CITY OF SAN ANGELO

STORMWATER DESIGN MANUAL

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this design manual is to establish standard principles and practices for the design and construction of storm drainage facilities within the City of San Angelo, Texas and within its extraterritorial jurisdiction. The design factors, formulas, graphs, and procedures described in the following pages are intended to serve as guidelines for the solution of drainage problems involving the volume and rate of flow, method of collection, storage, conveyance and disposal of stormwater and erosion protection from stormwater flows. Ultimate responsibility for actual design, however, remains with the design engineer. Sound engineering judgment must always be applied. Users of this manual should be knowledgeable and experienced in the theory and application of drainage engineering. Any deviation from the requirements of this manual must be approved by the City Engineer.

1.2 References

At certain points in the text, the reader will encounter superscript numbers, for example: [1]. These numbers correspond to the references listed in Appendix A.

1.3 Acknowledgements

The City of San Angelo would like to acknowledge the significant contribution to the development of this manual by the Stormwater Committee of the San Angelo Home Builders Association. Members of this committee dedicated many hours of meetings, review of policy and criteria, and development of specific recommendations to the City that are incorporated into this document.

2.0 CITY OF SAN ANGELO DRAINAGE POLICY

2.1 Purpose of Drainage Policy

The purpose of the City of San Angelo's drainage policy is intended to protect and provide for the safety and welfare of the general public and to mitigate flood damage to private and public property within the community.

2.2 Application of Drainage Policy

The City's policies for stormwater management govern the planning, design, construction and operation of storm drainage facilities within the City's jurisdiction. These drainage policies are based on the San Angelo Stormwater Ordinance and this manual. Each is considered effective on the date of acceptance by the San Angelo City Council. This stormwater management policy will apply to any stormwater management system improvement in accordance with the Section 12.412 (Adoption) of the San Angelo Stormwater Ordinance.

2.3 Regional Stormwater Management

The City of San Angelo may choose to implement regional stormwater management to limit the impact of development on runoff within the East Angelo Draw, North Fork of the Red Arroyo, South Fork of the Red Arroyo, North Fork of the Concho River, and South Fork of the Concho River watersheds and to provide flood mitigation in these areas. This will be a coordinated effort with other governmental entities within each watershed. The design engineer is encouraged to check with the City of San Angelo prior to the design of individual detention basins in these watersheds. City Council may authorize funding for regional stormwater facilities.

2.4 Downstream Impact Assessment

The downstream impacts of proposed development must be carefully evaluated to show that design criteria established in the Stormwater Ordinance and this Stormwater Design Manual are met. The purpose of the downstream assessment is to protect downstream properties from increased flooding damage, to protect downstream channels from increased erosion potential due to upstream development, and to protect public health and safety in accordance with the requirements of the Stormwater Ordinance and this Manual. The assessment shall extend from each outfall of a proposed development to a major stream (FEMA-defined floodplain), to a storm water facility identified by the City as having fully-developed flow capacity or to a point designated by the City Engineer, whichever is the nearest point downstream.

Runoff computations shall be based upon fully developed watershed conditions in accordance with the City's latest land use projections. Portions of the watershed which lie within the city limits and ETJ of San Angelo shall be analyzed and accommodated as if fully developed. Portions of the watershed which lie outside the City's limits and ETJ shall be analyzed for existing conditions.

2.5 Limitation of Runoff

Calculations using the following runoff limitation options shall be performed for the proposed development to demonstrate downstream adequacy in accordance with the design criteria in the Stormwater Ordinance and this Stormwater Design Manual:

1. If the necessary future capacities of the affected drainage systems within the downstream impact area are shown to adequately convey the fully developed design flows from the watershed that meet the design criteria and have adequate drainage easements, no limitation of development runoff is required.

2. If the downstream analysis demonstrates conditions that exceed the design criteria established in the Stormwater Ordinance and this Stormwater Design Manual within the downstream impact area, the developer shall:

   a. Identify the required upsizing of any affected downstream structures and any needed drainage easement within the downstream impact area to handle the fully developed watershed conditions. Determine the capacity of those downstream facilities to convey:

      i. existing watershed conditions;

      ii. existing watershed conditions with the proposed development; and

      iii. fully developed watershed conditions.

      The City may, at its discretion, participate in construction of facilities that correct existing drainage inadequacies and/or convey the fully developed watershed conditions, other than for the proposed development, if adequate funds or other funding agreements are available. Funding options are discussed in Section 12.410 of the Stormwater Ordinance; or

   b. Limit discharge to existing, predevelopment conditions or less; or

   c. Acquire suitable drainage easement on the City's behalf to contain the increased runoff to meet the design criteria in the Ordinance and this Stormwater Design Manual; or

   d. Participate in a regional facility that accommodates fully developed watershed conditions, as mutually agreed upon with the City. Funding alternatives are identified in Section 12.410 of the Ordinance.

2.6 Drainage Improvements Required for Development

1. All developments shall provide for any new drainage facilities, the improvement of any existing drainage facilities, channel improvements or grading, driveway adjustments, culvert improvements or any other improvement, drainage facility, or work which is necessary to provide for the stormwater drainage requirements specified in this Stormwater Design Manual.

2. The developer is required to dedicate drainage easements across the site that will accommodate fully developed watershed conditions. The developer is only required to construct that portion of the drainage system across the site that will

convey existing offsite flows and the flows from the proposed site development. Upsizing or phasing the construction of the drainage systems to accommodate fully developed watershed conditions shall be coordinated with the City. Funding for the increased sizing of these facilities shall be in accordance with Section 12.410 (Funding of Improvements) of the Ordinance.

3. The developer shall construct drainage improvements that convey stormwater runoff through the proposed development to the site outfall locations in accordance with the design criteria in the Ordinance and this Stormwater Design Manual.

2.7 Drainage Plan Submittals

A review process has been established by the City of San Angelo for development activities related to stormwater runoff through natural or manmade facilities. A Preliminary Drainage Study containing the conceptual layout of the proposed storm drainage system must be submitted as part of the preliminary platting process in accordance with the Ordinance and this Manual.

A Final Drainage Study shall be submitted as part of the final platting process. The Final Drainage Study shall include the appropriate computation sheets as required in the Stormwater Design Manual. Final Drainage Studies of all proposed improvements or land developments shall clearly identify the on-site and off-site drainage system improvements required to satisfy the San Angelo Stormwater Ordinance, and demonstrate the upstream and downstream limits to which the improvements or development comply with the design criteria in the Stormwater Ordinance and this Manual, if applicable.

2.8 Drainage System Requirements

2.8.A. Major Drainage Systems

The design storm, as defined by the 100-year storm event, must be contained within the right-of-way or dedicated easement of all major drainage systems, as defined in this Stormwater Manual, to provide for public safety and welfare.

2.8.B. Minor Drainage Systems

To enhance the quality of life and provide for public safety, minor drainage systems, as defined in this Stormwater Manual, are required to provide conveyance for design frequencies specified in Section 5 of this Manual.

2.9 Floodplain Development

Development within and improvements to the flood hazard area shall be consistent with the criteria set forth in Section 13 of this manual as appropriate.

2.10 Drainage Design Computations

Computations to support all drainage designs shall be submitted to the City Engineer for review as part of the Final Drainage Study and shall be summarized in the form of the standard computation sheets contained in this manual, unless approved otherwise by the City Engineer. Computer programs (other than spreadsheets) used to perform computations shall be limited to those referenced in this manual unless approved by the City Engineer.

All computations submitted shall be sealed by a professional engineer experienced in municipal stormwater drainage design and licensed in the State of Texas in accordance with the requirements set forth by the Texas Board of Professional Engineers and in accordance with the San Angelo Stormwater Ordinance. Stormwater runoff computations shall be based upon conditions representing fully developed watershed conditions in accordance with the San Angelo Stormwater Ordinance and this Manual.

2.11 Construction of Drainage Facilities

Development activities associated with the construction of drainage facilities must minimize erosion caused by the construction, in accordance with the San Angelo Stormwater Ordinance. The protection of existing vegetation should be emphasized during development of drainage plans.

2.12 Maintenance of Drainage Facilities

The hydraulic integrity of floodplain and drainage easements dedicated to and accepted by the City of San Angelo will be maintained by the City of San Angelo. The hydraulic integrity of drainage systems not dedicated to the City will require a maintenance agreement jointly agreed upon between the City and the property owner before being accepted by the City.

2.13 Drainage Plan Requirements

As part of the platting process, drainage studies shall be prepared for preliminary and final plats in accordance with Section 12.412 (Adoption) of the Stormwater Ordinance. These studies shall include drainage facilities for both off-site and on-site drainage, so that the proper transition between the two can be maintained. Criteria for on-site development shall also apply to off-site improvements. The construction of all improvements shall be in accordance with the current specifications and regulations adopted by the City of San Angelo. As applicable to the project, the items in the following lists shall be submitted for the Preliminary Drainage Study and the Final Drainage Study, as designated below. All references to hydraulic data refer to the 100-year storm event or the appropriate design discharge. All plans shall be submitted on 24" by 36" plan sheets, and shall include:

2.13.A. Drainage Area Map

Prelim	Final		Required Information
X	X	1.	Use a scale of one inch equals 200 feet maximum for the

			development and a scale of up to one inch equals 2,000 feet for creeks and off-site areas, provided that the scale is adequate for review, and show match lines between any two or more maps.
X	X	2.	Show existing and proposed storm systems.
X	X	3.	Indicate sub-areas at street intersections, inlets, off-site areas, major stream crossings, etc.
X	X	4.	Indicate existing contours on a map for on- and off-site drainage areas.
X	X	5.	Indicate zoning on drainage areas.
X	X	6.	Indicate post development contours or flow patterns including all crests, sags, and street and alley intersections with flow arrows.
X	X	7.	Provide runoff calculations for all areas showing acreage, runoff coefficient, inlet time and storm frequency.
X		8.	Show datum to establish location and elevation.
	X	9.	Show location of survey benchmarks.
X	X	10.	Show FEMA FIRM special flood hazard areas.
	X	11.	Identify which alternative criteria is proposed to satisfy the 100-year requirements for streets and alleys.

2.13.B. Storm Drain Pipe Plan and Profile Sheets

Prelim	Final	Required Information	
X	X	1.	Show plan and profile of all drainage elements including pipes, ditches, and channels (for preliminary study only the plan view is required, including showing pipe sizes and lengths).
	X	2.	Specify the type of storm drain pipe to be used.
X	X	3.	Indicate property lines along storm sewer and show easements with dimensions.
X	X	4.	Show all existing utilities in plan view, and show existing utilities in profile where possible conflicts may occur with the storm sewer.(Profile view not required for preliminary)
	X	5.	Indicate existing and proposed ground line and improvements on all street, alley and storm sewer system.
	X	6.	Show hydraulic grade lines with computations.
	X	7.	Show laterals on trunk profile with stations.

Prelim	Final	Required Information
	X	8. Number inlets according to the number designation given for the area or sub-area contributing runoff to the inlet.
	X	9. Indicate size and type of inlet on plan view, lateral size and flow line, paving station and top-of-curb elevation.
	X	10. Indicate quantity and direction of flows at all inlets, stub-outs, pipes and intakes.
	X	11. Show future streets and grades, where applicable.
X	X	12. Show water surface and exit velocity at outfall of storm drain. Include energy dissipation device, if needed.
	X	13. Where fill is proposed or trench cut in creeks or outfall ditches, specify compacted fill and compaction criteria.
	X	14. Show diameter of pipes, physical grade, design discharge, slope of hydraulic grade line and velocity in the pipe in the profile view.
	X	15. Show elevation of flow lines at 100-foot intervals on the profile.
X	X	16. Give benchmark information.
	X	17. Show design level elevations, flows, and velocities of the existing system into which the proposed system is being connected.
	X	18. Show details of all connection boxes, headwalls on storm sewer, flumes or any other item not in a standard detail sheet.
	X	19. Show headwalls and specify type for all storm sewers at outfall.
	X	20. Show horizontal and vertical curve data for all drainage elements.
	X	21. On all dead-end streets and alleys, show grades for drainage overflow path on the plan and profile sheets, and show erosion controls.

2.13.C. Erosion Control Plan

Prelim	Final	Required Information
	X	1. The names, addresses, and phone numbers of the owner, developer and engineer
	X	2. Site acreage noted and disturbed area noted (acres)

Prelim	Final	Required Information
	X	3. Vicinity map and N. arrow and graphic scale
	X	4. Title block, revision block, general notes, and symbol legend
	X	5. Site boundaries
	X	6. Existing and proposed ground contours
	X	7. Existing and proposed structures and pavement
	X	8. Existing and proposed utilities
	X	9. Existing and proposed drainage features
	X	10. FEMA 100-year storm event flood plain and elevations
	X	11. Benchmark
	X	12. Limits of trees/shrubs to remain and of undisturbed area
	X	13. Construction limits shown
	X	14. Borrow and spoil areas identified
	X	15. Best management practices (BMP) locations and calculations
	X	16. Best management practices (BMP) details

2.13.D. Bridge Plans

Prelim	Final	Required Information
X	X	1. Show the elevation of the lowest member of the bridge and the design level water surface elevation.
	X	2. Indicate borings on plans.
	X	3. Provide soils report.
X	X	4. Show a section at the bridge.
X	X	5. Provide hydraulic calculations on all sections.
	X	6. Provide vertical and horizontal alignment.

2.13.E. Creek Alteration and Channel Plans

Prelim	Final	Required Information
X	X	1. Show stationing in plan and profile (Profile not required for preliminary).
X	X	2. Indicate flow line and banks (design water surface elevation and velocity not required for preliminary).
X	X	3. Indicate the nature of banks, such as rock, earth, etc.

Prelim	Final	Required Information
	X	4. Provide cross sections with ties to property lines and easements.
	X	5. Show side slopes of creeks, channels, etc.
	X	6. Indicate any adjacent alley or street elevations on creek profile.
	X	7. Indicate existing and proposed velocities.
X	X	8. Show access and/or maintenance easements.
X	X	9. Identify the datum and benchmarks to which the flood and ground elevations are referenced.
	X	10. Show existing Finished Floor (F.F.) for existing structures, or proposed minimum F.F. of all structures, existing or proposed adjacent to creek or channel alternations.

2.13.F. Environmental Effects and Required Regulatory Permits Report

Prelim	Final	Required Information
X	X	1. The preliminary submittal of plans is to identify all permits that, in the design engineer's opinion, will or may be required by regulatory agencies. Such permits and agencies include, but are not limited to, NPDES (addressed in this manual in chapter 12), Section 404 permit from the U.S. Army Corps of Engineers, the Environmental Protection Agency (EPA), and Texas Commission on Environmental Quality (TCEQ).
	X	2. The final submittal of plans is to provide a list of all required permits necessary to construct the project and a copy of the submitted permit applications or notifications.

2.13.G. Detention and Retention Facilities

Prelim	Final	Required Information
X	X	1. Show plan view of detention/retention area and outlet structure.
	X	2. Delineate limits of conservation pool, sediment storage area, flood storage pool and/or freeboard.
	X	3. Indicate size, dimensions, total capacity and design discharge velocity of the outlet structure.
	X	4. Show erosion control features at the discharge point of the outlet structure.

Prelim	Final	Required Information
	X	5. Show existing or proposed structures or other facilities downstream of the outlet structure and emergency spillway, and provide information sufficient to show that the adjacent facilities will not be subjected to flooding (or increased flooding) or otherwise affected by the discharge from the basin.
X	X	6. Indicate locations and quantities of design inflows to the basin.
	X	7. State the design time to empty the basin.

2.13.H. Levees

Prelim	Final	Required Information
X	X	1. Show location, extent, nature, dimensions, etc. of levee embankments and associated interior and exterior drainage facilities.
	X	2. Show proposed erosion control of the interior and exterior faces of levee embankments.
	X	3. Provide engineering analysis of levee embankment stability
	X	4. Compaction of fill material should be performed in accordance with standard engineering practices.
X	X	5. Analyze interior drainage concerns. Identify sources of interior flooding and extent and depth of such flooding. Consider available storage, capacity of pumps and other drainage devices for evacuating interior waters.
	X	6. Write an operations manual which discusses the flood warning system to trigger closures if applicable; closure operations, procedures and personnel; operation plans for interior drainage facilities; at least an annual inspection program; and maintenance plans, procedures and frequency.
	X	7. Provide all other information required by the City of San Angelo Ordinances, and any other information requested or required by the City Engineer and/or FEMA.

3.0 CITY OF SAN ANGELO STORMWATER ORDINANCE

The City of San Angelo's Code of Ordinances contains requirements and guidelines for the design of stormwater management facilities. Where there is any conflict between this Manual and the current Stormwater Ordinance, the current Stormwater Ordinance shall take precedence.

4.0 DESIGN RAINFALL

4.1 Rainfall Intensity-Duration-Frequency

The design of all storm drainage facilities within the City of San Angelo shall be based on rainfall information from either Figure 4-1 or Tables 4-1 through 4-3, and standard technical information provided by USGS Water Resources Investigations Report 98-4044[2]. Rainfall information from these sources is provided in this manual for the convenience of the Engineer. If there are any discrepancies between the data in this manual and these references, the data from the referenced publications should be used.

Point rainfall intensity-duration-frequency curves for the San Angelo area are presented in Figure 4-1. The curves presented have been determined for rainfall durations ranging from five minutes to 24 hours and average return periods of 2-, 5-, 10-, 25-, 50-, 100- and 500-years. Point rainfall intensities can also be calculated using the equations listed in Table 4-1. For drainage areas greater than ten square miles, the depth-area relationship from Technical Paper #40, published by the U.S. Department of Commerce,[3] may be used to reduce rainfall amounts.

4.2 Probable Maximum Precipitation (PMP)

The design rainfall for NRCS dams or impoundments is based on a percentage of the Probable Maximum Precipitation (PMP), as specified in Section 5.2 of this manual. PMP rainfall depths for various durations and storm sizes can be obtained from Hydro-Meteorological Reports Nos. 51 and 52, respectively.[4,5] The computer program HMR52, written by the U.S. Army Corps of Engineers, may be used to distribute the PMP over the watershed, calculating the basin average precipitation for each basin. The rainfall temporal distribution and the design hydrograph shall be determined according to TCEQ's "Hydrologic and Hydraulic Guidelines for Dams in Texas."

Table 4-1. Point Rainfall Intensity Constants

Intensity for 5 min. to 7 Days I = b/(Td+d)^e			
Year	b	d	e
2	53.5	10.32	0.865
5	68.5	11.47	0.848
10	75.5	11.47	0.836
25	77.2	10.00	0.805
50	104.0	13.52	0.826
100	112.5	14.70	0.816
500	195.0	24.00	0.853

NOTE: I is rainfall intensity in inches per hour and Td is the duration of rainfall in minutes.

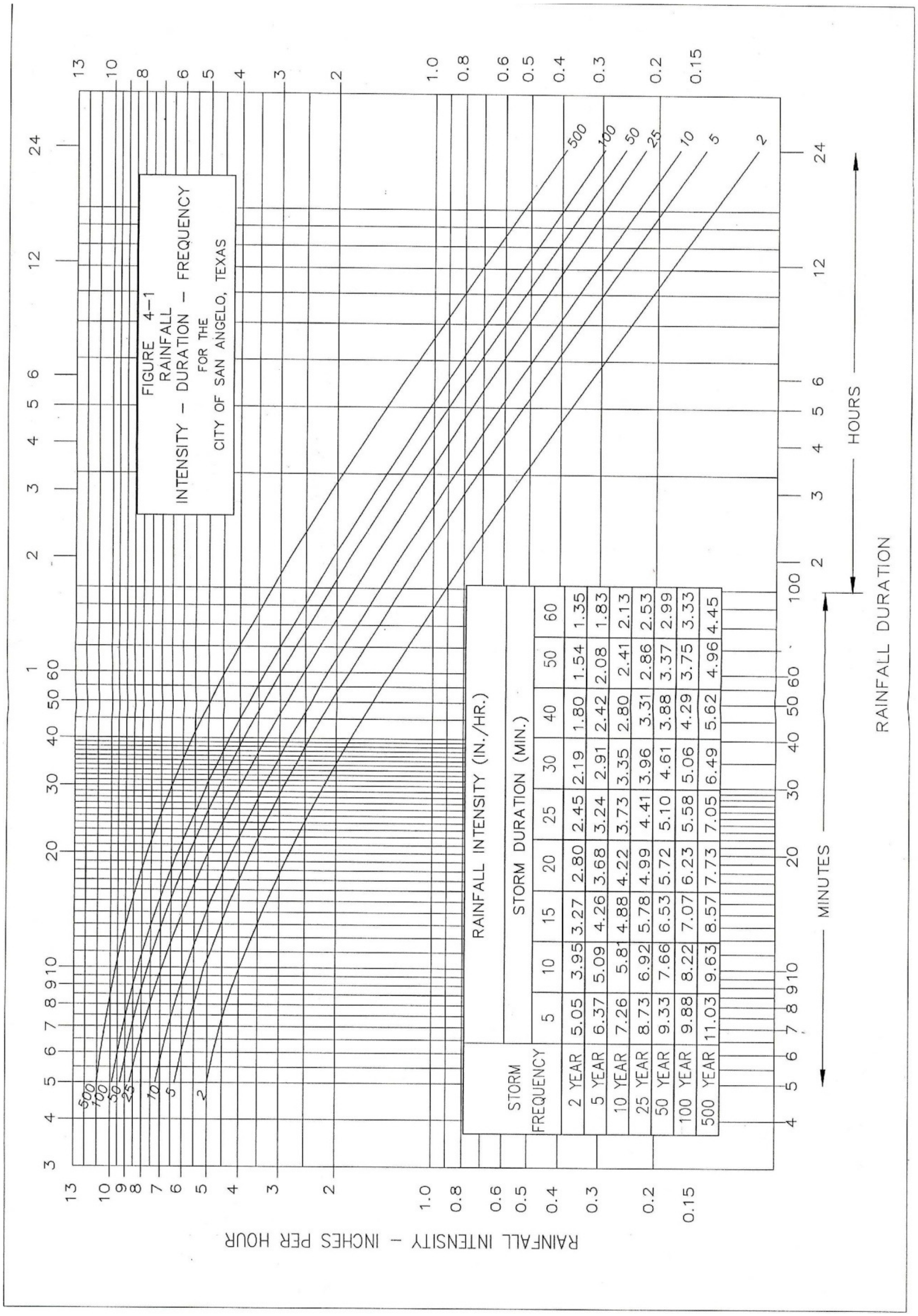
Table 4-2. Point Rainfall Depths

Depth	USGS 98-4044													
	5 (min)	10 (min)	15 (min)	30 (min)	60 (min)	2 (hour)	3 (hour)	6 (hour)	12 (hour)	24 (hour)	2 (days)	3 (days)	5 (days)	7 (days)
Year	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)
2	0.42	0.66	0.82	1.09	1.35	1.58	1.71	1.93	2.14	2.37	2.61	2.75	2.95	3.09
5	0.53	0.85	1.06	1.45	1.83	2.19	2.39	2.72	3.06	3.43	3.82	4.07	4.40	4.63
10	0.60	0.97	1.22	1.68	2.13	2.56	2.80	3.22	3.65	4.12	4.63	4.96	5.39	5.70
25	0.73	1.15	1.45	1.98	2.53	3.07	3.39	3.97	4.59	5.28	6.07	6.57	7.26	7.76
50	0.78	1.28	1.63	2.30	2.99	3.65	4.03	4.68	5.36	6.10	6.90	7.42	8.12	8.61
100	0.82	1.37	1.77	2.53	3.33	4.12	4.57	5.36	6.19	7.09	8.09	8.72	9.59	10.21
500	0.92	1.61	2.14	3.25	4.45	5.62	6.27	7.31	8.31	9.33	10.41	11.07	11.96	12.58

Table 4-3. Point Rainfall Intensities

Intensity	USGS 98-4044													
	5 (min)	10 (min)	15 (min)	30 (min)	60 (min)	2 (hour)	3 (hour)	6 (hour)	12 (hour)	24 (hour)	2 (days)	3 (days)	5 (days)	7 (days)
Year	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr	in/hr
2	5.05	3.95	3.27	2.19	1.35	0.79	0.57	0.32	0.18	0.10	0.05	0.04	0.02	0.02
5	6.37	5.09	4.26	2.91	1.83	1.09	0.80	0.45	0.26	0.14	0.08	0.06	0.04	0.03
10	7.26	5.81	4.88	3.35	2.13	1.28	0.93	0.54	0.30	0.17	0.10	0.07	0.04	0.03
25	8.73	6.92	5.78	3.96	2.53	1.53	1.13	0.66	0.38	0.22	0.13	0.09	0.06	0.05
50	9.33	7.66	6.53	4.61	2.99	1.83	1.34	0.78	0.45	0.25	0.14	0.10	0.07	0.05
100	9.88	8.22	7.07	5.06	3.33	2.06	1.52	0.89	0.52	0.30	0.17	0.12	0.08	0.06
500	11.03	9.63	8.57	6.49	4.45	2.81	2.09	1.22	0.69	0.39	0.22	0.15	0.10	0.07

Figure 4-1. Rainfall – Intensity – Duration – Frequency Curves



STORM FREQUENCY	5	10	15	20	25	30	40	50	60
	STORM DURATION (MIN.)								
2 YEAR	5.05	3.95	3.27	2.80	2.45	2.19	1.80	1.54	1.35
5 YEAR	6.37	5.09	4.26	3.68	3.24	2.91	2.42	2.08	1.83
10 YEAR	7.26	5.81	4.88	4.22	3.73	3.35	2.80	2.41	2.13
25 YEAR	8.73	6.92	5.78	4.99	4.41	3.96	3.31	2.86	2.53
50 YEAR	9.33	7.66	6.53	5.72	5.10	4.61	3.88	3.37	2.99
100 YEAR	9.88	8.22	7.07	6.23	5.58	5.06	4.29	3.75	3.33
500 YEAR	11.03	9.63	8.57	7.73	7.05	6.49	5.62	4.96	4.45

4.3 Standard Project Precipitation (SPP)

The design rainfall for projects which require the Corps of Engineers' Standard Project Flood (SPF) shall be obtained by applying 50 percent of the PMF or the runoff developed from the PMF, as described in Section 5.2.

4.4 Rainfall Loss Rates

Losses due to interception, infiltration and depression storage need to be deducted from the total design rainfall to obtain the effective design rainfall. The method used to calculate the rainfall losses will depend on the method used to compute the design discharge, as described in Section 5.3 of this manual. The Rational Method accounts for rainfall losses with the C coefficient, as described in Section 5.4. For the unit hydrograph methods described in Section 5.5, the initial-uniform constant loss method is recommended for the Snyder's method and the NRCS Runoff Curve Number method should be used with the NRCS Dimensionless Unit Hydrograph method. Recommended loss rates for the Snyder's method are found in Section 5.5.A.2.

5.0 DETERMINATION OF DESIGN DISCHARGE

5.1 Design Frequencies

The storm frequencies for fully developed watershed conditions shown in the following tables shall be used in storm drainage designs in the City of San Angelo. All drainage features must be designed to meet the following requirements.

Table 5-1. Design Criteria

Facility	Minor Storm (2-Year)	Major Storm (100-Year)
Streets and Alleys (Chapter 6)	Maximum water depth of 6” above gutter flowline or edge of pavement, within ROW	Maximum water depth of 8” above gutter flowline or edge of pavement, contained within ROW and easement. Alternative A: Building FFE must be 18” above highest adjacent gutterline to the property. Maximum water depth of 12” above gutter flowline or edge of pavement. Alternative B: Sealed engineering plan showing adequate conveyance of drainage in ROW and/or easements. Minimum of 6” of FFE freeboard for structures. Maximum water depth of 12” above gutter flowline or edge of pavement.
Enclosed Pipe System (Chapter 8)	Conveyance in the pipe with maximum HGL of 6” above gutter flowline	Maximum overflow of 12” above gutter flowline.
Channels and Creek Improvements (Chapter 9)	Maintain non-erosive conditions	Water surface elevation within easement with one foot (1’) freeboard
Culverts (Chapter 10)	U.S. headwater no more than top of pavement or curb at the low point of the crossing	Water surface elevation within easement and no more than twelve inches (12”) above the gutter flowline at the low point
Bridges (Chapter 10)	U.S. headwater at least one foot (1’) below low chord elevation	U.S. headwater below low chord elevation
Detention Pond and Sump Areas (Chapter 11)	Low flow outlet structure	Water surface elevation within easement with one foot (1’) freeboard

Table 5-1 continued on next page

Table 5-1. Design Criteria (continued)

Facility	Minor Storm (2-Year)	Major Storm (100-Year)
Levees		
Protecting 1-5 homes	Maintain non-erosive conditions	Water surface elevation within easement with one foot (1') freeboard
Protecting 5-20 homes	Maintain non-erosive conditions	Water surface elevation within easement with three feet (3') freeboard
Protecting >20 homes	Maintain non-erosive conditions	Standard Project Flood within easement

Velocity limitations are listed in Table 5-2 below.

Table 5-2. Velocity Limitations

Facility	Maximum Velocity of Design Discharge
Storm Drains	
Inlet laterals (shorter than 30 feet)	No Limit
Inlet laterals (longer than 30 feet)	20 fps
Trunk lines	20 fps
Last 20 diameter of length upstream of an outfall	6 fps or 120% of Receiving Stream*, whichever is greater
Open Channels	
Grass Lined Earthen	6.0 fps
Rock (Native)	10.0 fps
Gabion Lined	12.0 fps
Reinforced Concrete Lining	20.0 fps
Rock Riprap (Placed Rock)	Use US Army Corps of Engineer Guidelines
Prefabricated Lining Products	Use 90% of Manufacturer's Recommended Velocity Limits
Surface Drainage	
Streets and Alleys	12 fps at a 6" depth within the street

5.2 Design Frequencies for Dams or Impoundments

Lakes and dams will be designed utilizing Spillway Design Flood listed in subsection C of the criteria listed below, for fully developed watershed conditions. The design criteria for a dam is dependent on the size and hazard classification of the dam, as described by the Texas Commission on Environmental Quality (TCEQ) under 30 TAC 299.11 - 299.14,[6] which provides for the safe construction, maintenance, repair and removal of dams located in the State of Texas. The following criteria from current TCEQ regulations will be used to classify a dam. If the TCEQ criteria are updated, then the revised TCEQ regulations will apply.

5.2.A. Size

The classification for size is based on the height of the dam and storage capacity, whichever gives the larger size category, as shown in Table 5-2. "Height" is defined as the distance between the top of the dam and the existing streambed at the downstream toe. Storage is defined as the maximum water volume impounded at the top of the dam.

Table 5-2. Size Classification Impoundment

Category	Storage (acre-feet)	Height (feet)
Small	<1,000	<40
Intermediate	≥1,000 and <50,000	≥40 and <100
Large	≥50,000	≥100

5.2.B. Hazard Potential.

The hazard potential for a dam is based on the potential for loss of human life and property damage downstream from a dam in the event of failure. Table 5-3 shows the categories to be used:

Table 5-3. Hazard Potential Classification

Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
Low	None expected (no permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (no urban developments and no more than a small number of inhabitable structures)	Appreciable (notable agricultural, industry or structures)
High	More than few	Excessive (extensive community, industry or agriculture)

5.2.C. Spillway Design Flood.

The classification of a dam based on the above criteria will be used to determine the Spillway Design Flood (SDF). The total capacity of a dam structure, including principal and emergency spillways, shall be adequate to pass the SDF without a failure of the dam. The SDF is computed as a percentage of the PMF hydrograph for various dam classifications according to Table 5-4:

Table 5-4. Spillway Design Flood

Hazard	Size	SDF (Flood Hydrograph)
Low	Small	1/4 PMF
	Intermediate	1/4 PMF to 1/2 PMF
	Large	PMF
Significant	Small	1/4 PMF to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High	Small	PMF
	Intermediate	PMF
	Large	PMF

Where a range is given, the minimum flood hydrograph will be determined by straight-line interpolation within the given range. Interpolation shall be based on either hydraulic height or impoundment size, whichever is greater. In all cases, the minimum spillway design capacity is the 100-year storm event. In certain cases, a dam breach analysis may be required to determine the proper hazard classification of the structure. For all structures requiring a spillway design flood equal to the PMF, an emergency action plan, prepared according to TCEQ guidelines, is required prior to completion of construction.

5.2.D. Additional Design Requirements

1. An engineering plan for such construction, accompanied by complete drainage design information and sealed by a licensed professional engineer, shall be submitted to the City of San Angelo for review and approval.
2. The spillway and any emergency overflow areas shall be located so that floodwaters will not inundate any permanent habitable structures.
3. The minimum SPF should be the 100-year storm event, with a 24-hour storm duration regardless of critical inflow design storm peaks.
4. The design shall comply with all federal, state and county laws pertaining to the impoundment of surface water, including the design, construction and safety of the impounding structure. Copies of any federal, state or county permits issued for the proposed impoundments shall be submitted to the City Engineer.
5. Any existing dams, including NRCS dams, which are included as part of the development area shall comply with the applicable federal, state, county and city safety design requirements for structures. Dams not in compliance with applicable design requirements cannot be included as a stormwater management feature for the development area unless and until the structure has been upgraded to the currently adopted requirements. For existing dams in the drainage area, the drainage study shall include current capacity of the dam(s) and a comparison with applicable regulations.
6. Before removing, enlarging or altering any existing lake, the applicant will furnish a study of the effects of the alteration upon flooding conditions both upstream and downstream. The study shall be prepared by a professional engineer and submitted to the City Engineer for approval prior to making the proposed alteration.
7. Any improvements to existing dams or lakes or construction of new impoundments shall be made at the expense of the developer, prior to completion of the adjacent street, utilities and drainage improvements, as provided for under the subdivision regulations.

5.3 City Engineer Computation Methods

Prior to the design of drainage facilities, the design rainfall must be converted to a flow rate for the location in question. Two methods -- the Rational Method and the Unit Hydrograph Method -- are acceptable for use in design of storm drainage facilities in the

City of San Angelo. In either case, the determination of the design discharge is to be in accordance with this Manual.

5.4 Rational Method

The Rational Method, based on the direct relationship between rainfall and runoff, is applicable to small watersheds and shall be used to determine runoff for watershed with drainage areas of 200 acres or less. The discharge computed by the Rational Method is the peak discharge for a given frequency on the watershed in question, and is given by the following relationship:

$$Q = C * I * A \qquad \text{(Eq. 5-1)}$$

where:

- Q is the peak design discharge in cubic feet per second for a given frequency on the watershed at the desired design point.
- C is a dimensionless weighted runoff coefficient, representing ground cover conditions and/or zoned land use within the watershed area. (See Table 5-5.)
- I is the average rainfall intensity in inches per hour for a given rainfall duration which is generally equal to the time of concentration, associated with the desired design frequency. (See Figure 4-1 or Table 4-1)
- A is the drainage area in acres contributing runoff to the desired design point.

5.4.A. Runoff Coefficient

The American Society of Civil Engineers (ASCE) has compiled average runoff coefficients used in the Rational Method for various surface conditions.[7] The runoff coefficients shown in Table 5-5 are adapted from the ASCE values used in conjunction with the classifications outlined in the latest comprehensive Land Use Plan for the City of San Angelo. The coefficients are to be used in design to represent fully developed conditions according to current zoning. Adjustments are not needed for impervious areas. For un-zoned area, a weighted average of the pervious and impervious areas should be used with values from the Table.

Table 5-5. Values Of Runoff Coefficient "C"

Zone Type	Description	Value of C
Unzoned:		
None	Parks & Open Areas, Golf Courses	0.30
None	Pavement / Roofs / other impervious areas	0.95
Residential:		
R&E	Ranch and Estate District (>= 1 ac. lots)	0.40
R-1	Single Family Residence District (< 1 ac. lots)	0.50
R-2	Two Family Residence District	0.55
R-2A	Townhouse and Zero Lot Line Residential District	0.70
R-3	Multi-Family Residence District	0.75
MHP, MHS	Manufactured Home Park/Subdivision	0.75
Commercial:		
C-1	Neighborhood Retail District	0.75
C-2	Business District	0.85
C-3	Commercial District	0.85
C-N	Neighborhood Commercial	0.80
C-O	Commercial Office	0.85
C-G	General Commercial	0.85
C-H	Heavy Commercial	0.95
C-E&R	Commercial Entertainment and Recreation (Not open grassed areas)	0.90
C-BD	Commercial Business District	0.90
M-1, ML	Light Manufacturing District	0.90
M-2, M-3	Heavy Manufacturing District	0.95
PD	Planned Development	0.60

The drainage area under investigation may consist of several different drainage surfaces or zoning classifications. In such cases, an average coefficient weighted in accordance with the respective areas should be used, as outlined in Equation 5-2. The weighted runoff coefficient is as follows:

$C_w = (A_1 * C_1 + A_2 * C_2 + A_3 * C_3 + ... + A_n * C_n) / (A_1 + A_2 + A_3 + ... + A_n)$ (Eq. 5-2)

where:
- A Drainage Area (acre)
- C Runoff Coefficient
- C_w Weighted Runoff Coefficient

5.4.B. Time of Concentration

The time of concentration (T_c) is the amount of time required for surface runoff to travel from the most hydraulically remote point within the drainage basin to the drainage point under consideration. The most hydraulically remote drainage point refers to the route requiring the longest drainage travel time and not necessarily the greatest linear distance. The flow routes used in determining the time of concentration must take into consideration fully developed conditions as proposed by thoroughfare plans, zoning maps, the comprehensive plan, etc.

It should be noted that when determining the time of concentration, the flow route may consist of several segments. The total time of concentration is determined as follows:

$T_c = \text{Initial } T_c + L_1 / (V_1 * 60) + L_2/(V_2*60) + \ldots + L_n/(V_n *60)$ (Eq. 5-6)

Where:

T_c	=	Time of concentration (min.)
Initial T_c	=	the initial time of concentration from Table 5-6 (min.)
L_1, L_2, & L_n	=	the travel distance of Segment n (ft.)
V_1, V_2, & V_n	=	the average velocity of flow across Segment n or through pipe segment n (ft./sec.)
n	=	the number of segments in the flow route

Segments may include sheet flow, overland flow, and open channel flow. Overland and open channel flow segments can be subdivided as needed.

5.4.B.1. *Initial Tc Sheet Flow*

When computing the initial T_c the maximum length of sheet flow shall be 200 feet, at which point flow shall be considered overland flow. Table 5-6 contains the initial T_c values, which also represent the minimum T_c at all inlets (i.e., prior to entering a street, pipe system, roadside ditch, or other channel).

Table 5-6. Initial Times Of Concentration

Type of Area	Initial Inlet Time (Minutes)
Parks & Open Areas	20
Residential	
Single Family	15
Multi-Family	10
Commercial/Business	10
Roofs & Paved Areas	5

5.4.B.2. *Overland Flow Velocity for T_c*

The flow length beyond the sheet flow will be considered overland flow unless the sheet flow opens directly into an open channel or pipe system. Velocities for overland flow can be calculated by using the coefficient C from Table 5-7 in Equation 5-3. For flow lengths which are not concentrated as channelized or as street flow, velocities will be calculated using constants, from Table 5-3, for either paved or unpaved depending on the surface over which it flows. Non-concentrated flow lengths shall not exceed 1,000 feet. In areas where the flow is concentrated as street flow, the engineer is to estimate the depth of flow and use the appropriate street flow constants from Table 5-7.[8,9] :

$$V = C * S^{0.5} \qquad \text{(Eq. 5-3)}$$

where: V = is the average velocity of overland flow (ft/sec).

C = the overland flow coefficient from Table 5-3

S = is the slope of the land over which the runoff will flow (ft/ft).

