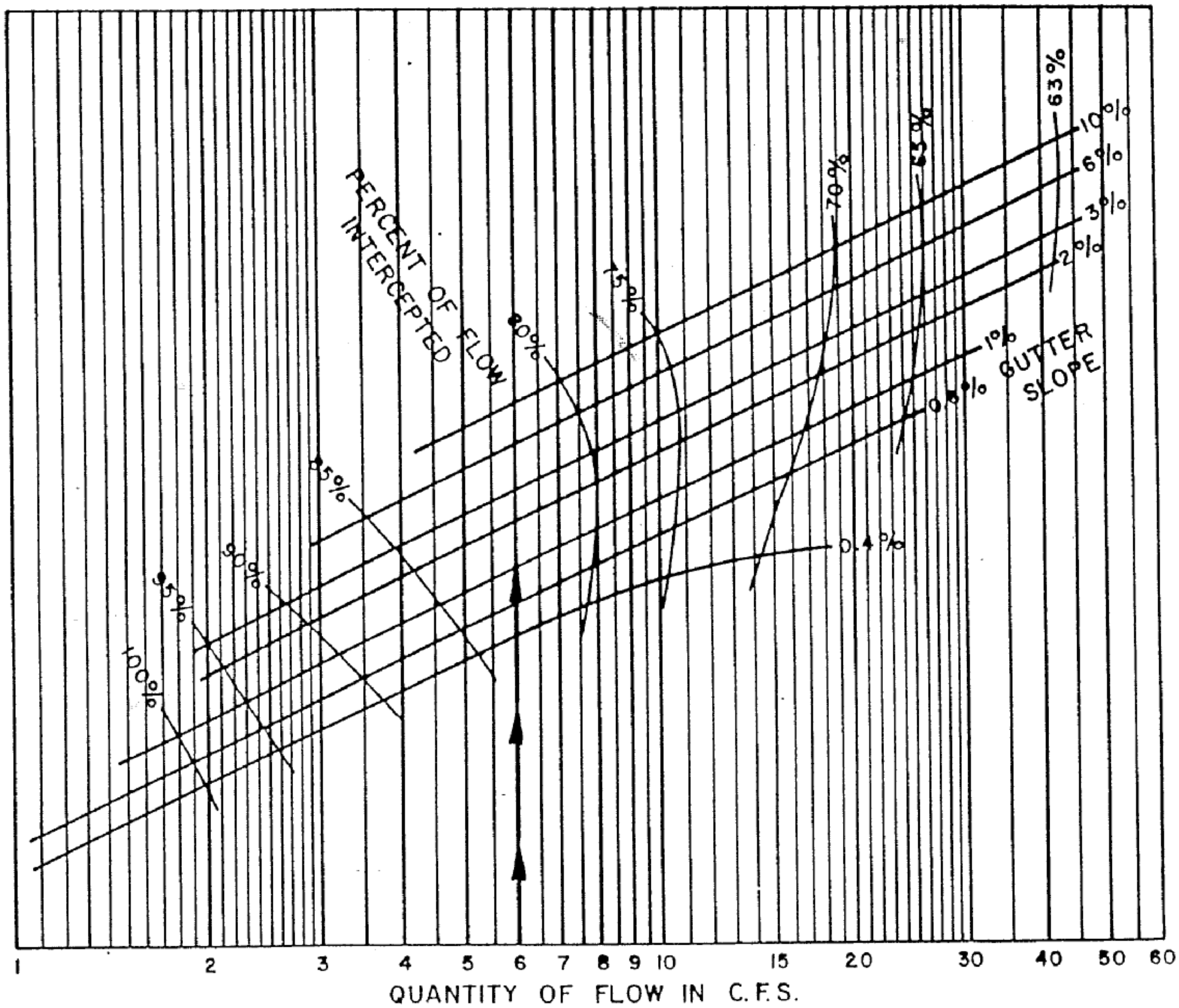
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0%

Find:

Capacity of Six Grate Inlet

Solution:

Enter Graph at 6.0 c.f.s.
Intersect Slope = 1.0%
Read Percent of Flow
Intercepted = 82%
82% of 6.0 c.f.s. = 4.9 c.f.s.
as Capacity of Six Grate Inlet
Remaining Gutter Flow =
6.0 c.f.s. – 4.9 c.f.s. = 1.1 c.f.s.



