City of Lancaster

Storm Water 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 Lancaster, 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 storm water, and erosion protection from storm water 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 Public Works Director.

1.2 City Of Lancaster Storm Water Management Ordinance

The City of Lancaster's Code of Ordinances contains requirements and guidelines for the design of storm water management facilities. Where there is any conflict between the Storm Water Management Ordinance and this manual, the Storm Water Management Ordinance must take precedence.

2.0 CITY OF LANCASTER STORM WATER MANAGEMENT POLICY

2.1 Purpose of Storm Water Management Policy

The purpose of the City of Lancaster's storm water management policy is to protect and provide for the safety and welfare of the community, to mitigate flood damage and soil erosion to private and public property within the community, and to provide for water quality protection.

2.2 Application of Storm Water Management Policy

The City's policies for storm water management govern the planning, design, construction, operation, and maintenance of storm drainage facilities within the City's jurisdiction. This storm water management policy is based on the City of Lancaster Storm Water Management Ordinance and this Storm Water Design Manual. Each is considered effective on the date of acceptance by the Lancaster City Council. This storm water management policy will apply to any storm water management system improvement plans that are released for construction after the date of Council approval of this manual.

2.3 Regional Storm Water Management

The City may choose to implement regional storm water management to limit the impact of development on runoff within the watersheds of water bodies with Federal Emergency Management Agency (FEMA)-defined floodplains. This will be a coordinated effort with other governmental entities within each watershed. The design engineer is encouraged to check with the City prior to the design of individual detention basins in these watersheds.

2.4 Drainage Plan Submittals

A review process has been established by the City to provide control of all development activities related to storm water runoff through natural or manmade facilities. As part of the review process, a Preliminary Drainage Plan containing the conceptual layout of the proposed storm drainage system must be submitted in accordance with the City of Lancaster Storm Water Management Ordinance preliminary platting process requirements.

A Final Drainage Plan must be submitted at the time of the final plat application. The Final Drainage Plan must include the appropriate computation sheets as required in Sections 2.8 and 2.11 of this manual.

2.5 Drainage System Classifications

2.5.1 Major Drainage Systems

Major drainage systems may include natural or improved channels, detention reservoirs, bridges or roadway culverts, overflow swales and street rights-of-way. In certain instances, an enclosed storm drain pipe system may be considered part of a major drainage system if it drains a sump area. The design storm, as defined by the 100-year frequency storm event, must be contained within the rights-of-way or dedicated easements of all major drainage systems to provide for public safety and welfare.

2.5.2 Minor Drainage Systems

Minor drainage systems are intended to provide conveyance for more frequent nuisance-type storm events and usually consist of streets, storm drain inlets and pipes (excluding those in sump areas), roadside ditches, and driveway culverts. To enhance the quality of life and provide for public safety, minor drainage systems are required to provide conveyance of the runoff from a 25-year frequency storm event.

2.6 Floodplain Development

Development within and improvements to the flood hazard area must be consistent with the criteria set forth in Section 13 (Floodway/Floodplain Development Criteria) of this manual and the most recent Storm Water Master Plan (SWMP).

2.7 Drainage Structure Aesthetics

Drainage design in the urban environment must also consider appearance as an integral part of the design. In an effort to maintain the natural aesthetics of its existing floodplains, the City strongly encourages preservation of the natural floodplains as greenbelt areas, and in some areas, the City may require the floodplain to be designated as a deed-restricted greenbelt area. When utilized, the design of drop structures and other hydraulic structures should blend with the natural surroundings as much as possible to maintain the aesthetics of the natural channel.

2.8 Drainage Design Computations

Computations to support all drainage designs must be submitted to the Public Works Director for review as part of the Final Drainage Plan and must be summarized in the form of the standard computation sheets contained in this manual, unless otherwise approved by the Public Works Director. Computer programs used to perform computations must be limited to those referenced in this manual, unless otherwise approved by the Public Works Director. The Lancaster Storm Water Management Ordinance requires that all computations submitted must be certified by an engineer experienced in municipal storm water drainage design and licensed in the State of Texas in accordance with the requirements set forth by the Texas Board of Professional Engineers. Storm water runoff computations must be based upon conditions representing ultimate watershed development conditions in accordance with the Lancaster Storm Water Management Ordinance and this manual.

2.9 Construction of Drainage Facilities

Erosion and sediment discharge associated with the construction of drainage facilities must be minimized in accordance with Section 15.10 of the Lancaster Storm Water Management Ordinance and Texas Commission on Environmental Quality (TCEQ) Texas Pollutant Discharge Elimination System (TPDES) requirements. Efforts must be made to maximize the protection of trees and vegetation during construction and development activities.

2.10 Maintenance of Drainage Facilities

Major and minor drainage system components dedicated to and accepted by the City will be maintained by the City. The Public Works Director may approve a request that a drainage system component not be dedicated to the City; however, the drainage system must be maintained by the property owner. Floodplain and drainage easements must be maintained by the property owner. The appearance of drainage channels, excluding the area between the top of each channel bank, and overflow swales must be maintained by the adjacent property owners.

2.11 Drainage Plan Requirements

To maintain a proper transition, storm drain plans must be prepared for both off-site and on-site drainage as part of the platting process. Criteria for on-site development must also apply to off-site improvements. The construction of all improvements must be in accordance with the current specifications and regulations adopted by the City. If applicable to the project, the final drainage plan must include the components discussed in the following sections.