Table 5-7. Overland Flow And Street Estimate Velocity Coefficients

Type of Ground Cover or Flow Condition	C
Unpaved areas, (pasture, lawn, park, etc.)	16
Paved areas, (parking lots, concrete, asphalt, compacted areas, etc.)	20
Street flow up to the top of curb	28
Street flow up to 6" above the top of curb	40
Street flow at 12" or more above the top of curb	70

5.4.B.3. *Open Channel Flow Velocity for T_c*

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets or where proposed channels including roadside ditches, are located. Manning's equation, Equation 5-4, or water surface profile information can be used

to estimate average flow velocity. Average flow velocity is usually determined for bank full elevation.[10]
Manning=s n values for channels are found in Table 9-2 of Section 9.

$V = [1.49 * R^{(2/3)} * S^{(1/2)}] / n$ (Eqn. 5-4)
$R = A / p_w$ (Eqn. 5-5)

where:
- V is average velocity (ft/sec)
- R is hydraulic radius (ft)
- S is slope of the hydraulic grade line (channel slope, ft/ft)
- n is Manning's roughness coefficient for open channel flow
- A is cross sectional flow area (ft^2)
- p_w is the wetted perimeter (ft)

5.4.C. Computation Sheet for the Rational Method

Data generated using the Rational Method shall be summarized using a spreadsheet similar to that shown in Computation Sheet 5-1. This sheet shall be included in all Preliminary (including conceptual sketch of system layout) and Final Drainage Study Submittals. Notes regarding the use of Sheet 5-1 are as follows:

Column 1 Inlet number of structure receiving inflow, which is the same as its primary contributing drainage area sub-basin.

Column 2 The street station of the structure receiving the inflow.

Column 3 The frequency design year for which inflow is being calculated.

Column 4 The designation of the drainage area or areas contributing runoff, which is the same as the inlet which receives a majority of its flow.

Column 5 The drainage area, in acres, of the contributing drainage area or areas identified in Col. 4.

Column 6 The Runoff Coefficient of the drainage area or areas identified in Col. 4.

Note: If more than one drainage area is contributing runoff, a weighted runoff coefficient is used, Equation 5-2.

Column 7 The initial time of concentration from Table 5-6

Column 8, 13 The total time of the overland flow not including the initial time of concentration, using equation 5-6.

Column 9 The total time of concentration, the summation of Cols. 7 + 8 (min)

Column 10 The amount of carry over C from Col. 28 of Computation Sheet 7-1

Column 11 The amount of carry over A from Col. 29 of Computation Sheet 7-1

Column 12 The time of concentration of the by-pass flow from Col. 31 of Computation Sheet 7-1

Column 14 The total time of concentration, the summation of Cols. 12 + 13

Column 15 The weighted product of the runoff coefficient of the antecedent precipitation coefficient. This is found as follows:

Weighted C = (Col. 5 * Col. 6 + Col.10 * Col.11) / (Col. 5 + Col.11) (Eq. 5-7)

Column 16 The summation of Cols. 5 + 11, (acre)

Column 17 The time of concentration to be used to determine intensity. Generally this the greater of the Total Tc, Col. 9, and Total Carry Over Tc, Col. 14.

Note: In some instances, the flow determined by the product of C*I*A of the local drainage area, or Cols. 6 * 18 * 5, where the time in Col. 9 is used to determine intensity (I), is greater than the flow of local drainage area plus the by-pass flow. In such a case the average of the Cols. 9 and 14 is to be used.

Column 18 Intensity for the frequency storm in Col. 3 is calculated from Equation 4-1, using the time in Col. 17 and the constants found in either Table 4-1 or at the top right hand corner of the computation sheet.

Column 19 The product of Cols. 15 * 16 * 18.

5.5 Unit Hydrograph Methods

Stormwater discharges produced by watersheds larger than 200 acres shall be computed using a unit hydrograph method. There are two acceptable unit hydrograph methods for drainage system design in the City of San Angelo: Snyder's Unit Hydrograph and the Natural Resource Conservation Service (NRCS) Dimensionless Unit Hydrograph. Regardless of the method used, rainfall losses due to interception, infiltration, and depression storage should be deducted from the total rainfall contributing to runoff when using a unit hydrograph method. The NRCS method should be used for drainage basins larger than 200 acres but less than 2,000 acres.[10] HEC-1 OR HEC-HMS and TR-20 are acceptable computer models for developing runoff hydrographs.

5.5.A. Snyder's Unit Hydrograph

Snyder's unit hydrograph method may be used for drainage areas 200 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations*[11] and Paul Rodman's paper, "The Effects of Urbanization on Various Frequency Peak Discharges",[12] utilizes the following equations:

$t_p = C_t\ (L\ L_{ca} / S^{1/2})^{0.34}$ (Eq. 5-8)

$t_r = t_p\ )\ 5.5$ (Eq. 5-9)

$q_p = C_p 640\ )\ t_p$ (Eq. 5-10)

$t_{pR} = t_p + 0.25(t_R - t_r)$ (Eq. 5-11)

$q_{pR} = C_p 640\ )\ t_{pR}$ (Eq. 5-12)

$q_{pR} = q_p\ t_p\ )\ t_{pR}$ (Eq. 5-13)

$Q_p = q_p\ A$ (Eq. 5-14)

The terms in the above equations are defined as:

- t_r is the standard unit rainfall duration, in hours.
- t_R is the unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.
- t_p is the lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
- t_{pR} is the lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.
- q_p is the peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.
- q_{pR} is the peak rate of discharge in cfs/sq mi. of unit in hydrograph for unit rainfall duration, t_R.
- Q_p is the peak rate of discharge of unit hydrograph in cfs.
- A is the drainage area in square miles.
- L_{ca} is the river mileage from the design point to the centroid of gravity of the drainage area.
- L is the river mileage from the given station to the upstream limits of the drainage area.
- S is the average slope of the streambed in ft/mi.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from about 0.5 to 2.0 and values for the San Angelo region average about 0.9.[13] C_t for a watershed can be estimated if the lag time, t_p, stream length, L, distance to the basin central, L_{ca}, and the streambed slope (ft/mi) are known. These values reflect no significant urbanization of the watershed.

The coefficient C_p is the peaking coefficient, which typically ranges from 0.4 to 0.8 and is related to the flood wave and storage conditions of the watershed. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 5-8.

Table 5-8. Typical Values of C_p

Drainage Area Characteristics	Typical Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

5.5.A.1. *Urbanization Curves*

To account for the effects of urbanization, a method was developed by the Corps of Engineers to adjust the t_p coefficient. Urbanization curves allow for the determination of t_p based on the percent urbanization and the type of soil in the study area. Urbanization curves for the San Angelo area are found in Figures 5-2 and 5-3, which can also be determined from the equations below[14]:

$$t_p = 10\text{^}(0.3833*\log(L*L_{ca}/(S_{st})\text{^}.5)+ \log(Ip)-(0.266 * \%Urb)/100) \quad \text{(Eqn. 5-15)}$$
$$S_{st} = (el_{85\%} - el_{15\%})/(0.7*L) \quad \text{(Eqn. 5-16)}$$

where:

- t_p is the lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
- L_{ca} is the river mileage from the design point to the centroid of the drainage area.
- L is the river mileage from the design point to the upstream limits of the drainage area.
- S_{st} is the weighted slope of the flow path (ft/mi)
- I_p is the calibration point, defined as the t_p where (L * L_{ca}/S_{st}^.5) =1 and urbanization = 0%. [1.09 for clays, 1.76 for sands]
- %Urb is a value representative of the degree to which urbanization has occurred in the basin, in percent.
- $el_{85\%}$ is the elevation at a location 85% upstream of the given station.
- $el_{15\%}$ is the elevation at a location 15% upstream of the given station.

For the San Angelo area, the I_p values used are 1.09 for clayey soils such as Tarrant Ector and Mereta (TE&M) soils, and 1.76 for sandy soils such as Kimbrough-Mereta-Angelo and Angelo soils. For a study area that is composed of both sand and clay, a weighted average of the two can be used. Design runoff may be determined for a given watershed by multiplying each ordinate of the unit hydrograph by the excess rainfall, which can be estimated as described below.

5.5.A.2. *Losses*

Only a portion of the total rainfall volume which falls on a drainage basin results in direct runoff, this is excess rainfall. The portion of rainfall which does not result in runoff is loss to infiltration or interception. These losses are caused by direct infiltration of rain into the ground, depression storage, or other mechanisms. The simplest means to account for these losses is the initial and uniform loss method. For the convenience of the design engineer, recommended initial and uniform loss rates are found in Table 5-9. There are other acceptable means to account for losses and these may be used at the discretion of the City Engineer.

Table 5-9. Initial and Uniform Losses

Soil Type	Initial	Uniform
Sand ~ Kimbrough-Mereta-Angelo and Angelo Soils	1.0	0.10
Clay ~ Tarrant-Ector and Mereta	0.5	0.05

Weighted averages can be used for additional soil types as appropriate.

5.5.B. Soil Conservation Service Unit Hydrograph Method

The procedures for the Natural Resources Conservation Service (NRCS) method are outlined in Section 4 of the National Engineering Handbook[15] and in numerous hydrology textbooks.

5.5.B.1. *NRCS Curve Number*

The NRCS method uses a dimensionless unit hydrograph applied to the peak discharge computed for a given watershed. The runoff curve number used in design is to be based on fully developed watershed conditions. Table 5-10 contains runoff curve numbers for Antecedent Moisture Condition II (AMC II) for various land uses.[10,16] For a listing of applicable soil types, refer to the United States Department of Agriculture, Soil Conservation Service, Soil Survey of Tom Green County, Texas.[17]

5.5.B.2. *NRCS Lag Time*

The NRCS lag time is based on the time of concentration. The time of concentration shall be determined as outlined in Section 5.4.C of this chapter. To convert the T_c to lag time use Equation 5-17.[18]

$$T_{LAG}\text{ (hr)} = 0.6 * T_c\text{ (min)} * [1\text{ (hr)} / 60\text{ (min)}] \qquad \text{(Eqn. 5-17)}$$

Figure 5-2. Kimbrough-Mereta-Angelo and Angelo Soils Urbanization Curve

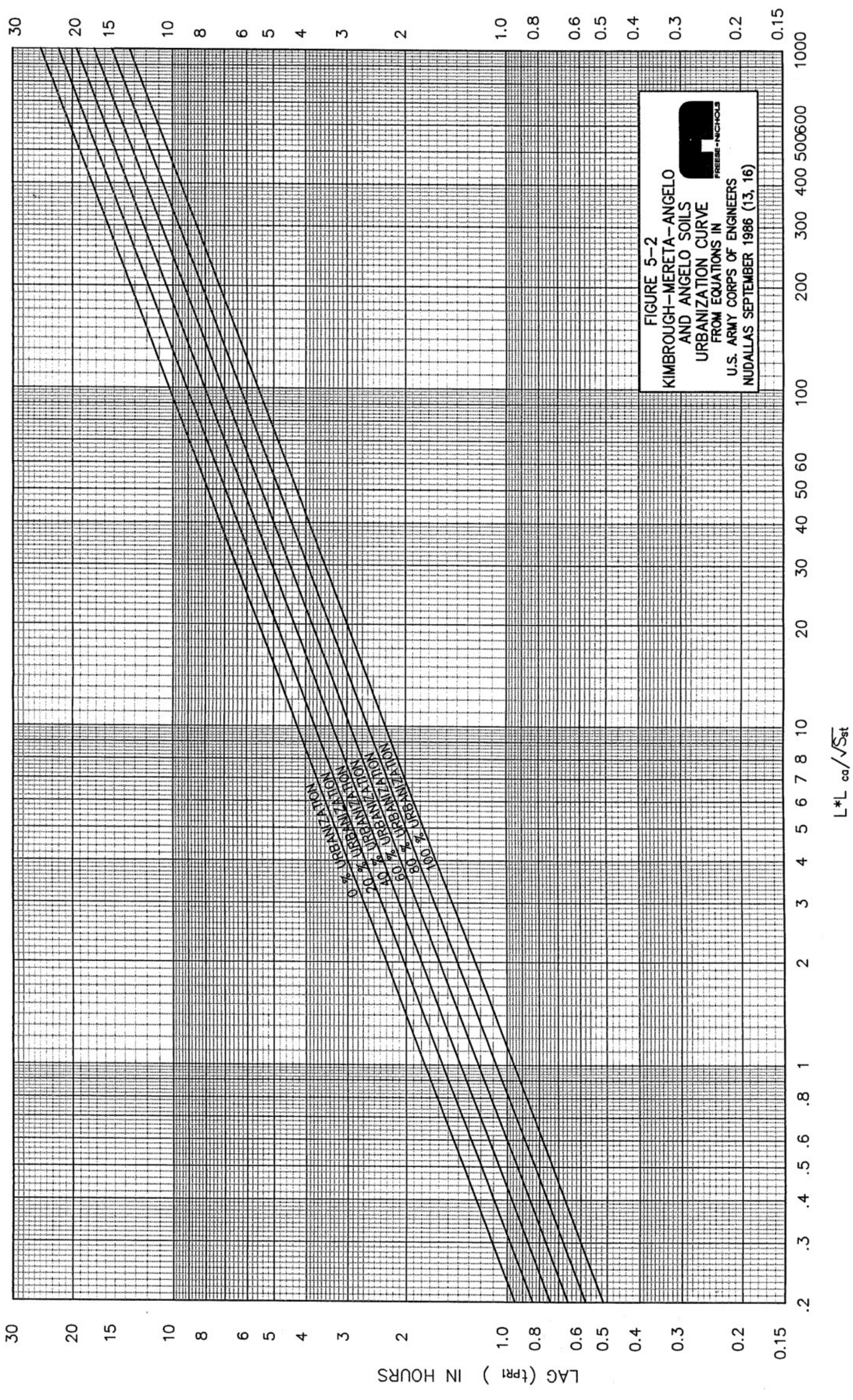


Figure 5-3. Tarrant Ector and Mereta Urbanization Curve

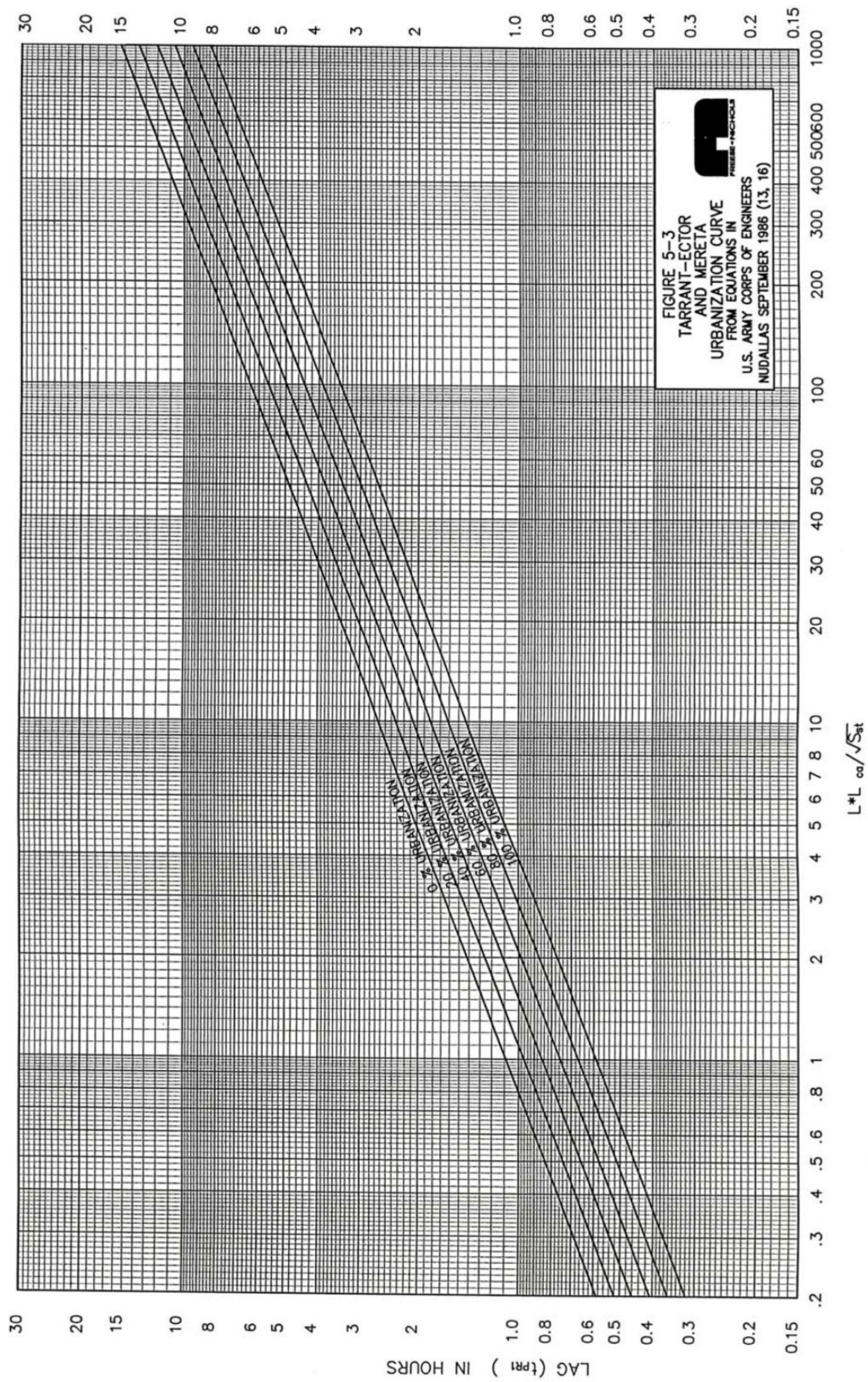


Table 5-10. SCS AMC I Runoff Curve Numbers

Zone Type	Description	CN for Soil Types A	B	C	D
	Parks & Open Areas, Golf Courses	39	61	74	81
Residential:					
R&E	Ranch and Estate District	51	68	79	84
R-1	Single Family Residence District	54	70	80	85
R-2	Two Family Residence District		57	72	81
					86
R-2A	Townhouse and Zero Lot Line Residential District	61	75	83	87
R-3	Multi-Family Residence District	77	85	90	92
MHP, MHS	Manufactured Home Park/Subdiv.	77	85	90	92
Commercial:					
C-1	Neighborhood Retail District	77	85	90	92
C-2	Business District	81	88	91	93
C-3	Commercial District	81	88	91	93
C-N	Neighborhood Commercial	79	87	91	93
C-O	Commercial Office	81	88	91	93
C-G	General Commercial	81	88	91	93
C-H	Heavy Commercial	95	95	95	95
C-E&R	Commercial Entertainment and Rec. (Not open grassed areas)	89	92	94	95
C-BD	Commercial Business District	89	92	94	95
M-1, ML	Light Manufacturing District	89	92	94	95
M-2, M-3	Heavy Manufacturing District	95	95	95	95
PD	Planned Development	57	72	81	86

5.6 Detention Requirements

Peak runoff flow rate from residential, commercial, business, or industrial areas may be reduced by the use of detention ponds, as outlined in Section 11 of this manual.

5.7 Hydrologic Computer Programs

The Corps of Engineers' HEC-1 OR HEC-HMS Program may be used to assist the designer when using the unit hydrograph methods. The procedures outlined in the HEC-1 Flood Hydrograph Package User's Manual[19] or HEC-HMS Technical Manual[20] shall be consulted when using this method for definitions and values for the various modeling parameters. Data generated for use with the HEC-1 OR HEC-HMS program and the results of the program shall be summarized on the drainage plans using a table similar to that shown in Computation Sheet 5-2. Hydrologic data obtained from the Corps of Engineers in the NUDALLAS format shall be converted to the HEC-1 OR HEC-HMS format.

The designer may also utilize the NRCS TR-20 program for hydrograph analysis. The procedures outlined in Technical Release No. 20[21] shall be followed when using this program. Computation Sheet 5-3, or a similar spreadsheet, should be used to summarize the hydrologic analysis made with the TR-20 program.

Computation Sheet 5-1. Rational Method Peak Runoff Rate

Rational Method Peak Runoff Rate Computation

Project Description: ____________________ ________ By: ____________________

____________________ ________ Checked By: ____________________

Freq. Yr.	b	d	e	Freq. Yr.	b	d	e
2	90.62	15.71	0.901	25	73.92	13.64	0.733
5	75.42	14.48	0.807	50	75.85	13.43	0.712
10	73.00	14.03	0.768	100	78.37	13.28	0.696

1 Inlet No.	2 Roadway Sta.	3 Freq. Year	4 Drainage Area	5 Area (acre)	6 C	7 Initial Tc (min.)	8 Time of Overland Flow (min.)	9 Total Tc (min.)	10 Carry Over C	11 Carry Over Area	12 Carry Over Tc (min.)	13 Time of C.O. Overland Flow (min.)	14 Total C.O. Tc (min.)	15 Weighted C	16 Total Area (acre)	17 Tc Used (min.)	18 I (in./hr) b / (Tc+d)^e	19 Q C*I*A (c.f.s.)

Computation Sheet 5-1

Computation Sheet 5-2. HEC-1 Hydrologic Summary Table

HEC - 1 HYDROLOGIC SUMMARY TABLE

Project: ____________________

	Drainage Area Characteristics											Output Summary			
Design Point	Area No.	Total Area (ac)	Weighted Stream Slope (%)	(%) Urbanization	(%) Sand	Imprvs Area (ac)	Total Hydrlc Length (ft)	Centroid Length (ft)	Design Frequency (yrs)	Rainfall Depth (in)	Peak Discharge (cfs)	Time of Peak (min)	Total Basin Area (sq mi)	Depth of Flow (ft)	Time of Max Stage (min)
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**

Computation Sheet 5-2

Computation Sheet 5-3. TR-20 Hydrologic Summary Table

TR-20 HYDROLOGIC SUMMARY TABLE

ALT NO	STORM NO	SECTION / STRUCTURE IDENTIFICATION	STANDARD CONTROL OPERATION	DARINAGE AREA (sq mi)	RAIN TABLE NO	ANTEC MOIST COND	PRECIPITATION AMOUNT (in)	RUNOFF AMOUNT (in)	ELEV (ft)	HYDROGRAPH INFORMATION: INFLOW PEAK (cfs)	INFLOW TIME (hr)	OUTFLOW PEAK (cfs)	OUTFLOW TIME (hr)	OUTFLOW + INTERV AREA PEAK (cfs)	OUTFLOW + INTERV AREA TIME (hr)	BASE-FLOW (cfs)	ROUTING PARAMETERS: VOLUME ABOVE BASE (in)	MAIN TIME INCREMENT (hr)	ITERATION NO	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O / I (Q*)	S / Q @ PEAK (K) (sec)	ATT-KIN COEFF (C_R)	PEAK TRAVEL TIME STORAGE (hr)	PEAK TRAVEL TIME KINETIC (hr)	NOTES
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29

COMPUTATION SHEET 5-3

6.0 STREET FLOW

6.1 Street Flow Limitations

Streets may be used to convey stormwater runoff in accordance with the limitations in this Stormwater Manual. Allowable depths of flow across street intersections for 2-and 100-year storm events are established as indicated in Table 6-1 and shown in Figure 6-1.

Table 6-1. Allowable Intersection Flows

Street Intersection	Maximum 2-Year Cross Flow	Maximum 100-Year Cross Flow
Major Arterial Streets	6” above gutter	12” above gutter
Collector Streets	6” above gutter	12” above gutter
Residential Streets	6” above gutter	12” above gutter

For intersections of differing types of streets, the street whose traffic is being crossed by the drainage flow is used in the table. The city standard street cross slope and all crown transitions, as required by the City, shall maintain adequate hydraulic capacity, through the intersection. Intersections shall comply with Table 6-1. Where additional hydraulic capacity is required, curb inlets and storm sewers may be installed to remove a portion of the flow.

6.2 Velocity Limitations

Maximum velocity of flow in streets shall be no more than 12 feet per second at a 6”..

6.3 Street Flow Calculations

Evaluation of street flow is based upon open channel hydraulics theory, with the Manning’s equation modified to allow direct solution, based on the street cross section. Generally, the street will have a straight or parabolic section. Figure 6-2 has been prepared for gutter flow capacity as divided into two types, triangular and triangular with gutter. These street flow calculations are dependent on the shape of the street. Flow calculations for inverted streets shall use the methods outlined in Section 6.3.

Figure 6-1. Street Cross Section

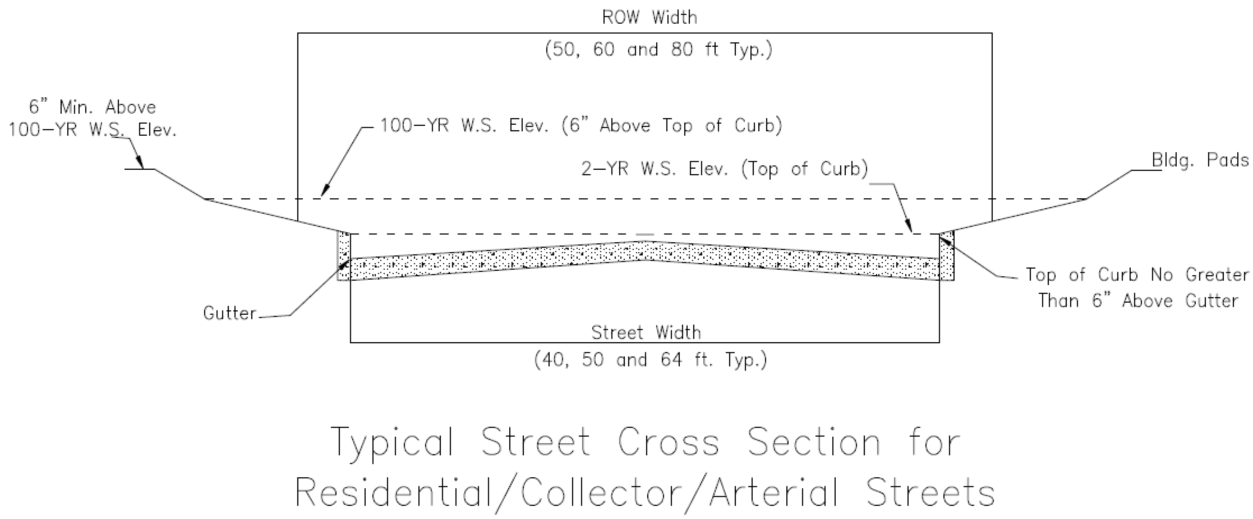
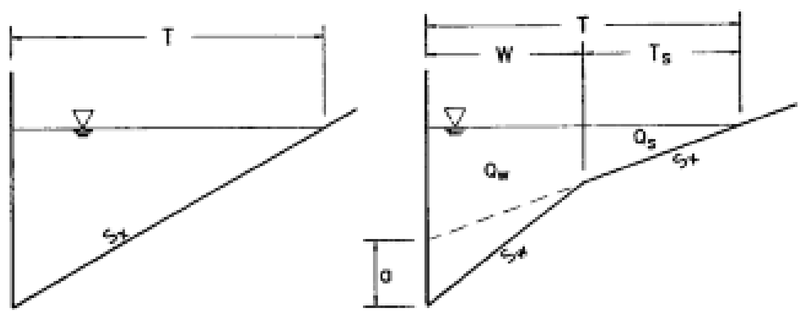


Figure 6-2. Typical Gutter Cross Sections



a. Triangular b. Triangular with Gutter

The direct solution for gutter flow depth for a given flow in straight sections (triangular channel) is based upon the following formula:

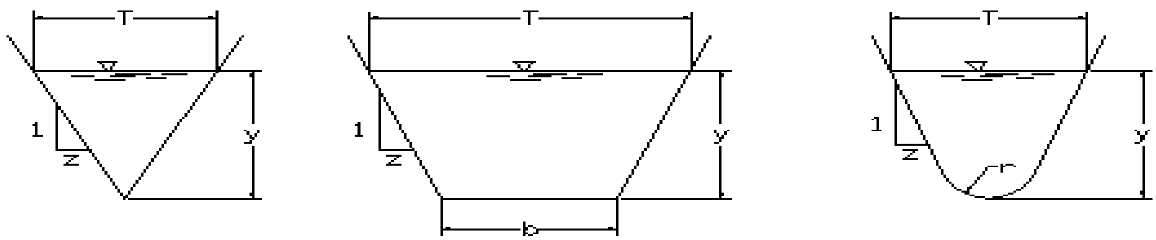
$$Q = 0.56 * S^{0.5} * y^{8/3} / (n * S_x) \quad \text{(Eq. 6-1)}$$
$$y = [(Q * n * S_x) / (0.56 * S^{1/2})]^{3/8} \quad \text{(Eq. 6-2)}$$
$$T = y / S_x \quad \text{(Eq. 6-3)}$$
$$V = (2 * Q) / (T^2 * S_x) \quad \text{(Eq. 6-4)}$$

where:
- Q is the gutter discharge, ($ft.^3$/sec.)
- y is the flow depth in the gutter, (ft.)
- S_x is the crown slope, (ft./ft.)
- n is Manning's coefficient of roughness, usually 0.016 for streets
- S is the slope of the gutter, (ft./ft.)
- T is the spread of flow or ponding width (ft.)
- V is velocity of flow, (ft./sec.)

6.4 Alley Flow

Alley flows shall not exceed criteria in Table 5-1 for the 2-year and 100-year storm events. Figure 6.3 is a representation of typical alley cross sections.

Figure 6-3. Typical Alley Cross Sections.



a. Triangular b. Trapezoidal c. Round Bottom

Flow in alleys is also based upon open channel hydraulics theory, with the Manning equation modified to allow direct solution, with regard to the alley cross section. The depth of flow for the triangular cross section in Figure 6-3a can be calculated by the following equation:

$$y = (Q * n*(1+z^2)^{1/3}/(0.936* z^{5/3} * S^{1/2}))^{3/8} \quad \text{(Eq. 6-20)}$$

$$T = 2 * z* y \quad \text{(Eq. 6-21)}$$

$$A = z * y^2 \quad \text{(Eq. 6-22)}$$

$$V = 0.936 * z^{2/3} *y^{2/3} * S^{1/2}/(n * (1+z^2)^{1/3}) \quad \text{(Eq. 6-23)}$$

$$Q = 0.936 * z^{5/3} *y^{8/3} * S^{1/2}/(n * (1+z^2)^{1/3}) \quad \text{(Eq. 6-24)}$$

where:
- Q is the discharge, ($ft.^3$/sec.)
- y is the depth of flow, (ft.)
- z is the inverse of the slope of the crown slope, (ft./ft.)
- n is Manning's coefficient of roughness, usually 0.016 for streets
- S is the slope of the gutter, (ft./ft.)
- T is the spread of flow or ponding width (ft.)
- V is the velocity of flow, (ft./sec.)

Trapezoidal cross-sections like the one shown in Figure 6-3b require an iterative process of depth using the Manning's equation for discharge. Once the depth of flow is determined, it can be used to calculate top width, area, and velocity using the following equations:

$$T = b + 2 * z * y \quad \text{(Eq. 6-25)}$$

$$A = (b + 2*z) * y \quad \text{(Eq. 6-26)}$$

$$V = Q / A \quad \text{(Eq. 6-28)}$$

$$Q = (1.486/n)*S^{1/2}*((b+z*y) * y)^{5/3} / ((b+z*y)*(1+z^2)^2) \quad \text{(Eq. 6-29)}$$

y must be solved iteratively using Equation 6-24

where:
- y is the depth of flow, (ft.)

z is the inverse of the slope of the crown slope, (ft./ft.)
T is the spread of flow or ponding width (ft.)
b is the bottom width of the trapezoid, (ft.)
n is Manning's coefficient of roughness, usually 0.016 for streets
S is the slope of the gutter, (ft./ft.)
V is the velocity of flow, (ft./sec.)
Q is the discharge, (ft.^{3}/sec.)

The normal depth for a round-bottomed, triangular cross-section shown in Figure 6-3c can be calculated if the top width is known. To determine the top width, a trial-and-error approach must be taken using the Manning's equation:

$$Q = \frac{(1.486/n)*S^{1/2}*((T/(4*z)) - (r^2/z)*(1 - z*\cot^{-1}z))^{5/3}}{((T/2*(1+z^2)^2) - (2*r/z)*(1 - z*\cot^{-1}z))^{2/3}} \quad \text{(Eq. 6-30)}$$

$$y = \frac{(T/2) - r*(1+z^2)}{z} + r \quad \text{(Eq. 6-31)}$$

$$A = (T/(4*z)) - (r^2/z) * (1 - z*\cot^{-1}z) \quad \text{(Eq. 6-32)}$$

$$V = Q / A \quad \text{(Eq. 6-33)}$$

where:
T is the spread of flow or ponding width (ft.)
b is the bottom width of the trapezoid, (ft.)
n is Manning's coefficient of roughness, usually 0.016 for streets
S is the slope of the gutter, (ft./ft.)
V is the velocity of flow, (ft./sec.)
Q is the discharge, (ft.^{3}/sec.)
r is the radius of curvature in the bottom of the alley, (ft.)

7.0 INLET DESIGN

7.1 Inlet Design Considerations

The hydraulic efficiency of storm drain inlets varies with the amount of gutter flow, street grade, street crown and the geometry of the inlet opening. The following are some considerations which must be given attention during inlet design:

A. Inlets must be located where the allowable street flow capacities are exceeded, at low points (sumps or sags) and upstream of transitions between normal and super-elevated street sections.

B. A bypass flow of not more than 50% of the 2-year flow will be allowed on streets except in sump or sag areas, where no bypass is allowed.

C. For a 2-year storm event or less, water flowing in arterial streets shall be intercepted by an inlet prior to super-elevated sections, to prevent water from flowing across the roadway.

D. In super-elevated sections of divided arterial streets, inlets placed against the center medians shall have no gutter depression. Interior gutter flow (flow along the median) shall be intercepted at the point of super-elevation transition, to prevent pavement cross flow.

E. At bridges with curbed approaches, gutter flow shall be intercepted prior to flowing onto the bridge.

F. The maximum approved vertical inlet opening is six inches. Openings larger than six inches require approval of the City Engineer and, if approved, must contain a bar or other form of restraint to limit entry.

G. The design and location of all inlets must take into consideration pedestrian and bicycle traffic. In particular, if grate inlets are used, they should be designed for safe passage of bicycles.

H. Grate inlets may be used only where space restrictions prohibit the use of other types of inlets. If used, the inlet opening should be designed to be twice as large as the theoretical required area, to compensate for clogging, and must be approved by the City Engineer.

I. Combination curb inlets (with opening in curb and grate opening in gutter) may be used only where space behind the curb prohibits the use of other inlet types.

J. Where recessed inlets are required, they shall not decrease the width of the sidewalk or interfere with utilities.

K. Recessed inlets must also be depressed, unless otherwise approved by the City Engineer. The maximum allowable inlet depression for recessed inlets shall be seven inches.

L. Non-recessed, depressed inlets shall have a maximum allowable inlet depression of five inches.

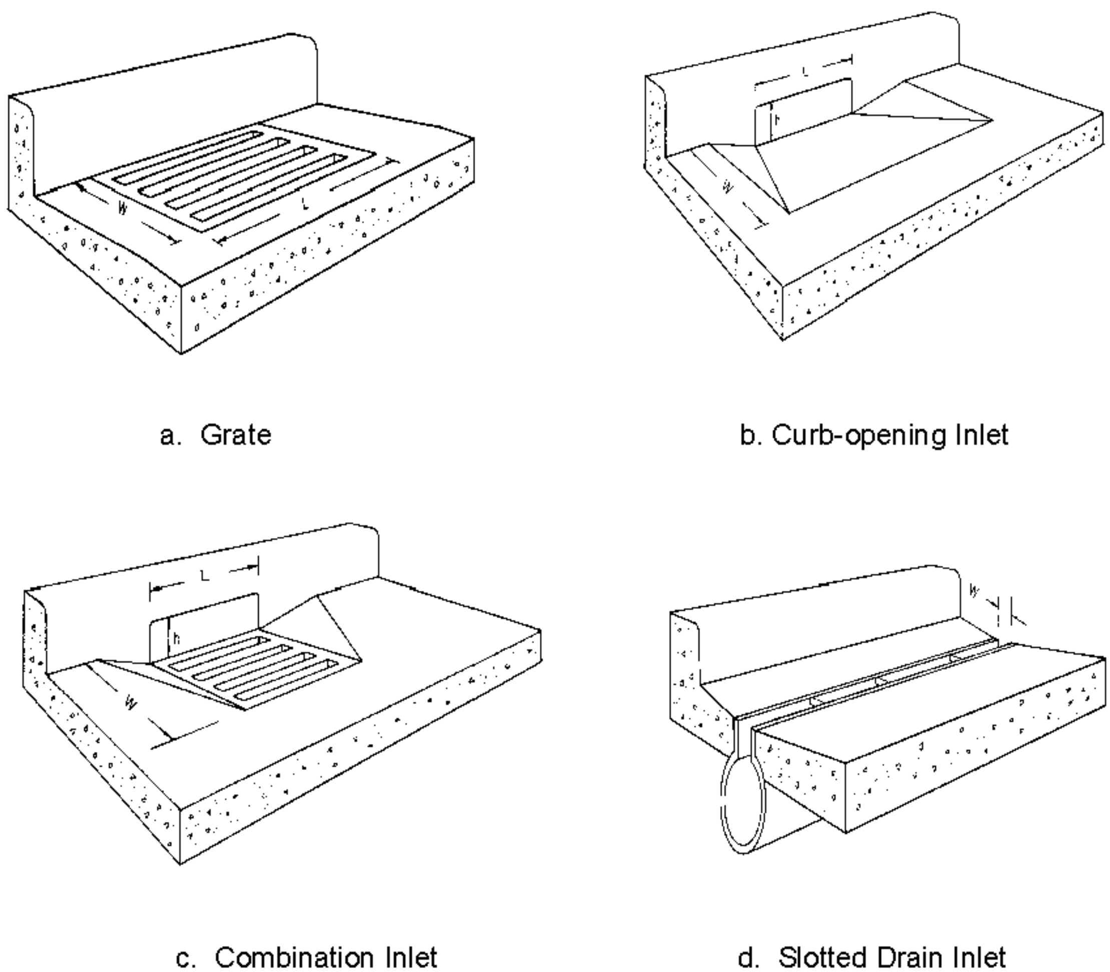
M. The use of slotted drains is not allowed except in instances where there is no alternative, in which case approval must be obtained from the City Engineer. If slotted drains are used, the inlet capacity shall be in conformance with design guidelines in this manual or the manufacturer’s design guidelines.

7.2 Inlet Types and Descriptions

Stormwater inlets are used to remove surface runoff and convey it to a storm drainage system. For the purposes of this manual, inlets are divided into four classes:

1. Grate inlets
2. Curb-opening inlets and type y-inlets
3. Combination inlets
4. Slotted inlets

Figure 7-1. Inlet Types



a. Grate

b. Curb-opening Inlet

c. Combination Inlet

d. Slotted Drain Inlet

7.2.A. Grate Inlets

Although grate inlets may be designed to operate satisfactorily in a range of conditions: however, they may become clogged by floating debris during storm events. In addition, they can produce a hazard to wheel-chair and bicycle traffic and must be designed to be safe for both. For these reasons, they may be used only at locations where space restrictions prohibit the use of other types of inlets, and must be approved by the City Engineer. Figures 7-2 through 7-7 provide examples of grates which are acceptable for use in the City of San Angelo.