SIX GRATE INLET
CAPACITY CURVES
ON GRADE

FIGURE M

EXAMPLE

Known:

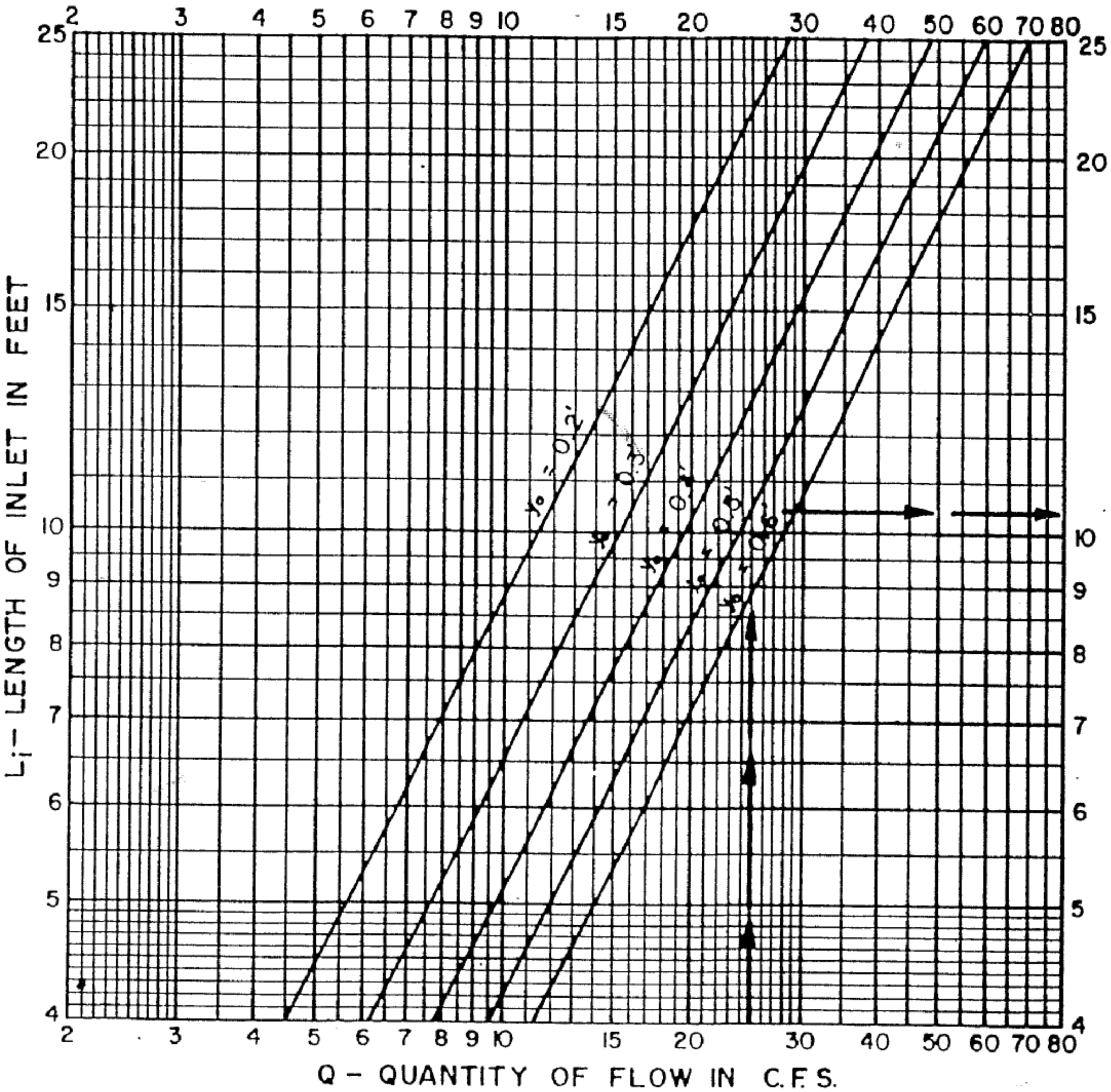
Quantity of Flow = 25.0 c.f.s.
Maximum Depth of Flow Desired
At Low Point (y_o) = 0.5'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 25.0 c.f.s.
Intersect y_o = 0.5'
Read L_i = 10.4'
Use 12' Inlet



ROUGHNESS COEFFICIENT n = .0175	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

COMBINATION INLET
CAPACITY CURVES
AT LOW POINT

FIGURE N

EXAMPLE

Known:

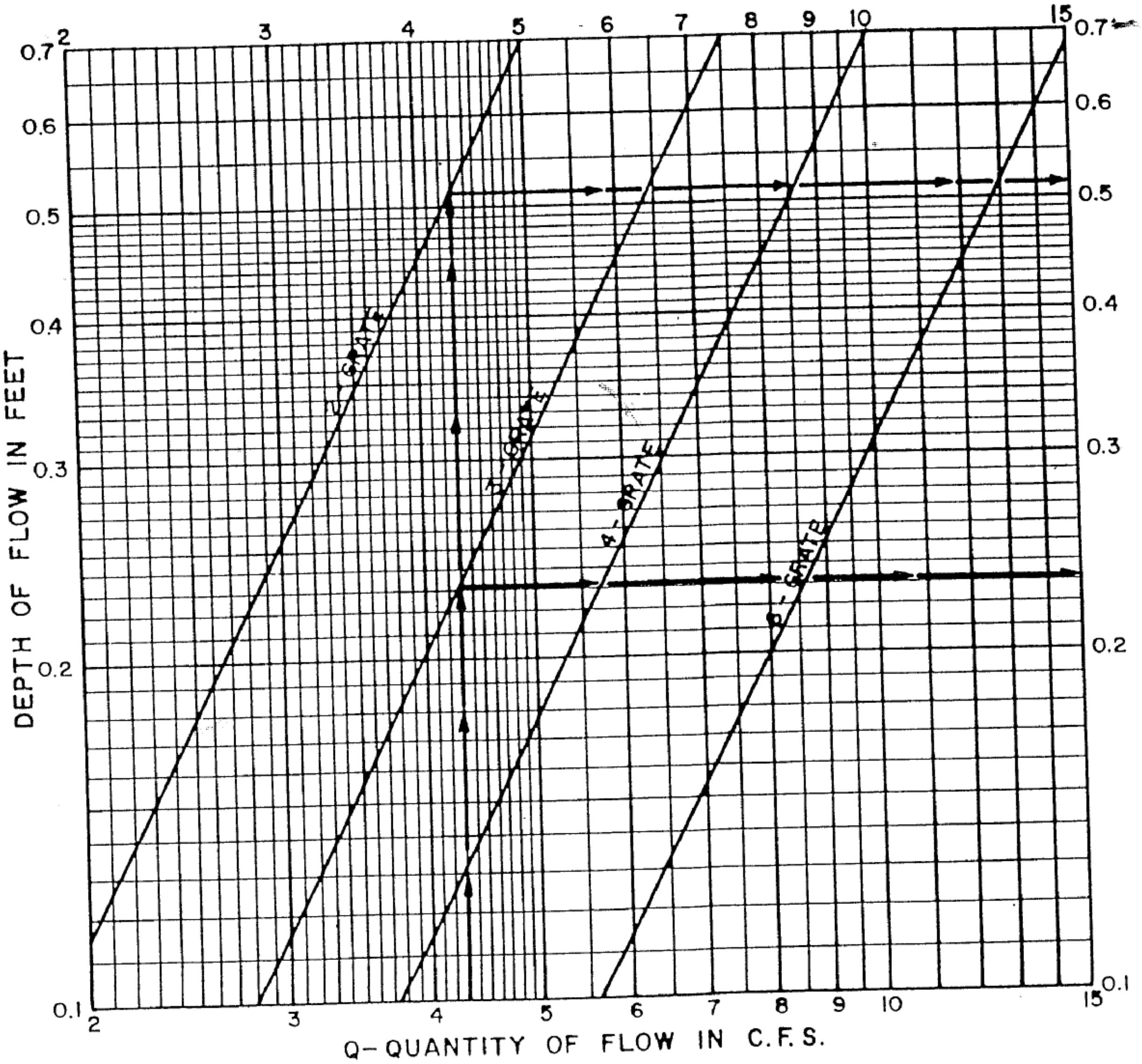
Quantity of Flow = 4.3 c.f.s.
Maximum Depth of Flow Desired at Low Point = 0.3'

Find:

Inlet Required

Solution:

Enter Graph at 4.3 c.f.s.
Intersect 3 - Grate at 0.23'
Intersect 2 - Grate at 0.51'
Use 3 - Grate



GRATE INLET
CAPACITY CURVES
AT LOW POINT

FIGURE O

EXAMPLE

Known:

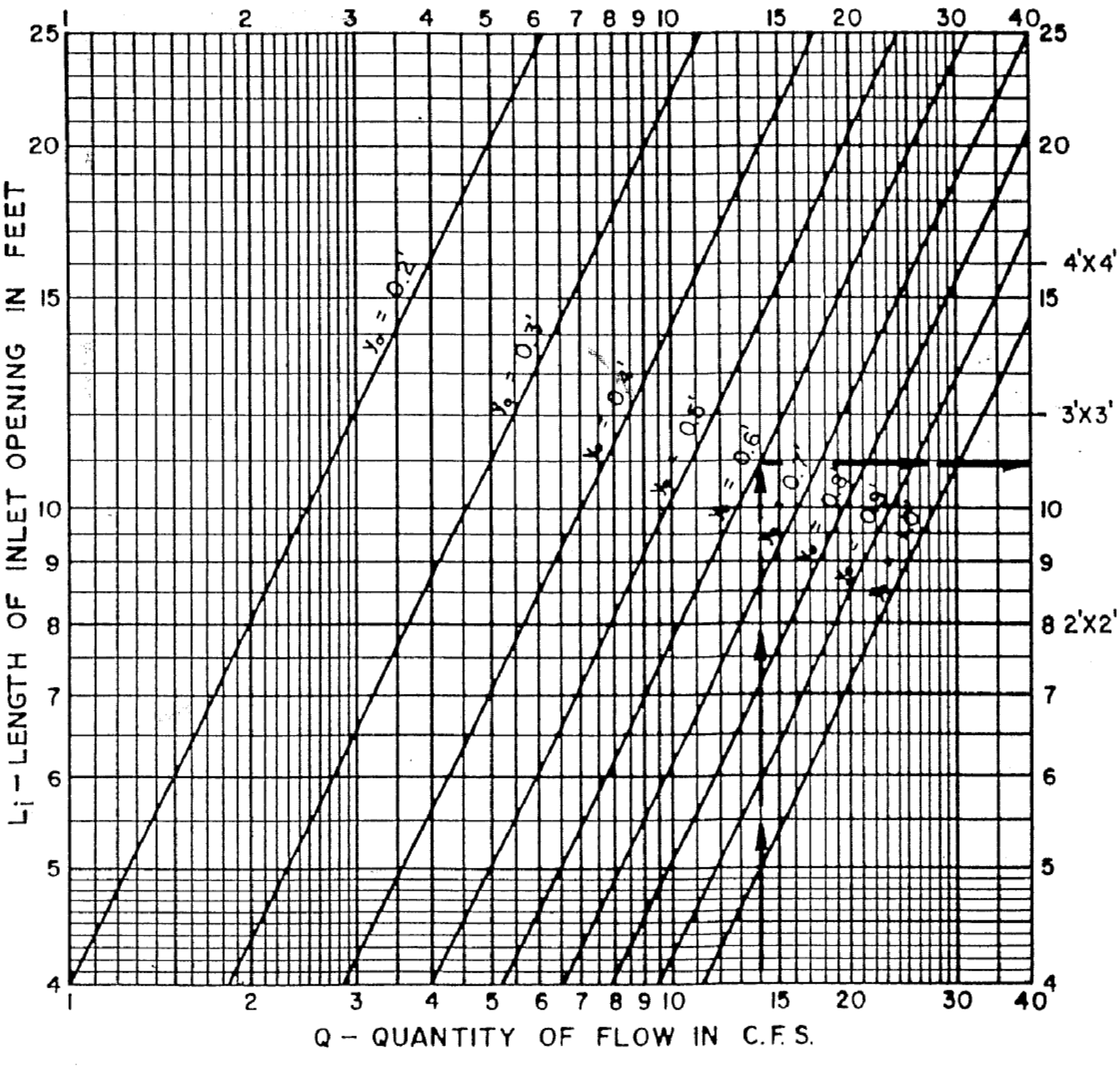
Quantity of Flow = 14.0 c.f.s.
Maximum Depth of Flow Desired
(y_o) = 0.6'

Find:

Length of Inlet Opening Required (L_i)

Solution:

Enter Graph at 14.0 c.f.s.
Intersect y_o = 0.6'
Read L_i = 10.9'
Use 12' of Inlet; 3'X 3'



Standard Drop Inlet Sizes:
2'X 2'; Li = 8'
3'X 3'; Li = 12'
4'X 4'; Li = 16'

DROP INLET
CAPACITY CURVES
AT LOW POINT

FIGURE P

APPENDIX B - REFERENCES

DRAINAGE DESIGN MANUAL

CITY OF KERRVILLE
KERR COUNTY
TEXAS

APPENDIX B – REFERENCES

The following sources were references were consulted directly or indirectly by reference in the development of this manual:

North Central Texas Council of Governments (NCTCOG), Draft integrated Storm Water Management (iSWM) Design Manual for Development/Redevelopment, 2004.

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