2.11.1 Drainage Area Map

1. Use a scale of one inch equals 200 feet 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.
2. Show existing and proposed storm drains and inlets.
3. Show all drainage easements proposed to be utilized by the development.
4. Indicate sub-areas for each alley, street, and inlet as well as off-site areas.
5. Indicate contours on map for on-site and off-site areas.
6. Indicate zoning in the drainage area.
7. Show points of concentration.
8. Indicate runoff direction at all inlets, dead-end streets, and alleys; to adjacent additions; and to undeveloped land, as applicable.
9. Provide runoff calculations for all areas showing acreage, runoff coefficient, inlet time and storm frequency.
10. Indicate all crests, sags and street and alley intersections with flow arrows.
11. Provide hydraulic length path corresponding to calculated time of concentration (T_c).

2.11.2 Storm Drain Plan and Profile Sheets

1. Show plan and profile of all storm drains on separate sheets from paving plans.
2. Specify the type of storm drain pipe to be used.
3. Indicate property lines along storm drains and show easements with dimensions.
4. Show all existing utilities in plan view of storm drains, and show those existing utilities in profile where possible conflicts may occur with the storm drains.
5. Indicate existing and proposed ground line and improvements on all street, alley and storm drains profiles.
6. Show hydraulic gradient with computations.
7. Show laterals on trunk profile with stations.
8. Number inlets according to the number designation given for the area or sub area contributing runoff to the inlet.
9. Indicate size and type of inlet on plan view, lateral size and flow line, paving station and top-of-curb elevation.
10. Indicate quantity and direction of flows at all inlets, stubouts, pipes and intakes.
11. Show future streets and grades, where applicable.
12. Show water surface, velocity and typical section of receiving water body at outfalls of storm drains.
13. Specify compacted fill and compaction criteria to a minimum 95 percent Standard Proctor density.
14. Show size of pipe, length of each pipe size, and stationing at 100-foot intervals in the plan view.
15. Begin and end each sheet with even or 50-foot stationing.

16. Show diameter of pipes, physical grade, discharge, capacity of pipe, slope of hydraulic gradient and velocity in the pipe in the profile view.
17. Show elevation of flow lines at 100-foot intervals on the profile.
18. Provide benchmark information corresponding to City's benchmark system.
19. Show capacities, flows, and velocities of the existing system into which the proposed system is being connected.
20. Show details of all connection boxes, headwalls on storm drains, flumes or any other item not in a standard detail sheet.
21. Provide profile where existing utility is crossed.
22. Show headwalls and specify type for all storm drains at outfall.
23. Show if curbing in alleys is needed to add extra capacity.
24. Runoff from alleys and other paved areas are not to cause street capacity to be exceeded.
25. Show curve data for all storm drains.
26. Tie storm drains stationing with paving stations.
27. On all dead-end streets and alleys, show grades for drainage overflow path on the plan and profile sheets, and show erosion controls.
28. Specify concrete strength for all structures.
29. Provide sections for road, railroad and other ditches with profiles and hydraulic computations. Show design water surface on profile.
30. Show outlet protection measures.

2.11.3 Bridge Plans

1. Show the elevation of the lowest member (i.e., low chord) of the bridge and the 100-year water surface elevation.
2. Indicate borings on plans.
3. Provide soils report.
4. Show a section at the bridge.
5. Provide hydraulic calculations on all sections.
6. Provide structural details and calculations with dead load deflection diagram.
7. Provide vertical and horizontal alignment.
8. Provide calculations and details for all erosion protection.
9. Provide scour analysis data on plans.

2.11.4 Creek Alteration and Channel Plans

1. Show stationing in plan and profile.
2. Indicate flow lines, banks, design water surface, and hydraulic computations.
3. Indicate the nature of banks, such as rock, earth, etc.

4. Provide cross sections at a minimum of 50-foot intervals with ties to property lines and easements.
5. Show side slopes of creeks, channels, etc.
6. Specify compacted fill, where fill is proposed, to a minimum 95 percent Standard Proctor density.
7. Indicate any adjacent alley or street elevations on creek profile.
8. Show any temporary or permanent erosion controls.
9. Indicate existing and proposed velocities.
10. Show access and/or maintenance easements.
11. Identify the datum, benchmarks and date of re-leveling the benchmarks to which the flood and ground elevations are referenced. FEMA benchmarks must be matched.
12. Show existing Finished Floor Elevation (FFE), or proposed minimum FFE of all structures, existing or proposed, adjacent to creek or channel alterations.

2.11.5 Environmental Effects and Required Regulatory Permits Report

A report will be submitted with the preliminary and final plans and will include the following:

1. A description of the existing environmental conditions of the creek and overbank areas of the project site. The description of these conditions is to include the characterization of creek features, such as bed quality and material, magnitude and continuity of flow, bank quality and material, bank erosion, topographic relief, disturbances to the natural character of the creek, soil types and existing and proposed land uses of the site and surrounding areas. The report is also to include a description of whether wetlands or other special aquatic sites are present or not.
2. The assessment will include a description of proposed strategies to mitigate adverse impacts. Examples of strategies include tree wells, temporary construction and permanent erosion and sedimentation controls, vegetative buffers, and replacement plantings.
3. For preliminary submittal of plans, the report is to identify all environmental permits that will or may be required by regulatory agencies. Such permits include, but are not limited to the TCEQ TPDES storm water permit for construction activities and U.S. Army Corps of Engineers (USACE) Section 404 permits. Sediment and erosion control requirements are discussed in more detail in Section 12 of this manual.
4. For the final submittal of plans, the report is to provide a list of all required permits necessary to construct the project and a copy of the approved permits.

2.11.6 Detention and Retention Facilities

1. Show plan view of detention/retention area and outlet structure.
2. Delineate limits of conservation pool, sediment storage area, flood storage pool and/or freeboard.
3. Indicate size, dimensions, total capacity and design discharge velocity of the outlet structure.
4. Show any erosion control features at the discharge point of the outlet structure.
5. Specify side slopes of basin and outlet structure.