Figure 7-2. P-50 and P 50 x 100 Grate Inlets

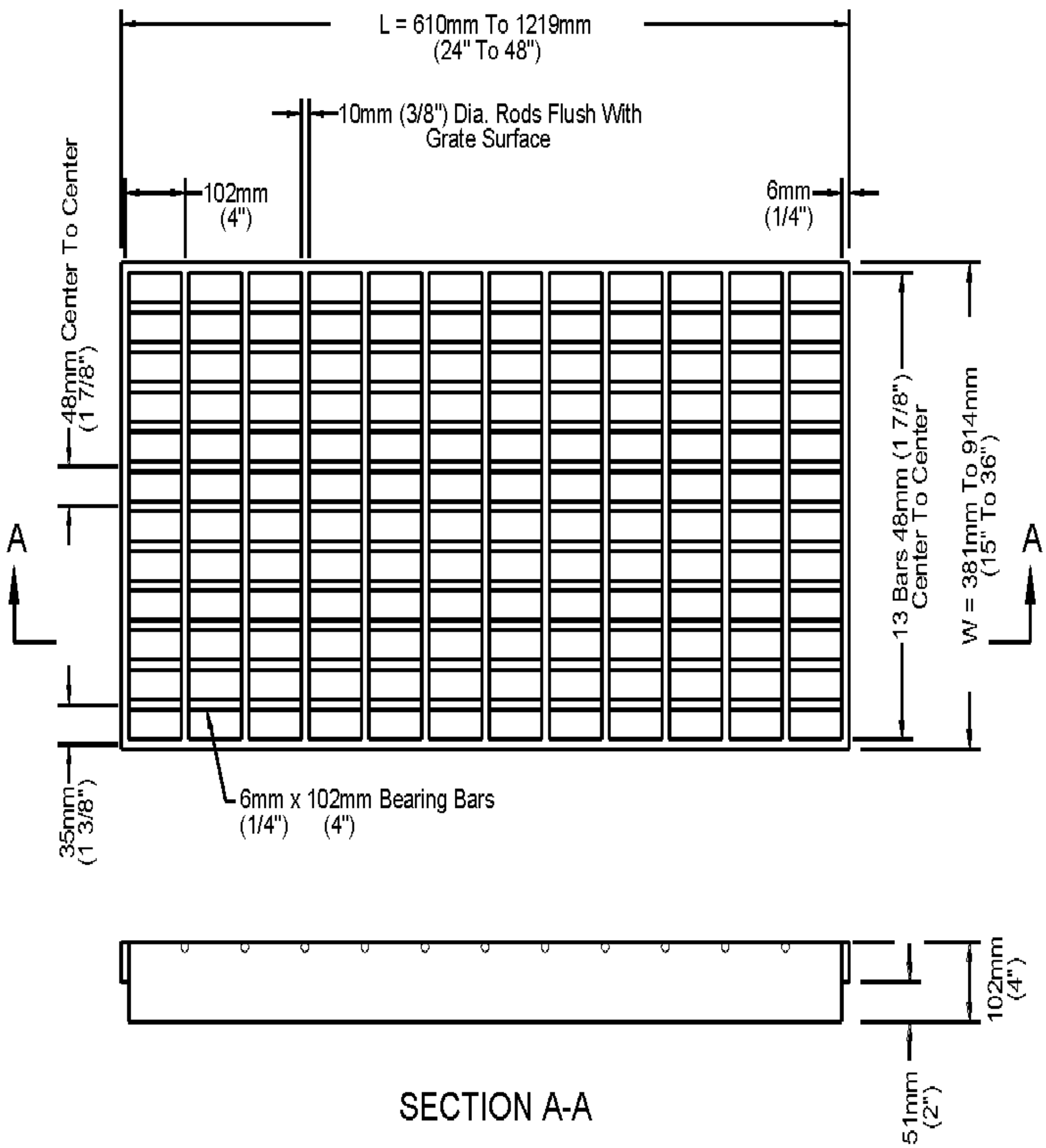


Figure 7-3. P-30 Grate Inlet

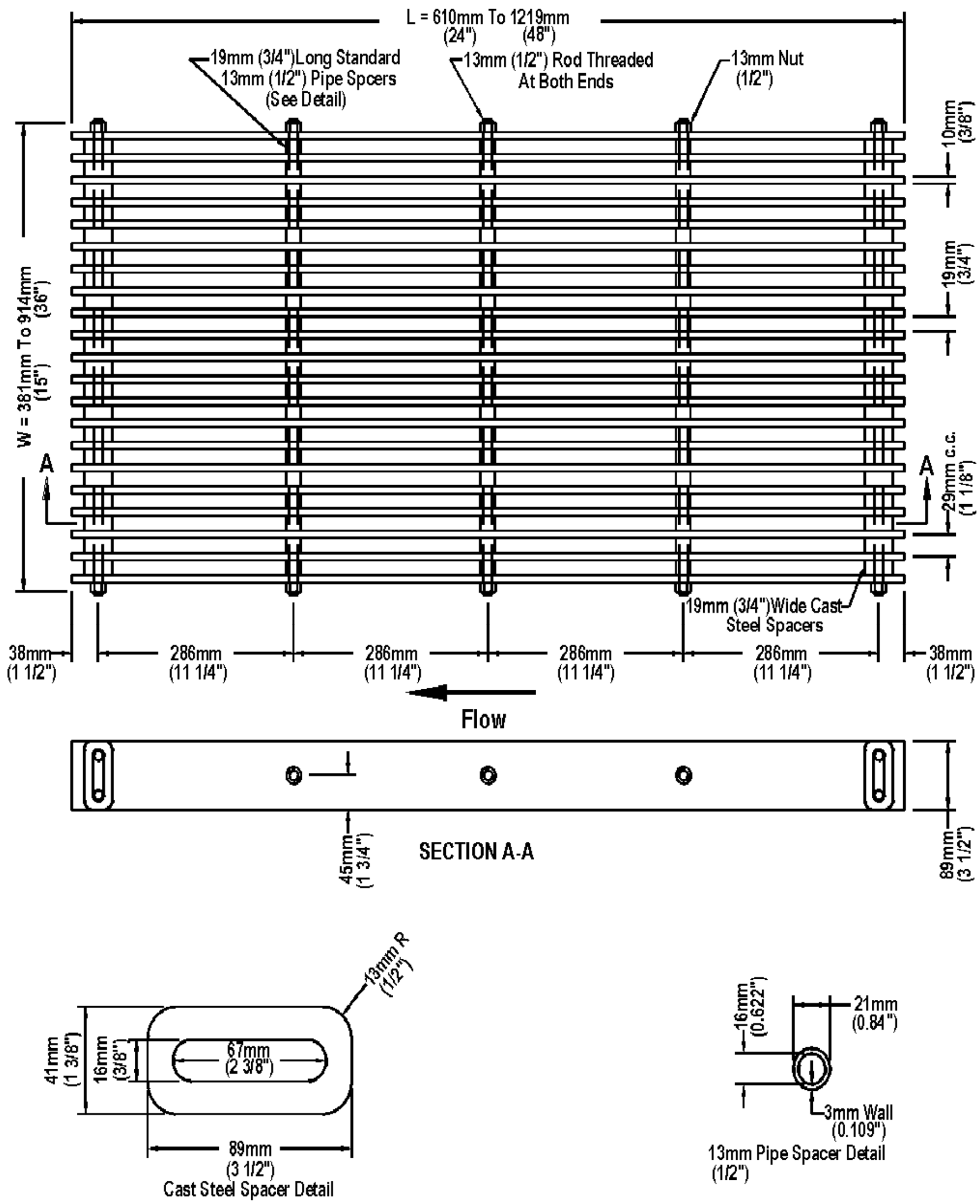


Figure 7-4. Curved Vane Grate Inlet

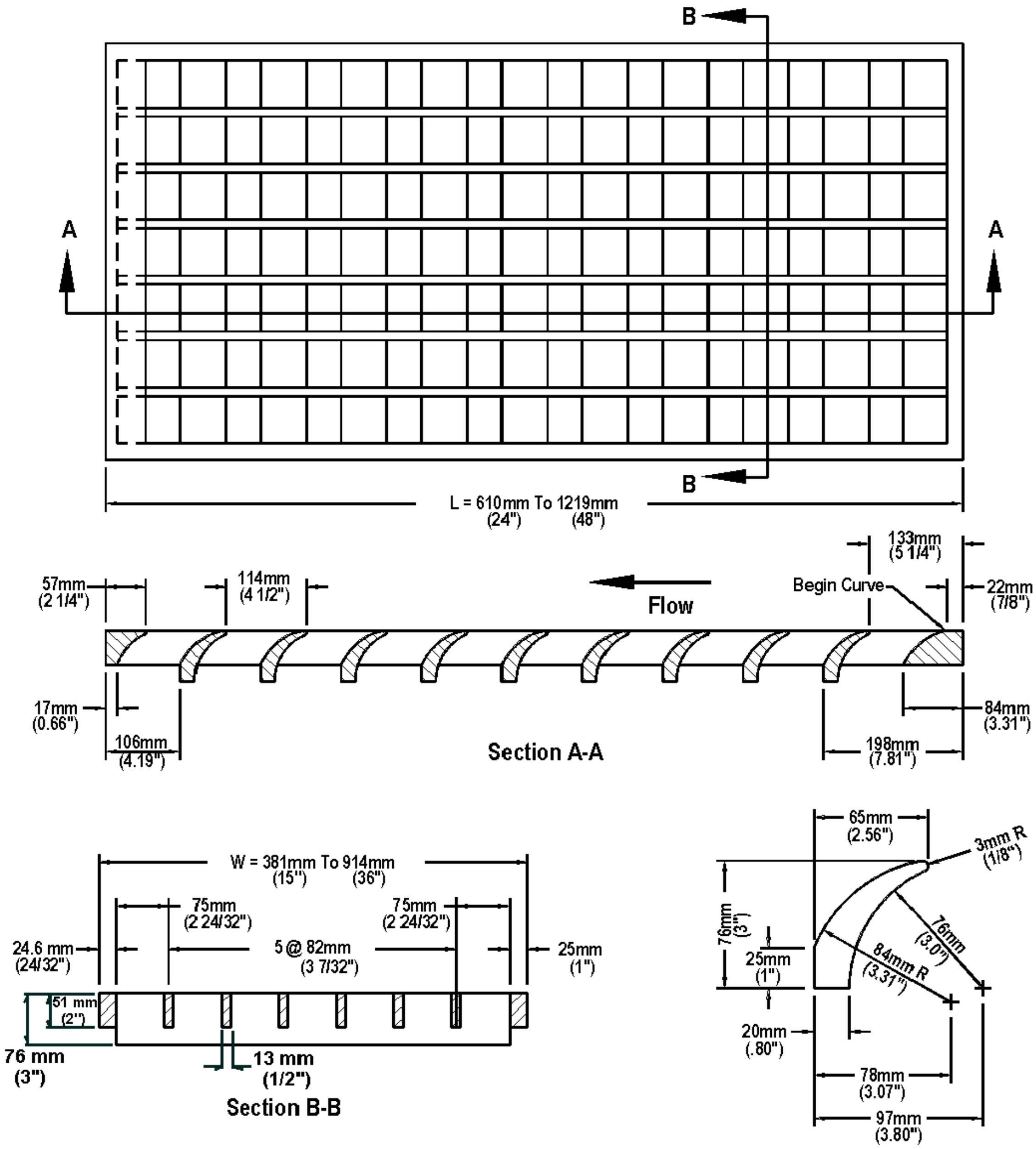
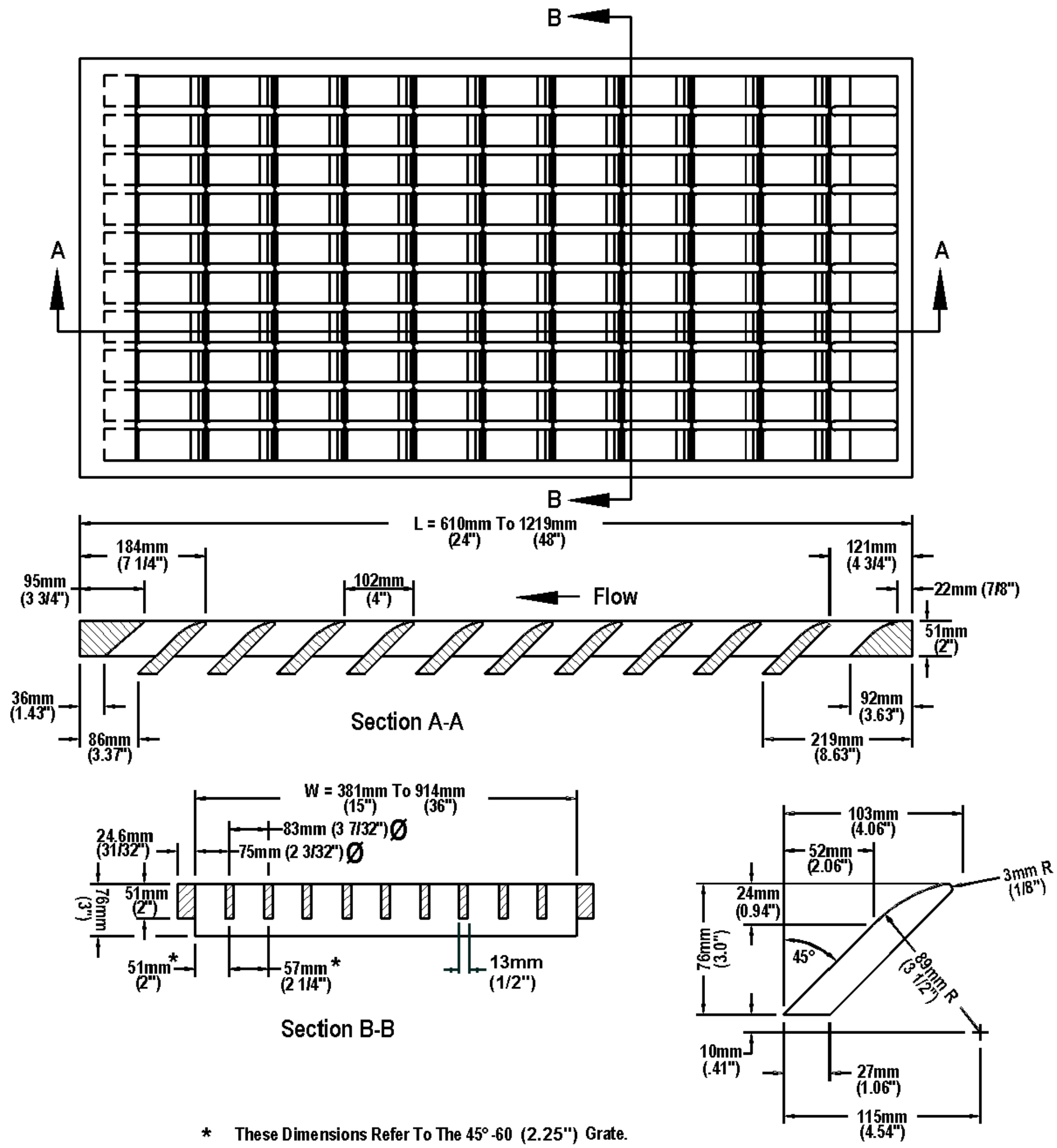


Figure 7-5. Transverse 45° Tilted Grate Inlet



* These Dimensions Refer To The 45°-60 (2.25") Grate.

Ø These Dimensions Refer To The 45°-85 (3.25") Grate.

Figure 7-6. Transverse 30^o Tilted Grate Inlet

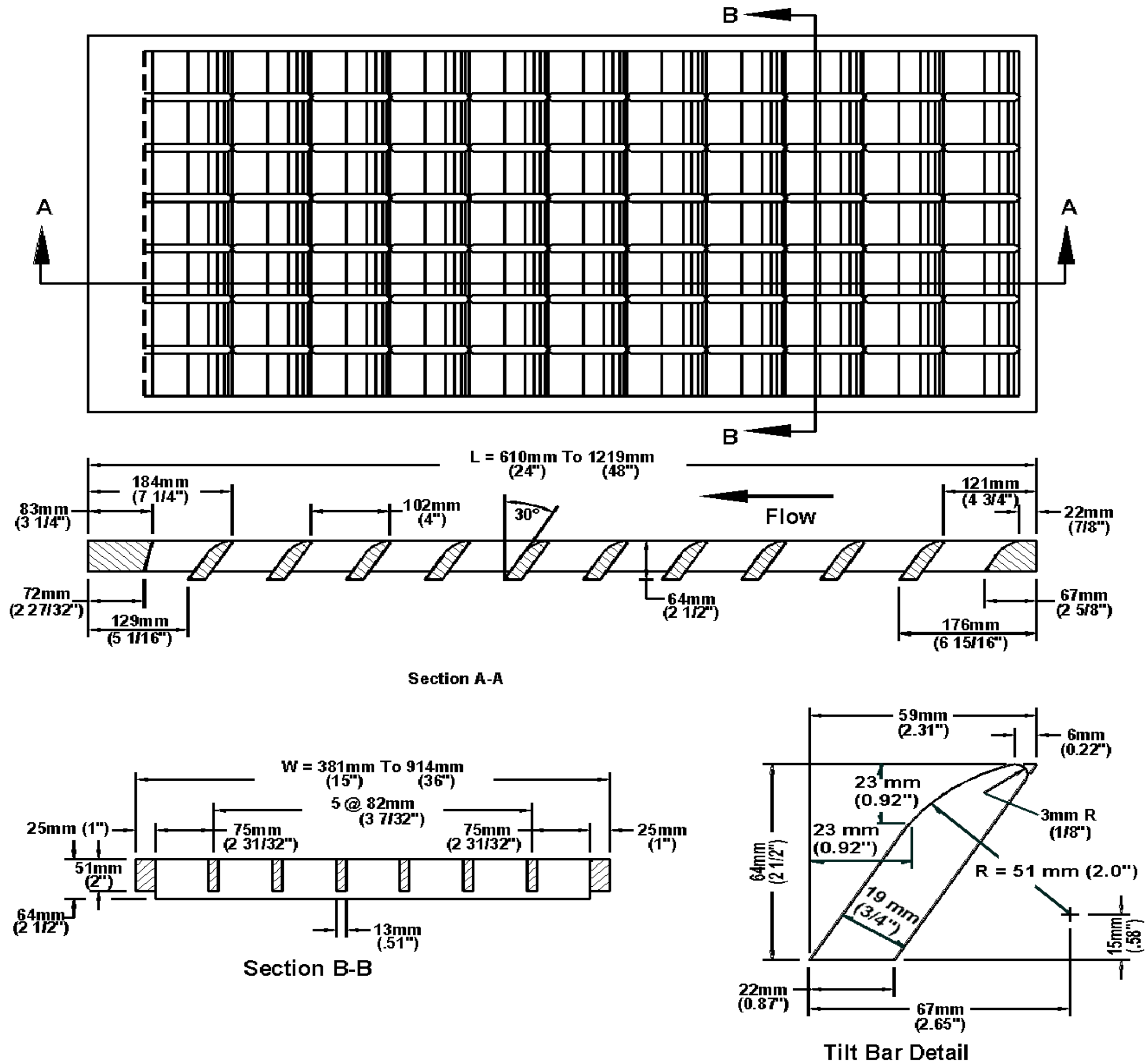
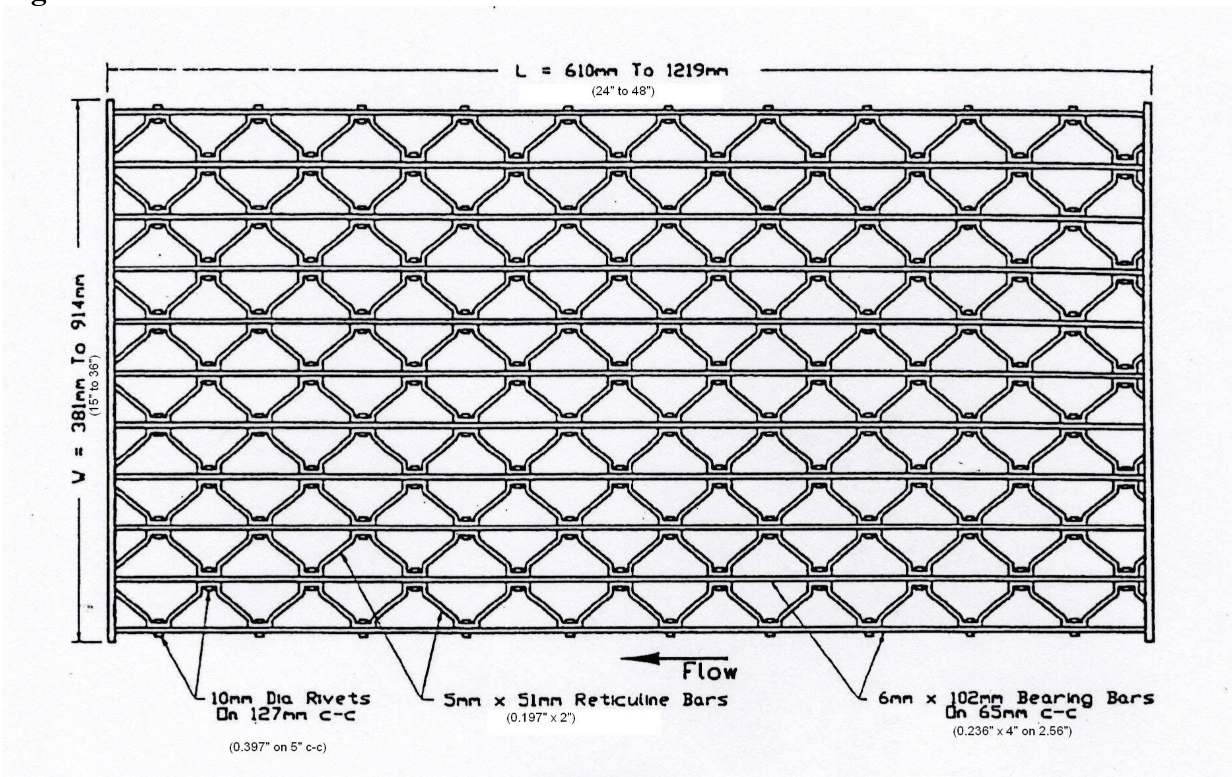


Figure 7-7. Reticuline Tilted Grate Inlet



7.2.B. Curb Inlets and Type-Y Inlets

Curb inlets are the most effective type of inlet on slopes flatter than 3%, in sag locations, and with flows which typically carry large amounts of debris. Similar to grate inlets, curb inlets also tend to lose capacity as street grades increase, but to a lesser degree than grate inlets. Curb inlets are recommended in grades less than 3% and in sag locations. Type-Y inlets are most often used in drainage of swales and sags.

7.2.C. Combination Inlets

A combination inlet consists of both the grate inlet and the curb inlet. This configuration provides many of the advantages of both inlet types. The combination inlet also reduces the chance of clogging by debris with flow into the curb portion of the inlet. If a curb opening is extended on the upstream side of the combination inlet it will act as a “Sweeper”, and remove debris before it reaches the grate portion of the inlet.

7.2.D. Slot Inlets

Although slotted drains can be used to intercept sheet flow, or flow in wide sections, they are not recommended for use in the City of San Angelo since they are very susceptible to clogging from debris. Slot inlets may only be used with the permission of the City Engineer. If slot inlets are allowed, the inlet capacity shall be calculated by both equations for a curb inlet found in Section 7.3-B.23 and the manufacturer's design guidelines, and the lesser inlet capacity, or more conservative method, shall be used for design.

7.2.E. Inlet Abbreviations

Table 7-1. Inlet Types

Inlet Type	Abbreviation	Inlet Type	Abbreviation
On Grade Curb	ONGC	Curb & Grate T-30	CG30
Depressed Curb	DEPC	Curb & Grate T-CV	CGCV
Recessed Curb	RECC	Curb & Grate 50X100	CGPT
Slotted Drain	SLOT	Curb & Recticuline	CGRT
Grate P-50	FRP5	Sag Grate P-50	SGP5
Grate P-30	GRP3	Sag Grate P-30	SGP3
Grate Trans. 45	GR45	Sag Grate 50X100	SCPT
Grate Trans. 30	GR30	Sag Recticuline	SGRT
Grate Trans. CV	GFCV	Sump Curb	SMPC
Grate 50X100	GRPT	Sag Curb & Grate P-50	SGC5
Recticuline	GRRT	Sag Curb & Grate P-30	SGC3
Curb & Grate P-50	CGP5	Sag Curb & Grate P-30	SGC3
Curb & Grate P-30	CGP3	Sag Curb & Recticuline	SCRT
Curb & Grate P-45	CG45	Sweeper	SWEP

7.3 Inlet Capacity Calculations

Stormwater inlets can be further classified into three groups: sump inlets, non-depressed inlets on grade, and depressed inlets on grade. Calculation of the capacity for each inlet type and group which pertains to it are discussed in this section. Many of the equations used for the calculation of inlet capacity came directly from or are modified forms of equations found in Hydraulic Engineering Circular No. 22[9].

7.3.A. Inlets in a Sump

Inlets in a sump are to be designed to have sufficient capacity to capture all of the flow from the 100-year storm event. This includes the flow from the area that contributes directly to the inlets in a sump, as well as any by-pass flow from inlets upstream of the sump. It should be noted that the longitudinal slope of the roadway decreases in the vertical curve near a sump which may cause additional ponding and spread width. Also, no by-pass flow is allowed from a sump location, and the depth of water required to create sufficient head for the inlet to capture all the flow may be greater than the normal flow depth for a given roadway. As a result, the depth, y, may require adjustment to build up sufficient head. This larger "y" will increase the ponding and spread width of the water. Additional length of the sump inlet, or additional inlets or inlet length upstream of the sump inlet may be required to limit the spread width to the acceptable limits specified in Section 6.1.

7.3.A.1. *Grate Inlets in a Sump*

Grate inlets are subject to clogging by debris during storm events, and shall not be used in sumps or sags unless they have a minimum of a five-foot "Sweeper" curb inlet on both sides of the grate inlet. When a "Sweeper" curb inlet is used, the capacity of that portion of inlet which does not have a grate will be calculated as if it was operating alone. The flow which will bypass the "Sweeper" curb inlets will then be used for the sizing of the grate inlet in the sump. A combination inlet is discussed in Section 7.3-A.3 of this manual and may be more efficient than a grate with "Sweeper" curb inlets.

A grate inlet in a sump or sag operates under either weir or orifice flow. Capacity calculations for both conditions will be performed and the lesser of the two capacities will be the design capacity of the grate inlet. Due to the fact that grate inlets in a sump are prone to clog, only 50% of the perimeter shall be used for the weir calculations and 50% of the surface area shall be used for the orifice calculations.

The only grate types that are acceptable in a sump location are the P-50, P-30, P-50X100, and Recticuline grates, as shown in Figures 7-2, 7-3, and 7-7. Capacity of a grate inlet in a sump under weir conditions shall be calculated by the following equation:

$$Q_i = C_w * P * y^{1.5} \qquad \text{(Eq. 7-1)}$$

where:

- Q_i is the capacity in cfs of grate inlet under weir conditions, (ft^3/sec.)
- C_w is the weir coefficient of 3.0
- P is the perimeter of the grate inlet, = 2 * (width + length), (ft)
- y is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in Equations 6-2, 6-5, 6-10, and 6-15 or it is the adjusted head required to accept the 100-year storm event, (ft), which ever is the greater.

Capacity of a grate inlet in a sump location under orifice flow conditions shall be calculated by the following equations:

$$Q_i = C_0 * A * (2 * g * y)^{0.5} \qquad \text{(Eq. 7-2)}$$

where:
Q_i is the capacity in cfs of grate inlet under weir conditions, (ft^3/sec.)
C_o is the orifice coefficient of 0.67
A is 50% of the open surface area of the grate inlet opening, (ft^2/sec). Effective area, 50% of total open area, for the different grate inlet types can be calculated from the Equations 7-3 through 7-6 in Table 7-2.

g is the acceleration due to gravity, = 32.2, (ft/sec^2)
y is the head oat the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in Equations 6-2, 6-5, 6-10, and 6-15 or it is the adjusted head required to accept the 100-year storm event, (ft), whichever is greater.

Table 7-2. Grate Effective Area

Inlet	Abbreviation	Area	
Sag Grate P-50	SGP5	L * W * 0.729 * 50%	(Eq. 7-3)
Sag Grate P-30	SGP3	(L* 0.948) * (W * 0.655) * 50%	(Eq. 7-4)
Sag Grate 50X100	SGPT	(L* 0.910) * (W * 0.729) * 50%	(Eq. 7-5)
Sag Grate Recticuline	SGRT	L * W * 0.800 * 50%	(Eq. 7-5)

7.3.A.2. *Curb Inlets and Type-Y Inlets in a Sump*

Sump locations are low points in the street grade. In general, curb inlets are to be used along paved streets and Type-Y inlets (drop inlets) are to be used in unpaved areas and drainage ditches. Curb inlets, recessed curb inlets, and Type-Y inlets that are located in a sump or a low point can generally be considered to function as rectangular broad-crested weirs. The capacity of an inlet in a sump should be based on the following weir equation:

$$Q_i = C_w (L + 1.8 W) y^{3/2} \qquad \text{(Eq. 7-7)}$$

where:
Q_i is the interception capacity in cfs of curb opening inlet or drop inlet, (ft^3/sec.)
C_w is the weir coefficient, 2.3, for on grade curb gutters, and 3.0 for depressed curb gutters and Type-Y inlets.

L is the length of curb opening, or the portion of perimeter of inlet opening through which water enters the drop inlet, (ft)

W is the width of the depression or the gutter. This is zero if there is no depression, or if the inlet length is greater than 12.0 feet, (ft)

y is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in equations 6-2, 6-5, 6-10, and 6-15, or it is the adjusted head required to accept the 100-year storm event, (ft), whichever is greater.

Inlets should be located frequently enough along the street that the inlet openings do not become submerge. When the depth of flow is more than 1.4 times the height of the opening of the inlet, the inlet operates under completely submerged conditions and the orifice equation should be used to compute the inlet capacity. The capacity of a completely submerged inlet is derived from following orifice equation:

$$Q_i = Co * A * (2 * g * (y * (h / 2)))]^{0.5} \qquad \text{(Eq. 7-8)}$$

where:

Q_i is the capacity in cfs of curb opening inlet or drop inlet under submerged conditions, (ft^3/ft)

Co is the orifice coefficient of 0.67

h is the height of the curb opening, (ft.)

A is the area of inlet opening, L* h,(ft.^{2})

g is the acceleration due to gravity, = 32.2, (ft./sec.^{2})

y is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in Equations 6-2, 6-5, 6-10, and 6-15, or it is the adjusted head required to accept the 100-year storm event, (ft.), whichever is greater.

7.3.A.3. *Combination Inlets in a Sump*

Combination Inlets used in a sump shall have a minimum of a five-foot "Sweeper" curb inlet on both sides. The "Sweeper" curb inlet capacity, that portion of inlet which does not have a grate, shall be calculated as if it was operating alone. The flow which bypasses the "Sweeper" inlets will then be used for the sizing of the grate inlet in the sump. The capacity of the combination inlet is equivalent to the sum of 50% of the capacity of the grate inlet in a sump as determined in section 7.3-A.2. The only grate inlets which are allowed in a sump are the P-50, P-30, P-50X100, and Recticuline grate inlets.

7.3.A.4. *Slot Inlets in a Sump*

Slot inlets are not allowed in a sump location due to their susceptibility to clogging.

7.3.B. Inlets on Grade

Inlets on grade are to be placed to provide sufficient capacity to capture the flow from a 2-year or 100-year storm event as outline in Table 6-2. Inlets on grade generally do not suffer diminished capacity due to floating debris. They do, however, suffer from diminished capacity from excessive street grades. In general, more inlet length will be required to remove the same flow from a steeper roadway than from a flatter roadway.

7.3.B.1. *Grate Inlets On Grade*

Grate inlets on grade are an effective means of conveying flow from the roadway to the drainage system. Although there is less chance of clogging g from floating debris on grade than in a sump, it may still occur. Therefore, grate inlets are to only be used in areas where floating debris will not be a problem and with the approval of the City Engineer.

Each grate inlet type has a splash over velocity, V_0, which is used to determine the amount of the flow which will be intercepted by the front of the inlet, based on the spacing of the bars, length of the inlet, and longitudinal slope of the road. The equations for V_0 can be found in Table 7-3. If the velocity of the flow in the gutter is less than V_0 then essentially all of the frontal flow will be intercepted by the grate. If the velocity of flow is greater than V_0, then only a portion of the flow will be intercepted.

Table 7-3. Splash Over Velocity V_0, For Various Grate Inlets

Grate Type	Grate Abbreviation	Splash Over Velocity, V_0	
P-50	GRP5	$2.218 + 4.031 * L - 0.65 * L^2 + 0.06 * L^3$	(Eq. 7-9)
P-30	GRP3	$1.762 + 3.117 * L - 0.45 * L^2 + 0.03 * L^3$	(Eq. 7-10)
Curved Vane	GRCV	$1.381 + 2.780 * L - 0.30 * L^2 + 0.02 * L^3$	(Eq. 7-11)
45° Tilt Bar	GR45	$0.988 + 2.625 * L - 0.36 * L^2 + 0.03 * L^3$	(Eq. 7-12)
30° Tilt Bar	GR30	$0.505 + 2.344 * L - 0.20 * L^2 + 0.01 * L^3$	(Eq. 7-13)
P-50x100	GRPT	$0.735 + 2.437 * L - 0.26 * L^2 + 0.02 * L^3$	(Eq. 7-14)
Reticuline	GRRT	$0.030 + 2.278 * L - 0.18 * L^2 + 0.01 * L^3$	(Eq. 7-15)

The frontal flow or the gutter flow is the portion of the total gutter flow that is found between the curb and the outer edge of the grate, or between the curb and the point where the depression/gutter begins. The ratio of frontal flow or gutter flow to the total flow, E_0, is found by the following equation:

$$E_0 = 1/ \{(1+ ((S_w/S_x)/([1+(S_w/S_x)/((T/W)-1)]^{2.67} -1)\} \quad \text{(Eq. 7-16)}$$

For non-depressed inlets, the equation can be simplified to:

$$E_0 = 1-(1-W/T)^{2.67} \quad \text{(Eq. 7-17)}$$
$$Q_s/ Q = 1- (Q_w/Q) = 1-E_o \quad \text{(Eq. 7-18)}$$

where:

E_0 is the ratio of frontal flow or gutter flow to total flow
S_x is the cross slope of the roadway, (ft./ft.)
W is the width of the grate or the depression, (ft.)
S_w is S_x + [a * 12 (ft./in.)] /W, (ft./ft.) (Eq. 7-19)
a is the amount of depression, (in.)
T is the spread width of the flow in the roadway, y/ S_x for straight streets

Q_w is the flow in the gutter or depressed section, (ft.^{3}/sec.)
Q_s is the side flow that does not flow in the gutter or depressed section

and will flow into or along the side of the grate, (ft.^{3}/sec.)

Q is the total flow, (ft.^{3}/sec.)

The ratio of flow intercepted by the grate to frontal flow, R_f, is given by the equation:

$$R_f = 1 - K_c (V - V_o) \quad \text{(Eq. 7-20)}$$

K_c is a coefficient of 0.09

V is the velocity of flow in the gutter,
$2Q / (T^2 * S_x)$, (ft./sec.) (Eq. 7-21)

V_o is the splash over velocity which can be determine from the equations 7-9 through 7-15 found in Table 7-3, (ft./sec.)

Note: R_f can not exceed 1.0

The ratio of side flow intercepted to total flow, R_s, is given by the equation:

$$R_s = 1 / [1 + ((K_s * V^{1.8}) / (S_x * L^{2.3}))] \quad \text{(Eq. 7-22)}$$

where: K_s is the coefficient 0.15

The efficiency, E, of a grate inlet is given by the equation:

$$E = R_f * E_0 + R_s * (1 - E_o) \quad \text{(Eq. 7-23)}$$

Therefore the total interception (capacity) of a grate inlet on grade is given by the equation:

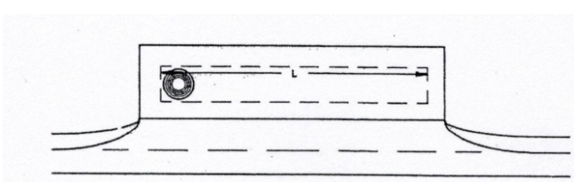
$$Qi = E * Q = Q * [R_f * E_0 + R_s * (1 - E_o)] \quad \text{(Eq. 7-24)}$$

7.3.B.2. *Curb Inlets and Type-Y Inlets On Grade*

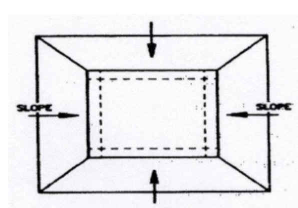
Curb inlets on grade are classified into three groups:

a) Curb inlets which have no depression and are not recessed
b) Depressed curb inlets which are depressed but not recessed
c) Recessed curb inlets which are both depressed and recessed

Figure 7-8 Curb Inlet Types



a. Recessed Inlet



b. Type-Y Inlet

For curb inlets on grade in San Angelo, the recessed curb inlet is recommended due to superior interception efficiency. In areas where there is insufficient room to construct the recessed inlet, or it poses the possibility of a traffic hazard, other inlet types may be used with the permission of the City Engineer. The other inlet types on grade are listed below, in order from the most desirable to the least, as follows:

a) Depressed, non-recessed inlets

b) Combination inlets, with sweeper inlet

c) Combination inlets, without sweeper inlet

d) Grate inlets

e) Non-depressed, non-recessed curb inlets, and

f) Slot inlets

Due to possible traffic hazards, depressed non-recessed inlets should not be used on major thoroughfares, designated as arterials and collectors.

The calculation of the amount of flow intercepted by a curb inlet on grade is the same for all three types of curb inlets. It requires the calculation of the length of inlet required to intercept the entire flow, L_T, which is give by the equation:

$$L_T = K_c * Q^{0.42} * S_L^{0.3} * (1 / (n * S_c))^{0.9} \qquad \text{(Eq. 7-25)}$$

where:	K_c	is the coefficient 0.6
	Q	is the total flow, (ft.^{3}/sec.)
	S_L	is the longitudinal slope of the roadway, (ft./ft.)
	n	is Manning's roughness coefficient, usually = 0.016 for streets
	S_e	is the equivalent cross slope in cross sections with a depression, this is S_x if there is no depression, (ft./ft.)
	S_e	is $S_x + S'_w * E_o$ (Eq. 7-26)
	S_x	is the cross slope of the roadway, (ft./ft.)
	S'_w	is the cross slope of the gutter measured from the cross slope of the pavement, S_x, (ft./ft.)
	S'_w	is a / 12 (in./ft.), (ft.)
	a	is the gutter depression, (in.)
		Is the ratio of frontal or gutter flow to total flow from equation 7-16 or 7-17

The amount of flow that a curb inlet on grade will intercept is equivalent to the product of the total flow and the efficiency of the inlet, E. The inlet efficiency, E, is dependent on the actual inlet length, L, and the required inlet length, L_T, and is determined by the equation:

$$E = 1 - (1 – (L / L_T)^{1.8}$$

Therefore the total amount of flow intercepted by a curb inlet on grade is:

$$Q_i = Q * E = Q * [1 - (1 – (L / L_T)^{0\ 1.8}]$$

7.3.B.3. *Combination Inlets On Grade*

Combination inlets may be used, with the permission of the City Engineer, in areas where a depressed inlet cannot be constructed. The chance of clogging due to floating debris can be lowered with the use of a “Sweeper” curb inlet on the upstream side of the combination inlet. A combination inlet will have a minimum of a five-foot “Sweeper” curb inlet on both sides, unless specified otherwise by the City Engineer. The “Sweeper” curb inlet capacity, that portion of inlet which does not have a grate, will be calculated as if it was operating along. The flow which bypasses the “Sweeper” inlets will then be used for the sizing of the combination inlet on grade. The capacity of the combination inlet on grade is equivalent to the sum of 50% of the capacity of the grate inlet on grade as determined in Section 7.3-B.1 and 50% of the capacity of the curb inlet on grade as determined in Section 7.3-B.2. All of the grate inlets shown in Figures 7-2 through 7-7 can be used in combination inlets on grade.

7.3.B.4. *Slot Inlets On Grade*

Slot inlets may only be used with the permission of the City Engineer. If slot inlets are allowed, the inlet capacity shall be calculated by equations for a curb inlet found in Section 7.3-B.2 and the manufacturer’s design guidelines. The more conservative method of the two shall be used.