6. Show existing or proposed structures or other facilities downstream of the outlet structure and emergency spillway, and provide information sufficient to show that the downstream facilities will not be subjected to inundation (or increased inundation) or otherwise affected by the discharge from the basin. Calculations to verify downstream adequacy shall be performed to the nearest major receiving stream or downstream to the point where the developed property is no more than 10% of the total drainage area for each proposed development.
7. Indicate locations and quantities of all inflows to the basin.
8. State the design time to empty the basin.
9. Indicate total volume of flood pool.

2.11.7 Levees

1. Show location, extent, nature, dimensions, and other relevant features of levee embankments and associated interior and exterior drainage facilities.
2. Provide engineering analysis addressing potential erosion of the levee embankments, including levee embankment stability and seepage through the levee during flood events.
3. Perform compaction of fill material in accordance with standard engineering practices.
4. Analyze interior drainage concerns. Identify sources of interior flooding and extent and depth of such flooding. Consider capacity of pumps and other drainage devices for evacuating interior waters.
5. Write an operations manual that discusses
    a. the flood warning system to trigger closures,
    b. closure operations,
    c. procedures and personnel,
    d. operation plans for interior drainage facilities
    e. an inspection program with at least an annual frequency, and
    f. maintenance plans, procedures and frequencies.
6. Provide all other information requested or required by the Public Works Director and/or the Federal Emergency Management Agency (FEMA).

2.11.8 Storm Water Utility Impervious Area Calculations

For non-single family residential developments, submit a calculated area (in square feet) with support documentation showing the total impervious area for the development. This information will serve as the basis for the establishment of the development’s storm water utility fee rate. Features to calculate in the impervious area determination include:

1. Rooftops
2. Paved parking lots
3. Packed gravel areas
4. Driveways
5. Patios

6. Any other permanent surface feature that increases surface runoff by inhibiting precipitation from entering directly into the ground, except for the following:
    a. Trees, shrubs, ground cover, and other vegetation
    b. Swimming pools
    c. Detention/retention ponds

2.12 Erosion Hazard Setback Regulation

An erosion hazard setback zone determination is necessary for the banks of streams in which the natural channel is to be preserved. The purpose of the setbacks is to reduce the amount of structural damage caused by the erosion of the bank. With the application of streambank erosion hazard setbacks, an easement is dedicated to the City such that no structure can be located, constructed, or maintained in the area encompassing the erosion hazard setback.

Variations to the setback policy are allowed by the City only with the approval of the Public Works Director. The City may allow for streambank stabilization as an alternative to dedicating the erosion hazard setback zone.

Streambank erosion hazard setbacks may be required to extend beyond the limits of the regulatory floodplain. The procedure for determining the streambank erosion hazard setback zone is as follows:

1. Locate the toe of the natural stream bank.
2. From this toe, construct a line sloping at 4 horizontal to 1 vertical towards the bank until it intersects natural ground.
3. From this intersection, add 15 feet in the direction away from the stream to locate the outer edge of the erosion hazard setback.

The erosion hazard setback area may be reduced in places where the streambanks are composed partially or entirely of rock. In these areas, the interface of the natural streambank with the top of the unweathered rock strata should be located with the assistance of a qualified geotechnical engineer or geologist. From this point, a line sloping at 3 horizontal to 1 vertical is constructed until its intersection with natural ground. The erosion hazard setback is located 15 feet in the direction away from the stream from this intersection.

3.0 DETERMINATION OF DESIGN DISCHARGE

3.1 Design Frequencies

The storm frequencies listed in Table 3-1 for fully developed watershed conditions must be used in storm drain designs in the City. Alternative approaches are only permitted with the approval of the Public Works Director.

Table 3-1. Design Frequencies

Storm Drainage Facility	Frequency	Freeboard	Point of Reference
Enclosed Pipe System	25 year	1 foot	Throat of inlet
Enclosed Pipe System Draining Sump Areas	100 year	Top of Curb	Curb
Street With Curb	100 year	Top of Curb	Curb
Street Without Curb	100 year	Top of Crown or within ROW, whichever is more restrictive	Crown/ROW
Channels	100 year	1 foot	Top of bank
Culverts	100 year	1 foot	Top of curb
Bridges	100 year	1 foot	Low chord of bridge
Floodway and Floodplains	100 year	1 foot	FFE
Levees	FEMA's design requirements. Lesser freeboard will be allowed under special circumstances with the approval of the Public Works Director.		

3.2 Design Frequencies for Dams or Impoundments

Lakes and dams will be designed using the size and hazard classification adopted by the TCEQ under 30 TAC §299 (TXSOS 2003), which provides for the safe construction, maintenance, repair and removal of dams located in the State of Texas. The design criteria will be determined by the Public Works Director based on information furnished by the owner. At a minimum, the following criteria will be used by the Public Works Director to classify a dam:

- Size
- Hazard potential
- Spillway design flood (SDF)
- Additional design requirements
- Maintenance and liability criteria

3.2.1 Size

The classification for size, as summarized in Table 3-2, is determined by the height of the dam or storage capacity, whichever gives the larger size category. Dam height is defined as the vertical distance from the lowest point on the crest of the dam (excluding spillways) to the lowest elevation on the centerline or downstream toe of the dam (including the natural stream channel). Storage is defined as the maximum water storage capacity at the lowest point on the crest of the dam (excluding spillways).

Table 3-2. 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

3.2.2 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. The categories listed in Table 3-3 will be used.

Table 3-3. Hazard Potential Classification

Category	Extent of Development Loss of Life	Extent of Development Economic Loss
Low	None expected (no permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agricultural improvements)
Significant	Possible, but not expected(A small number of inhabitable structures)	Appreciable (Notable agricultural, industrial or commercial development)
High	Expected (Urban development or large number of inhabitable structures)	Excessive (Extensive public, industrial,commercial or agricultural development) or agriculture)

Hazard classification pertains to potential loss of human life and/or property damage within either existing or potential developments in the area downstream of the dam in event of failure or malfunction of the dam or appurtenant facilities. Hazard classification does not indicate any condition of the dam itself.

Dams in the low hazard potential category are normally those in rural areas where failure may damage farm buildings, limited agricultural improvements, and county roads. Significant hazard potential category dams are usually those in predominantly rural areas where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use (including the design purpose of the facility) of relatively important public utilities. Dams in the high hazard potential category are usually those in or near urban areas where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial, or commercial facilities, important public utilities (including the design purpose of the facility), main highways, or railroads.

3.2.3 Spillway Design Flood (SDF)

The classification determined from the above criteria will be used to determine the SDF for the dam. The total capacity of a dam structure, including principal and emergency spillways, must be adequate to pass the SDF without a failure of the dam. The SDF is computed as a percentage of the Probable Maximum Flood (PMF) hydrograph for various dam classifications, as shown in Table 3-4.

Table 3-4. 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 must be based on either hydraulic height or impoundment size, whichever is greater. In all cases, the minimum principal spillway design capacity is the 100-year design flood. In certain cases, a dam breach analysis may be required to determine the proper classification of the structure. For all structures requiring a spillway design flood (SDF) equal to the PMF, a dam breach analysis is required to determine the downstream consequences of a failure of the dam. All dams must be constructed at the SDF elevation without freeboard.

3.2.4 Additional Design Requirements

1. An engineering plan for such construction, accompanied by complete drainage design information and sealed by a licensed professional engineer, must be submitted for approval by the City.
2. The spillway and any emergency overflow areas must be located so that floodwaters will not inundate any permanent habitable structures.
3. The minimum total flood storage must include the 100-year, 24-hour storm without requiring emergency spillway operation, regardless of critical inflow design storm peaks.
4. The design must 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 must be submitted to the Public Works Director.

5. Any existing dams which are included in the project drainage area must comply with applicable federal, state, county, and city safety requirements for such structures. Improvements may be required to upgrade the structure to the currently adopted guidelines.
6. A breach analysis of the structure should be performed in order to determine the breach hazard area downstream. For projects being constructed downstream of the dam, the project should stay out of the breach hazard area. For projects constructed upstream of the dam, the project owner should demonstrate no increase in the breach risk or hazard area due to construction of the project.
7. Before removing, enlarging or altering any existing impoundment, the applicant must furnish to the City a study of the effects of the alteration upon flooding conditions both upstream and downstream. The study must be prepared by a licensed professional engineer and submitted to the Public Works Director for approval prior to making the proposed alteration.
8. Any improvements to existing dams or lakes or construction of new impoundments must be made at the expense of the developer prior to acceptance of the adjacent street, utilities, and drainage improvements, as provided for under the subdivision regulations.

3.2.5 Maintenance and Liability Criteria

The owner or developer must retain ownership of the constructed lake, pond, lagoon or basin and must assume full responsibility for the protection of the general public from any health or safety hazards related to the lake, pond, lagoon, or basin constructed.

The owner or developer must assume full responsibility for the maintenance of the lake, pond, lagoon, or basin constructed. The owner or developer must keep the Public Works Director advised of the agent currently responsible for this maintenance.

4.0 DESIGN RAINFALL

4.1 Rainfall Intensity-Duration-Frequency

The U.S. Geological Survey (USGS), in cooperation with the Texas Department of Transportation (TxDOT) recently conducted a study of the depth-duration frequency (DDF) of precipitation for Texas. The North Central Texas Council of Governments, as part of their integrated Storm Water Management (iSWM) Program, contracted with USGS to obtain the depth-duration frequency relationships for each county in the North Central Texas region. Using this latest rainfall relationship data, intensity-duration-frequency (IDF) relationships were developed.

Equation 4-1 is the formula used to determine IDF data for Dallas County which are applicable to the City of Lancaster. Table 4-1 lists the appropriate coefficients to use in Equation 4-1 at specified frequency intervals. Table 4-2 was developed using Equation 4-1 and the parameters from Table 4-1. Table 4-2 has the data for durations ranging from 5 minutes to 24 hours with average return periods of one, two, five, 10, 25, 50, and 100 years. Point rainfall intensities can also be calculated using Equation 4-1.

$$I = b/(T_d + d)^e \qquad \text{(Equation 4-1)}$$

where:

I = Point rainfall intensity

T_d = Duration of rainfall in minutes

b,d,e = Coefficients for intensity-duration-frequency data as determined using Tables 4-1 and 4-2

Table 4-1. Rainfall IDF Equation Coefficients

	Return Period (Years)						
Coefficient	**1**	**2**	**5**	**10**	**25**	**50**	**100**
e	0.833	0.815	0.804	0.798	0.782	0.770	0.759
b	47.679	55.179	70.024	79.931	87.970	94.058	100.079
d	9	10	12	13	13	13	13

4.2 Probable Maximum Precipitation (PMP)

The design rainfall for dams or impoundments is based on a percentage of the probable maximum precipitation (PMP), as specified in Section 5.2. The Probable Maximum Precipitation (PMP) is developed using Hydro-Meteorological Report Nos. 51 and 52 (NWS 1978, 1982). Rainfall depths for various durations and storm area sizes are taken from these reports and input into HMR52, a computer model developed by the U.S. Army Corps of Engineers that distributes the precipitation over the watershed and calculates the basin average PMP rainfall during each 15-minute time step for each of the sub-basins in the watershed. The storm center should be varied in HMR52, producing incremental rainfall amounts for input into HEC-1. Initially the storm should be centered at the centroid of the entire drainage area. Additional runs should then be made with storms centers moving away from the centroid in 0.1 mile increments to verify the critical storm location and configuration that produces the maximum rainfall amount.