7.4 Inlet Interception Computation Sheet

In order to facilitate the computations required in determining the various hydraulic properties for curb inlets and drop inlets, Computation Sheet 7-1 has been prepared. A list of abbreviations for inlets is found in Table 7-1 of Section 7.2-E of this manual. The number columns of Computation Sheet 7-1 are as follows:

Column 1 Inlet number and designation.

Column 2 The weighted runoff coefficient, C, that produces the flow in the roadway. This can be taken from Col. 15 of Computation Sheet 5-1.

Column 3 The drainage area, A, that produces the flow in the roadway. This can be taken from Col. 16 of Computation Sheet 5-1.

Column 4 Total flow, Q, (ft.^{3}/sec.), in the roadway, flow is derived from both the contributing area and the carry-over flow from the inlet or inlets upstream. This can be taken from Col. 16 of Computation Sheet 5-1

Column 5 The roadway type from Table 6-3

Column 6 The longitudinal slope of the roadway, S, (ft./ft.)

Column 7 The cross slope of the roadway, if the roadway is trapezoidal, S_x, (ft./ft.)

Column 8 The depression of the gutter or inlet, a, (in.)

Column 9 The gutter or grate width, W, (ft.)

Column 10 S'_w is the cross slope of the gutter measured from the cross slope of the pavement, S_x, (ft./ft.). $S'_w = a / (12 \text{ (in./ft.)} * W)$, (Eq. 7-26)

which is: Col. 8 / (12 * Col. 9)

Column 11 $S_w = S_x + S'_w$ (ft./ft.), (Eq. 7-19)

which is: Col. 7 + Col. 10

Column 12 E_o is the ratio of flow in a chosen width, Col. 9, of roadway, usually the width of depression or the grate, to the total flow in the roadway,
(Eq. 7-26)

$E_o = 1/ \{(1 + ((S_w / S_x) / [1 + (S_w / S_x) / ((T/W) - 1)]^{2.67} - 1)\}$
depressed inlets (1-16)

$E_o = 1 -\{1- W/T)^{2.67}$ non-depressed inlets (7-17)

Column 13 S_e is the equivalent cross slope, $S_e = S_x + S'_w * E_o$ (ft./ft.), (Eq. 7-26), which is: Col. 7 + (Col. 10 * Col. 12). Note: If Col. 9 = 0, then Col. 13 = 7.

Column 14 The depth, y, of flow in the roadway at the curb, (ft.)

TRAPEZOIDAL $y = [(Q * n * S_x) / (0.56 * S^{½})]^{3/8}$ (Eq. 6-2)

$[(\text{Col. } 4 * \text{Col. } 5 * \text{Col. } 7) / (0.56 * (\text{Col. } 6)^{½})]^{3/8}$

PARABOLIC $y = ((\text{Col. } 4 / (\text{Col. } 6)^{0.5})^{C2})/C_1$ (Eq. 6-5)

Column 15 The width of the spread or the ponding, T, of the flow in the roadway (ft.).

TRAPEZOIDAL $T = y/ S_x$ = Col. 14/ Col 7 (Eq. 6-2)

PARABOLIC $T = B - (C_3 - C_4 * \text{Col. } 14)^{0.5}$ (Eq. 6-5)

If the depth of flow is greater than the crown, T = B

Column 16 The velocity of the flow in the roadway for the given spread width, T from Col. 14, width of the spread or the ponding, T, of the water in the roadway, (ft.^{2}/sec.).

TRAPEZOIDAL $V = (2 * \text{Col. } 4) / ((\text{Col. } 15)^2 * \text{Col. } 7)$ (Eq. 6-4)

PARABOLIC V = (Col. 4) / ((Col. 15 * Col. 14) / 3) (Eq. 6-9)

Column 17 The splash over velocity, V_o for a given grate from the equations in Table 7-3.

Column 18 Inlet type from Table 7-1.

Column 19 The length of the curb inlet or the grate inlet, or the open perimeter of the Type-Y inlet, (ft.).

Column 20 The total inlet length required, L_T, (ft.), for curb and slot inlets given by the equation:

$$L_T = K_c * Q^{0.42} * S_L^{0.3} * (1 / (n * S_c))^{0.9} \quad \text{(Eq. 7-25)}$$

which is: $0.6 * (\text{Col. } 4)^{0.42} * (\text{Col. } 6)^{0.3} * (1 / (\text{Col. } 5 * \text{Col. } 13))^{0.9}$

Column 21 The ratio of flow intercepted by the grate to frontal flow, R_f, is given by the equation:

$$R_f = 1 - K_c (V - V_o) \quad \text{(Eq. 7-20)}$$

$1 – 0.09 * (\text{Col. } 16 – \text{Col } 17)$

Note: R_f can not exceed 1.0

Column 22 The ratio of side flow intercepted to total flow, R_f given by the equation:

$$R_s = 1 / [1 + ((K_s * V^{1.8}) / (S_x * L^{2.3}))] \quad \text{(Eq. 7-22)}$$

$1 / [1 + ((0.15 * \text{Col. } 16^{1.8}) / (\text{Col } 7 * (\text{Col. } 19)^{2.3}))$

Column 23 The amount of flow intercepted by the inlet, (ft.^{3}/sec.).

7.4.A. <u>Sump Inlets:</u>

<u>Grate</u>: the smaller of $Q_i = C_w * P * y^{1.5}$ (Eq. 7-1)

$Q_i = 3.0 * (\text{Col. } 9 + \text{Col. } 19 * 0.5) * (\text{Col. } 14)^{1.5}$ and

$Q_i = C_0 * A * (2 * g * y)^{0.5}$ (Eq. 7-2)

$Q_i = 0.67 *)\text{Col. } 9 * \text{Col. } 19 * 0.5) * (2 * 32.2 * \text{Col. } 14)^{0.5)}$

<u>Curb</u>: *if* $y < 1.4h$, $Q_i = C_w (L + 1.8 W) y^{3/2}$ (Eq. 7-7)

if $a > 0$ *and* $L < 12$,

$Q_i = 2.3 * (\text{Col. } 19 + 1.8 * \text{Col. } 9) * (\text{Col. } 14)^{3/2}$

if $a > 0$ *and* $L > 12$,

$Q_i = 2.3 * \text{Col. } 19 * (\text{Col. } 14)^{3/2}$

if $a = 0$, $Q_i = 3.0 * \text{Col. } 9 * (\text{Col. } 14)^{3/2}$

if $y > 1.4h$, $Q_i = C_o * A * [2 * g * (y – h/2))]^{0.5}$ (Eq. 7-8)

$(Q_i = 0.67 * (0.5 * \text{Col. } 19) * [2 * 32.2 * (\text{Col. } 14 – (0.5/2))]^{0.5}$

Combination $Q_i =$ 50% Grate in sump + 50% Curb in sump

7.4.B. <u>On Grade Inlets:</u>

<u>Grate</u>: $Qi = E * Q = Q * [R_f * E_0 + R_s * (1 – E_o)]$ (Eq. 7-24)

$(Qi = \text{Col. } 2 * [(\text{Col. } 21 * \text{Col. } 12) + (\text{Col. } 22 * (1 – \text{Col. } 12))])$

<u>Curb</u> $Qi = Q * [1 - (1 - (L / L_T)^{1.8}]$ (Eq. 29)

(Qi = Col. 2 * [1 - (1 - (Col. 19 / Col. 20))$^{1.8}$])

Slot Qi = Q * [1 - (1 - (L / L_T)$^{1.8}$] (Eq. 29)

(Qi = Col. 2 * [1 - (1 - (Col. 19 / Col. 20))$^{1.8}$])

Combination Q_i = 50% Grate on grade + 50% Curb on grade

Column 24 The percent of flow that is intercepted by the inlet, which is Cols. 23 / Col.4.

Column 25 The amount of C intercepted by the inlet, which is Cols. 2 * Col. 24.

Column 26 The amount of drainage area, A, which produces flow for the inlet that is intercepted by the inlet, which is: Col 3 * Col. 24.

Column 27 The lateral or line to which intercepted flow is discharge to.

Column 28 The amount of C that bypasses the inlet, which is, Col. 2 - Col. 25.

Column 29 The amount of drainage area, A, which produces flow for the inlet that is bypassed by the inlet, which is: Col. 3 - Col. 26.

Column 30 The inlet to which the bypass flows.

Column 31 The time of concentration to the inlet from Col 17 of Computation Sheet 5-1.

Computation Sheet 7-1. Inlet Interception Computation Sheet

Inlet Interception Computation Sheet

Project Description: ______________

____

____

Project No.: ______________

Date: ______________

By: ______________

Checked By: ______________

manning's n = 0.016

1 Inlet No.	2 Weighted C	3 A (acre)	4 Q (c.f.s.)	5 Road-way Type	6 Street S (ft./ft.)	7 Sx (ft./ft.)	8 a (in.)	9 Gutter or Grate Width (ft.)	10 S'w	11 Sw	12 Eo	13 Se	14 y (ft.)	15 Spread T (ft.)	16 V (ft./sec)	17 Vo (ft./sec)	18 Inlet Type	19 Inlet L (ft.)	20 Lt (ft.)	21 Rf	22 Rs	23 Qi Flow Intercp.	24 % Intercp.	25 C Intercp.	26 A Intercp. (acre)	27 Inlet or Line Disc. to	28 C By-Pass	29 A By-Pass (acre)	30 Inlet By-Pass to	31 By-Pass Tc (min.)

Computation Sheet 7-1

8.0 STORM DRAIN DESIGN

8.1 Applicable Design Criteria

Storm drain systems are needed where the water depth, water spread, and/or intersection cross flow limits specified in this manual are exceeded. The following are guidelines which must be considered and met in storm drainage design:

A. The minimum lateral storm drain pipe diameter shall be 18 inches, except in sump areas, which shall be at least 21 inches in diameter. The minimum pipe diameter for a trunk line pipe shall be 24 inches.

B. In the case that a road side ditch is used, it must be designed to convey the 100-year storm event within the right of way or easement, and not to exceed the maximum roadway depths outlined in Section 5 of this manual.

C. Pipe diameters shall increase downstream, unless otherwise approved by the City Engineer.

D. Storm drain pipe in roadways or underneath pavement shall be designed for HS-20 traffic loading.

E. Storm drain pipes under roadways within public ROW or dedicated in a public easement under pavement shall be reinforced concrete pipe, Class III or stronger.

F. Storm drain pipes used for driveway crossing within ROW shall be;

   i. reinforced concrete pipe, Class III or stronger; or

   ii. corrugated metal pipe; or

   iii. plastic pipe (HDPE or corrugated).

G. If corrugated metal pipe or plastic pipe is used, the manufacturers design guidelines should be followed. Concrete lining shall be used with corrugated metal pipes with diameters of 36” or greater.

H. Storm drain pipes dedicated in public easements not under pavement shall be;

   i. Reinforced concrete pipe, or Class III or stronger; or

   ii. Corrugated metal pipe; or

   iii. Plastic pipe (HDPE or corrugated).

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

J. For roadway culvert crossing refer to Chapter 10.

Table 8-1. Roughness Coefficient "n" For Storm Drains

Materials of Construction	Minimum Roughness Coefficient "n"
Concrete Pipe	0.013
Corrugated-metal pipe*	
Plain or Coated	0.024
Concrete Lined	0.013
Plastic Pipe*	
Smooth	0.011
Corrugated	0.024

*Requires approval of City Engineer

Table 8-2. Maximum Spacing Of Manholes And Junction Boxes

Pipe Diameter (in.)	Max. Spacing (ft.)
< 36	500
36 < dia < 60	800
60 and larger	1,000

8.2 Design Parameters

In addition to the criteria listed above, there are several general design guidelines to be observed when designing storm drains that will tend to alleviate or eliminate common problems of storm drain performance:

A. Select the pipe size and slope such that the velocity of flow will increase progressively down the system or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration.

B. For all pipe junctions other than manholes and junction boxes, manufactured wye connections should be used, and the angle of intersection shall not be greater than 45 degrees. This includes discharges into box culverts and channels. Special circumstances may require cut-ins instead of manufactured wye connections; the use of cut-ins must be approved by the City Engineer.

C. Inlet laterals will normally connect only one inlet to the trunk line. Special circumstances requiring multiple inlets to be connected with a single lateral shall be approved by the City Engineer.

D. Storm drain pipes under roadways and parking lots shall be reinforced concrete pipe, minimum Class III, or stronger as determined by the engineer.

E. Plastic pipe (HDPE, corrugated plastic, etc.) can be used in areas outside the street or paved areas.

F. Corrugated metal and plastic pipe can only be used for individual driveways and will not be allowed beneath streets and paved areas in public easements and rights-of-way.

G. The cover over the crown of circular pipe under paved areas should be at least 18” and should be based on the type of pipe used, the expected loads and the supporting strength of the pipe. Box sections should normally have a minimum of one foot of cover; however, direct traffic may be allowed in special situations with the approval of the City Engineer. Exceptions will be considered for structural solutions sealed by a professional engineer.

Maximum velocities in conduits are important because of the possibility of excessive erosion of the storm drain pipe material. Table 8-3 lists the maximum velocities allowed. Maximum flow velocities at the downstream end of pipe systems shall be consistent with the maximum allowable velocities for the receiving channel (refer to Section 9, Open Channels). The most downstream length of pipe, for twenty (20) pipe diameters from the outfall, will be at such a grade as to result in velocities which are equal or less than 120% of the receiving stream maximum velocities.

Outfalls to natural channels should be analyzed for erosion impact assuming full pipe flow and the natural low water level of the creek or channel. Discharges that have been concentrated in a drainage system shall be conveyed in improvements to the flow line of the channels. Erosion protection is required for disturbed banks of natural channels. Exit velocities greater than 6 fps shall require additional dissipation elements in accordance with Section 9.8 of this Manual.

Table 8-3. Maximum Velocity in Storm Drains

Storm Drain Type	**Maximum Velocity of Design Discharge**
Inlet laterals (shorter than 30 feet)	No Limit
Inlet laterals (longer than 30 feet)	20 fps
Trunk lines	20 fps
Last 20 diameters of length upstream of an outfall	6 fps, 120% of Receiving Stream*, or provide energy dissipation

* Velocity of receiving stream defined as the average velocity of the stream at its design level discharge at the location of the pipe outfall

8.3 Calculation of the Hydraulic Grade Line

The 2-year and 100-year frequency hydraulic grade lines (HGL) shall be computed and plotted for all storm drain systems. For designs that contain sumps, the 100-year hydraulic grade line is required from the system outfall to the most upstream sump. The determination of friction losses and minor losses are required for these calculations.

8.3.A. Starting Tailwater Conditions

The designer must determine the tailwater conditions at the downstream end of the proposed storm drain system when calculating the hydraulic performance of the system. When proposed storm drains are to discharge into existing watercourses, the tailwater elevation used in hydraulic calculations of the proposed storm drain system will be determined by the design engineer and approved by the City Engineer. The tailwater elevation shall be the greater of the water surface of the receiving stream and the top of the outlet pipe.

The water surface of the receiving stream, shall be based on the “Coincidence Occurrence Frequency Table”, Table 8-4, obtained from the Federal Highway Administration HEC-22.[8] If there is a difference in the drainage area of the receiving stream and the contributing stream there will most likely be a difference in the time the peak flow from both systems will reach the point where the contributing tributary discharges into the receiving stream. In an effort to determine what the water surface will be in the receiving stream a statistical analysis has been compiled to show, given the ratio of drainage areas, what storm frequency should be used in the receiving stream to determine its water surface elevation.

Table 8-4. Frequencies For Coincidental Occurrence

Drainage Area Ratio Receiving Stream to Contributing Stream	**Frequencies For Coincidental Occurrence**			
	10 Year Design		100 Year Design	
	Receiving Stream	Contributing Tributary	Receiving Stream	Contributing Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

8.3.B. Friction Losses

Friction losses shall be computed using Manning’s equation, below, with the Manning's "n" values consistent with Table 8-1.

$$Q = (1.486 / n) * A^{(5/3)} / P^{(2/3)} * S^{0.5} \quad \text{(Eq. 8-3)}$$

$$V = (1.486 / n) * A^{(2/3)} / P^{(2/3)} * S^{0.5} \quad \text{(Eq. 8-4)}$$

$$S = ((Q * n * P^{(2/3)}) / (1.486 * A^{(5/3)}))^2 \quad \text{(Eq. 8-5)}$$

where:
- Q is the flow in the conduit, (cfs)
- V is the velocity of the flow in the conduit, (ft/sec)
- S is the slope of the conduit in the direction of flow, (percent)
- n is the Manning's roughness coefficient from Table 8-1
- A is the cross sectional area of the flow from the equations in Table 8-5, (ft^2)
- P is the wetted perimeter of the flow from the equations in Table 8-5, (ft)

Table 8-5. Hydraulic Geometric Elements Of Storm Drain Conduits

Conduit Shape	Flow Area A	Wetted Perimeter P	Top Width T	Hydraulic Depth D
Circular	$1/8\ (\theta - \sin\theta)\ d_o^2$	$2\ \theta\ d_o$	$(\sin 2\ \theta)$ do	$1/8 * d_o * \frac{(\theta - \sin\theta)}{(\sin 2\ \theta)}$
Box, $y < 0.99*d_o$	y * b	y * 2 + b	b	y
Box, $y \geq 0.99*d_o$	y * b	(y + b) * 2	b	y

Note: $\theta = \pi + 2 * (\arcsin((y-d_o/2)/(d_o/2))$

8.3.C. Minor Losses

There are three types of minor losses that must be considered in hydraulic calculations: losses at pipe junctions, losses due to bends, and losses at pipe transitions.

8.3.C.1. *Junction Losses, Without a Manhole or Junction Box*

Junction losses, losses incurred when a lateral or trunk line flows into a trunk line, without the use of a manhole or junction box, shown in Figure 8-1, shall be computed using the following equation:

$$Hj = \frac{(Q_o * V_o) - (Q_i * V_i) - (Q_l * V_l \cos\theta)}{0.5 * g * (A_o + A_i)} + h_i - h_o \quad \text{(Eq. 8-6)}$$

where:
- Hj is the head loss incurred in junction (ft)
- Q_o, Q_i, and Q_l is the outlet, inlet, and lateral flows respectively, (ft^3 /sec)
- V_o, V_i, and V_l is the outlet, inlet, and lateral velocities respectively, (ft/sec)

h_o, h_i	is the outlet and inlet velocity heads respectively, (ft)
A_o, A_i	is the outlet and inlet cross-sectional areas respectively, (ft^2)
θ	is the angle between the inflow and outflow pipes
g	is the acceleration due to gravity (32.2 ft/sec^2)

8.3.C.2. *Junction Losses, With a Manhole or Junction Box*

Junction losses incurred when a lateral or trunk line flows into a trunk line, concurrent with the use of a manhole or junction box, shown in Figure 8-2, shall be computed using the following equations:

$$H_{ah} = K_j \, (V_o^2 / 2g) \qquad \text{(Eq. 8-7)}$$

Figure 8-1. Junction Losses without a Manhole or a Junction Box

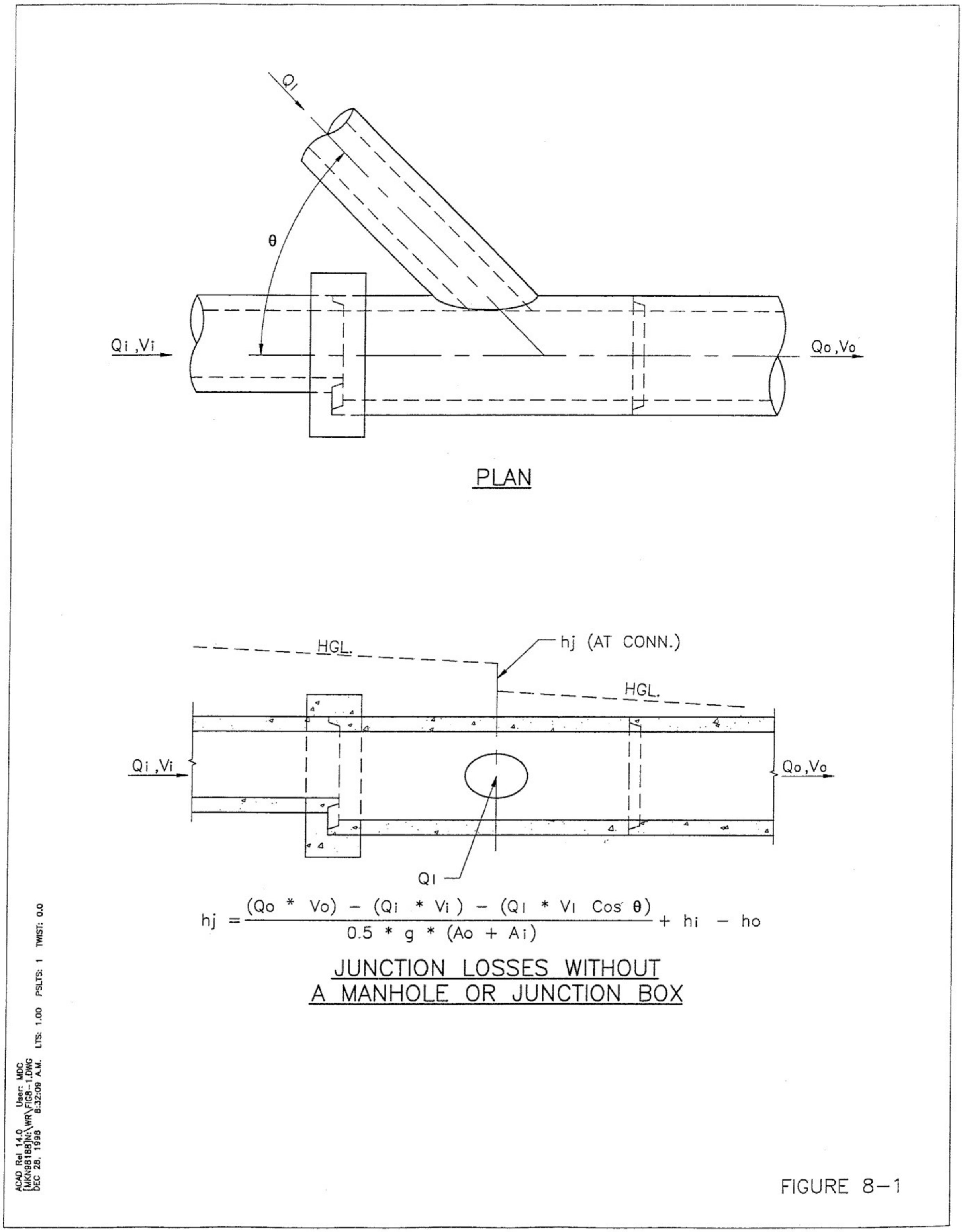
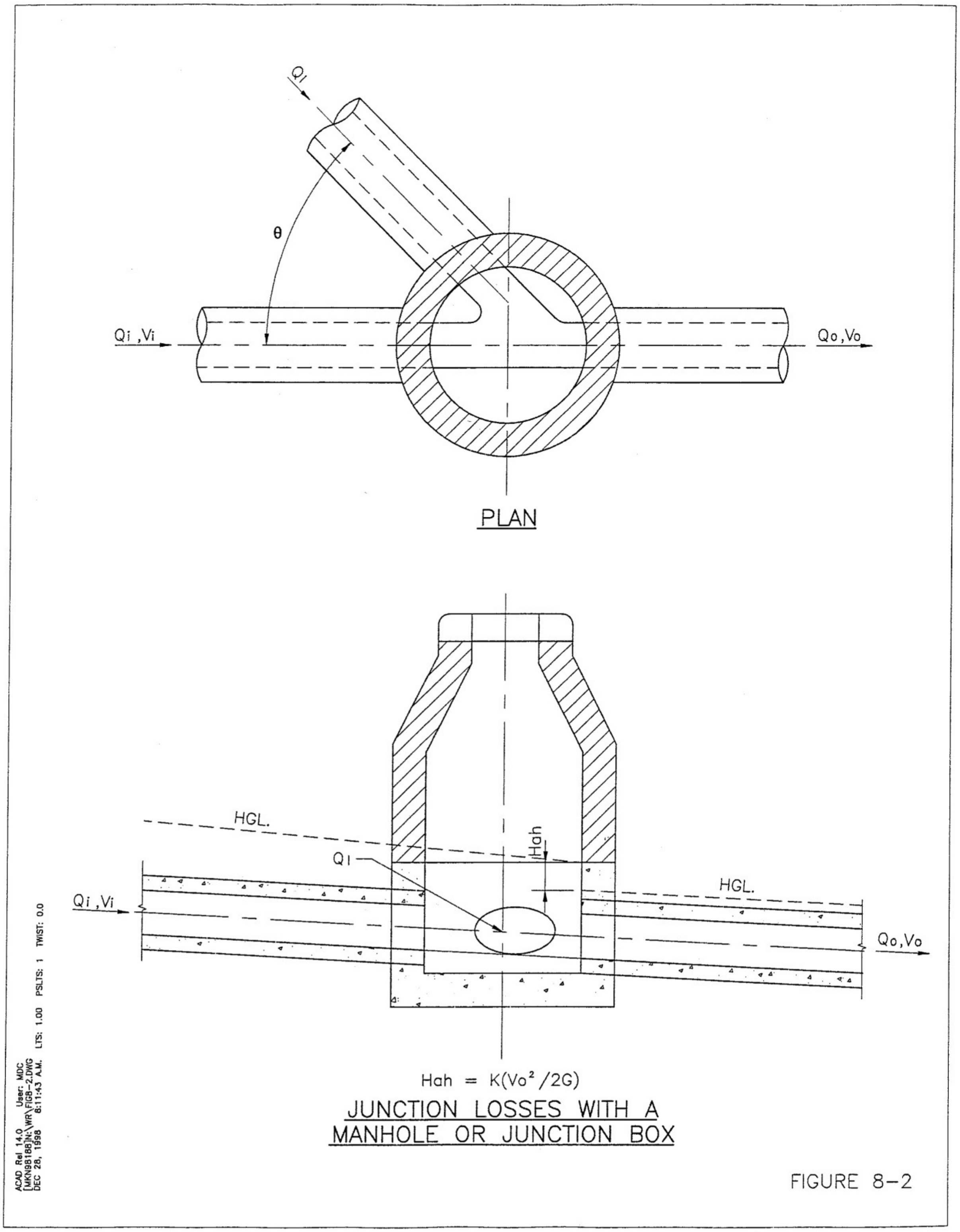


Figure 8-2. Junction Losses with a Manhole or a Junction Box



$$Kj = Ko * CD * CQ \quad \text{(Eq. 8-8)}$$

where: H_{ah} is the head loss incurred in the junction (ft)

V_o is the velocity of the flow in the outlet pipe (ft/sec)
g is the acceleration due to gravity (32.2 ft/sec^2)
K_j is the adjusted loss coefficient
K_o is the initial head loss coefficient based on relative access hole size
C_D is the correction factor for pipe diameter and pipe depth
C_Q is the correction factor for relative flow

The coefficient of head loss due to relative access hole size, K_o, is determined by the following equation:

$$K_o = 0.1 (b / D_o) * (1- \sin \theta) + 1.4 (b / D_o)^{0.15} \sin \theta \quad \text{(Eq. 8-9)}$$

where: K_o is the coefficient of head loss due to relative access hole size
b is the manhole or junction box diameter
D_o is the outlet pipe diameter (ft)
θ is the angle between the inflow and outflow pipes

A head loss due to pipe diameter is only significant in pressure flow, likewise head loss due to flow depth is only significant in non-pressure flow. Therefore, the coefficient of head loss for pipe diameter and pipe depth, C_D, is determined by the following equations:

If $d_{aho} / D_o > 3.2$ then $C_D = (D_o / D_i)^3$ (Eq. 8-10)

If $d_{aho} / D_o < 3.2$ then $C_D = 0.5 * (d_{aho}/ D_o)^{0.6}$ (Eq. 8-11)

where: C_D is the coefficient of head loss due to for pipe diameter and pipe depth
d_{aho} is the water depth in the access hole above the outlet pipe invert, (ft)
D_o is the outlet pipe diameter, (ft)
D_i is the inflowing pipe diameter, (ft)

A coefficient of head loss due to the flow relative to an incoming pipe, C_Q, is a function of the angle of the incoming flow as well as the ratio of the flow from the inflow pipe to the total outflow. The coefficient is given by the following equation:

$$C_Q = (1 - 2 \sin \theta) * (1- (Q_i / Q_o))^{0.75} + 1 \quad \text{(Eq. 8-12)}$$

where: C_Q is the coefficient of head loss due to the flow relative to an incoming pipe
Q_i and Q_o is the inlet and outlet flows respectively, (ft^3 /sec)
θ is the angle between the inflow and outflow pipes

If the flow line of the inflow lateral or trunk line is above the hydraulic grade line of the outflow pipe, then the initial depth is equivalent to the minimum depth, y_m, of the inflow pipe and is determined by the Equation 8-2 found in Section 8.3-A.

8.3.C.3. *Losses in a Bend*

The head loss at pipe bends is related to the velocity head and can be computed using the following equation:

$$h_b = K_a * K_r * V^2/2g$$

where:
- h_b is the head loss at the bend, (ft.)
- K_a is the bend loss coefficient due to the angle of the bend, Table 8-6
- K_r is the bend loss coefficient due to the diameter of the radius the bend is pulled , Table 8-6
- $V^2/2g$ is the velocity head using the velocity at the downstream end of the bend.

The coefficients K_a and K_r vary with the angle of the bend. Table 8-6 and Figure 8-3 contain different K_r coefficients used in bend losses calculations.

Table 8-6. Coefficients Of Loss Due To A Bend: K_a AND K_b

Bend Angle Degrees	Bend Loss Coefficient, K_a	Radius / Diameter	Bend Loss Coefficient, K_r
$\theta \leq 15^O$	-	R/D < 2	1.000
$15^O < \theta \leq 22.5^O$	0.20	2 ≤ R/D < 4	0.500
$22.5^O < \theta \leq 45^O$	0.35	4 ≤ R/D < 6	0.433
$45^O < \theta \leq 60^O$	0.43	6 ≤ R/D < 8	0.367
$60^O < \theta$	0.50	8 ≤ R/D < 20	0.300
		20 ≤ R/D	-

Note: Minimum radius of bend shall be specified by the manufacturer.

4. <u>Losses Due To Transitions (Sudden Expansion or Contraction)</u>

The head losses due to sudden enlargements and contractions are calculated using the same equation as for junction losses, Equation 8-7. The values for K_j for sudden enlargements and contractions are contained in Table 8-7.

Figure 8-3. Losses in a Horizontal Bend

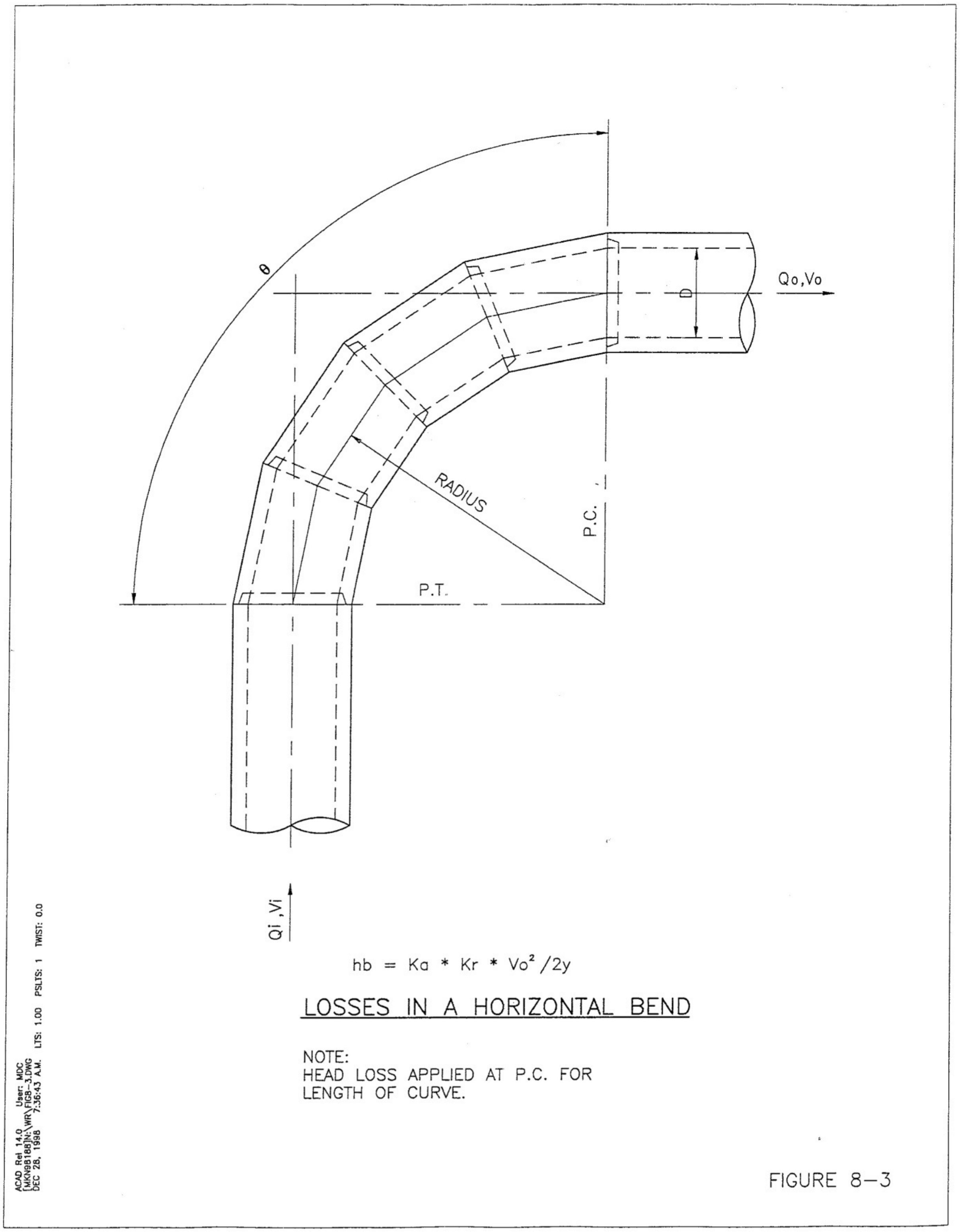


Table 8-7. Head Loss Coefficients Due To Sudden Expansion And Contraction

D_2/D_1*	Sudden Expansion K_j	Sudden Contraction K_j
1.2	0.10	0.08
1.4	0.23	0.18
1.6	0.35	0.25
1.8	0.44	0.33
2.0	0.52	0.36
2.5	0.65	0.40
3.0	0.72	0.42
4.0	0.80	0.44
5.0	0.84	0.45
10.0	0.89	0.46
4	0.91	0.47

*D_2/D_1 = Ratio of larger to smaller diameter

8.4 Hydraulic Grade Line Computation Sheet

In order to facilitate computations required in determining the hydraulics of a drainage system, Computation Sheet No. 8-1 has been prepared or a similar spreadsheet should be used. Notes regarding Sheet 8-1 are as follows:

Column 1: Design point, that is, the first junction point upstream

Column 2: Junction point immediately downstream of design point

Column 3: Distance between the design point in Column 1 and the design point in Column 2

Column 4: Incremental drainage sub-area size contributing to the design point in Column 1, from Computation Sheet 7-1, Col. 26.

Column 5: Total drainage sub-area size contributing to the design point in Column 1

Column 6: Weighted runoff coefficient for incremental drainage sub-area

Column 7: Column 4 times Column 6

Column 8: Total CA for all drainage sub-areas contributing to the design point in Column 1

Column 9: Time of concentration to the design point in Column 1

Column 10: For the upstream most storm drain, this value will be the same as Column 9, for all others, the value will be the sum of Column 10 and Column 17 from the previous run

Column 11: Rainfall intensity determined by using the larger of the two times of concentration in Columns 9 and 10 and the IDF coefficients found in Section 4.

Column 12: Total discharge, Column 8 times Column 11

Column 13: Pipe size using the criteria found in Section 8.2 and calculations found in Section 8.3, pipe should be sized as close as possible to full gravity flow

Column 14a: Width of box culvert

Column 14b: Height of box culvert

Column 15: Full flow capacity of selected pipe (box) size

Column 16: Full flow velocity. Value is equal to pipe discharge divided by cross-sectional area of pipe (box)

Column 17: Design flow velocity. If pipe is flowing full, this value will be equal to that in Column 15. If the pipe is not flowing full, the velocity can be determined from Eq. 8-4 or by Chart 26 in HEC-22.

Column 18: Travel time in the pipe, Column 3 divided by Column 16

Column 19: Upstream invert elevation (including changes in pipe diameter)

Column 20: Downstream invert elevation

Column 21: Approximate crown drop, from Eq. 8-7 and Computation Sheet 8-2 Col.27

Column 22: Friction slope, from Eq. 8-5

Complete the table in Computation Sheet 8-1 including each run of pipe in the system. Check the design by calculating the hydraulic grade line using Computation Sheet 8-2 or similar spreadsheet. Each line in the computation sheet represents a junction or structure and its associated pipe. The calculations begin at the outfall and work upstream with each junction. Computation Sheet 8-2 is used to calculate HGL and EGL elevations and pipe and structure losses. In sub-critical flow, the losses are summed to determine upstream HGL levels. In the case of supercritical flow, pipe and manhole losses are not carried upstream. Should supercritical flow occur, the designer should advance to the next section upstream to determine the flow regime at that point. This process continues until the system returns to a subcritical flow regime. Before completing the columns in Sheet 8-2, the designer should determine a hydraulic grade line (HGL) at the outlet of the structure. Section 8.3-A describes the method for determining the starting tailwater elevation. Notes regarding Computation Sheet 8-2 are as follows:

Column1: Structure identification

Column 2: Pipe diameter, (ft)

Column 3: Discharge, (ft^3/s)

Column 4: Conduit length (ft)

Column 5: Pipe velocity (ft/s). If the pipe is flowing full, enter the full flow velocity from Column 15 in Computation Sheet 8-1. If the pipe is part full, refer to Eq. 8-4 or Chart 26 in HEC-22 to find the velocity of the flow.

Column 6a: Depth of flow, determined by an iterative procedure using Eq. 8-3 or by using Chart 26 in HEC-22.

Column 6b: Critical depth, from Eq. 5-2 or Chart 27 in HEC-22. From this, determine the state of flow in the conduit. If the depth of flow is less than critical depth, the flow is supercritical. If the depth of flow is greater than critical depth, the flow is subcritical.

Column 7: Velocity Head, calculated using velocity from Column 5. If the flow regime is supercritical, a zero is entered for the value. If the flow is supercritical, Columns 1-7 must be filled out for the next structure upstream before advancing to Column 8. If the flow is supercritical for the next section upstream as well, place zeros in Columns 11,12, and 27. Compute the HGL at that section to be the outlet invert plus the outlet pipe flow depth and place that value in Column 14 of the previous structure line. Fill in columns 15 and 16 and then repeat the procedure until the flow is subcritical.

Column 8: Friction slope of the pipe. If pipe is flowing full, S_f is computed by Eq. 8-5. If the pipe is not flowing full, the friction slope is set equal to the pipe slope.