4.3 Rainfall Loss Rates

To obtain the effective design rainfall, losses due to interception, infiltration and depression storage are deducted from the total 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 runoff coefficient (C), as described in Section 5.4. For the unit hydrograph methods described in Section 5.3, the initial-uniform constant loss method or the U.S.

Department of Agriculture Soil Conservation Service (USDA SCS) runoff curve number method may be used with either the SCS dimensionless unit hydrograph or Snyder's method.

Table 4-2. Rainfall IDF Data

Hours	Minutes	Return Period (Years)						
		1	2	5	10	25	50	100
		Rainfall Intensity (inches per hour)						
0.083	5	5.30	6.06	7.17	7.96	9.18	10.15	11.17
	6	5.00	5.75	6.85	7.62	8.80	9.74	10.72
	7	4.74	5.48	6.55	7.31	8.45	9.36	10.31
	8	4.51	5.23	6.29	7.03	8.14	9.02	9.94
	9	4.30	5.00	6.05	6.78	7.85	8.70	9.59
	10	4.11	4.80	5.82	6.54	7.58	8.41	9.27
	11	3.94	4.61	5.62	6.32	7.33	8.14	8.98
	12	3.78	4.44	5.43	6.12	7.10	7.88	8.70
	13	3.64	4.28	5.26	5.93	6.89	7.65	8.45
	14	3.50	4.13	5.09	5.76	6.69	7.43	8.21
0.250	15	3.38	4.00	4.94	5.59	6.50	7.22	7.99
	16	3.27	3.87	4.80	5.44	6.32	7.03	7.78
	17	3.16	3.75	4.66	5.29	6.16	6.85	7.58
	18	3.07	3.64	4.54	5.15	6.00	6.68	7.39
	19	2.97	3.54	4.42	5.03	5.85	6.52	7.22
	20	2.89	3.45	4.31	4.90	5.72	6.37	7.05
	21	2.81	3.35	4.20	4.79	5.58	6.22	6.89
	22	2.73	3.27	4.10	4.68	5.46	6.08	6.74
	23	2.66	3.19	4.01	4.57	5.34	5.95	6.60
	24	2.59	3.11	3.92	4.48	5.23	5.83	6.46
	25	2.53	3.04	3.83	4.38	5.12	5.71	6.34
	26	2.47	2.97	3.75	4.29	5.02	5.60	6.21
	27	2.41	2.90	3.68	4.21	4.92	5.49	6.09
	28	2.36	2.84	3.60	4.12	4.82	5.39	5.98
	29	2.31	2.78	3.53	4.05	4.73	5.29	5.87
0.500	30	2.26	2.73	3.46	3.97	4.65	5.19	5.77
	31	2.21	2.67	3.40	3.90	4.56	5.10	5.67
	32	2.17	2.62	3.34	3.83	4.48	5.01	5.57
	33	2.12	2.57	3.28	3.76	4.41	4.93	5.48
	34	2.08	2.52	3.22	3.70	4.33	4.85	5.39
	35	2.04	2.48	3.16	3.64	4.26	4.77	5.31
	36	2.00	2.43	3.11	3.58	4.20	4.69	5.22
	37	1.97	2.39	3.06	3.52	4.13	4.62	5.14
	38	1.93	2.35	3.01	3.46	4.07	4.55	5.07
	39	1.90	2.31	2.96	3.41	4.01	4.48	4.99
	40	1.87	2.27	2.92	3.36	3.95	4.42	4.92
	41	1.84	2.24	2.87	3.31	3.89	4.36	4.85
	42	1.81	2.20	2.83	3.26	3.83	4.30	4.79
	43	1.78	2.17	2.79	3.22	3.78	4.24	4.72
	44	1.75	2.13	2.75	3.17	3.73	4.18	4.66
0.750	45	1.72	2.10	2.71	3.13	3.68	4.12	4.60
	46	1.70	2.07	2.67	3.08	3.63	4.07	4.54
	47	1.67	2.04	2.63	3.04	3.58	4.02	4.48
	48	1.65	2.01	2.60	3.00	3.54	3.97	4.42
	49	1.62	1.98	2.56	2.96	3.49	3.92	4.37
	50	1.60	1.96	2.53	2.93	3.45	3.87	4.32
	51	1.58	1.93	2.50	2.89	3.41	3.82	4.27
	52	1.56	1.91	2.47	2.85	3.36	3.78	4.22
	53	1.53	1.88	2.44	2.82	3.32	3.73	4.17
	54	1.51	1.86	2.41	2.79	3.29	3.69	4.12
	55	1.49	1.83	2.38	2.75	3.25	3.65	4.07
	56	1.48	1.81	2.35	2.72	3.21	3.61	4.03
	57	1.46	1.79	2.32	2.69	3.17	3.57	3.99
	58	1.44	1.77	2.30	2.66	3.14	3.53	3.94
	59	1.42	1.75	2.27	2.63	3.11	3.49	3.90
1	60	1.40	1.73	2.24	2.60	3.07	3.45	3.86
2	120	0.83	1.04	1.38	1.61	1.92	2.18	2.45
3	180	0.61	0.76	1.02	1.20	1.44	1.63	1.85
6	360	0.35	0.44	0.60	0.71	0.86	0.98	1.12
12	720	0.20	0.26	0.35	0.41	0.51	0.58	0.67
24	1440	0.11	0.15	0.20	0.24	0.30	0.35	0.40

Reference: NCTCOG iSWM Manual

5.0 HYDROLOGIC 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. In either case, the determination of the design discharge is to be based on fully developed watershed conditions.

When determining design flow rates and water surface elevations for rivers and creeks in the City, the ultimate development design discharges must be obtained from the most recent Storm Water Master Plan.

5.1 Downstream Hydrologic Assessment

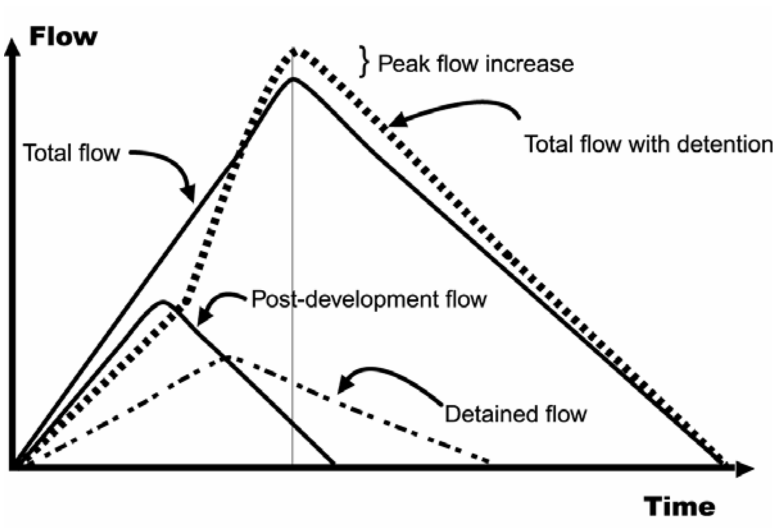
The purpose of the channel protection and flood protection criteria is to protect downstream properties from flood increases due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually make flooding problems worse downstream. The reasons for this have to do with the timing of the flow peaks and the total increase in volume of runoff. Further, due to a site's location within a watershed, there may be very little reason for requiring flood control from a particular site. This section outlines the procedure for determining the impacts of post-development storm water peak flows and volumes on downstream flows.

5.1.1 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 5-1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained. This is most likely to happen if detention is placed on tributaries towards the bottom of the watershed, holding back peak flows and adding them as the peak from the upper reaches of the watershed arrives.

Figure 5-1. Detention Timing Example

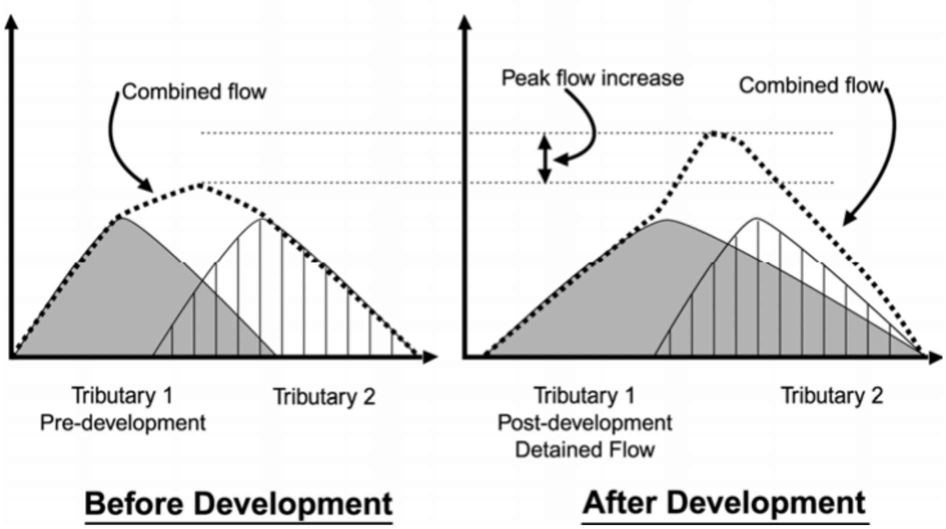


Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 5-2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

Figure 5-2. Hydrograph Effect of Increased Post-Development Runoff Volume with Detention



5.1.2 The Ten-Percent Rule

In this Manual, the "ten percent" criterion has been adopted as the most flexible and effective approach for ensuring that storm water quantity detention ponds actually attempt to maintain pre-development peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises ten percent (10%) of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in the application of the ten-percent rule are:

1. Determine the target peak flow for the site for predevelopment conditions.
2. Use a topographic map to determine the lower limit of the zone of influence (10% point).
3. Use a hydrologic model to determine the pre-development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.

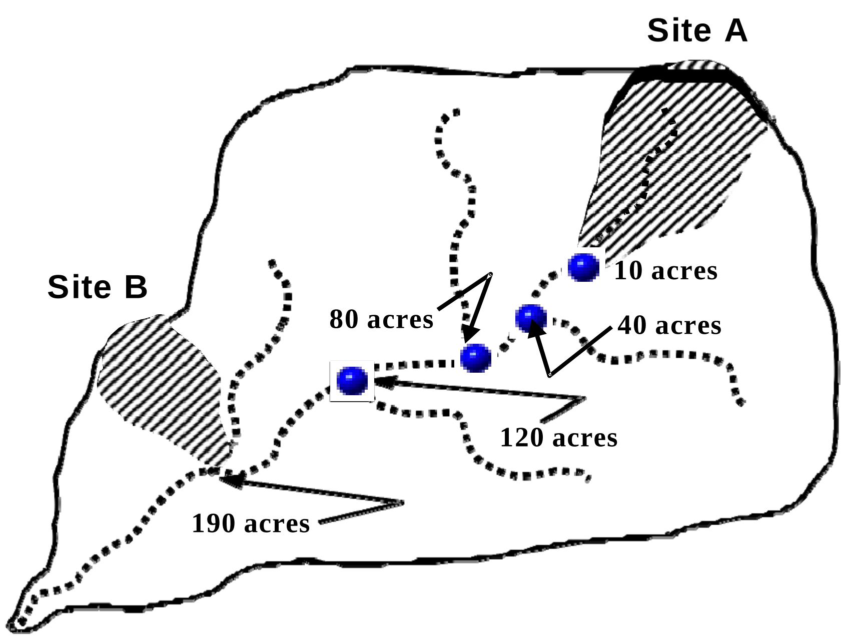
4. Change the land use on the site to post-development and rerun the model.
5. Design the structural control facility such that the flood protection post-development flow does not increase the peak flows at the outlet and the determined tributary junctions.
6. If it does increase the peak flow, the structural control facility must be redesigned or one of the following options considered:
   - Control of the flood protection volume (Q_f) may be waived by the local authority saving the developer the cost of sizing a detention basin for flood control. In this case the ten-percent rule saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
   - Work with the local government to reduce the flow elevation through channel or flow conveyance structure improvements downstream.
   - Obtain a flow easement from downstream property owners to the 10% point.