Column 9: Total pipe losses, from Column 22

Column 10: Energy grade line at the upstream end of the pipe, computed as the HGL from the previous structure (Column 14) plus the total pipe losses (Column 9) plus the velocity head (Column 7)

Column 11: Adjusted loss coefficient, from Column 27

Column 12: Total structure losses, Column 11 times Column 7

Column 13: Energy grade line at the structure, Column 12 plus column 10

Column 14: Hydraulic grade line at structure, Column 13 minus Column 7

Column 15: Top of conduit value, Col. 2 + Computation Sheet 8-1 Col. 20

Column 16: Ground surface or top of grate elevation at the structure

Column 17: Friction losses, Column 4 times Column 8

Column 18: Bend losses, from Eq. 8-13

Column 19: Head loss due to a contraction, obtained from Eq. 8-7 using the coefficients contained in Table 8-7

Column 20:	Head loss due to an expansion, obtained from Eq. 8-7 using the coefficients contained in Table 8-7
Column 21:	Head loss due to a junction, computed using either Eq. 8-6 or Eq. 8-7
Column 22:	Total pipe losses, Col. 21 + Col. 20 + Col.19 + Col. 18 + Col. 17
Column 23:	Depth of water in access hole, Column 10 minus Column 7 minus the pipe invert from Computation Sheet 8-1.
Column 24:	Initial head loss coefficient based on relative access hole size, from Eq. 8-9
Column 25:	Correction factor for pipe diameter and flow depth, from Eq. 8-10 or 8-11
Column 26:	Correction factor for relative flow, from Eq. 8-12
Column 27:	Adjusted loss coefficient, from Eq. 8-8

Computation Sheet 8-1. Conduit Computation Sheet 1of 2

Conduit Computation Sheet 1 of 2

Project Description: ____________________

____

____

Project No.: ____________

Date: ____________

By: ____________

Checked By: ____________

manning's n = 0.016

Freq. Yr.	b	d	e
2	90.62	15.71	0.901
5	75.42	14.48	0.807
10	73.00	14.03	0.768

Freq. Yr.	b	d	e
25	73.92	13.64	0.733
50	75.85	13.43	0.712
100	78.37	13.28	0.696

1 Station of Juction or Des. Point	2 Length to U.S. Calc. Point	3 Remarks	4 Contributing Inlet or Line	5 C of Contributing Line	6 Area of Contributing Line	7 Weighted C	8 Total Area	9 Time of Concentration: Contributing Line	10 Time of Concentration: Total	11 Freq. Year	12 Rainfall I (in./hr) b/(Tc+d)^e	13 Flow Q C*I*A (c.f.s.)	14 Pipe Size Dia. or H (ft.)	15 Width (If Box) (ft.)	16 Slope %	17 Capacity Full (c.f.s.)	18 Velocity: Full (ft./sec)	19 Velocity: Design (ft./sec)	20 Section Time (min.)	21 d (ft.)	22 dc (ft.)	23 V^2 / 2g	24 Sf %

Computation Sheet 8-1

Computation Sheet 8-2. Conduit Computation Sheet 2 of 2

Conduit Computation Sheet 2 of 2

Project Description: ______________

Project No.: ______________ Date: ______________

By: ______________ Checked By: ______________

manning's n = 0.013

	Receiving Stream or Line	Line
W.S	________	________
F.L.	________	________

1	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
Station of Juction or Des. Point	d (ft.)	dc (ft.)	V^2 / 2g	Sf %	Flow Line		Top of Pipe		EGLo ft. msl	K	K(V^2/2g)	EGLi ft. msl	HGL ft. msl	U.S. TOC ft. msl	Surface Elevaiton ft. msl	Hf	hb	Hc	He	Hj	d aho	Ko	Cd	Cq	K
					U.S. ft. msl	D.S. ft. msl	U.S. ft. msl	D.S. ft. msl																	

Computation Sheet 8-2

9.0 OPEN CHANNELS

9.1 Applicable Design Criteria

All channel improvements shall be in accordance with the San Angelo Master Drainage Plan, if applicable. Any variation from the Master Drainage Plan must be approved by the City Engineer. Open channels may be in the form of natural channels or they may be lined. Lining materials may include, but are not limited to: reinforced concrete, gabions, concrete segmental retaining walls, and interlocking concrete block.

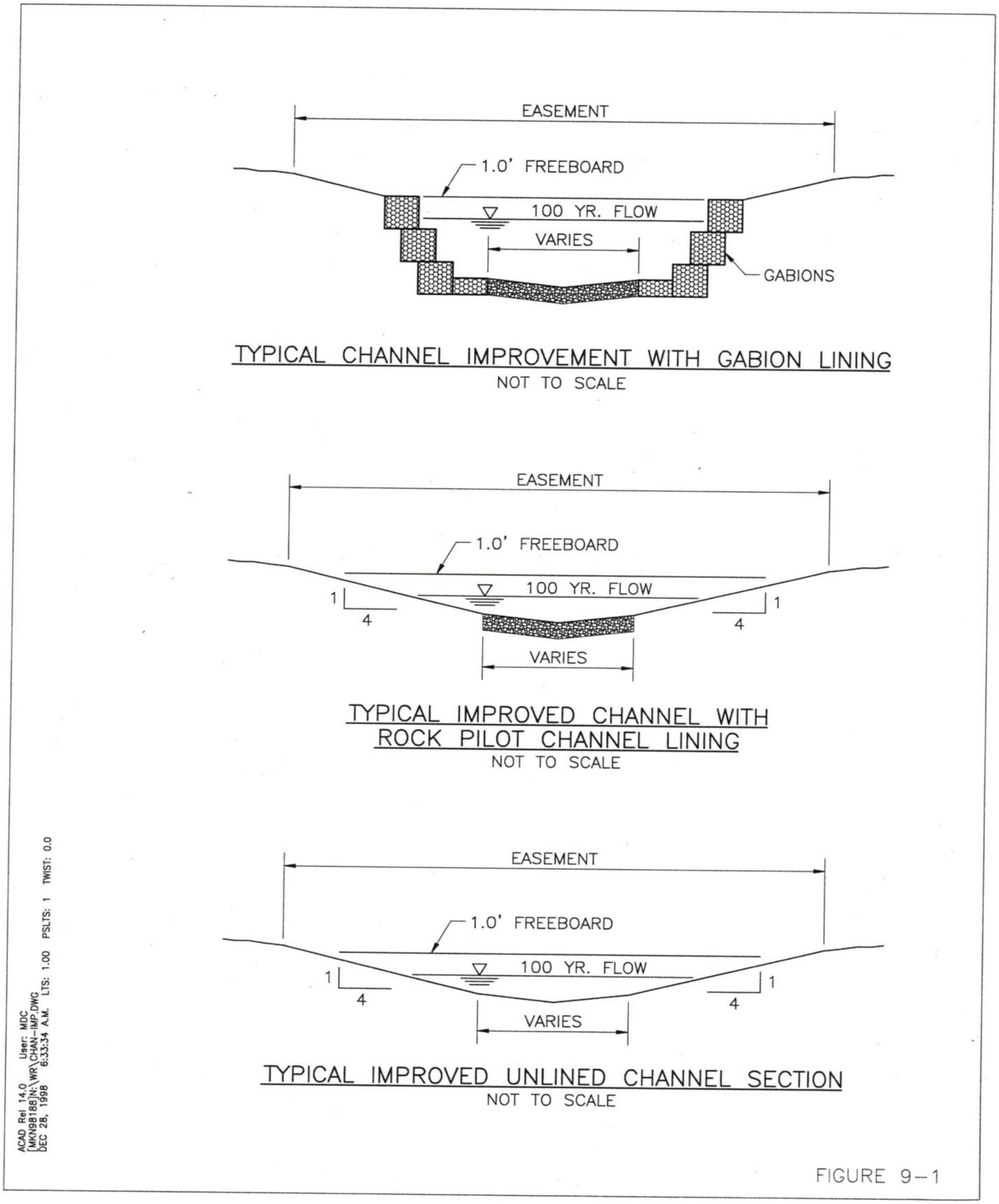
Due to recent changes in the environmental permitting review process, few sizable concrete lined channels are able to be constructed. Although concrete lined channels are allowed by the City of San Angelo, the design engineer is reminded that it may be extremely difficult to obtain the proper permits from the State and Federal authorities for concrete channel designs. Also, developers are responsible for acquisition of all regulatory agency permits.

The developer shall be responsible for the channel modifications, as required by State and Federal agencies. Clearing of debris, small trees, brush, vines, etc. from floodways and floodplains of channels shall be the responsibility of the developer as allowed by the current permitting requirements. In addition, the developer may dedicate the floodway and/or floodplain as a deed-restricted greenbelt area.

All channels shall be designed to carry the 100-year storm event, based on ultimate watershed development, and shall have one foot of freeboard as illustrated in Figure 9-1. Channels that require concrete lining shall be lined up to an elevation of the water surface resulting from the 100-year storm event, based on ultimate watershed development. Freeboard along the outside of channel bends shall include the increased water surface due to super-elevation (refer to Section 9.3).

Unlined channels that contain bends shall be designed such that erosion at the bends is minimized. Erosion protection at bends shall be determined based on the velocity along the outside of the channel bend (refer to Section 9.3). Unlined channels shall have side slopes no steeper than 4:1 and lined channels shall have side slopes no steeper than 2:1, unless authorized by the City Engineer. A soil stability analysis prepared by a licensed geotechnical engineer may be required to determine the maximum slope that the soil at the channel improvement site will sustain without failure. Roadside ditches, if used, shall be designed to carry the 100-year storm event within the right-of-way or drainage easement.

Figure 9-1. Freeboard Requirements and Channel Section Illustrations



9.2 Design Parameters

Design flows in channels and through bridges, culverts, weirs, or other structures associated with a particular channel shall be based on the 100-year storm event. The procedures described in Section 5 shall be used to determine the design flows.

Channels, either lined or unlined, shall normally have a trapezoidal cross section. The channel section should have adequate flow area to account for the uncertainties in runoff estimates, seasonal changes in channel roughness coefficients, channel obstructions, and silt accumulations. Figure 9-1 illustrates the basic criteria defining shape and lining requirements for channels.

Where possible, channels should have sufficient gradient, depending upon the type of soil or channel lining material, to provide velocities that will be self-cleaning at the 2-year storm event but will not be so great as to create erosion. Maximum permissible velocities are shown in Table 9-1.

Table 9-1. Maximum Velocity In Open Channels

Channel Material	Maximum Flow Velocity, fps
Grass Lined Earthen	6.0
Rock (Native)	10.0
Gabion Lined	12.0
Reinforced Concrete Lining	20.0
Rock Riprap (Placed Rock)	Use U.S. Army Corps of Engineers Guidelines
Prefabricated Lining Products	Use 90 % of Manufacturer's Recommended Velocity Limits

*Sheet flow into channels and streams shall not exceed 6.0 feet per second

Appropriate energy dissipating structures may be used to control erosion due to high velocities at pipe system outfalls and steep grades and shall be designed in accordance with accepted design practices such as outlined by the Soil Conservation Service, the Corps of Engineers, the Bureau of Reclamation, or the Texas Department of Transportation. The design of energy dissipators may utilize a geotechnical investigation of the site.

9.3 Supercritical Flow

The Froude Number provides a relationship between flow velocity and the hydraulic depth of flow, and gravitational action and shall be calculated for all channel improvement designs. Subcritical flow conditions occur when the Froude number is less than 1.0 and supercritical flow conditions exist in lined channels when the Froude Number exceeds 1.0. The Froude number may be calculated by the following equations:

$$Fr = V / (g * D)^{0.5} \quad \text{(Eq. 9-1)}$$

where:
- V is velocity of flow, (ft./sec.)
- g is the acceleration due to gravity (ft./sec^2)
- D is the hydraulic depth (ft.)

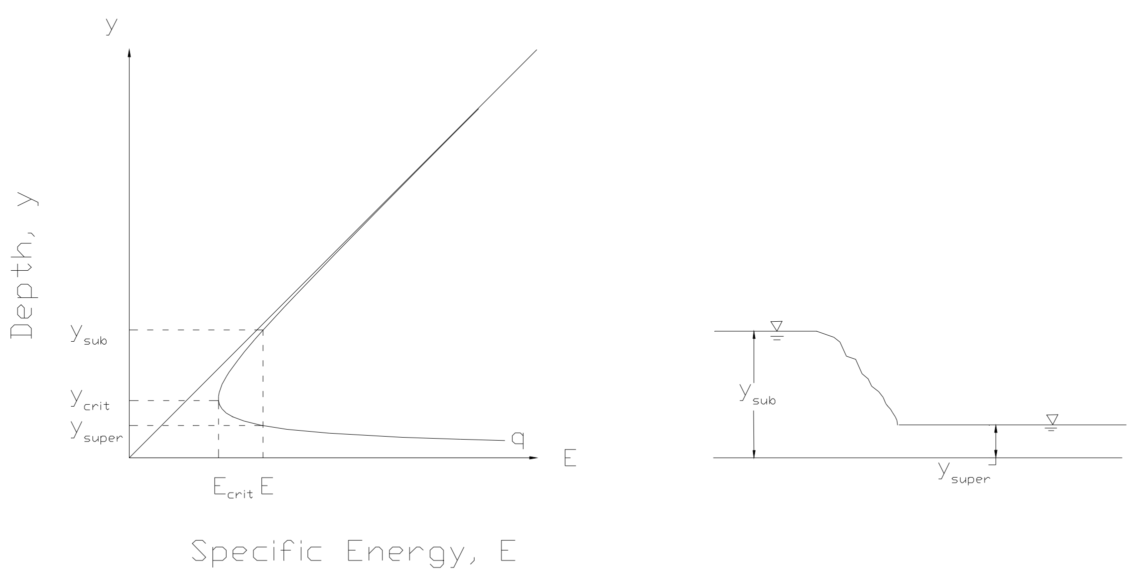
and

$$D = A / T \quad \text{(Eq. 9-2)}$$

where:
- A is the cross-sectional area of the flow (ft^2)
- T is the top width of the flow (ft.)

Each channel cross section has two flow depths, the normal depth and the alternate depth. Although the depths, velocities, and Froude Number differ, the specific energy of the two depths are equivalent. Figure 9-2 shows the relationship of specific energy to depth. If a channel's normal depth is supercritical, its alternate depth is a deeper subcritical depth. Obstructions that may enter a stream during a storm event may cause supercritical flows to experience a hydraulic jump and become subcritical flows. Due to this fact, channels that are designed for supercritical conditions must have freeboard equal to the alternate depth plus one foot of freeboard for design flow conditions. Supercritical flow must be contained in straight sections of the channel. No supercritical flow is allowed in bends. All channels in which supercritical flow occurs must be adequately reinforced.

Figure 9-2. Alternate Depths on the Specific Energy Curve.



$$\mathbf{E = V^2_{sub}/2g + y_{sub} = V^2_{super}/2g + y_{super}} \quad \text{(Eq. 9-3)}$$

Supercritical flow conditions are to be avoided if at all possible. Subcritical flow conditions are recommended for all channel designs in the City of San Angelo, as supercritical flow tends to have high velocities and high potential for channel erosion. Supercritical flow conditions will not be allowed in unlined channels. Subcritical flow conditions may be achieved by using energy dissipators in unlined channels in areas where the existing topography will not allow subcritical flow conditions to occur naturally. In all cases, the channel improvements shall be designed to avoid the unstable transitional flow conditions that occur when the Froude Number is between 0.9 and 1.1.

9.4 Flow in Bends

When a channel changes direction, the depth of flow along the outside edge of the curve is higher than the average channel flow depth, and the water surface is commonly referred to as superelevated. Therefore, additional freeboard must be provided to prevent the outside channel bank from being overtopped. The amount of super-elevation along the outside of the bend can be estimated using Equation 9-4:[20]

$$\Delta H = \frac{C^2}{2g r_o^2 r_i^2} \left(r_o^2 - r_i^2\right) \qquad \text{(Eq. 9-4)}$$

where:

- ΔH is the increase in water surface elevation along the outside of the channel bend due to super-elevation in feet.
- C is the circulation constant (ft^2/sec).
- r_o is the outside radius of the channel bend in feet.
- r_i is the inside radius of the channel bend in feet.
- g is the acceleration due to gravity (32.2 ft/sec).

If the discharge, depth of flow at the approach to the bend, average flow velocity in the approach to the bend, and the inner and outer radii of the bend are known, the value of the circulation constant can be approximated by solving Equation 9-5 for C :

$$Q = C \left[y_a + \frac{V_a^2}{2g} - \frac{C^2}{2g r_o r_i} \right] \ln\left(r_o / r_i \right) \qquad \text{(Eq. 9-5)}$$

where:

- Q is the total flow in the channel in cubic feet per second.
- V_a is the average velocity in the approach to the bend in feet per second.
- y_a is the depth of flow in the approach to the bend in feet.

The flow velocity along the outside of the bend, V_o (in feet per second), can then be approximated by:

$$V_o = C / r_o \qquad \text{(Eq. 9-6)}$$

V_o shall not exceed the maximum values established in Table 9.1.

9.5 Drop Structures

The function of a drop structure is to reduce flow velocities by dissipating some of the kinetic energy of the flow at the drop structure, and also providing flatter channel slopes upstream and downstream of the drop structure. Sloping channel drops and vertical channel drops are two commonly used drop structure types. The flow velocities in the channel upstream and downstream of the drop structure shall satisfy the permissible velocities allowed for channels (Table 9-1). The velocities shall be checked for flows produced by the 2- and 100-year storm events.

An apron shall be constructed immediately upstream and downstream of a drop structure to protect against the increasing velocities and turbulence. The upstream apron shall extend at least ten feet upstream from the point where flow becomes supercritical and shall include a concrete toe into the ground. The downstream apron shall extend a minimum of twenty feet beyond the anticipated location of the jump and shall include a concrete toe into the ground. The toe at each end shall extend a minimum of twenty-four inches into the ground to minimize scour at the transition to natural ground.

The design of drop structures shall be based on the height of the drop, the normal depths upstream and downstream of the drop structure, and the flow rate. All drop structures shall be constructed of reinforced concrete, and the bottom and walls (if any) shall have a minimum thickness of six inches. To facilitate maintenance, drop structures should be located near bridges or culverts if possible, as directed by the City Engineer.

9.5.A. Vertical Drop Structures

The drop structure should have sufficient height to stabilize the hydraulic jump. The drop length and the hydraulic jump length of the drop structure should be calculated to determine the length of the downstream apron required to prevent erosion.[21,22] In order to utilize a vertical drop structure, vehicular access must be provided to both the upstream and downstream ends of the structure.

9.5.B. Sloping Drop Structures

The location of the hydraulic jump should be determined based on the upstream and downstream flow depths and channel slopes.[21,22] The length of the hydraulic jump should be calculated to determine the length of the downstream apron required to prevent erosion. When utilizing a sloping drop structure, a minimum slope of 6:1 shall be used to allow vehicular access from one end across the structure. If the slope of the drop structure is less than 6:1, vehicular access must be provided to both the upstream and downstream ends of the structure.

9.6 Flumes

Flumes may be used to carry runoff from streets and alleys to earthen channels provided that adequate erosion protection is provided. A flume may be used to direct overflow runoff along property lines until the runoff can be intercepted by streets, alleys or storm drains. Flumes shall be contained in a dedicated drainage easement with sufficient width to allow future maintenance accessibility.

9.7 Standard Step Backwater Computation Sheet

Water surface profiles for the design frequency floods shall be computed for all channels and shown on all final drawings. The Standard Step Method for Backwater Calculations shall be used to determine water surface profiles for steady uniform flow equal to the design discharge. Computation Sheet 9-1 has been provided for summarizing standard step backwater computations. The Corps of Engineers HEC-2 or HEC-RAS Water Surface Profile Programs may also be used to perform standard step backwater calculations, and if used, a summary table consistent with Computation Sheet 9-1 shall be submitted to the City. Losses due to changes in velocity, drops, bridge openings, and other obstructions shall be considered in the backwater computations, as described in the HEC-2 and HEC-RAS User's Manuals.

Computation of the water surface profile shall be based on the Standard Step Method, which essentially consists of the solution of the energy equation between two points along the channel. For natural channels, the energy slope S_e, is approximated by the friction slope, S_f, which is calculated by rearranging Manning's equation, as shown in Equation 9-7. A table of Manning's (n) roughness coefficients is found in Table 9-2.[21] Referring to Figure 9-3 and writing equations between points one and two, the Standard Step Method for determining water surface profiles can be performed using Computation Sheet 9-1.

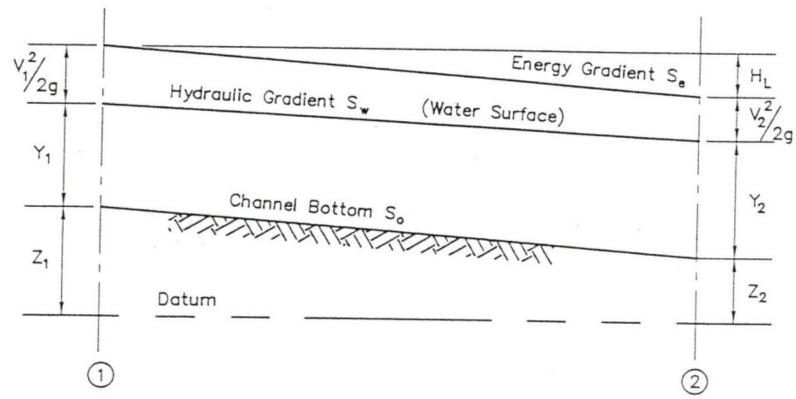
$$S_f = \frac{n^2 V^2}{2.22\ R^{4/3}} = S_e \qquad \text{(Eq. 9-7)}$$

where:

- S_f is the friction slope (ft./ft.)
- S_e is the energy slope (ft./ft.)
- n is Manning's n coefficient, see table 9-2
- V is the average velocity of flow in the channel (ft./sec.)
- R is the hydraulic radius which is the cross sectional area (A) divided by the wetted perimeter (P)

Figure 9-3. Standard Step Backwater Method for Determining Water Surface Profiles

STANDARD STEP BACKWATER METHOD FOR DETERMINING WATER SURFACE PROFILES



Y = Depth Of Flow, Ft.
V = Velocity Of Flow, Ft./Sec.
S_o = Invert Channel Slope, Ft./Ft.
S_e = Energy Gradient Slope, Ft./Ft.
S_w = Hydraulic Gradient Slope Ft./Ft.

H = Energy Losses, Ft.
$V^2/2g$ = Velocity Head, Ft.
Z = Distance Above Arbitrary Datum, Ft.

$$V = \frac{1.486}{n}(R^{2/3})(S^{1/2}) \quad \text{Eq. 1}$$

$$S_oL + Y_1 + V_1^2/2g = Y_2 + V_2^2/2g + H_L \quad \text{Eq. 2}$$

$$W_1 = S_oL + Y_1 + Z_2 \quad \text{Eq. 3}$$

$$W_2 = Y_2 + Z_2 \quad \text{Eq. 4}$$

$$H_L = S_eL = 1/2\,(S_{e_1} + S_{e_2})L = \bar{S}_eL \quad \text{Eq. 5}$$

$$S_e = \frac{(n^2)(V^2)}{2.22(R^{4/3})} \quad \text{Eq. 6}$$

$$H_1 = W_1 + V_1^2/2g \quad \text{Eq. 7}$$

$$H_2 = W_2 + V_2^2/2g \quad \text{Eq. 8}$$

$$H_1 = H_2 + H_L \quad \text{Eq. 9}$$

FIGURE 9-3

Table 9-2. Manning's (n) Roughness Coefficients [20]

Type of channel description	Minimum	Normal*	Maximum
Concrete or asphalt streets	0.012	0.016	0.20
Reinforced concrete pipes	0.011	0.013	0.020
Corrugated metal pipes	0.017	0.024	0.030
Concrete lined channel	0.011	0.013	0.015
Dry rubble or riprap lined channel	0.023	0.033	0.036
Gabion basket/mattress lined channel	0.025	0.030	0.035
Earth channel maintained,			
straight and uniform with short grass and few weeds	0.022	0.027	0.033
winding with grass and some weeds	0.025	0.030	0.033
winding with earth bottom and rubble sides	0.028	0.030	0.035
winding with cobble bottom and clean sides	0.030	0.040	0.050
Earth channel not maintained, weeds and brush uncut,			
dense weeds, high as flow depth	0.050	0.080	0.120
clean bottom, brush on sides	0.040	0.050	0.080
dense brush, high stage	0.080	0.100	0.140
Flood plains			
pasture short grass	0.025	0.030	0.035
pasture high grass	0.030	0.035	0.050
cultivated area with mature row crop	0.025	0.035	0.045
cultivated are with mature field crop	0.035	0.040	0.050
scattered brush with heavy weeds	0.035	0.050	0.070
light brush and trees	0.035	0.055	0.080
medium to dense brush	0.045	0.085	0.160
dense willows	0.110	0.150	0.200
cleared land with tree stumps, no sprouts	0.030	0.040	0.050
heavy stand of trees with few down trees, little undergrowth flood stage below branches	0.080	0.100	0.120
heavy stand of trees with few down trees, little undergrowth flood stage above branches	0.100	0.120	0.160

Note: The values under the column marked Normal* shall be used in all design in the City of San Angelo, unless detailed analysis, approved by the City Engineer, proves otherwise.

The columns of Sheet 9-1 are as follows:

Column 1	Cross-section identified by station number.
Column 2	Invert flow line elevation of channel or stream, (ft. m.s.l.).
Column 3	Design discharge Q, (cfs)
Column 4	Assumed depth of flow, Y (ft.).
Column 5	Water surface elevation, Col. 2 + Col. 4, (ft. m.s.l.).

Column 6	Area of flow at depth Y, (ft.^{2}).
Column 7	Mean velocity, design discharge (Col. 3) divided by area (Col. 6), (ft./sec.).
Column 8	Velocity head, (Col. 6) 2/2 * g (ft.).
Column 9	Elevation of the total head, Col. 5 + Col. 8, (ft.).
Column 10	Wetted perimeter of flow at depth Y, (ft.).
Column 11	Hydraulic radius, Col. 6 divided by Col. 10, (ft.).
Column 12	Hydraulic radius to the 4/3 power.
Column 13	Energy slope at section, (ft./ft.), (See Eq. 9-7.)
Column 14	Average energy slope between cross-sections, approximately equal to average of energy slope (Col. 13) and the energy slope of the previous cross-section. (See Eq. 9-7.)
Column 15	Length between cross-sections, (ft.).
Column 16	Friction loss between cross-sections. Col. 14 times Col. 15.
Column 17	Elevation of total energy head in feet. [This step equates the total energy head (H) from Col. 9 to the total energy head found by adding Col. 16 with Col. 17 of the previous cross-section. If the elevation obtained does not closely agree with the elevation in Column 9, then a new flow depth (Col. 4) must be assumed and the procedure repeated until agreement is obtained. When agreement is reached, proceed to the next cross-section.]

Computation Sheet 9-1. Standard Step Backwater Computation Sheet

Standard Step Backwater Calculations

Project Description:________________ Project No.: ________________ By: ________________

Date: ________________ Checked By: ________________ manning's n = 0.016

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Station	Invert Elevation (ft MSL)	Design Q (cfs)	Depth Y (ft)	Water Surface Elev, W (ft MSL)	Flow Area A (ft^2)	Flow Velocity V (ft/sec)	Velocity head $V^2/2g$ (ft)	Total Head Elev @ Sect 1 H_1 (ft MSL)	Wetted Perimeter P (ft)	Hydraulic Radius R (ft)	$R^{4/3}$	Energy Slope S_e (ft/ft)	Ave Energy Slope S_e (ft/ft)	Length L (ft)	Head Loss H_L (ft)	Total Head Elev @ Sect 2 H_2 (ft MSL)

Computation Sheet 9-1

9.8 Energy Dissipation

The outlets of pipes and lined channels are points of critical erosion potential. Storm water transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at storm water outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

Energy dissipators are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of storm water conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow. Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosion velocity of the downstream area channel system. Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence. For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

The Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, is recommended for the design procedures of energy dissipators. If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

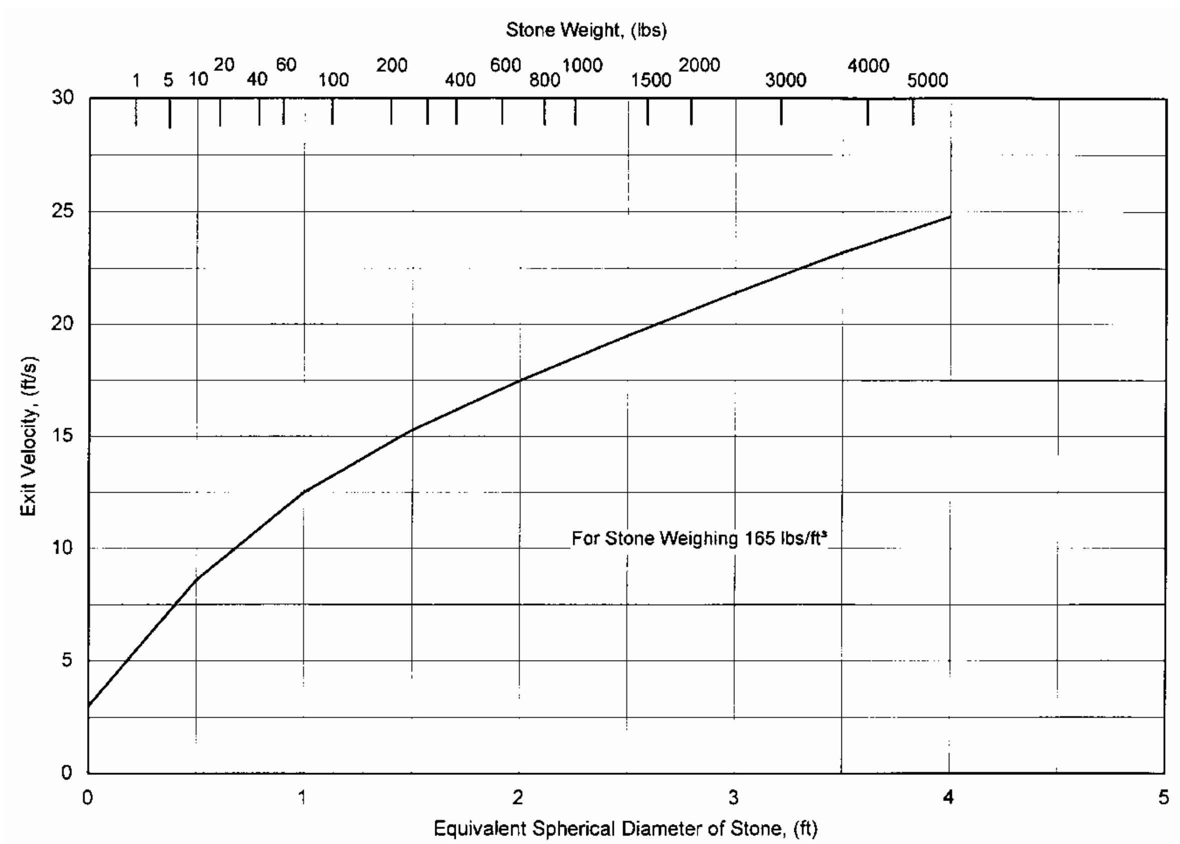
b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered. If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 9.4 provides the riprap size recommended for use downstream of energy dissipators.

Figure 9.4 Riprap Size for Use Downstream of Energy Dissipator



(Source: Searcy, 1967)

10.0 BRIDGE AND CULVERT DESIGN

10.1 Applicable Design Criteria

A. Bridges and culverts shall be designed to pass the 100-year storm event, at not more than the100-year street depth across the roadway, per design criteria in Table 5-1. All bridge and culverts shall be contained within a right-of-way or drainage easement.

B. Headwalls or mitered end treatments and necessary erosion protection shall be provided at all culverts and shall comply with the Texas Department of Transportation standards; or the design parameters in 10.2.

C. End treatments shall be provided at all culverts for vehicular safety and pedestrian safety, and shall comply with the Texas Department of Transportation standards; or the design parameters in 10.2.

D. Proposed reinforced concrete box culverts, bridges, and related structures may be adaptations of the Texas Department of Transportation standards.

10.2 Design Parameters

Where a proposed culvert crossing will eventually become part of a planned storm drainage system, the alignment, location and grade of the proposed culvert must be consistent with planned development of the drainage system for that watershed.

In the event the particular watershed or waterway is not covered by a planned storm drainage system, the designer should proceed with the design from the nearest downstream hydraulic control (i.e. bridge, culvert dam, etc.), as agreed upon with the City Engineer, and design the proposed drainage system improvements anticipating future system expansion due to fully developed watershed conditions.

Several hydraulic parameters should be considered in culvert and bridge design. These considerations include, but are not limited to the following:

(Information to be provided by city officials)

A. Channel transitions into and out of the bridge or culvert opening.

B. Overall length and height of bridge or culvert.

C. Cross-sectional opening of bridge or culvert.

D. Bridge alignment relative to general flow of main channel (i.e., is it a "skewed" crossing).

E. Number of crossings ("dual" or multiple bridges or culverts).

F. Other obstructions to flow (e.g., piers and abutments).

G. Design flows for bridge or culvert opening to pass design flow.

H. Any freeboard requirements for channel design.

I. Erosion protection at piers and abutments.

The most commonly used backwater program for modeling hydraulic conditions at existing or proposed bridge crossings are the HEC-2 and HEC-RAS programs developed by the U.S. Army Corps of Engineers.[22,23]

All headwalls shall be constructed of reinforced concrete. Wingwalls, if used, may be either straight-parallel, flared, or tapered. Approach and discharge aprons shall be provided for all culvert headwall designs. The guidelines listed in Table 10-1 are intended to aid in determining when to use various types of wingwalls.

Table 10-1. Guidelines For Wingwall Use

Conditions	Wingwall Type
Small culverts with flat slopes.	Straight (parallel), flared or tapered.
Abrupt change in flow direction is necessary.	Straight with one perpendicular wingwall (not recommended for large culverts) or flared.
Approach velocities below 6 fps, approach channel undefined, formation of backwater pools acceptable.	Straight, flared or tapered.
Approach velocities 6-10 fps, approach channel well defined.	Flared (wingwalls located with respect to axis of the approach channel)

*Wingwalls with concrete riprap at each end are acceptable options, assuming that the above conditions are met.

Precast headwalls and endwalls may be used if all other criteria are satisfied; generally precast headwalls/endwalls are available for smaller culverts (18 inches to 24 inches diameter).

The ends of culverts, headwalls and endwalls, shall be grated and marked with reflector posts as directed by the City Engineer.

10.3 Culvert Outlet Protection

High discharge velocities from culverts can cause eddies or other turbulence which could damage unprotected downstream channel banks and roadway embankments. To prevent damage from scour and erosion in these conditions, culvert outlet protection is needed.

The outlet protection should extend downstream to a point where non-erosive channel velocities are established in accordance with Table 9-1. The outlet protection should be placed sufficiently high on the adjacent banks to provide protection from wave wash under design flow conditions.

10.4 Culvert Hydraulics

The hydraulic design of culverts shall be based upon design guidelines set forth by the Texas Department of Transportation, the U.S. Department of Transportation, or other suitable material. Figures 10-1 through Figure 10-9 (24, 25) are provided as design guides and may be used to complete Computation Sheet 10-1. Table 10-2 contains the culvert entrance loss coefficients (K_e) for use with Figures 10-2, 10-4 and 10-6. The numbered columns of Computation Sheet 10-1 are as follows:

Column 1 Culvert description.

Column 2 Design discharge in cfs.

Column 3 Culvert size.

Column 4 Ratio of headwater depth to culvert height from Figure 10-1, 10-3 or 10-5.

Column 5 Headwater depth for inlet control conditions in feet, which is obtained by multiplying the culvert height times the value in Column 4.

Column 6 Entrance loss coefficient, which is based on the configuration of the culvert entrance.

Column 7 Head in feet between the upstream and downstream sides of the culvert, which is obtained from Figure 10-2, 10-4 or 10-6.

Column 8 Critical depth in feet. The critical depth of a culvert can be obtained from Figure 10-7 or 10-8, or the following:

Circular Conduits

$$Q^2/g = A^3/b$$

Where,

Q is the design discharge in cfs.
A is the flow area in sq. ft.
b is the water surface width in feet.
g is the acceleration due to gravity (32.2 ft/sec^2).

To find critical depth in a circular conduit, assume trial values of critical depth and then calculate the dependent variables and verify the above equation.

Rectangular Conduits

$$d_c^3 = Q^2 / (gw^2)$$

Where,

d_c is the critical depth in feet.
Q is the design discharge if cfs
w is the width of the conduit in feet.
g is the acceleration due to gravity (32.2 ft/sec^2)

Column 9 One half of the sum of the critical depth and the culvert height.

Column 10 The tailwater depth in feet at the culvert outlet.

Column 11	If the tailwater depth is greater than the culvert height, h_o equals the value in Column 10. If the tailwater depth is less than the culvert height, h_o is equal to the greater of the values in Columns 9 and 10.
Column 12	Elevation difference in feet between the upstream and downstream ends of the culvert, which is determined by multiplying the culvert slope (S_o) and the culvert length (L).
Column 13	Headwater depth in feet for outlet control conditions, which is determined by summing the values in Column 7 and Column 11 and subtracting the value in Column 12.
Column 14	Actual headwater depth in feet at the culvert, which is the greater of the values in Column 5 and Column 13.
Column 15	Outlet flow velocity in fps based on the depth of flow in the culvert.

Computation Sheet 10-2 should be used to summarize the bridge design parameters when the Corps of Engineers HEC-2 Water Surface Profile program is used to design a bridge or culvert.