Even if the flood protection requirement is eliminated, the water quality treatment (WQ_v) and channel protection (CP_v) criteria will still need to be addressed.

5.1.3 Example Problem

Figure 5-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

Figure 5-3. Example of the Ten-Percent Rule



5.1.4 Discussion

Site A is a development of 10 acres, all draining to a wet extended detention storm water pond. The flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase at the 80-acre point then there will be no increase through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not vital for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows that a detention facility, in this case, will actually increase the peak flow in the stream.

5.2 Rational Method

The Rational Method, based on the direct relationship between rainfall and runoff, is applicable to small watersheds and must be used to determine runoff for watersheds with drainage areas of 160 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 calculated using Equation 5-1.

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

where:

Q = Peak design discharge in cubic feet per second
Measured for a given frequency on the watershed at the desired design point.

K = Antecedent precipitation coefficient

C = Runoff coefficient

I = Average rainfall intensity in inches per hour
Measured during the time of concentration, associated with the desired design frequency.

A = Drainage area in acres

5.2.1 Antecedent Precipitation Coefficient

The runoff computations should include the antecedent precipitation coefficient (K), as identified in Table 5-1. This coefficient is intended to reflect the additional runoff that results from saturated ground conditions. In no case should the product of the runoff coefficient and the antecedent precipitation coefficient exceed 1.0.

Table 5-1. Values Of Antecedent Precipitation Coefficient (K)

Frequency	K
25-year	1.1
50-year	1.2
100-year	1.25

(TXDOT 2002)

5.2.2 Runoff Coefficient

The Federal Highway Administration (FHWA) has compiled average runoff coefficients used in the Rational Method for various surface conditions (FHWA 1996). The runoff coefficients shown in Table 5-2 are adapted from the FHWA values.

The coefficients are to be used in design to represent fully developed conditions, based on the most recent City of Lancaster land use map. A minimum runoff coefficient of 0.6 must be used for areas not covered by the proposed land use maps or current City zoning.

The drainage area under investigation may consist of several different drainage surfaces or zoning classifications. If more than one drainage area is contributing runoff, the weighted runoff coefficient (C_w) is calculated using Equation 5-2.

$$C_w = (A1 * C_1 + A_2 * C_2 + ... + A_n * C_n) / (A_1 + A_2 + A_3 + ... + A_n) \quad \text{(Equation 5-2)}$$

where:

C_w = Weighted runoff coefficient

A_n = Drainage area in acres for each drainage area contributing runoff

C_n = Runoff coefficient for each drainage area contributing runoff

5.2.3 Time of Concentration

The time of concentration (T_c) is the longest time of travel for water to flow from the upstream portion of the sub-basin to the downstream point of design. Typical site conditions will dictate that T_c is the minimum time to inlet per Table 5-3. In special cases, T_c in excess of the minimum time to inlet presented in Table 5-3 may be calculated with the following procedure and such calculations and flow paths should be included with the data submitted for review by the City. Following the procedures specified herein, T_c can be computed with NRCS *TR-55*, *Urban Hydrology for Small Watersheds*.

In using the calculated procedure for determining T_c the following issues shall be considered. First, care shall be taken to ensure that the longest time of travel chosen is characteristic of the overall drainage within the sub-basin. Second, the interface between overland flow and shallow concentrated flow shall be carefully evaluated considering shallow concentrated flow paths on lawns, in swales, between structures, etc.

For undeveloped pre-project conditions, T_c shall always be calculated and overland flow shall be assumed to occur for the first 300 ft of flow, unless there is a defined stream depicted on City topographic maps. If the calculated T_c is less than 20 minutes, then the 20 minute minimum time to inlet shall apply. This 20 minute minimum time to inlet shall only be used for undeveloped pre-project conditions.

T_c is composed of four basic components, overland flow, shallow concentrated flow, channelized flow to inlet, and channelized flow downstream of the inlet to the point of design. Either this method or the minimum time to inlet must be used when determining T_c downstream of an inlet. Time of concentration at a design point is calculated as:

$$T_c = T_o + T_s + T_h + T_t \quad \text{(Equation 5-3)}$$

where:

T_c = Time of concentration, minutes (min);

T_0 = Overland flow travel time, min;

T_s = Shallow concentrated flow travel time, min;

T_h = Channelized flow travel time to inlet, min; and

T_t = Channelized flow time of travel downstream of inlet to the design point, min.