Figure 10-1. Headwater Depth for Concrete Pipe Culverts with Inlet Control

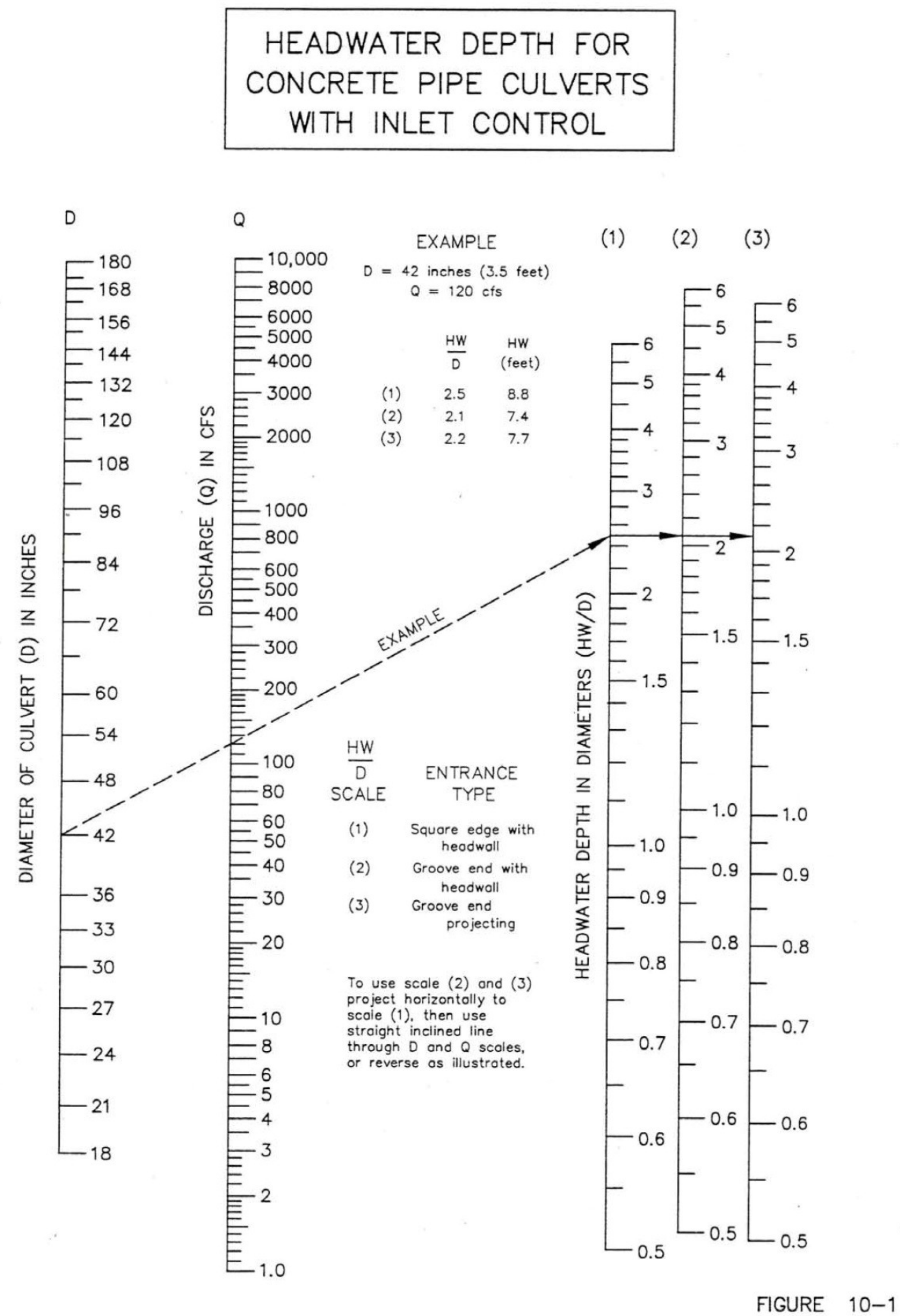


Figure 10-2. Head for Concrete Pipe Culverts with Outlet Control

HEAD FOR CONCRETE PIPE CULVERTS
WITH OUTLET CONTROL

n = 0.012

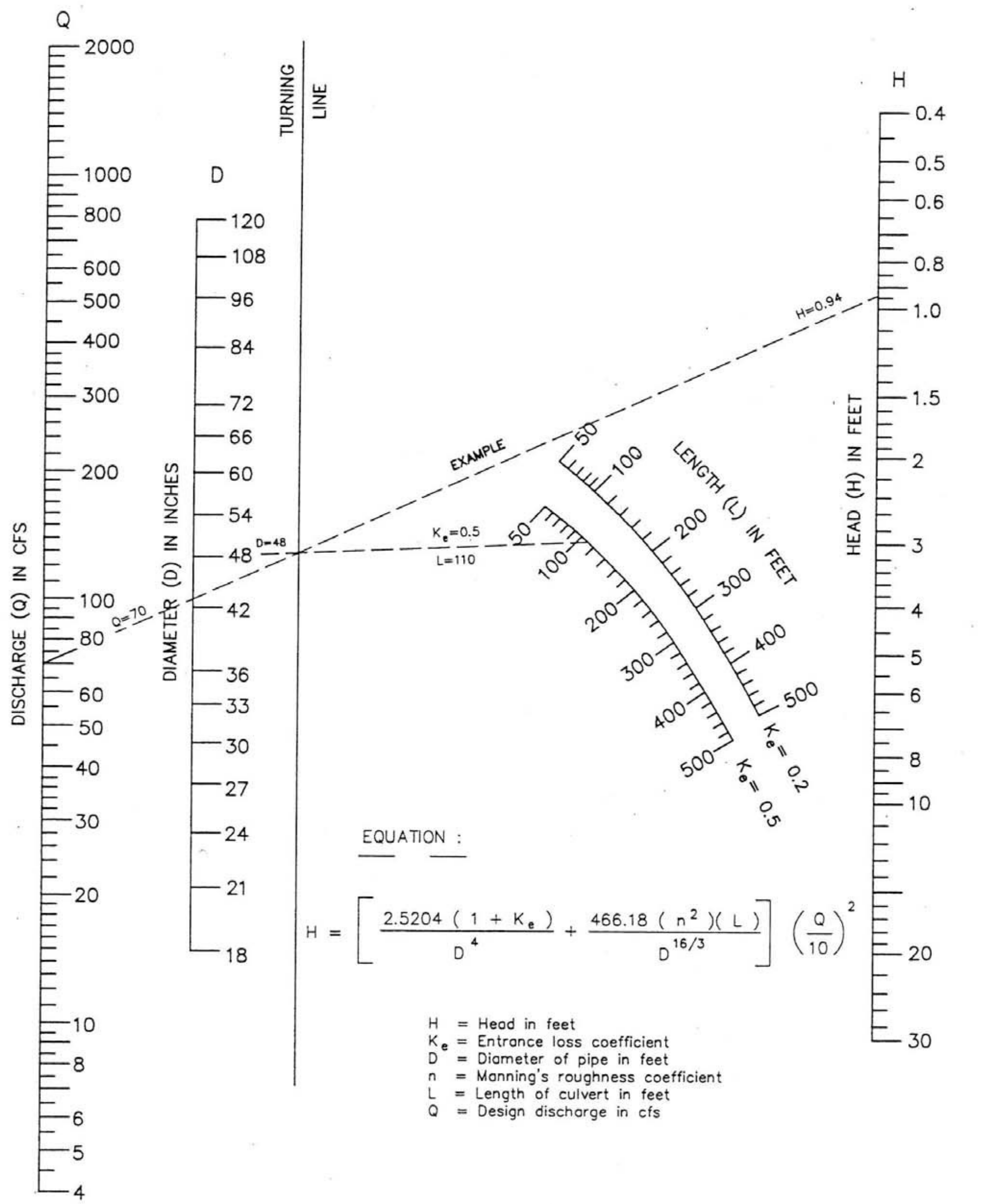


FIGURE 10–2

Figure 10-3. Headwater Depth for C.M. Pipe Culverts with Inlet Control

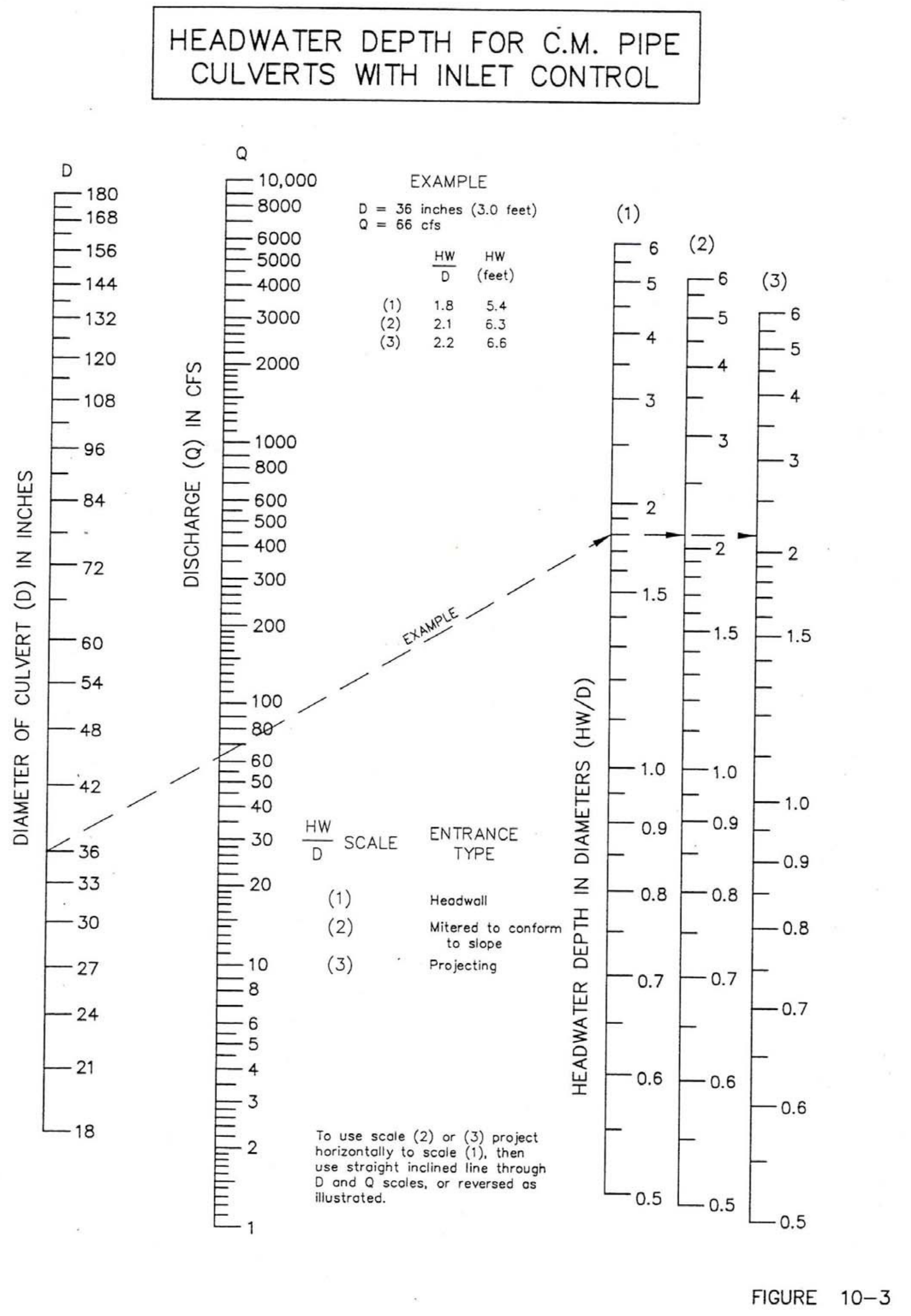


Figure 10-4. Head for C.M. Pipe Culverts with Outlet Control

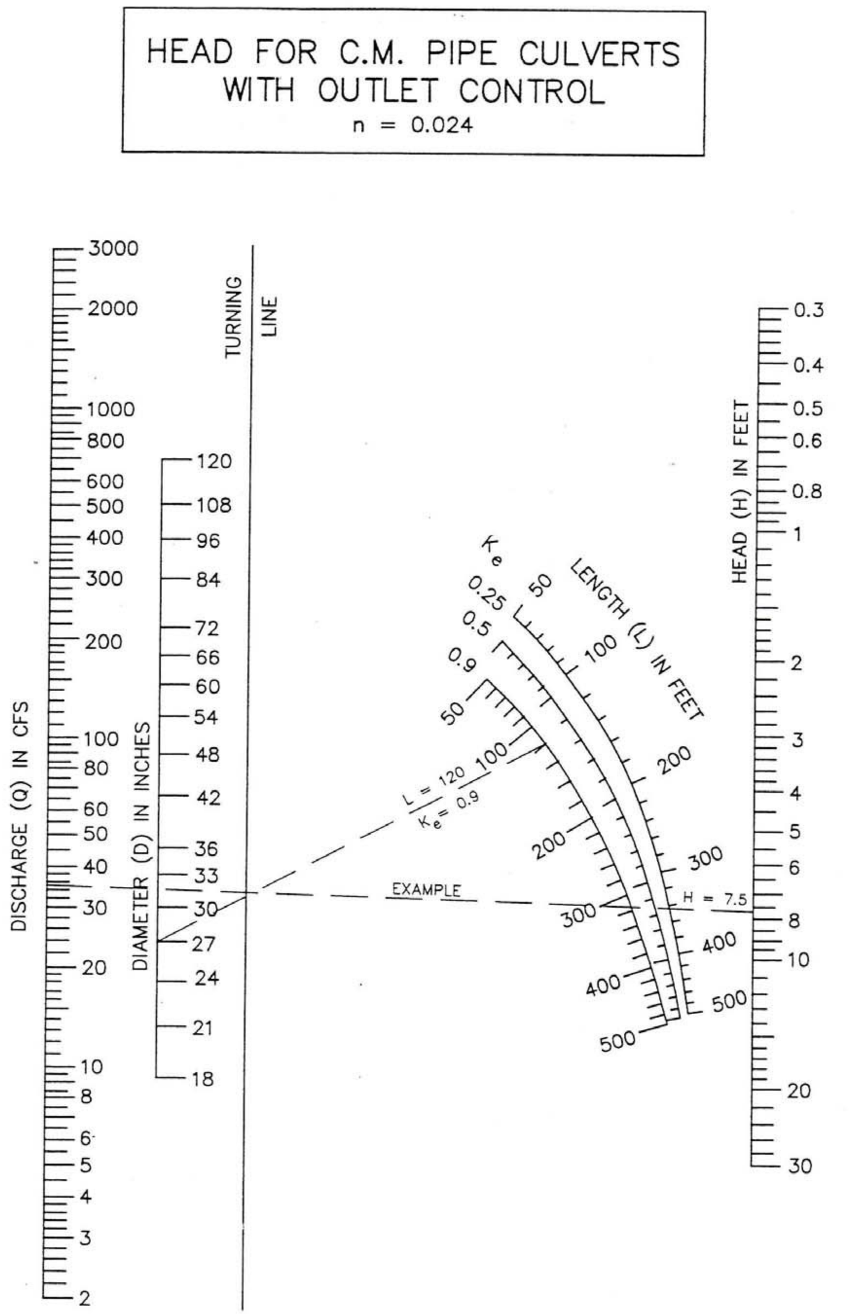


FIGURE 10–4

Figure 10-5 Headwater Depth for Box Culverts with Inlet Control

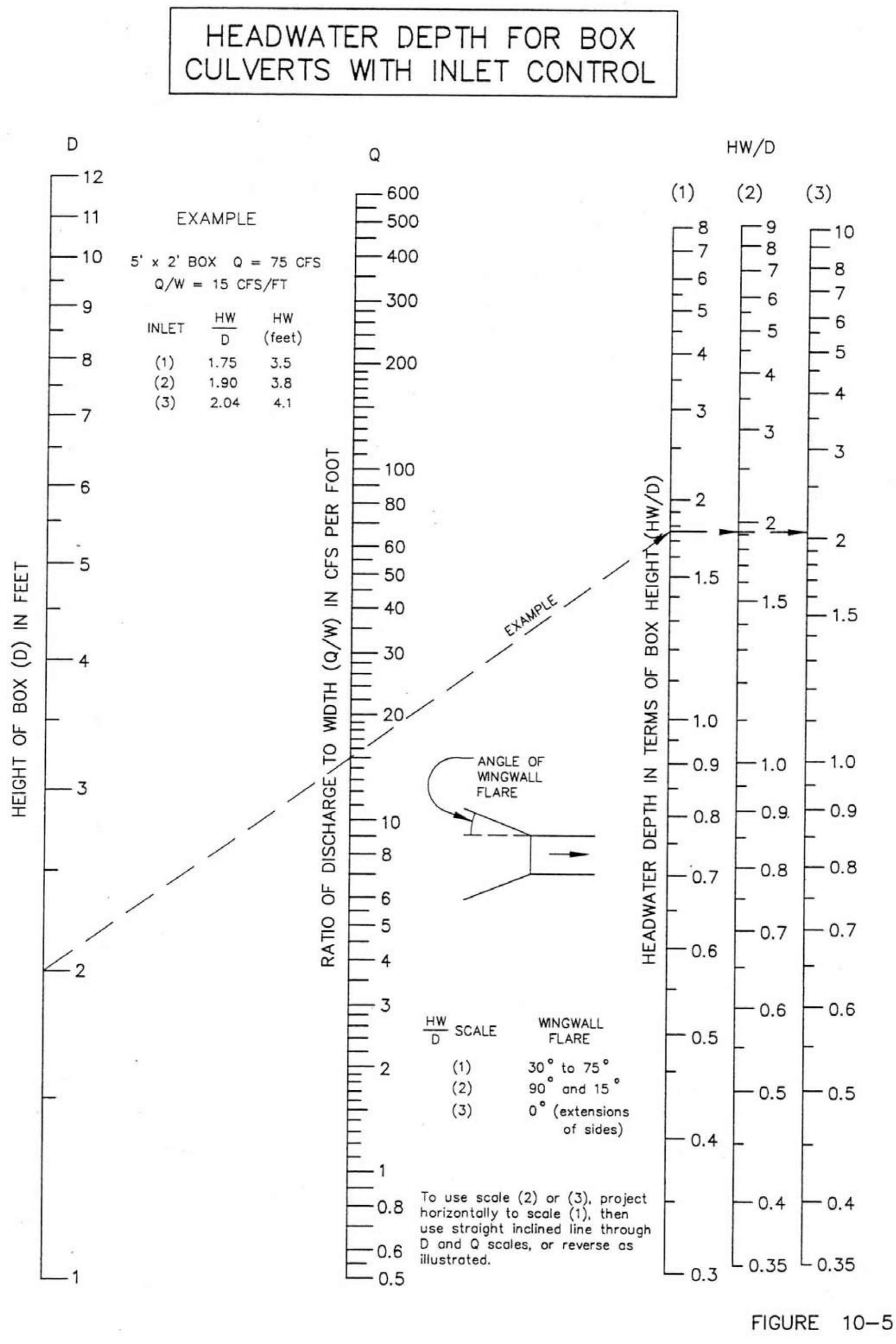


Figure 10-6. Head for Concrete Box Culverts with Outlet Control

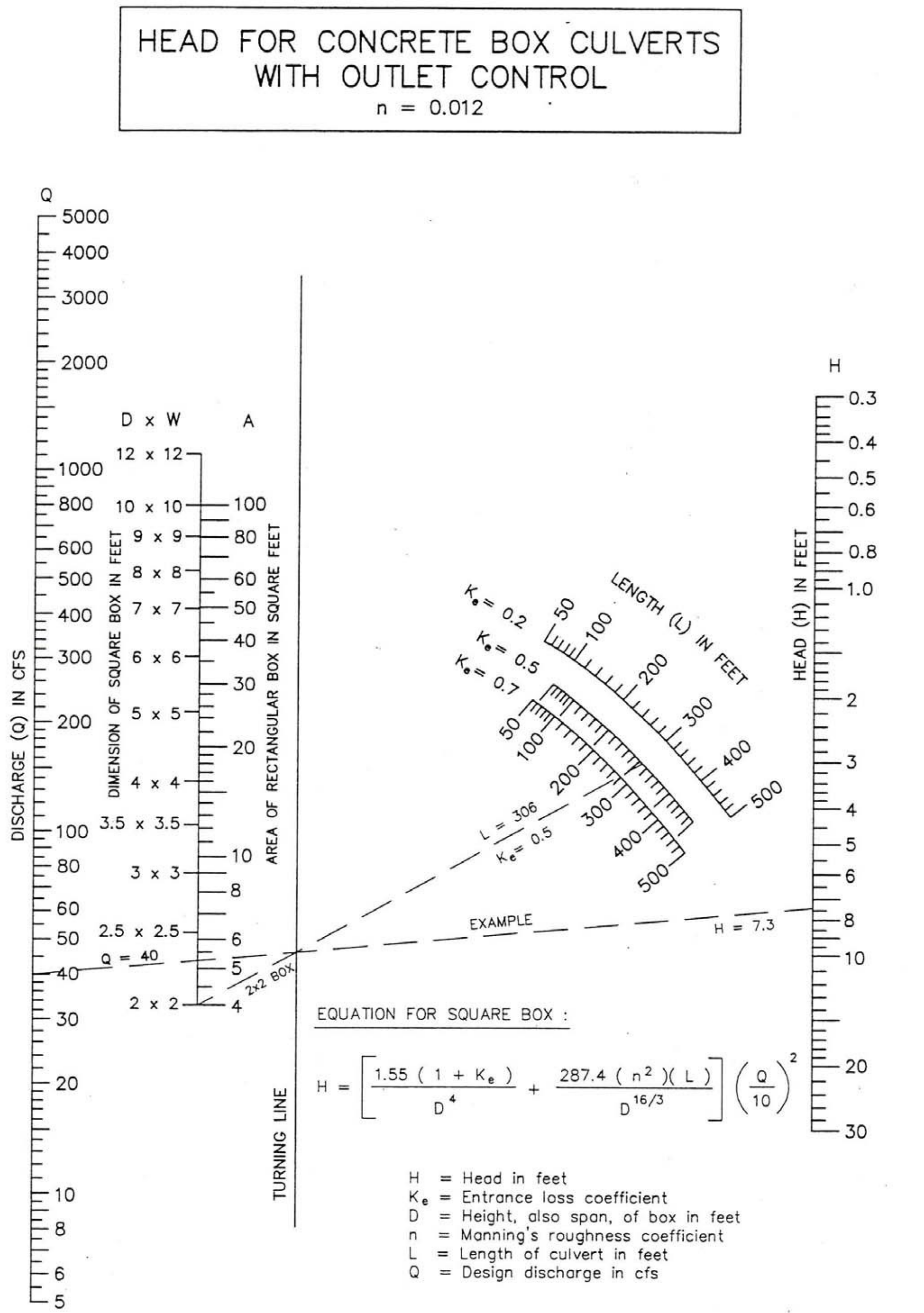
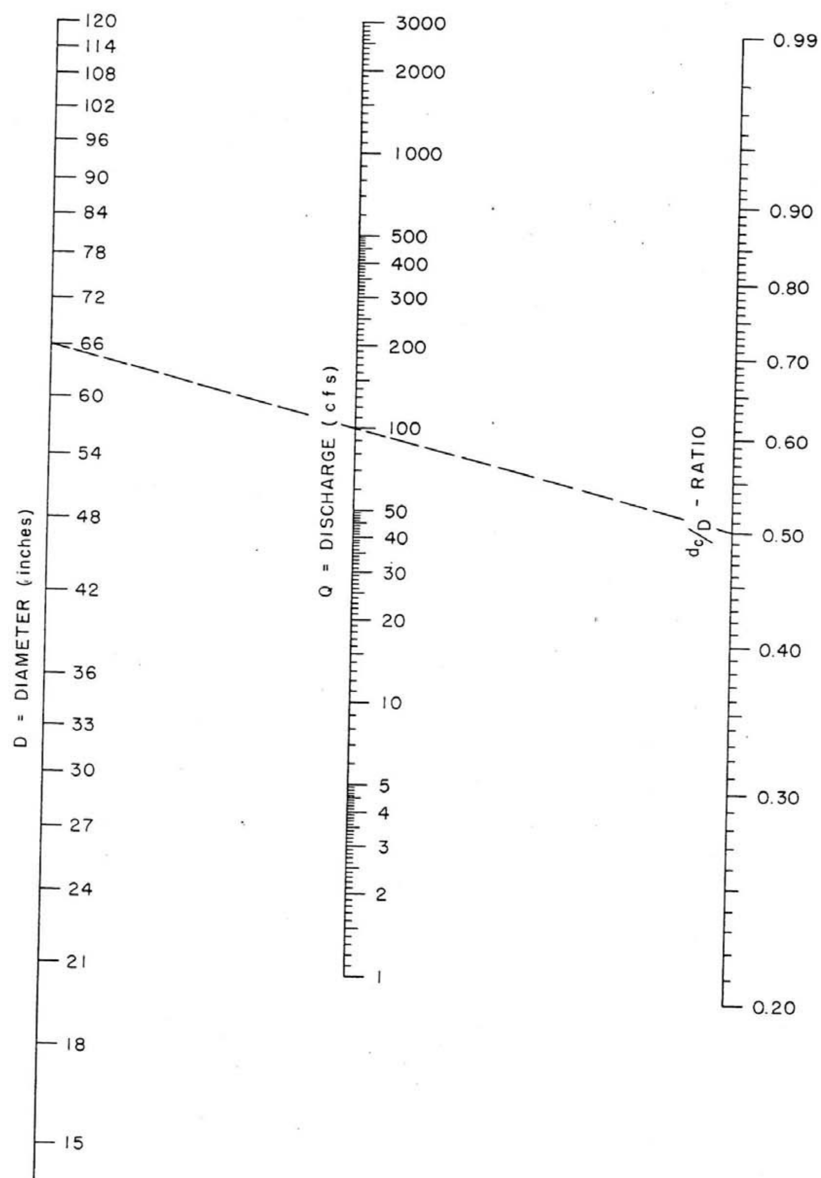


FIGURE 10–6

Figure 10-7. Critical Depth of Flow for Circular Conduits



DHT

CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

FIGURE 10-7

Figure 10-8. Velocity in Pipe Conduits

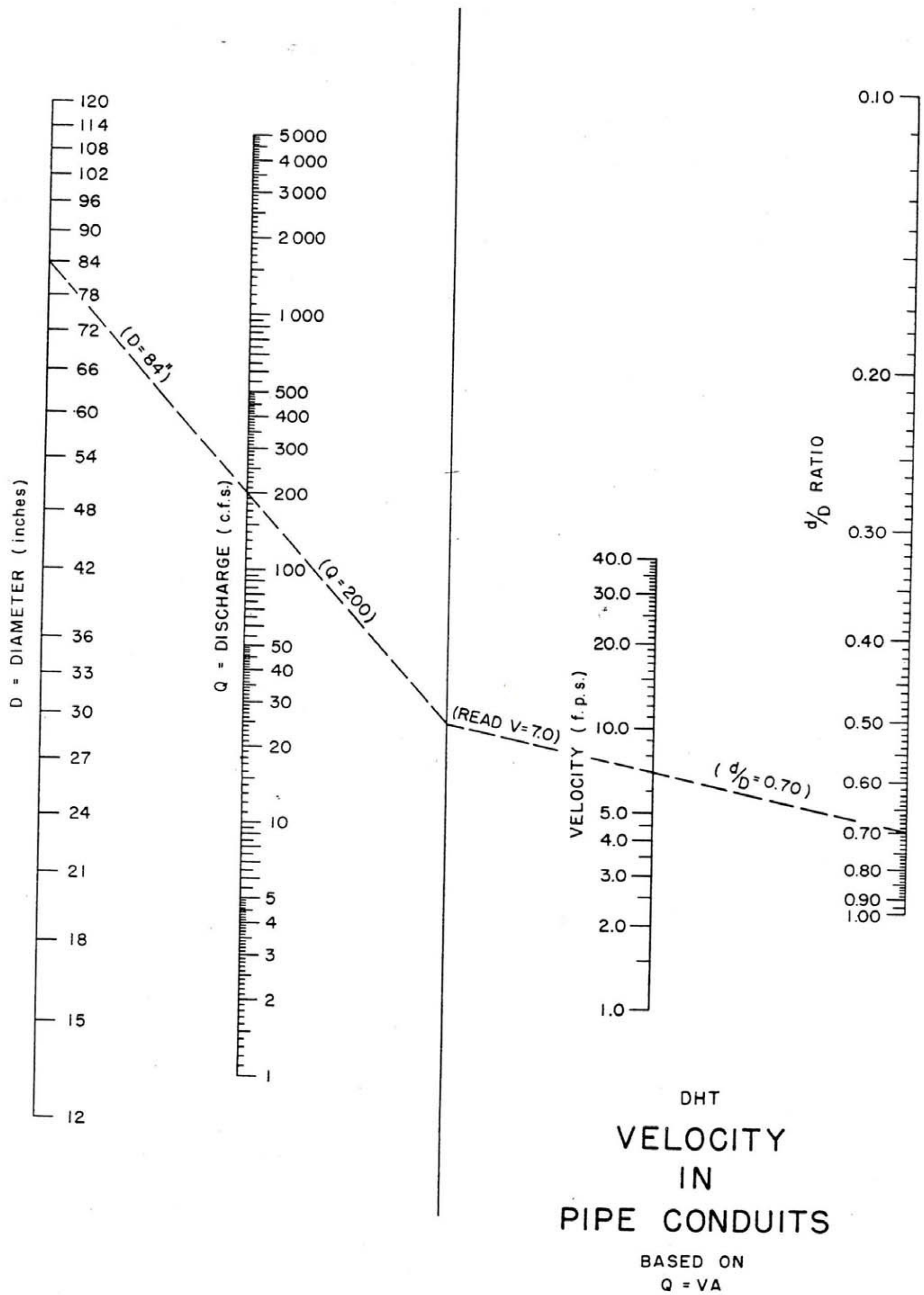


FIGURE 10-8

Figure 10-9. Critical Flow for Box Culverts

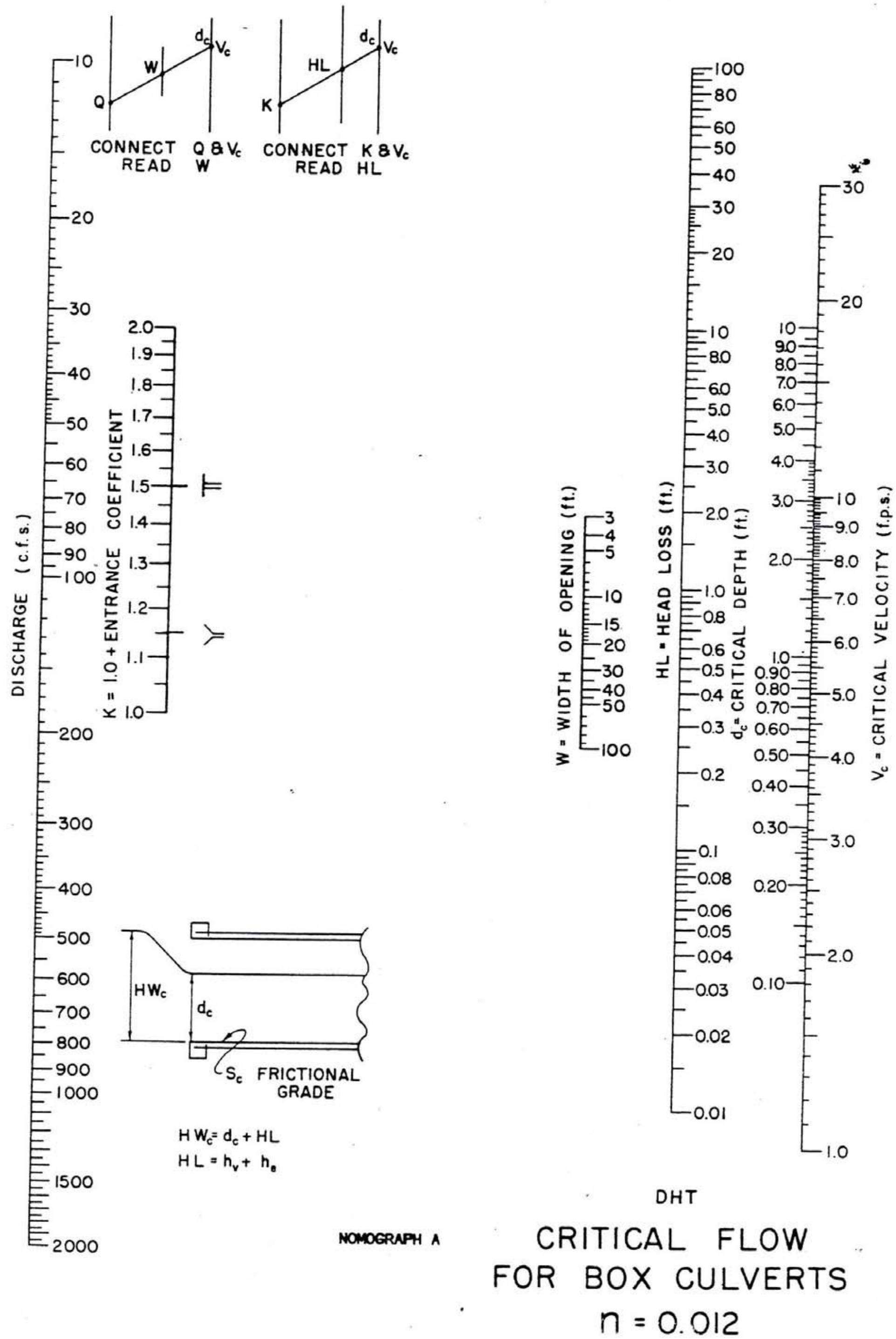
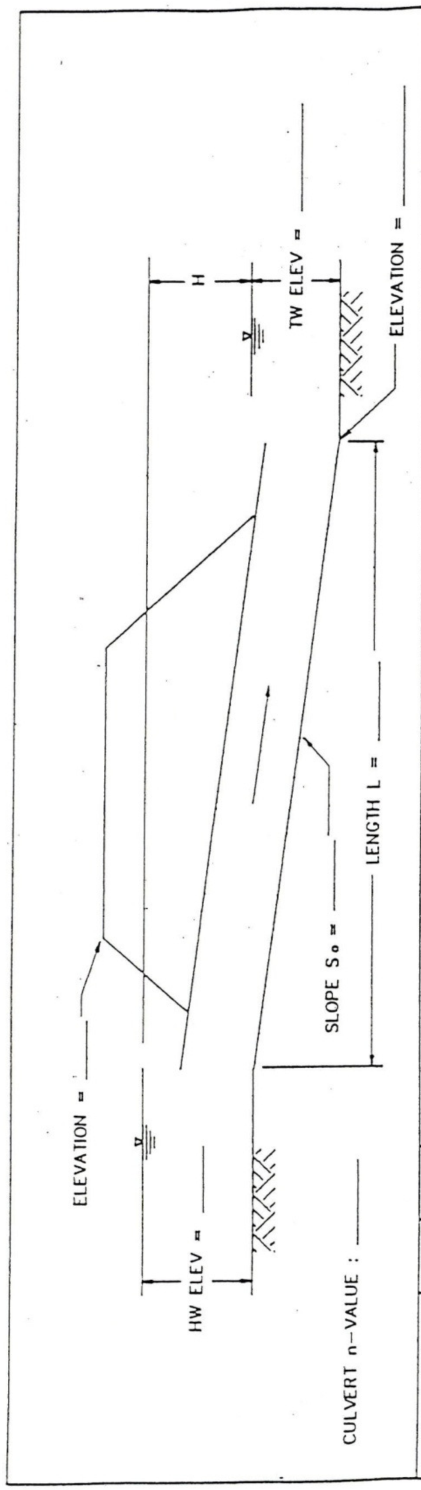


FIGURE 10-9

Computation Sheet 10-1. Culvert Design Computation Sheet

COMPUTATION SHEET 10-1

CULVERT DESIGN



CULVERT DESCRIPTION (ENTRANCE TYPE)	FLOW (cfs)	CULVERT SIZE	HEADWATER COMPUTATION										CONTROLLING HEADWATER DEPTH	OUTLET VELOCITY	COMMENTS
			INLET CONTROL		OUTLET CONTROL $HW = H_o - LS_o$										
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c - D}{2}$	TW	h_o	LS_o	HW			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

COMPUTATION SHEET 10-1

Table 10-2. Culvert Entrance Loss Coefficients

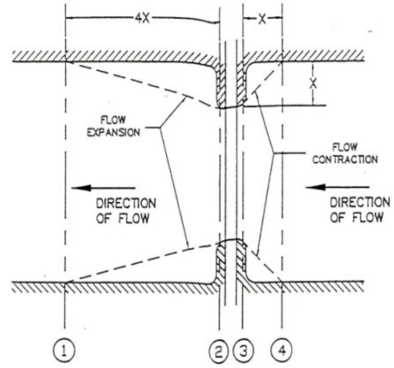
Type of Culvert	K_e
Reinforced Concrete Pipe:	
projecting from fill, socket end (groove end)	0.2
projecting from fill, square cut end	0.5
headwall or headwall with wingwalls	
socket end of pipe (groove end)	0.2
square edge	0.5
rounded edge (radius ∃ 0.0833D)	0.2
mitered to conform to fill slope	0.7
bevelled edges, 33.7E or 45E bevels	0.2
side or slope tapered inlet	0.2
Corrugated Metal Pipe or Arch-Pipe:	
projecting from fill (no headwall)	0.9
headwall or headwall with wingwalls, square edge	0.5
mitered to conform to fill slope, paved or unpaved slope	0.7
bevelled edges, 33.7E or 45E bevels	0.2
side or slope tapered inlet	0.2
Reinforced Concrete Box:	
headwall parallel to embankment (no wingwalls)	
square-edged on three sides	0.5
rounded on three sides to radius of 1/12 barrel dimension, or bevelled edges on three sides	0.2
wingwalls at 30E-70E to barrel	
square-edged at crown	0.4
crown edge rounded to radius of 1/12 barrel dimension, or bevelled top edge	0.2
wingwall at 10E-25E to barrel, square-edged at crown	0.5
wingwalls parallel (extension of sides), square-edged at crown	0.7
side or slope-tapered inlet	0.2

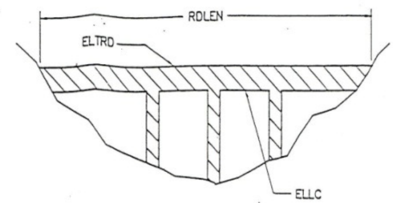
Computation Sheet 10-2. HEC-2 Bridge Modeling Data

10.5

HEC-2 BRIDGE MODELING DATA

PROJECT DESCRIPTION : ____________________
____
____
PROJECT NO. : ____________________
DATE : ____________________
BY : ____________________
CHECKED BY : ____________________





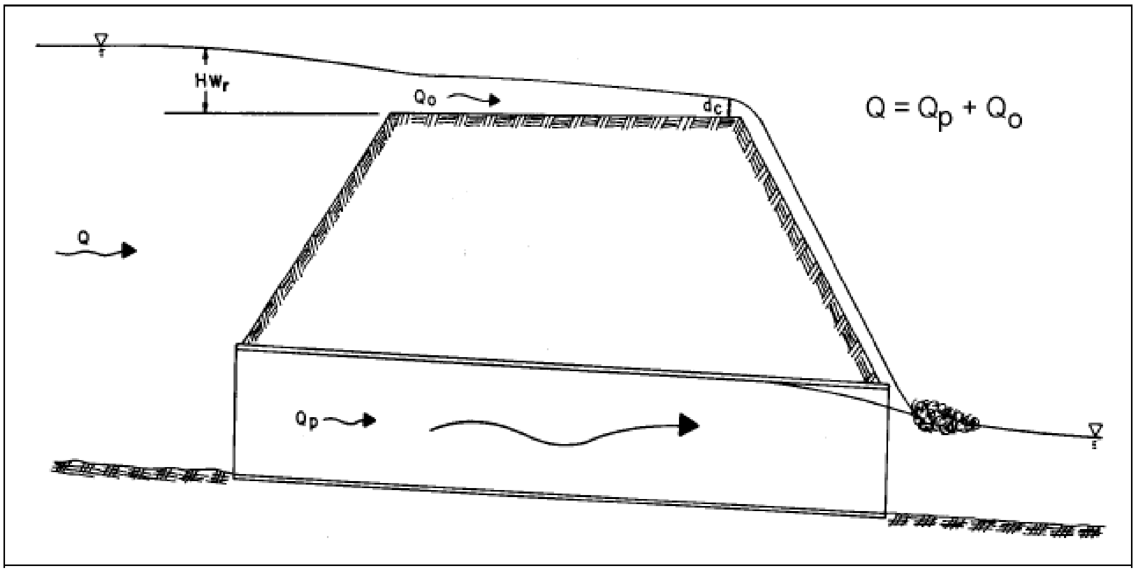
BRIDGE LOCATION	METHOD: SPECIAL OR NORMAL	DOWN-STREAM SECT ② LOCATION	UP-STREAM SECT ③ LOCATION	PIER SHAPE COEFF. (XK)	ORIFICE LOSS COEFF. (XKOR)	WEIR DISCHARGE COEFF. (COFQ)	AVERAGE ROADWAY LENGTH (RDLEN)	TOTAL WIDTH OF BRIDGE OPENING (BWC)	TOTAL WIDTH OF OBSTRCT(S) (BWP)	NET AREA OF OPENING (BAREA)	SIDE SLOPES (SS)	UPSTREAM CHANNEL ELEVATION (ELCHU)	DOWNSTREAM CHANNEL ELEVATION (ELCHD)	TOP OF ROADWAY ELEVATION (ELTRD)	LOW CHORD ELEVATION (ELLC)	TOTAL PRESSURE FLOW (QPR)	TOTAL WEIR FLOW (QWEIR)	UPSTREAM WATER SURFACE ELEVATION	DOWNSTREAM WATER SURFACE ELEVATION
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

COMPUTATION SHEET 10-2

Roadway Overtopping

Overtopping will begin when the headwater rises to the elevation of the roadway (Figure 10-10). The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir. Flow coefficients for flow overtopping roadway embankments are found in Federal Highway Administration (FHWA) *Hydraulic Design Standard (HDS) No. 1, Hydraulics of Bridge Waterways*, as well as in the documentation of *HY-7, the Bridge Waterways Analysis Model*. Curves from HY-7 are shown in Figure 10-11. Figure 10-11a is for deep overtopping, Figure 10-11b is for shallow overtopping, and Figure 10-11c is a correction factor for downstream submergence.

Figure 10-10. Roadway Overtopping



Equation 10-1 defines the flow across the roadway.

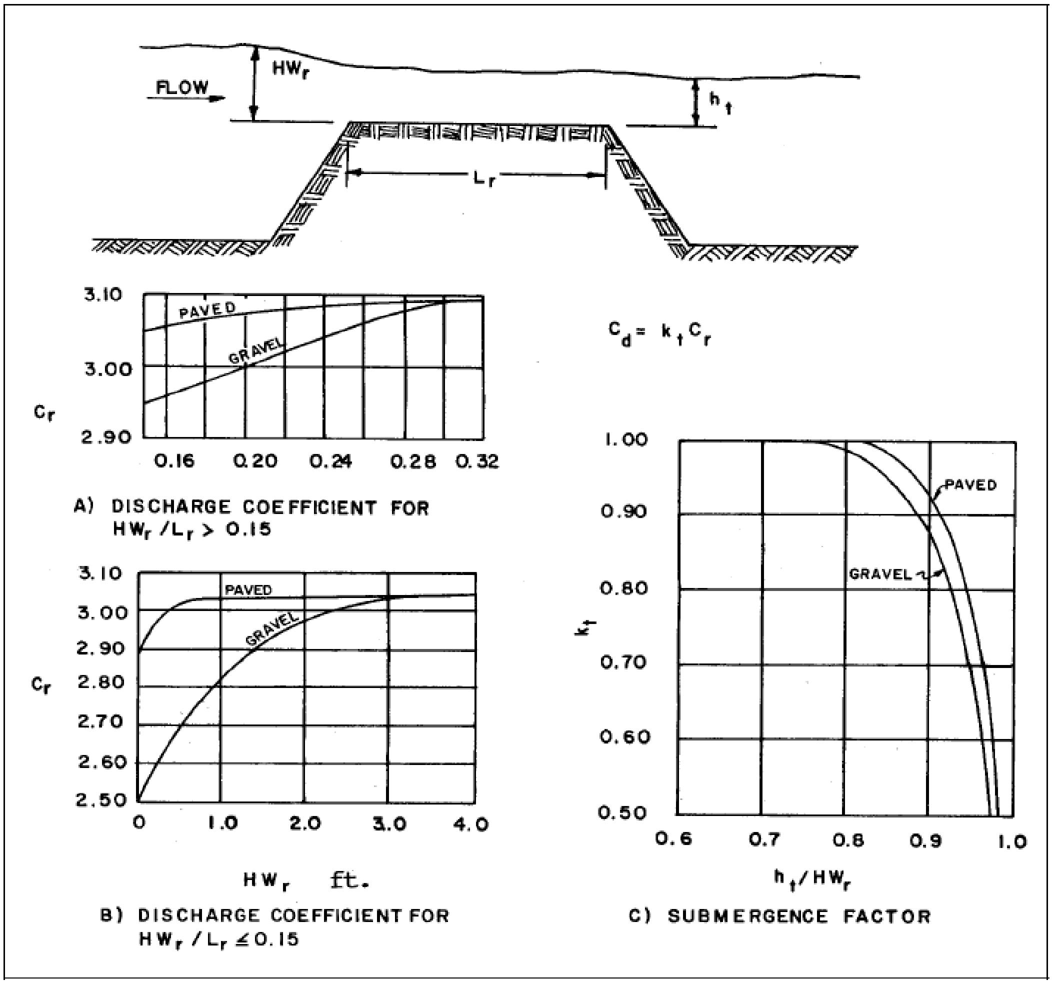
$$Q_o = C_d * L * HW_r^{1.5} \qquad \text{(Eq. 10-1)}$$

where:

Q_o = the overtopping flow rate in m³/s (ft³/s)
*C_d = the overtopping discharge coefficient (*for use in SI units, see note)
L = the length of the roadway crest, m (ft)
HW_r = the upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown, m (ft)

*Note: C_d determined from Figure 10-11 and other English unit research must be corrected by a factor of 0.552 [C_d (SI) = 0.552 (C_d English)]

Figure 10-11. English Discharge Coefficients for Roadway Overtopping

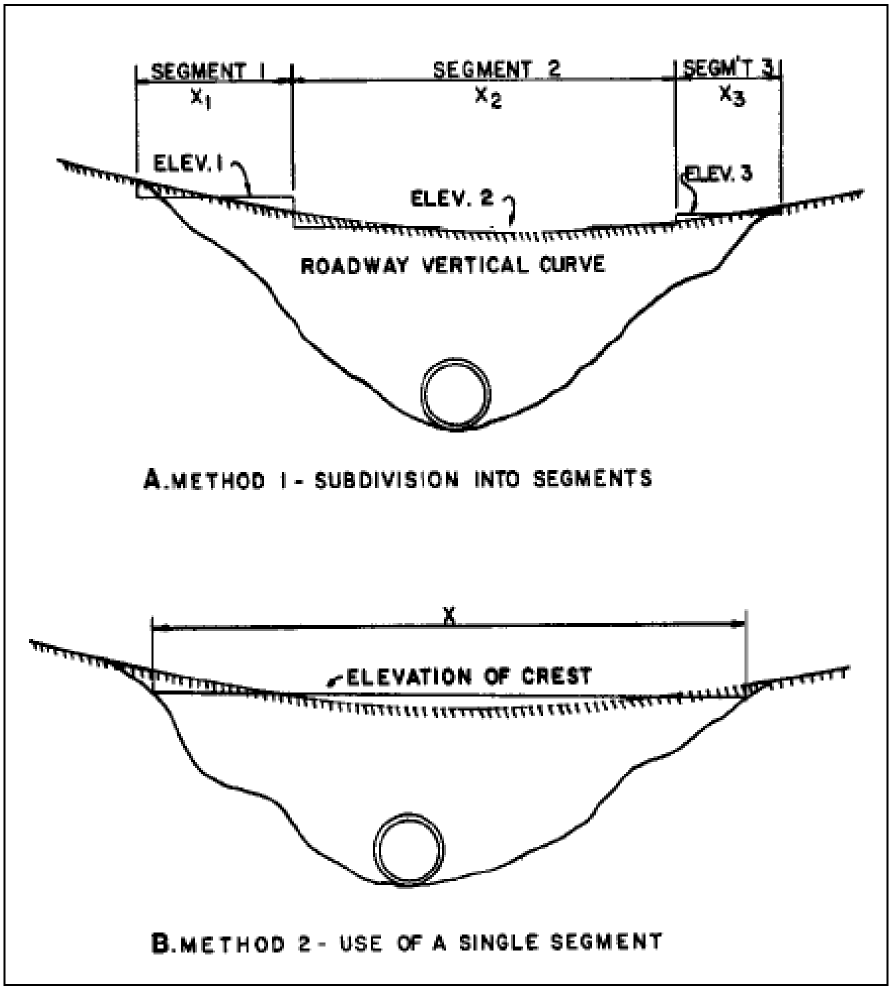


The length and elevation of the roadway crest are difficult to determine when the crest is defined by a roadway sag vertical curve. The sag vertical curve can be broken into a series of horizontal segments as shown in Figure 10-12a. Using Equation 10-1, the flow over each segment is calculated for a given headwater. Then, the incremental flows for each segment are added together, resulting in the total flow across the roadway.

Representing the sag vertical curve by a single horizontal line (one segment) is often adequate for culvert design (Figure 10-12b). The length of the weir can be taken as the horizontal length of this segment or it can be based on the roadway profile and an acceptable variation above and below the horizontal line. In effect, this method utilizes an average depth of the upstream pool above the roadway crest for the flow calculation.

It is a simple matter to calculate the flow across the roadway for a given upstream water surface elevation using Equation 10-1. The problem is that the roadway overflow plus the culvert flow must equal the total design flow. A trial and error process is necessary to determine the amount of the total flow passing through the culvert and the amount flowing across the roadway. Performance curves may also be superimposed for the culvert flow and the road overflow to yield an overall solution.

Figure 10-12. Weir Crest Length Determinations for Roadway Overtopping



11.0 DETENTION BASIN DESIGN

11.1 Applicable Design Criteria

Stormwater detention basins are used to temporarily impound (detain) excess stormwater, thereby reducing peak discharge rates. These basins can be used to provide the detention required to reduce the proposed peak discharge rate down to that of the existing conditions to satisfy the following requirements: compliance with City ordinances, preservation of existing floodplains along major creeks, prevention of overloading inadequate downstream storm drainage facilities, and prevention of erosive conditions in water courses. Detention basins also may improve water quality by allowing some sediment to settle out. Either regional detention/retention ponds or on-site detention/retention ponds may be used to provide the required detention. Detailed hydrologic studies of the entire watershed upstream of the detention site shall be required to evaluate the timing of inflow and outflow hydrographs from regional facilities.

Basins without upstream detention areas and with drainage areas of 200 acres or less can be designed using the Modified Rational Method, as described in Section 11.2 of this manual. Basins with drainage areas greater than 200 acres or where the Modified Rational Method is not applicable are to be designed using the Unit Hydrograph Method to determine the critical duration (the duration requiring the largest detention volume for a given frequency). The hydrograph routings through the detention basins are to be done using the Modified Puls Method.

All detention pond designs shall be performed by an engineer experienced in detention pond design and licensed in the State of Texas. The following criteria shall serve as minimum requirements for detention pond design in the City of San Angelo:

A. The 100-year storm event shall be used to determine the volume of detention storage required. In addition, the outlet structure shall be designed such that the peak discharge is not increased for the 2-year storm as well as the 100-year storm event described in Section 5. This will reduce the erosion that can result from the more frequent events.

B. The design discharge shall be determined as described in Section 5 of this manual.

C. The Modified Rational Method may be used to develop runoff hydrographs for detention storage design when the contributing drainage area is 200 acres or less. A combination of a unit hydrograph method and storage routing method shall be used when the contributing drainage area exceeds 200 acres.

D. The design discharge shall be determined as for the 24 hour storm when the discharge is determined by a synthetic unit hydrograph. Ponds shall be designed to drain within a 24 hour period.

E. An emergency spillway or overflow area shall be provided at the maximum 100-year pool level and shall be capable of conveying discharges as required by the Texas Commission on Environmental Quality (TCEQ). The spillway shall be constructed of a non-erodible material.

F. The discharge velocities of any detention pond outflow structure shall be consistent with the velocity requirements of the receiving channel and/or adequate riprap or other energy dissipation shall be provided at the outfall location.

G. Where the outflow structure conveys flow through the embankment in a conduit, the conduit shall be reinforced concrete, designed to support the external loads. The conduit is to withstand the internal hydraulic pressure without leakage under full external load or settlement and must convey water at the design velocity without damage to the interior surface of the conduit. Cut-off collars shall be provided for all conduits that discharge through the embankment. If pipe materials other than reinforced concrete are proposed, the engineer shall submit documentation that the material will meet the use intended.

H. The outflow structure shall discharge flows into the natural stream or unlined channels at a non-erosive rate in accordance with Section 9 of this Manual.

I. Detention basins to be excavated shall provide positive drainage through the pond with a minimum slope of 0.30%. The steepest side slope permitted around the detention pond is 4:1.

J. Earthen embankments used to impound a required detention volume must have a minimum top-width of 8 feet, shall contain a low permeability core, and shall be based on a geotechnical investigation for the site. The geotechnical investigation shall be performed by a licensed engineer and shall include, as a minimum, the type of material on-site, water content, liquid limit, plasticity index and desired compaction. Earthen embankments higher than 6 feet shall conform to Title 30, Texas Administrative Code (TAC) Chapter 299 and other applicable dam safety requirements.

K. Security fencing may be required to encompass the detention storage area if the location, velocity, depth, or slopes justify restricted access to the general public as determined by the City Engineer. The fence shall be designed to allow access for maintenance as well as not to restrict stormwater flow into or out of the detention basin.

L. Maintenance access shall be provided in accordance with the requirements in Section 14 (Maintenance).

M. For parking lots used as detention basins, the maximum depth of ponding allowed shall not exceed one foot, and a maintenance/indemnity agreement may be required.

N. Underground detention storage facilities may be allowed when approved by the City Engineer.

O. Basins with permanent storage must include dewatering facilities to provide for maintenance for the storage areas.

P. The design of detention facilities shall include provisions for collecting and removing sediment deposited after collecting and releasing stormwater.

Q. Non-erosive conditions shall be provided to convey runoff from entry points of concentrated flow into the pond to the outlet structure of the pond during low flow conditions. Erosion protection must be provided as necessary to prevent erosion of the pilot channel due to scour.

11.2 Detention Basin Calculations

Detention basins without upstream detention areas and with drainage areas of 200 acres or less can be designed using the Modified Rational Method to determine the critical duration. The detention volume for a given duration in this method is calculated as the difference between the triangular inflow hydrograph and the trapezoidal outflow hydrograph. A range of storm durations is required to determine the critical duration for the detention basin. See the next section for example of this method.

11.3 Detention Pond Computation Sheet

Computations performed for detention pond sizing using the Unit Hydrograph Method shall be submitted in a format consistent with Computation Sheet 11-1. Listed below is an example of the calculations of the sizing of a detention pond. A copy of Computation Sheet 11-1 is also provided showing this example.

GIVEN: A 53.91-acre site which is currently zoned as agricultural use, and is to be developed for Zone R-1, Single-Family Residence District with a Rational Method C of 0.50. The existing field has a C of 0.30.

DETERMINE: Maximum release rate and required detention storage.

SOLUTION:

Step 1. Determine 100-year peak runoff rate prior to site development.

Calculate Peak Discharge for Present Conditions (Agriculture)

$Q = CIA$

$C = 0.30$

$T_c = 26 \text{ minutes} = T_d$

For 100-year b = 112.5, d = 14.70, and e = 0.816

$i_{100} = b/((T_d+d)^e) = 112.5 / ((26.00 + 14.70)^{0.816}) = 5.47$ in./hr.

$Q_{100} = 0.30 * (5.47) * 53.91 = 88.5$ cfs (Maximum release rate)

Step 2. Determine inflow hydrograph for storms of various durations in order to determine maximum volume required with release rate determined in Step 1.

NOTE: Incrementally increase durations to next 10-minute time and increase by 10-minutes for each additional time step to determine maximum required 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.

Future Conditions (R-1)

$C = 0.50$

T_c = 21 minutes = T_d

For 100-year b = 112.5, d = 14.70, and e = 0.816

$i_{100} = b/((T_d+d)^e) = 112.5 / ((21.00 + 14.70)^{0.816}) = 6.08$ in./hr.

$Q_{100} = 0.50 * (6.08) * 53.91 = 163.9$ cfs

Step 3. Determine Maximum Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow for each storm duration.

Step 3A. Calculate the intensity, i, and peak flow, Q, for the various duration storms:

21 minutes	i = 6.08	Q = 0.50 * (6.08) * 53.91 = 163.9	cfs
30 minutes	i = 5.06	Q = 0.50 * (5.06) * 53.91 = 136.4	cfs
40 minutes	i = 4.29	Q = 0.50 * (4.29) * 53.91 = 115.6	cfs
50 minutes	i = 3.75	Q = 0.50 * (3.75) * 53.91 = 101.1	cfs
60 minutes	i = 3.33	Q = 0.50 * (3.33) * 53.91 = 89.8	cfs
70 minutes	i = 3.01	Q = 0.50 * (3.01) * 53.91 = 81.1	cfs
80 minutes	i = 2.74	Q = 0.50 * (2.74) * 53.91 = 73.9	cfs
90 minutes	i = 2.53	Q = 0.50 * (2.53) * 53.91 = 68.2	cfs
100 minutes	i = 2.35	Q = 0.50 * (2.35) * 53.91 = 63.3	cfs

Step 3B. Determine the required storage for each storm duration:

Inflow $= T_d * Q * 60$ sec/min

Outflow $= 0.5 * (T_d + T_c) * Q_0 * 60$ sec/min

Storage = Inflow - Outflow

where:

T_d = Time of storm duration (min)

Q = Flow for that T_d, (cfs)

T_c = Time of concentration of the basin

Q_0 = Original flow, pre-development conditions

21 min. Storm	Inflow = 21 * (163.9) * 60 sec/min	=	206,514 cf
	Outflow = (0.5) * (21+21) * (88.5) * 60 sec/min	=	<u>111,510 cf</u>
	Storage = (cf)	=	95,004 cf
	Storage (acre-ft) = Storage (cf) / 43,560	=	2.18 acre-ft

This is repeated for the other durations as shown in the example Computation Sheet 11-1

Step 4 Determine the greatest amount of storage required and at what storm duration the greatest amount of storage occurs at.

Maximum volume required is 118,438 cubic feet or 2.72 acre-feet at the 40 min. storm duration, with a maximum release rate equivalent to the existing flow of 88.4 cfs.

11.4 Playa Lakes

Playa Lakes are natural formations which retain runoff but lack the ability to sufficiently drain after a storm event. A designer may use a playa lake as part of a stormwater control plan if there is suitable area-capacity information describing storage availability. Excavation and fill are allowed, if necessary, but under no circumstances may the storage capacity be reduced without the appropriate compensation of storage loss within the immediate area of the development. When creating a hydraulic model of the playa, assume the lake is full to the lowest natural outlet point prior to rainfall.

Computation Sheet 11-1. Detention Pond and Example

Modified Rational Method Detention Pond Design

Project Description: ____________________ Project No.: ________________ By: ________________

____________________ Date: ________________ Checked By: ________________

Step		C	ensity, I = b/(Tc+d)^e					A	Q peak
			b	d	e	Tc	I (in/hr)	(acres)	(cfs)
1	Exist. Cond.		112.50	14.70	0.816				
2	Prop. Cond.		112.50	14.70	0.816				

Step 3									
Time Step	Td Duration (min)	C	I (in/hr)	A (acres)	Q peak (cfs)	Volume (ft^3)			Volume (acre-ft)
						Inflow Td*Q*60	Outflow .5*(Tc+Td)*Qo*60	Req. Storage Inflow - Outflow	Previous Col. / 43,560
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									

Note: 1. The use of the Modified Rational Method for detention design, is limited to drainage basins with less than 200 acres.
2. The detention pond shall be designed using the 100-year storm peak flows, (K=1.25, b=112.5, d=14.70, and e=0.816).

Step 4. Max Required Storage (acre-ft) ____________________

Computation Sheet 11-1

Modified Rational Method Detention Pond Design

Project Description: Example of detention pond calculations. Project No.: ______ By: ______

Date: ______ Checked By: ______

Step		C	ensity, I = b/(Tc+d)^e					A	Q peak
			b	d	e	Tc	I (in/hr)	(acres)	(cfs)
1	Exist. Cond.	0.30	112.50	14.70	0.816	26.00	5.47	53.91	88.41
2	Prop. Cond.	0.50	112.50	14.70	0.816	21.00	6.08	53.91	165.40

Step 3

Time Step	Td Duration (min)	C	I (in/hr)	A (acres)	Q peak (cfs)	Volume (ft^3) Inflow Td*Q*60	Volume (ft^3) Outflow .5*(Tc+Td)*Qo*60	Volume (ft^3) Req. Storage Inflow - Outflow	Volume (acre-ft) Previous Col. / 43,560
1	21.00	0.50	6.08	53.91	165.40	208,400	111,400	97,000	2.23
2	30.00	0.50	5.06	53.91	137.67	247,815	135,271	112,544	2.58
3	40.00	0.50	4.29	53.91	116.76	280,233	161,795	118,438	2.72
4	50.00	0.50	3.75	53.91	101.81	305,442	188,319	117,124	2.69
5	60.00	0.50	3.33	53.91	90.55	325,971	214,843	111,128	2.55
6	70.00	0.50	3.01	53.91	81.72	343,243	241,366	101,877	2.34
7	80.00	0.50	2.74	53.91	74.61	358,134	267,890	90,244	2.07
8	90.00	0.50	2.53	53.91	68.74	371,213	294,414	76,799	1.76
9	100.00	0.50	2.35	53.91	63.81	382,872	320,938	61,934	1.42
10									
11									
12									

Note: 1. The use of the Modified Rational Method for detention design, is limited to drainage basins with less than 200 acres.

2. The detention pond shall be designed using the 100-year storm peak flows, (b=112.5, d=14.70, and e=0.816).

Step 4. Max Required Storage (acre-ft) 2.72

Computation Sheet 11-1

12.0 STORM WATER POLLUTION PREVENTION DURING CONSTRUCTION

12.1 General

It is the responsibility of the Developer to provide stormwater pollution prevention for the duration of the construction period. The Developer must comply with the current Texas Commission on Environmental Quality general permit for stormwater discharges from construction activities under the Texas Pollutant Discharge Elimination System (TPDES) program as well as the City of San Angelo's Stormwater Ordinance and Design Manual. This section provides current guidelines and Best Management Practices (BMPs) information for adhering to all Local, State, and Federal environmental regulations with respect to stormwater pollution prevention during construction activity.

The following items generally summarize the stormwater pollution prevention requirements for construction activity. The Developer and/or his designees must adhere to these requirements to comply with applicable State and City requirements.

A. Comply with applicable requirements of all governing authorities having jurisdiction. The Specifications and the Plans are not intended to be prescriptive but rather to convey the intent to provide complete slope protection, erosion control, and stormwater pollution prevention for both the Owner's property and adjacent properties.

B. Develop and implement a stormwater pollution prevention plan in accordance with the current TCEQ general permit for stormwater discharges from construction activities prior to the beginning of construction activity.

C. Establish stormwater pollution prevention measures maintained during the entire length of construction until final stabilization has been achieved for the area protected.

D. All land-disturbing activities are required to be planned and conducted to minimize the area to be exposed at any one time as well as time of exposure, off-site erosion, sedimentation, and adverse water quality impacts.

E. Surface water runoff originating upgrade of an exposed area is required to be managed to minimize erosion and sediment loss during the period of exposure.

F. Install measures to control both the velocity and rate of release so as to minimize erosion and sedimentation of the receiving water body (i.e., ditch, channel, stream) in accordance with regulatory requirements.

G. Periodically clean out and dispose of all sediment and other pollutants as necessary to maintain adequate treatment capacity of each pollution control feature. Clean out and properly dispose of all sediment and other stormwater pollutants at the time of completion of the Work.

12.2 Documentation

For small and large projects, copies must be maintained of a schedule of major construction activities, inspection reports, and revision documentation with the stormwater pollution prevention plan (SWPPP) required under the current TPDES general permit for stormwater discharges from construction activities.

12.2.A. Small Construction Activity (>1 Acre but <5 Acres)

A. On small construction projects (disturbed area equal to or greater than one acre and less than five acres), the Developer submit a copy of the completed Construction Site Notice to the City prior to beginning construction activity.

B. A copy of the Construction Site Notice must be posted at the construction site in a location where it can be viewed by the general public and Local, State, and Federal authorities prior to starting construction activities.

C. The Construction Site Notice posting must be maintained until completion of the construction activities.

12.2.B. Large Construction Activity (>5 Acres)

On large construction projects (five acres or more of disturbed area), the following must be submitted to the TCEQ and the City:

A. Notice of Intent (NOI) at least 48 hours prior to beginning construction activity.

B. Notice of Change (NOC) letter when relevant facts or incorrect information was submitted in the NOI, or if relevant information in the NOI changes during the course of construction activity.

C. Notice of Termination (NOT) when the construction project has been completed and stabilized in accordance with the requirements of the current TPDES general permit for stormwater discharges from construction activities.

A copy of the NOI must be posted at the construction site in a location where it viewable by the general public and Local, State, and Federal authorities prior to starting construction activities and maintain the posting until completion of the construction activities.

12.3 Best Management Practices

Following are reference sources for stormwater pollution prevention measures to be selected, as appropriate, for the construction site and activity in the City of San Angelo:

A. Environmental Protection Agency Stormwater Guidance for Construction Activities: Developing Pollution Prevention Plans and Best Management Practices

B. Texas Department of Transportation (TxDOT) Erosion Control Report and/or the most current annual Approved Products List for TxDOT

C. City of Austin Environmental Criteria Manual

D. North Central Texas Council of Governments (NCTCOG) integrated Stormwater Management (iSWM) Design Manual for Construction

E. Harris County/Harris County Flood Control District/City of Houston Stormwater Management Handbook for Construction Activities

F. Center for Watershed Protection

All materials and construction methods used for stormwater pollution prevention shall meet the minimum requirements of the current TCEQ general permit for stormwater discharge from construction activities.

The best management practices identified below are for commonly used sediment loss prevention practices and are referenced from the North Central Texas Council Of Governments (NCTCOG) integrated Stormwater Management (iSWM) Design Manual for Construction. Appropriate control devices should be selected to protect against stormwater pollution from construction site activity.

12.3.A. Erosion Control Blankets

Erosion Control Blankets are designed to hold seed and soil in place until vegetation is established on disturbed areas.

12.3.A.1. *Material Selection*

The type and class of erosion control mat must be specified as appropriate for the slope of the area to be protected and the anticipated length of service.

Erosion control blankets must meet the applicable TxDOT Minimum Performance Standards for TxDOT as provided in its Erosion Control Report and/or be listed on the most current annual Approved Products List for TxDOT applicable to TxDOT Item 169 Soil Retention Blanket and its Special Provisions.

12.3.A.2. *Placement*

Locate anchor trenching along the entire perimeter of the installation area, except for small areas with less than 2 percent slope.

Joints and overlapping material shall be securely fastened.

12.3.B. Silt Fences

Silt fences are intended for use as perimeter controls located downstream of disturbed areas. Following are the minimum design requirements for silt fence materials.

12.3.B.1. *Material Selection*

If 50% or less soil by weight passes the U.S. Standard sieve No. 200, select the apparent opening size (A.O.S.) to retain 85% of the soil.

If 85% or more of soil by weight passes the U.S. Standard sieve No. 200, silt fences are required to not be used unless the soil mass is evaluated and deemed suitable by a soil scientist or geotechnical engineer concerning the erodibility of the soil mass, dispersive characteristics, and the potential grain-size characteristics of the material that is likely to be eroded.

Silt fence fabric must meet the following minimum criteria:

1. Tensile Strength, ASTM D4632 Test Method for Grab Breaking Load and Elongation of Geotextiles, 90-lbs.
2. Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 60-lbs.
3. Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 280-psi.
4. Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 70 (max) to No. 100 (min).
5. Ultraviolet Resistance, ASTM D4355. Minimum 70 percent.

Filter stone for an overflow structure is required to be 1-1/2" washed stone containing no fine material. Angular shaped stone is preferable to rounded shaped stone.

Fence posts are required to be galvanized steel or equivalent and may be T-section or L-section, 1.3 pounds per linear foot minimum, and 4 feet in length minimum. Wood Posts may be used depending on anticipated length of service and provided they are 4 feet in length minimum and have a nominal cross section of 2 inches by 4 inches for pine or 2 inches by 2 inches for hardwoods.

Silt fence is required to be supported by galvanized steel wire fence fabric as follows:

1. 4" x 4" mesh size, W1.4/1.4, minimum 14-gauge wire fence fabric;
2. Hog wire, 12-gauge wire, small openings installed at bottom of silt fence;
3. Standard 2" x 2" chain link fence fabric; or
4. Other welded or woven steel fabrics consisting of equal or smaller spacing as that listed herein and appropriate gauge wire to provide support.

12.3.B.2. *Placement*

A. Locate fences along a line of constant elevation (along a contour line if possible).

B. Maximum drainage area is required to be 0.25 acre per 100 linear feet of silt fence.

C. Maximum flow to any 20-foot section of silt fence is required to be 1 CFS.

D. Maximum distance of flow to silt fence is required to be 200 feet or less. If the slope exceeds 10 percent, the flow distance is required to be less than 50 feet.

E. Maximum slope adjacent to the fence is required to be 2:1.

F. Stone overflow structures or other outlet control devices are required to be installed at all low points along the fence or spaced at approximately 300 feet if there is no apparent low point.

G. A 6-inch wide trench is to be cut 6 inches deep at the toe of the fence to allow the fabric to be laid below the surface and backfilled with compacted earth or gravel to prevent bypass of runoff under the fence. Fabric is required to overlap at abutting ends a minimum of 3 feet and is required to be joined such that no leakage or bypass occurs.

H. Sufficient room for the operation of sediment removal equipment is required to be provided between the silt fence and other obstructions in order to properly maintain the fence.

I. The ends of the fence are required to be turned upstream to prevent bypass of stormwater.

12.3.C. <u>Inlet Protection</u>

Inlet protection is necessary in new developments that include new inlets or roads with new curb inlets or during repairs to existing roadways.

12.3.C.1. *Material Selection*

1. Filter fabric protection is required to be designed and maintained in a manner similar to a silt fence.
2. Where applicable, filter fabric, posts, and wire backing are required to meet the material requirements specified in the silt fence design requirements.
3. Filter gravel is required to be ¾ inch (Block and Gravel Protection) or 1-1/2 to 2 inch (Excavated Impoundment Protection) washed stone containing no fines. Angular shaped stone is preferable to rounded shapes.
4. Concrete blocks are required to be standard 8” x 8” x 16” concrete masonry units.

12.3.C.2. *Placement*

1. Ensure that inlet protection is properly designed, installed, and maintained to avoid flooding of the roadway or adjacent properties and structures.
2. Maximum depth of flow is required to be 8 inches or less.
3. Positive drainage is critical in the design of inlet protection. If overflow is not provided for at the inlet, excess flows are required to be routed through established swales, streets, or other watercourses to minimize damage due to flooding.
4. Filter Barrier Protection – Silt fence is required to consist of nylon geotextile supported by wire mesh, W1.4 X W1.4, and galvanized steel posts set a minimum of 1 foot depth and spaced not more than 6 feet on center. A 6-inch wide trench is to be cut 6 inches deep at the toe of the fence to allow the fabric to be laid below the surface and backfilled with compacted earth or gravel. This entrenchment prevents any bypass of runoff under the fence. If the inlet is installed within a paved area, provide sufficient material overlap at the base to allow for anchorage of the fabric to the concrete inlet slab by sand bags or other means in order to prevent bypass or runoff under the fence.
5. Block and Gravel Protection (Curb and Drop Inlets) – Concrete blocks are to be placed on their sides in a single row around the perimeter of the inlet, with ends abutting. Openings in the blocks should face outward, not upward. ½" x ½" wire mesh is required to then be placed over the outside face of the blocks covering the holes. Filter stone is required to then be piled against the wire mesh to the top of the blocks with the base of the stone being a minimum of 18 inches from the blocks. Alternatively, where loose stone is a concern (streets, etc.), the filter stone may be placed in appropriately sized geotextile fabric bags. Periodically remove and clean the stone or replace it with new stone when the stone filter becomes clogged.
6. Excavated Impoundment Protection – An excavated impoundment is required to be sized to provide a storage volume of between 1800 and 3600 cubic feet per acre of disturbed area. The trap is required to have a minimum depth of one foot and a maximum depth of 2 feet as measured from the top of the inlet and is required to have sideslopes of 2:1 or flatter. Install weep holes in the inlet walls to allow for the complete dewatering of the trap. When the storage capacity of the impoundment has been reduced by one-half, remove the silt and dispose of it at an approved location.

12.3.D. Stone Outlet Sediment Traps

Stone outlet sediment traps are used in situations where flows are concentrated in a drainage swale or channel.

12.3.D.1. *Material Selection*

The embankment is required to be placed on geotextile fabric meeting the following minimum criteria:

1. Tensile Strength, ASTM D4632 Text Method for Grab Breaking Load and Elongation of Geotextiles, 250-lbs.
2. Puncture Rating, ASTM D4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products, 135-lbs.
3. Mullen Burst Rating, ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, 420-psi.
4. Apparent Opening Size, ASTM D4751 Test Method for Determining Apparent Opening Size of a Geotextile, U.S. Sieve No. 20 (max).

12.3.D.2. *Placement*

1. The maximum drainage area contributing to the trap is limited to 10 acres. For larger drainage areas a sediment basin is required to be used.
2. The minimum storage volume is 1800 cubic feet per acre of disturbed land draining to the device.
3. The surface area of the design storage is 1% of the area draining to the device.
4. The maximum embankment height is 6 feet as measured from the toe of the slope on the downstream side.
5. Minimum width of the embankment at the top is required to be 2 feet.
6. Embankment slope is required to be 1:5:1 or flatter.
7. The embankment is required to have a depressed area to serve as the outlet with a minimum width of 4 feet.
8. A six inch minimum thickness layer of ¾ to 2 inch (1-½ inch nominal) well graded filter stone is required to be placed on the face of the embankment.
9. The embankment is required to be comprised of well graded stone with a size range of 6 to 12 inches in diameter. The stone may be enclosed in wire mesh or a gabion basket and anchored to the channel bottom to prevent washing away.
10. The outlet is required to be designed to have a minimum freeboard of 6” at design flow.
11. Geotextile fabric, covered with a layer of stone, is required to extend past the base of the embankment on the downstream side a minimum of 2 feet.

12.3.E. Sediment Basins

Sediment basins are used as treatment devices for sites with disturbed areas of 10 acres and larger that is part of a common drainage area. The sediment basin is required to have minimum design dewatering time of 36 hours.

12.3.E.1. *Placement*

1. Minimum capacity of the basin is required to be the calculated volume of runoff from a 2-year, 24-hour duration storm event.
2. Deposited sediment is required to be removed when the storage capacity of the basin has been reduced by 20%.
3. Minimum width of the embankment at the top is required to be 8 feet.
4. Embankment slope is required to be 3:1 or flatter.
5. Maximum embankment height is required to be 6 feet as measured from the toe of slope on the downstream side. Sediment basins with embankments exceeding 6 feet are regulated by the Texas Commission on Environmental Quality (TCEQ) and must meet specific requirements for dam safety.
6. The basin outlet is required to be designed to accommodate a 25-year design storm without causing damage to the containment structure.
7. The basin must be laid out such that the effective flow length of the basin should be at least twice the effective flow width.
8. The outlet of the outfall pipe (barrel) is required to be stabilized with riprap or other form of stabilization with design flows and velocities based on 25-year design storm peak flows. For velocities in excess of 6 feet per second, energy dissipation measures should be used to reduce outfall velocities.
9. The effectiveness of sediment basins may be increased by using baffles to prevent short-circuiting of flow through the basin.

12.3.F. Check Dams

Check dams are used for long drainage swales or ditches to reduce erosive velocities. Use geotextile filter fabric under check dams exceeding 18 inches in height. The fabric is required to meet the material specified for the Stone Outlet Sediment Trap discussed above.

12.3.F.1. *Material Selection*

A. Rock Check Dams

1. Stone is required to be well graded with size range from 1-1/2 to 3-1/2 inches in diameter depending on expected flows.

2. Rock Check Dams should be triangular in cross section with side slopes of 1:1 or flatter on the upstream side and 2:1 or flatter on the downstream side.

B. Sand Bag Check Dams

1. Sand Bag Check Dams should have a maximum flow through rate of 0.1 cfs per square foot of surface with a minimum top width of 16 inches and bottom width of 48 inches. Bags should be filled with coarse sand, pea gravel, or filter stone that is clean and free of deleterious material.
2. Bag length is required to be 24-inches to 30-inches, width is required to be 16-inches to 18-inches and thickness is required to be 6-inches to 8-inches and having an approximate weight of 40-pounds.
3. Bag material is required to be polypropylene, polyethylene, polyamide, or cotton burlap woven fabric, minimum unit weight 4-ounces-per-square-yard, Mullen burst strength exceeding 300-psi as determined by ASTM D3786 Standard Test Method for Hydraulic Bursting Strength of Textile Fabrics-Diaphragm Bursting Strength Tester Method, and ultraviolet stability exceeding 70 percent.
4. PVC pipes may be installed through the Sand Bag Dam near the top to allow for controlled flow through the dam. Pipe should be schedule 40 or heavier polyvinyl chloride (PVC) having a nominal internal minimal diameter of 4 inches.

12.3.F.2. *Placement*

1. Check Dams are required to be placed at a distance and height to allow small pools to form between each one. Typically, dam height should be between 18" and 36". Dams are required to be spaced such that the top of the downstream dam is at the same elevation as the toe of the upstream dam.
2. Flows must pass the check dam without causing any significant upstream impact.
3. Check dams should be used in conjunction with other sediment reduction techniques prior to releasing flow offsite.

12.3.G. Stabilized Construction Entrance

Stabilized construction entrances are used for sites in which significant truck traffic occurs on a daily basis. The stone is required to be a minimum of 3 to 5-inch coarse aggregate.

12.3.G.1. *Placement*

1. Stabilized Construction Entrances are to be designed such that drainage across the entrance is directed to a controlled, stabilized outlet on site with provisions for storage, proper filtration, and removal of wash water.

2. The entrance must be sloped away from the paved surface so that stormwater is not allowed to leave the site onto roadways.
3. Minimum width of entrance is required to be 15 feet.
4. Stone is required to be placed in a layer of at least 12-inch thickness. The stone is required to be a minimum of 3 to 5-inch coarse aggregate.
5. Prevent construction vehicles from shortcutting the full length of the construction entrance by installing barriers as necessary.
6. Vehicles are required to not be permitted to track or drop sediment onto paved roads, streets, or parking lots. When necessary, vehicles must be cleaned to remove sediment prior to entrance onto paved areas. When washing is required, it is required to be done on a constructed wheel wash facility that drains into an approved sediment trap or sediment basin or other sedimentation/filtration device.
7. Minimum dimensions for the entrance are recommended to be as follows:

Tract Area	**Average Tract Depth**	**Recommended Width of Entrance**	**Recommended Depth of Entrance**
<1 Acre	100 feet	15 feet	20 feet
<5 Acres	200 feet	20 feet	50 feet
>5 Acres	>200 feet	25 feet	75 feet

13.0 FLOODWAY/FLOODPLAIN DEVELOPMENT CRITERIA

All development within the floodplain shall be approved by the City Engineer. Listed below are the procedures that shall be followed when developing within a floodplain, whether or not the floodplain has been designated as such by the Federal Emergency Management Agency (FEMA).

13.1 Floodplain Development

Development in the floodway fringe will be allowed provided the following criteria are met:

A. Development within the 100-year floodplain shall be in accordance with the requirements of the City of San Angelo Stormwater Ordinance.

B. Construction of any above ground structure (including fences) which would impede flow will not be allowed within the floodway and/or floodplain easement.

C. Minimum finished floor elevations for proposed development areas within the floodway fringe shall be two feet above the base flood elevation, or one foot above the 100-year flood elevation at fully developed conditions, or one foot above the 500-year storm event, as defined in the FEMA Flood Insurance Study or in Section 5 of this manual for fully developed watershed conditions if a FEMA study does not exist.

D. In no event shall floodplain modifications and/or reclamations increase the water surface upstream of the modifications more than one foot above the base flood elevation as defined by current effective FEMA FIRM Maps and any FEMA approved revisions.

13.2 Floodway Realignment

Floodway realignments require approval by the City Engineer. If a floodway realignment request has been approved by the City Engineer, the developer or his engineer shall submit all necessary data to the City of San Angelo and pay all fees required to obtain a Conditional Letter of Map Revision (CLOMR) from FEMA.

Floodway realignment requests must be approved by the City Engineer and sent to FEMA for review and a CLOMR must be obtained from FEMA prior to issuance of a Floodplain Development Permit.

Subsection 13.2-A contains the Sequence of Action for Reclamation in a FEMA Floodplain and Subsection 13.2-B contains the Sequence of Action for Reclamation in a Floodplain Not Designated by FEMA.

13.2.A. <u>13.2-A Sequence of Action for Reclamation in a FEMA Floodplain</u>

Step 1 The Developer may submit a Preliminary Plat to the Planning Department for review and approval.

Step 2 The Developer submits:

1. Dual element Reclamation Plan for submittal to FEMA including existing and

proposed HEC-2 or HEC-RAS data for existing condition 100-year flows (FEMA) and existing and proposed HEC-2 or HEC-RAS data for fully developed 100-year flows (City). (Cross-sections spaced a maximum of every 500 feet.)

2. Floodplain Development Permit Application to the City.

Step 3 The City reviews and comments on the effects of the Reclamation Plan.

Step 4 The Developer submits a revised Reclamation Plan to the City if required.

Step 5 The Developer submits a final Reclamation Plan to the City for submittal to FEMA with existing FEMA flows for a Conditional Letter of Map Revision (CLOMR) and a final Reclamation Plan to the City for staff approval with the 100-year fully developed flows. All fees required by FEMA for the review shall be paid by the developer.

Step 6 FEMA issues a CLOMR if the submittal is in a form and reflects a design approach and design parameters acceptable to FEMA.

Step 7 The City can then approve the Floodplain Development Permit and Grading Plan, and release the fill activities.

Step 8 The Developer submits a Final Plat to Planning Department for review and approval. In those cases where the Final Plat is submitted prior to the completion of the Reclamation Plan activities, the Final Plat submittal shall identify as a Drainage Easement, all of the property which falls within the fully developed 100-year floodplain before the reclamation. This easement can be removed after the Reclamation activities have been completed and the property has been removed from the Floodplain through a Letter of Map Revision (LOMR) issued by FEMA. All fees required by FEMA for LOMR submittal and approval shall be paid by the developer.

Step 9 Record Drawings of the Reclamation Project must be submitted to the City and FEMA reflecting existing, as built conditions and allowance for fully developed conditions. This submittal must include HEC-2 or HEC-RAS data for the existing and fully developed conditions, compaction results from a Geotechnical firm certifying compaction of the fill to no less than the density specified and sealed by a licensed geotechnical engineer, and any other information required by FEMA.

Step 10 The City will forward the Record Drawings submittal, including the necessary certifications from the developer's engineer, to FEMA for a LOMR to remove the area from the Area of Special Flood Hazard.

Step 11 FEMA issues LOMR. If FEMA does not approve the submittal for the LOMR, alterations to the floodway or floodplain may be required to be removed or changed. Revisions to the Final Plat may be required to modify the drainage easements to reflect the adjustments required by FEMA.

LEGEND

FEMA	-	Federal Emergency Management Agency
FBFM	-	Flood Boundary and Floodway Maps
FIRM	-	Flood Insurance Rate Map
FIS	-	Flood Insurance Study
NFIP	-	National Flood Insurance Program
LOMA	-	Letter of Map Amendment
LOMR	-	Letter of Map Revision
CLOMR	-	Conditional Letter of Map Revision
SFHA	-	Special Flood Hazard Area
FHBM	-	Flood Hazard Boundary Map
BFE	-	Base Flood Elevation

13.2.B. Sequence of Action for Reclamation in a Floodplain Not Designated by FEMA

Step 1 The Developer may submit a Preliminary Plat to the Planning Department for review and approval.

Step 2 The Developer submits:

1. HEC-RAS data for existing condition 100-year flows (FEMA) and existing and proposed HEC-2 or HEC-RAS data for fully developed 100-year flows (City). (Cross-sections spaced a maximum of every 500 feet.)
2. Floodplain Development Permit Application to the City.

Step 3 The City reviews and comments on the effects of the Reclamation Plan.

Step 4 The Developer submits a revised Reclamation Plan to the City if required.

Step 5 The Developer submits a final Reclamation Plan to the City for staff approval with 100-year fully developed condition flows.

Step 6 The City can then approve the Floodplain Development Permit.

Step 7 The Developer submits a Final Plat to Planning Department for review and approval. In those cases where the Final Plat is submitted prior to the completion of the Reclamation Plan activities, the Final Plat submittal shall identify as a Drainage Easement, all of the property which falls within the fully developed 100-year floodplain before the reclamation. This easement can be removed after the Reclamation activities have been completed and the property has been removed from the Floodplain.

Step 8 As Built (Record) Drawings of the Reclamation Project must be submitted to the City reflecting existing and fully developed conditions. This submittal must include existing and proposed HEC-RAS data for the existing and fully developed conditions, compaction results from a geotechnical firm certifying compaction of the fill to no less than the density specified and sealed by a licensed professional engineer.

14.0 MAINTENANCE

Public drainage improvements dedicated in right-of-way or easement and accepted by the City will be, maintained and operated by the City. A maintenance agreement between the Developer and the City may be required for drainage systems not dedicated to and/or accepted by the City. The City is not obligated to accept public drainage improvements dedicated to the City that are not designed in accordance with the criteria in the Stormwater Ordinance and this Manual.

14.1 Floodplain and Drainage Easements

Maintenance of floodplain and drainage easements accepted by but not dedicated to the City is the responsibility of the property owner. Required maintenance includes maintaining the appearance of drainage channels, excluding the area between the top of each channel bank, and overflow swales.

14.2 Maintenance and Liability Criteria for Privately Maintained Facilities

For drainage systems not dedicated to and accepted by the City, the following provisions apply:

A. The owner or developer shall retain their private ownership of the constructed drainage facility (i.e. lake, pond, lagoon, basin) and shall assume full responsibility for the protection of the general public from any health or safety hazards related to the lake, pond or lagoon constructed.

B. The owner or developer shall assume full responsibility for the maintenance of the drainage facility. The owner or developer shall keep the City Engineer advised of the currently responsible agent for this maintenance.

C. For parking lots used as detention basins, the City may require a maintenance/indemnity agreement.

14.3 Maintenance Access Requirements

The following must be provided to allow for proper maintenance of channels and ponds.

A. Access roads and/or ramps shall be provided for all channels and ponds to allow vehicular access for proper maintenance. The location and design of access roads and ramps shall be in accordance with the criteria contained in the San Angelo Code of Ordinances.

B. Access roads shall have a width of at least 8 feet and a cross slope no greater than two percent.

C. Ramps on access roads shall have a vertical grade no steeper than ten percent.

D. Retention basins designed for permanent water storage must include dewatering facilities to allow the basin to be drained for maintenance.

E. Surface on-site detention basins shall be designed to allow for proper maintenance, side slopes, and outlet work operation.

Alternatively, the developer may submit to the City for City approval a detailed plan describing procedures to conduct proper maintenance of channels and ponds dedicated to the City

APPENDIX A. LIST OF REFERENCES

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(21) U.S. Soil Conservation Service, Technical Release No. 20, “TR-20, Project Formulation - Hydrology,” August 1972.

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(24) U.S. Army Corps of Engineers, “HEC-2 Water Surface Profile User's Manual,” 1982.

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(26) Federal Highway Administration, “Hydraulic Design of Highway Culverts,” Washington, D.C., September 1985.

(27) Texas State Department of Highways and Public Transportation, Bridge Division, “Hydraulic Manual,” Third Edition, Austin, Texas, December 1985.

(28) Federal Highway Administration, “HDS No. 1. Hydraulics of Bridge Waterways,” Washington, D.C. December 1985.

(29) Federal Highway Administration, “HY-7, Bridge Waterways Analysis Model,” 1985.

APPENDIX B. LIST OF ABBREVIATIONS

Θ	Slope of inlet gutter depression.
a	Gutter depression.
ac	Acres.
A	Area.
AMC	Antecedent moisture condition.
ASCE	American Society of Civil Engineers.
B	Half width of streets.
b	Width of partial flow in circular conduit.
C	Dimensionless weighted runoff coefficient used in the Rational Method to account for ground cover and/or land use within the watershed.
C1	Parabolic street constant 1
C2	Parabolic street constant 2
CA	Product of runoff coefficient and drainage area used in Rational Method.
cfs	Cubic feet per second.
c_h	crown height (ft)
CLOMR	Conditional Letter of Map Revision.
C_p	Coefficient of peak discharge used in Snyder's unit hydrograph method to account for flood wave and storage conditions.
C_t	Dimensionless coefficient used in Snyder's unit hydrograph method related to the watershed slopes and storage.
D	Diameter.
d_c	Critical depth.
FEMA	Federal Emergency Management Agency.
fps	Feet per second.
Ft	Feet.
g	Acceleration due to gravity (32.2 fps).
SAMDP	San Angelo Master Drainage Plan.
h, H	Head.
h_b	Head loss at a bend.
H_j	Head loss at a junction.
H_L	Total head loss.
HW	Headwater depth.

I	Rainfall intensity.
K_b	Head loss coefficient at a bend.
K_e	Entrance loss coefficient.
K_j	Head loss coefficient at a junction.
L	Length.
L_a	Length of curb inlet required for 100% interception.
L_{ca}	River mileage from design point to center of gravity of drainage area.
L_g	Gutter flow length.
L_o	Overland flow length.
Min.	Minimum or minutes.
MSL	Mean sea level.
n, N	Roughness coefficient used in Manning's formula.
P	Wetted perimeter of flow.
PMF	Probable Maximum Flood.
q	Peak design discharge per unit area.
Q, Q_p	Peak design discharge.
Q_a	Approach flow in gutter upstream of curb inlet.
q_p	Peak rate of discharge of unit hydrograph for unit rainfall duration, t_r.
q_{pR}	Peak rate of discharge of unit hydrograph for unit rainfall duration, t_R.
ROW	Right-of-way.
S, S_o, S_g	Ground slope, overland ground slope, or gutter flow ground slope.
SCS	Soil Conservation Service, (now Natural Resource Conservation Service)
S_e	Slope of energy gradient.
S_f	Slope of frictional gradient.
S_p	Spread of water from curb toward the street centerline for peak flow.
S_w	Slope of hydraulic gradient.
t_c	Time of concentration.
t_p	Hydrograph lag time from midpoint of rainfall duration, t_r, to peak of unit hydrograph.
t_{pR}	Lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph.
t_r	Standard unit rainfall duration.
t_R	Unit rainfall duration in hours other than the standard unit t_r.
T	Top width of flow.

v, V	Velocity.
w, W	Width.
y, Y	Flow depth.
z	Reciprocal of street cross slope.

APPENDIX C. DEFINITIONS OF TECHNICAL TERMS

2-Year Storm Event Given fully developed watershed conditions, the flood having a fifty percent (50%) chance of being equaled or exceeded in any given year. This is also the 2-year mean recurrence interval storm event.

100-Year Storm Event Given fully developed watershed conditions the flood having a one percent chance of being equaled or exceeded in any given year. This is also the 100-year mean recurrence interval storm event based on fully developed watershed conditions, (see also “Base flood,” and “Design flood”).

Abstractions The fractions of precipitation lost to evaporation, transpiration, interception, depression storage and infiltration.

Abutment A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth.

Active agricultural use The presently ongoing use of land for cropping or livestock.

Angle of flare The angle between the direction of a wingwall and the centerline of a culvert or storm drainage outlet or inlet.

Appeal A request for review of the floodplain administrator’s interpretation of any provision of San Angelo Stormwater Ordinance or a request for a variance.

Applicant Any firm, entity, partnership, company, public utility company or individuals, who plan to clear, grub, fill, excavate, grade or otherwise remove the vegetative cover of land, or who plan to either subdivide land and install the appropriate infrastructure or renovate existing structures, shall become applicants for a development permit upon submission of the appropriate application materials.

Apron A floor or lining of concrete, timber, or other suitable material at the toe of a dam, entrance or discharge side of spillway, a chute, or other discharge structure, to protect the waterway from erosion from falling water or turbulent flow.

Area of shallow flooding A designated AO or AH zone on the Flood Insurance Rate Map (FIRM) with a one percent or greater annual chance of flooding to an average depth of one to three feet, where a clearly defined channel does not exist, and the path of flooding is unpredictable and indeterminate. Such flooding is characterized by ponding or sheet flow.

Area of special flood hazard The land in the floodplain within a community subject to a one percent or greater chance of flooding in any given year. This area maybe designated as Zone A on the Flood Hazard Boundary Map (FHBM). After detailed rate making has been completed in preparation for publication of the FIRM, Zone A usually is refined into Zones A, AE, AH, AO, A1-99, VO, VI-30, VE, or V.

As-built plan A set of engineering or site drawings that delineate the specific permitted stormwater management facility as actually constructed.

Backwater The rise of the water level upstream due to an obstruction or constriction in the channel.

Backwater Curve The term applied to the longitudinal profile of the water surface in an open channel when flow is steady but non-uniform.

Baffle Chute A drop structure in a channel with baffles for energy dissipation to permit the lowering of the hydraulic energy gradient in a short distance to accommodate topography.

Baffles Deflector vanes, guides, grids, gratings, or similar devices constructed or placed in flowing water, to: (a) check or effect a more uniform distribution of velocities; (b) absorb energy; (c) divert, guide, or agitate the liquids; and (d) check eddy currents.

Base flood The flood having a one percent chance of being equaled or exceeded in any given year, determined based upon the FEMA guidelines and as shown in the current effective Flood Insurance Study, (FIS). This 100-year mean recurrence interval storm event is based on existing watershed conditions, (see also “100-year storm event,” and “Design flood”).

Base flood elevation The water surface elevation resulting from the base flood.

Basement Any area of the building having it floor subgrade (below ground level) on all sides.

Best management practices A wide range of management procedures, schedules of activities, and prohibitions on practices which have been demonstrated to effectively control the quality and/or quantity of stormwater runoff and which are compatible with the planned land use.

Builder A person, partnership or corporation engaged in clearing, grubbing, filling, excavating, grading, constructing a pad, installing service utility lines and/or constructing or placing a building(s) or other structure(s) on a lot or other type of tract of land that is owned by the person, partnership or corporation, and that will not be further subdivided into other lots.

Calibration Process of checking, adjusting, or standardizing operating characteristics of instruments and model appurtenances on a physical model or coefficients in a mathematical model. The process of evaluating the scale readings of an instrument in terms of the physical quantity to be measured.

Channel A natural or artificial stream that conveys water. Channels are often further classified by their size and purpose. For example, there are primary and secondary channels based on size, but diversions, waterways and chutes are also channels.

Channel improvement The improvement of the flow characteristics of a channel by clearing, excavating, realigning, lining or other means in order to increase its capacity. The term is sometimes used to mean channel stabilization.

Channel Roughness Irregularities in channel configuration which retard the flow of water and dissipate its energy.

Channel stabilization	Erosion prevention and stabilization of velocity distribution in a channel using jetties, drops, revetments, vegetation and other measures.
Check dam	A small dam constructed in a gully or other small watercourse to decrease the streamflow velocity, minimize channel scour and promote deposition of sediment.
Chute	An inclined conduit or structure used for conveying water to a lower level.
City of San Angelo Jurisdiction	All land located within the corporate limits of the City if San Angelo or its extra-territorial jurisdiction.
City-maintained land	Any land in actual ownership of the City of San Angelo; it does not include any type of easements that remain in private ownership.
Conduit	Any open or closed device for conveying flowing water.
Continuity	Continuity of flow exists between two sections of a pipe or channel when the same quantity of water passes the two cross sections and all intermediate cross sections at any one instant.
Crest	The top of a dam, dike, spillway or weir, frequently restricted to the overflow portion.
Critical feature	An integral and readily identifiable part of a flood-protection system, without which the flood protection provided by the entire system would be compromised.
Critical Flow	The state of flow which exhibits the following characteristics: (a) the specific energy is a minimum for a given discharge; (b) the discharge is a maximum for a given specific energy; (c) the specific force is a minimum for a given discharge; (d) the velocity head is equal to half the hydraulic depth in a channel of small slope; (e) the Froude Number is equal to unity.
Crown	(a) The highest point on a transverse section of conduit; (b) the highest point of a roadway cross section.

Culvert Large pipe or other conduit through which a stormwater flows under a road or street.

Curb A vertical or sloping rim along the edge of a roadway, normally constructed integrally with the gutter, which strengthens and protects the pavement edge and clearly defines the pavement edge to vehicle operators.

Curb Inlet A vertical opening in a curb through which the gutter flow passes. The gutter may be undepressed or depressed in the area of the curb opening.

Curb Split The elevation difference between curbs on opposite sides of a street.

Dam A barrier constructed across a watercourse for the purposes of (a) creating a reservoir; (b) diverting water from a conduit or channel.

Degradation The progressive general lowering of a stream channel by erosion.

Depression Storage Collection and storage of rainfall in natural depressions (small puddles) after exceeding infiltration capacity of the soil.

Design flood When in the context of floods, floodplains or flood hazards, the design flood is that level of flood upon which a structure impacted by that flood is designed to withstand. This is assumed to be the flood with a one percent change of being equaled or exceeded in any given year, based upon fully developed watershed conditions, unless specifically stated otherwise, (see also “100-year storm event,” and “Base flood”).

Design Storm or Flood The storm or flood which is used as the basis for design, i.e., against which the structure is designed to provide a stated degree of protection or other specified result.

Detention The storage of storm runoff for a controlled release during or immediately following the design storm.

a. Off-site detention - A detention pond located outside the boundary of the area it serves.

	b. On-site detention - A detention pond which is located within and serves only a specific site or subdivision.
	c. On-stream detention - Detention facilities provided to control excess runoff based on a watershed-wide hydrologic analysis.
Detention basin	A dry basin or depression constructed for the purpose of temporarily storing stormwater runoff and discharging all of that water over time at a rate reduced from the rate that would have otherwise occurred.
Developer	A person, partnership or corporation who owns a tract of land and who is engaged in clearing, grubbing, filling, mining, excavating, grading, installing streets and utilities to be dedicated to or accepted by the City of San Angelo and/or otherwise preparing that tract of land for the eventual division of the tract into one or more lots on which building(s) or other structure(s) will be constructed or placed.
Development	Any manmade change to improved or unimproved real estate, including, but not limited to, adding buildings or other structures, mining, dredging, filling, grading, paving, excavation, drilling operations, grading, clearing or removing the vegetative cover.
Discharge (hydraulics)	The rate of fluid flow, expressed as the volume of fluid passing a point per unit time, commonly expressed as cubic feet per second.
Disturbance	Any operation or activity, such as clearing, grubbing, filling, excavating, mining, cutting, grading, or removing channel linings, which results in the removal or destruction of the protective cover of soil, including vegetative cover, channel linings, retaining walls, and slope protection.
Disturbed areas	Any area or tract of land in which a disturbance is occurring or has occurred but that has not been stabilized.
Downstream Impact Area	The downstream area between the proposed development and the point at which the total drainage reaches a defined Major Stream or City-approved structural outfall (i.e., regional detention pond).
Drainage area	The land area from which water drains to a given point.

Drainage Easement	A delineated portion of land set aside for the overland or underground transfer of stormwater. This area shall not have any permanent structures, fences, or other obstacles hindering the safe transfer of water through the easement.
Drainage System	Drainage systems shall include streets, alleys, storm drains, drainage channels, culverts, bridges, overflow swales and any other facility through which or over which stormwater flows.
Drop Inlet	A storm drain intake structure typically located in unpaved areas. The inlet may extend above the ground level with openings on one or more sides or it may be flush with the ground with a grated cover.
Drop Structures	A sloping or vertical section of a channel designed to reduce the elevation of flowing water without increasing its velocity.
Elevated building	In the case of FEMA-designated zones A1-30, AE, A, A99, AO, AH, B, C, X and D, "elevated building" includes a building elevated by means of fill, so that the lowest finished floor of the building is at least two feet above the water surface elevation of either the 100-year storm event or base flood, which ever is greater.
Emergency spillway	A spillway built to carry runoff in excess of that carried by the principal spillway, preventing overtopping of the structure by the design flood.
Entrance Head	The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.
Entrance Loss	Head lost in eddies or friction at the inlet to a conduit, headwall or structure.
Equal conveyance	The principle of reducing stream conveyance for a proposed alteration with a corresponding reduction in conveyance to the opposite bank of the stream. The right of equal conveyance applies to all owners and uses and may be relinquished only by written agreement.
Erosion	The wearing away of land by action of wind and water.

Evaporation	Process by which water is transferred from land and water masses to the atmosphere.
Exceedance Probability	The statistical probability that an event will equal or exceed a specific magnitude.
Federal Emergency Management Agency (FEMA)	The federal agency which administers the National Flood Insurance Program.
Finished floor	The lowest floor of the lowest enclosed area (including basement). An unfinished or flood-resistant enclosure, usable solely for the parking of vehicles, building access or storage in an area other than a basement area is not considered a building's lowest floor, provided that such enclosure is not built so as to render the structure in violation of the applicable non-elevation design requirements of FEMA 60.3.
Flash Flood	A flood of short duration with a relatively high peak rate of flow, usually resulting from high intensity rainfall over a small area.
Flexible Pipe	Any corrugated metal pipe, pipe-arch, sectional plate pipe, sectional plate pipe-arch or plastic (polyethylene) pipe.
Flood or flooding	A general and temporary condition of partial or complete inundation of normally dry land areas from either the overflow of inland waters and/or the unusual and rapid accumulation or runoff of surface waters from any source.
Flood Control	The elimination or reduction of flood damage by the construction of flood storage reservoirs, channel improvements, dikes and levees, bypass channels, or other engineering works.
Flood Hazard Area	Area subject to flooding by 100-year frequency floods.
Flood Hazard Mitigation	See Stormwater Management.
Flood Insurance Rate Map (FIRM)	The official map on which the Federal Emergency Management Agency has delineated both the areas of special flood hazard and the risk premium zones applicable to the community.
Flood Insurance Study (FIS)	The official report provided in which the Federal Emergency Management Agency has provided flood

profiles, as well as the flood boundary/floodway map and the water surface elevation of the base flood.

Flood Management See Stormwater Management.

Floodplain Geographically the entire area subject to flooding. In usual practice, it is the area subject to flooding by the 100-year frequency flood. In this manual, the "100-year floodplain" refers to the floodplain resulting from a 100-year storm event based on fully watershed development conditions. The "FEMA floodplain" shall refer to the area subject to flooding resulting from the 100-year base flood for current watershed development conditions.

Floodplain Development Permit A permit required before development shall begin in an area designated as within the limits of the base flood.

Floodway The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation by more than a designated height. In this manual, the floodway refers to the floodway resulting from a 100-year flood event based on fully development conditions with a cumulative increase of no more than one foot.

Floodway Fringe Part of the flood hazard area within the floodplain but outside of the floodway.

Floodplain management The operation of an overall program of corrective and preventive measures for reducing flood damage, including but not limited to emergency preparedness plans, flood control works, and floodplain management regulations.

Flood proofing Any combination of structural and non-structural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property, water and sanitary facilities, structures, and their contents.

Flood protection system Those physical structural works for which funds have been authorized, appropriated and expended and which have been constructed specifically to modify flooding in order to reduce the extent of the areas within a community subject to a "special flood hazard" and the extent of the depths of associated flooding. Such a system typically includes dams, reservoirs, levees or dikes. These specialized flood-modifying works are those constructed in conformance with sound engineering standards.

Flume Any open conduit on a prepared grade, trestle or bridge.

Freeboard The distance between the normal operating level and the top of the side of an open conduit left to allow for wave action, floating debris, or any other condition or emergency without overtopping the structure.

Frequency (of storms, floods) Average recurrence interval of a given flood event over long periods of time. Mathematically, frequency is the reciprocal of the exceedance probability.

Froude Number A flow parameter which is a measure of the extent to which gravitational action affects the flow. A Froude number greater than one indicates supercritical flow and a value less than one indicates subcritical flow. The simplest form of the Froude number is given by the equation:

$$F = V/(gD)^{0.5}$$

where: V = Velocity in ft/sec
g = the acceleration due to gravity (32.2 ft/sec^2)
D = depth in ft.

Fully Developed Conditions The level of development anticipated when all of the land within a watershed is developed to the maximum extent allowable, typically determined by comparing existing and projected land uses on vacant and nonconforming properties based upon existing zoning or the latest Land Use Plan, whichever is more intense.

Functionally dependent use A use which cannot perform its intended purpose unless it is located or carried out in proximity to water.

Gabion A wire basket containing earth or stones, deposited with others to provide protection against erosion.

Grade (a) The inclination or slope of a channel, canal, conduit, etc., or natural ground surface, usually expressed in terms of the percentage of number of units of vertical rise (or fall) per unit of horizontal distance. (b) The elevation of the bottom of a conduit, canal, culvert, sewer, etc. (c) The finished surface of a canal bed, road bed, top of an embankment, or bottom of excavation.

Grading Any stripping, cutting, filling, stockpiling or combination thereof which modifies the existing land surface contour.

Grass Any member of the botanical family Poaceae; herbaceous plants with blade like leaves arranged in two ranks on a round to flattened stem. Common examples are Buffalograss, Curly Mesquite, Little Bluestem, and Sideoats Grama.

Grate Inlet An opening in the gutter covered by one or more grates through which the water falls. As with all inlets, grated inlets may be either depressed or undepressed and may be located either on a continuous grade of in a sump.

Gutter A generally shallow waterway adjacent to a curb, used or suitable for drainage of water.

Head The amount of energy per pound of fluid.

Headwater (a) The upper reaches of a stream near its sources; (b) the region where ground waters emerge to form a surface stream; (c) the water upstream from a structure.

High Intensity Node Areas of existing or proposed development that contain a large concentration of buildings and large amounts of pavement. High Intensity nodes typically generate large volumes of stormwater runoff.

Highest adjacent grade The highest natural elevation of the ground surface prior to construction next to the proposed walls of a structure.

Histogram Representation of statistical data by means of rectangles whose widths represent rainfall, runoff, etc. and whose height represents frequency.

Historic structure Any structure that is:

1. Listed individually in the National Register of Historic Places (a listing maintained by the Department of Interior) or preliminarily determined by the secretary of interior as meeting the requirements for individual listing on the National Register;

2. Certified or preliminarily determined by the secretary of the interior as contributing to the historical significance of a registered historic district or a district preliminarily determined by the Secretary to qualify as a registered historic district;

3. Individually listed on a state inventory of historic places in states with historic preservation programs which have been approved by the Secretary of Interior; or;

4. Individually listed on a local inventory or historic places in communities with historic preservation programs that have been certified either:

    A. By an approved state program as determined by the Secretary of the interior or;

    B. Directly by the Secretary of the Interior in states without approved programs.

Hydraulic Control The hydraulic characteristic which determines the stage-discharge relationship in a flowing stream or conduit. The control is usually critical depth, tailwater depth or uniform depth.

Hydraulic Grade Line A line representing the pressure head available at any given point within the system.

Hydraulic Gradient A hydraulic profile of the piezometric level of the water, representing the sum of the depth of flow and the pressure head. In open channel flow it is the water surface.

Hydraulic Jump The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is sometimes used as a means of energy dissipation.

Hydraulics A branch of science that deals with practical applications of the mechanics of water movement.

Hydrograph A graph showing flow (or sometimes stage, velocity or other properties of water) versus time at a given point on a stream or conduit.

Hydrology The science that deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth.

Hyetograph A histogram or graph of rainfall intensity versus time usually during a storm.

Impervious A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

Infiltration (a) The entering of water through the interstices or pores of a soil or other porous medium; (b) the quantity of groundwater which leaks into a sanitary or combined sewer or drain through defective joints, breaks or porous walls; (c) The absorption of water by soil, either as it falls as precipitation or from a stream flowing over the surface.

Inlet (a) An opening into a storm sewer system for the entrance of surface storm runoff, more completely described as a storm sewer inlet; (b) a structure at the diversion end of a conduit; (c) the upstream connection between the surface of the ground and a drain or sewer, for the admission of surface or stormwater.

Intensity As applied to rainfall, a rate usually expressed in inches per hour.

Interception As applied to hydrology, refers to the process by which precipitation is caught and held by foliage, twigs, and branches of trees, shrubs and buildings, never reaching the surface of the ground, and then lost by evaporation.

Invert The floor, bottom, or lowest portion of the internal cross section of a conduit. Used particularly with reference to storm drains, sewers, tunnels, channels and swales.

Lag Time The time difference between two occurrences, such as between rainfall and runoff or pumping of a well and effect on the stream. See Time of Concentration.

Levee A manmade structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control or divert the flow of water so as to provide protection from temporary flooding.

Levee System A flood protection system which consists of a levee or levees and associated structures, such as closure and drainage devices, which are constructed and operated in accordance with sound engineering practices.

Lining	Impervious material such as concrete, clay, grass, plastic, etc., placed on the sides and bottom of a ditch, channel, and reservoir to prevent or reduce seepage of water through the sides and bottom and/or to prevent erosion.
Lip	A small wall on the downstream end of an apron, to break the flow from the apron.
Lowest Floor	The lowest floor of the lowest enclosed area (including basement). An unfinished or flood resistant enclosure, usable solely for parking or vehicles, building access or storage in an area other than a basement area is not considered a building=s lowest floor; provided that such enclosure is not built so as to render the structure in violation of the applicable no-elevation design requirement of Section 60.3 of the National Flood Insurance Program regulations.
Major Drainage Systems	Major drainage facilities may include natural or improved channels, detention or retention ponds, bridges or roadway culverts, overflow swales, street rights-of-way, and others determined by the City. In certain instances, an enclosed storm drain pipe system may be considered part of a major drainage facility if it drains a sump area or is required to prevent flooding a habitable structure by the 100-year storm event.
Manning Coefficient	The coefficient of roughness used in the Manning Equation for flow in open channels.
Manning Equation	A uniform flow equation used to relate velocity, hydraulic radius and the energy gradient.
Minor Drainage Systems	Minor drainage systems are intended to provide conveyance for more frequent nuisance-type flooding and usually consist of streets, storm drain inlets (excluding those in sump areas) and pipes, and roadside ditches and driveway culverts.
Major Stream	A stream or channel with a FEMA-defined floodplain, based on the current FEMA flood insurance study.
Model	Mathematical systems analysis by computer, applied to evaluate rainfall-runoff relationships; simulate watershed characteristics; predict flood and reservoir routings; or for other aspects of planning.

Nappe	The sheet or curtain of water overflowing a weir or dam. When freely overflowing any given structure, it has a well-defined upper and lower surface.
Natural drainage	The dispersal of surface waters through ground absorption and by drainage channels formed by the existing surface topography which exists at the time of adoption of this Ordinance or formed by any manmade change in the surface topography.
Non-erodible	A material, such as natural rock, riprap, concrete, plastic, etc., that will not experience surface wear due to the natural forces of wind, water, or ice.
Open Channel	A conduit in which water flows with a free surface.
Orifice	(a) An opening with closed perimeter and regular form in a plate, wall, or partition, through which water may flow; (b) the end of a small tube, such as a Pitot tube, piezometer, etc.
Outfall	The point where water flows from a stream, river, lake or artificial drain.
Peak Flow or Discharge	The maximum instantaneous flow from a given storm condition at a specific location.
Percolation	To pass through a permeable substance such as ground water flowing through an aquifer.
Permanent erosion controls	Stabilization of erosive or sediment-producing areas by the use of means or techniques that will provide protection against erosion losses for an indefinite time period.
Permeability	The property of a material which permits movement of water through it when saturated and actuated by hydrostatic pressure.
Permissible velocity (hydraulics)	The highest velocity at which water may be carried safely in a channel or other conduit (see sections 8 and 9 of the Stormwater Manual).
Pervious	Applied to a material through which water passes relatively freely.

Porosity	(a) An index of the void characteristics of a soil or stratum as pertaining to percolation; degree of perviousness; (b) the ratio, usually expressed as a percentage, of the volume of the interstices in a quantity of material to the total volume of the material.
Post-development	The condition of the given site and drainage area after the anticipated development has take place.
Precipitation	Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.
Pre-development	The condition of the given site and drainage area prior to development.
Principal spillway	Generally, is constructed of permanent material and designed to regulate the normal water level, provide flood protection and reduce the frequency of operation of the emergency spillway.
Prismatic Channel	A channel with unvarying cross section and constant bottom slope.
Probable Maximum Flood (PMF)	The flood that may be expected from the most severe meteorological and hydrologic conditions that are reasonably possible in the region.
Probable Maximum Precipitation (PMP)	The critical depth-duration-area rainfall relationship for a given area during a storm containing the most critical meteorological conditions considered probable of occurring.
Public erosion nuisance	A situation in which erosion of or sediment from one location is causing a bothersome or unsightly condition on another property owned by a different individual or entity. A bothersome or unsightly condition or burden includes silt, mud or similar debris originating from one property but being deposited onto a second off-site property in which that off-site owner may have to remove or clean up the deposit due to liability, statutory, aesthetic, drainage or property damage concerns. The adversely affected off-site property owner could be a private citizen, corporation, government or other entity.
Rainfall Duration	The length of time over which a single rainfall event occurs.

Rainfall Frequency The average recurrence interval of rainfall events, averaged over long periods of time.

Rainfall Intensity The rate of accumulation of rainfall, usually in inches or millimeters per hour.

Rational Formula A traditional means of relating runoff from a drainage basin to the intensity of the storm rainfall, the size of the basin, and the characteristics of the basin (such as land use, impervious cover).

Reach Any length of river or channel. Normally refers to sections which are uniform with respect to discharge, depth, area or slope, or sections between gauging stations.

Recurrence Interval The average interval of time within which a given event is statistically predicted to be equaled or exceeded once. For an annual series (as opposed to a partial duration series), it is the probability of occurrence interval. Thus a flood having a recurrence interval of 100 years has a one percent probability of being equaled or exceeded.

Regulatory floodway The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the "base flood," as calculated by the Federal Emergency Management Agency, without cumulatively increasing the water surface elevation more than a designated height. This floodway is used by FEMA to determine compliance with its regulations.

Retention basin A pond or other water body which has been designed to have both a conservation pool for holding some water indefinitely and a flood storage pool for storing stormwater runoff on a temporary basis for the purpose of reducing the peak discharge from the basin for improving the water quality of the stream flow.

Return Period See Recurrence Interval

Reynold's Number A flow parameter which is a measure of the viscous effects on the flow. Typically defined as:

$$Re = VD/\mu$$

where,

V = Velocity
D = Depth
μ = Kinematic viscosity of the fluid

Rigid Pipe Any concrete, clay or cast iron pipe.

Riprap (Revetment) Forms of bank channel protection, usually using rock or concrete. Riprap is a term sometimes applied to stone which is dumped rather than placed more carefully.

Routing Routing is a technique used to predict the temporal and spatial variations of a flood wave as it traverses a river reach or reservoir. Generally, routing technique may be classified into two categories - hydrologic routing and hydraulic routing.

ROW (Right-of-Way) A strip of land dedicated for public streets and/or related facilities, including utilities, drainage systems and other transportation uses.

ROW Width The shortest horizontal distance between the lines which delineate the limits of right-of-way of a street.

Runoff That part of the precipitation that exceeds abstractions and reaches a stream or storm drain.

Runoff Coefficient A decimal number used in the Rational Formula, which defines the runoff characteristics (i.e., land use impervious cover) of the drainage area under consideration. It may be applied to an entire drainage basin as a composite representation or it may be applied to a small individual area such as one residential lot.

Runoff Total The total volume of flow from a drainage area for a definite period of time such as a day, month, year, or for the duration of a particular storm.

Scour The erosive action of running water in streams or channels in excavating and carrying away material from the bed and banks.

NRCS Runoff Curve Number Index number used by the Natural Resources Conservation Service (formerly Soil Conservation Service) as a measure of the tendency of rainfall to run off into streams rather than evaporate or infiltrate.

Sediment	Material of soil and rock origin transported, carried, or deposited by flowing water.
Sheet flow	Water, usually storm runoff, flowing in a thin layer over the ground surface. Synonymous with "overland flow."
Significant rise	Any rise in the design flood water surface elevation at a particular location along a stream.
Sidewalk	A paved area within the street right-of-way specifically designed for pedestrians and/or bicyclists.
Slope, Critical	(a) The slope or grade of a channel that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; (b) the slope of a conduit which will produce critical flow.
Slope, Friction	The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradient. Where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction scope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.
Soffit	The top of the inside of a pipe. In a pipe, the uppermost point on the inside of the structure.
Soil	The unconsolidated mineral and organic material on the immediate surface of the earth that serves as a natural medium for the growth of plants.
Spillway	A waterway in or about a dam or other hydraulic structure for the escape of excess water.
Stabilized	Protected from possible erosion losses, usually by the use of vegetative cover.
Standard Project Flood	A flood that has a magnitude of approximately one-half of the Probable Maximum Flood, as determined on a case-by-case basis using the accepted engineering methods.
Steady Flow	Open channel flow is said to be steady if the depth of flow does not change and can be assumed to be constant during the time interval under consideration.

Stilling Basin Pool of water conventionally used, as part of a drop structure or other structure, to dissipate energy.

Storm frequency An expression or measure of how often a hydrologic event of given size or magnitude should, on an average, be equaled or exceeded.

Storm Hydrology The branch of hydrology that concentrates on the calculation of runoff from storm rainfall.

Stormwater Management The control of storm runoff by means of land use restrictions, detention storage, erosion control, and/or drainage systems.

Stormwater Model Mathematical method of solving stormwater problems by computer technology.

Street Classifications:

Alley An alley is a passageway designed primarily to provide access to or from the rear or side of property otherwise abutting on a public street.

Arterial Street Arterial streets are designed to carry high volumes of through traffic. Arterial streets serve as a link between major activity centers within the urban area.

Collector Street The primary function of a collector street is to serve abutting traffic from intersecting local streets and expedite the movement of this traffic in the most direct route to an arterial street or other collector street.

Freeway Freeways are divided arterial highways designed with full control of access and grade separations at all intersections. Freeways provide movement of high volumes of traffic at relatively high speeds. This system carries most of the trips entering and leaving the urban area, as well as most of the through movements bypassing the central city.

Residential Street The primary function of a residential street is to serve abutting land use and traffic within a neighborhood or limited residential district. A local street is not generally continuous through several districts.

Parkway	(a) a freeway which does not have continuous frontage roads; (b) greenspace buffer between the roadway and adjacent development.
Subcritical Flow	Relatively deep, tranquil flow with low flow velocities. The Froude Number is less than 1.0 for subcritical flow conditions.
Supercritical Flow	Relatively shallow, turbulent flow with high velocities. The Froude Number is greater than 1.0 for supercritical flow conditions.
Tailwater	The depth of flow in the stream directly downstream of a drainage facility or other man-made control structure.
Temporary erosion protection	The stabilization of erosive or sediment-producing areas for a specific time period, usually during a construction job.
Time of Concentration	The estimated time in minutes required for runoff to flow from the most hydraulically remote section of the drainage area to the point at which the flow is to be determined. Hydraulically remote refer to the travel path with the longest flow travel time, not necessarily the longest linear distance.
Total Head Line (Energy Line)	A line representing the energy in flowing water. The elevation of the energy line is equal to the elevation of the flow line plus the depth plus the velocity head plus the pressure head.
Trash Rack	Racks, gratings, or mesh designed so as to prevent tree limbs, water-borne debris and rubbish from plugging the outlets from a dam or detention basin.
Trunk Line	The main line of a storm drain system, extending from manhole to manhole or from manhole to outlet structure.
Ultimate Development	The condition of the watershed after the entire watershed has undergone development.
Uniform Channel	A channel with a constant cross section, slope and roughness.
Uniform Flow	Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel.

Unit Hydrograph The direct runoff hydrograph resulting from one inch of precipitation excess, distributed uniformly over a watershed for a specified duration.

Velocity Head The energy per unit weight of water due to its velocity. The velocity head also represents the vertical distance water must fall freely under gravity to reach its velocity. The velocity head can be computed from:

$$H_v = V^2/2g$$

where ,

H_v = Velocity head in ft

V = Velocity in ft/sec

g = acceleration due to gravity 32.2 ft/sec^2)

Warped Headwall The wingwalls are tapered from vertical at the abutment to a stable bank slope at the end of the wall.

Water surface elevation The height, in relation to the National Geodetic Vertical Datum (NGVD) of 1929 (or other datum, where specified), of floods of various magnitudes and frequencies in the floodplains of riverine areas.

Water Year The water year commonly used in the United States is the period from October 1 of the previous calendar year to September 30 of the numbered calendar year.

Watershed The area contributing storm runoff to a stream or drainage system. Other terms are drainage area, drainage basin and catchment area.