Table 5-2. Values Of Runoff Coefficient (C)

Description of Area	C
Lawns	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Unimproved areas (agriculture/forest)	0.15
Business	
Downtown areas	0.95
Neighborhood areas	0.70
Residential	
Single Family (1/8 acre lots)	0.65
Single Family (1/4 acre lots)	0.60
Single Family (1/2 acre lots)	0.55
Single Family (1+ acre lots)	0.45
Multi-Family Units, (Light)	0.65
Multi-Family, (Heavy)	0.85
Commercial/Industrial	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 – 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

(TXDOT 2002)

5.2.4 Time to Inlet

The time to inlet is the time of travel of the water flow to the inlet considering overland flow, shallow concentrated flow, and channelized flow. Minimum times of travel to the inlet are specified in Table 5-3.

Table 5-3. Initial Times Of Concentration

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

These minimum times to inlet may be used for the time of concentration (T_c) at inlets in lieu of calculating T_c for post project conditions. However, the calculated time to inlet shall be used when determining the time of concentration downstream of an inlet.

5.2.5 Time of Travel

T_c for design points downstream of inlets shall be calculated using the time to inlet (i.e., the calculated T_c, minimum times to inlet shall not be used) plus the time of travel (T_t) of the flow through the channelized flow segments downstream of the inlet. For small drainage systems with short times of travel, the channelized flow segments downstream of the inlet for post project conditions may be neglected for design purposes. Time of travel (T_t) downstream of inlets shall be computed using the hydraulic procedures as previously specified for channelized flow (T_h).

5.2.6 Overland Flow

The time of travel for the overland flow component (T_0) is computed using Manning's kinematic solution:

$$T_o = 0.007 * [(n * L)^{0.8} / (R_2^{0.5} * S^{0.4})] \qquad \text{(Equation 5-4)}$$

where:

T_0 = Overland flow time of travel, min;
n = Manning's coefficient for sheet flow;
L = Flow length, feet (ft);
R_2 = 4.0 inches which is the 2-year, 24-hour rainfall; and
S = Slope of the hydraulic grade (assume it is equal the ground slope), ft/ft.

Manning's coefficient (*n*) for overland flow is based on soil cover. Values for *n* are presented in Table 5-4. Overland flow length (*L*) is based on City topographic maps (or more detailed site survey data) for pre-project conditions and proposed grading plan for post project conditions. *L* shall not exceed the lengths presented in Table 5-5. Larger *L* values for undeveloped and agricultural land use can be used for undeveloped pre-project conditions. The 300 ft maximum is set because after that distance, the flow is usually considered shallow concentrated flow.

5.2.7 Shallow Concentrated Flow

Overland flow becomes shallow concentrated flow in reels, shallow gullies, or swales, such as those between houses or businesses. Such flow in undeveloped areas extends from the overland flow to a stream as defined on a City topographic map or the most detailed topographic maps available (if it is outside the City). In developed areas, shallow concentrated flow extends from the overland flow to the curb. Flow in a gutter shall be treated as channelized flow. Areas with shallow concentrated flow with varying slopes or soil surfaces can be broken down into segments to better estimate the travel time. The total time of travel of the shallow concentrated flow is the sum of the times of travel for each segment.

Table 5-4. Manning's *n* for Overland Flow

Soil Cover	n Value
Undeveloped - Cultivated soil, dense grass, range, or woods	0.24 - 0.410
Developed - Lawns, dense grass, or woods	0.240
Concrete, asphalt, gravel, or bare soil	0.011

Table 5-5. Maximum Overland Flow Lengths

Land Use	Maximum L (ft)
Undeveloped, agricultural*	300
Parks, permanent open space, playgrounds	60
Single family residential (less than 3 lots per acre)	50
Single family residential, schools	40
Multi-family residential, commercial, industrial, manufacturing	20
Central business district (CBD), strip centers	10

* This length is a minimum, unless there is a defined stream on City topographic maps. An undeveloped site can assume a minimum time of concentration at 20 minutes with a run-off coefficient of 0.30.

Shallow concentrated flow is characterized by the soil cover as either paved or unpaved. The flow velocity is calculated using the following formula:

$$V_S = K * S^{0.5} \qquad \textit{(Equation 5-5)}$$

where:

V_S = Average velocity of flow, fps;
K = 16.1 for unpaved and 20.3 for paved soil cover; and
S = Slope of the watercourse, ft/ft.

The time of travel for shallow concentrated flow is calculated as:

$$T_S = L / (60 * V_S) \qquad \textit{(Equation 5-6)}$$

where:

T_S = Shallow concentrated flow travel time, min;
L = Flow length, ft; and
V_S = Average velocity of flow, fps.

5.2.8 Channelized Flow

Channelized flow is drainage in gutters, storm drains, channels, and streams. Generally, in the analysis of channelized flow it is necessary to breakdown the flow into a series of reaches, each reach having its own characteristics, to better estimate the travel time. The total time of travel of the channelized flow is the sum of the times of travel for each segment. Flow velocities are calculated using the Manning equation with Q_p for the 2-year flood.

For natural and constructed channels and street gutters, the velocity (V_h) may be calculated by assuming uniform bank full flow. For closed conduit systems on flat grades not being hydraulically analyzed for the project, it may be reasonable to calculate V_h assuming uniform half-full flow. After computing the velocity, the time of travel for channelized flow is calculated with the following equation:

$$T_h = L / (60 * Vh) \qquad \text{(Equation 5-6)}$$

where:

T_h = Channelized flow travel time, min;
L = Flow length, ft; and
V_h = Average velocity of flow, fps.

Flow through ponds or lakes and where the calculated velocity for channelized flow for post project conditions is less than 3 fps, then the flow should be assumed to travel at wave celerity:

$$T_h = c = (g * d_m)^{0.5} \qquad \text{(Equation 5-7)}$$

where:

c = Wave celerity, fps;
g = 32.2 = Acceleration of gravity, feet per second per second (ft/sec^2); and
d_m = Average depth of flow, ft.

5.3 Unit Hydrograph Methods

Storm water discharges produced by watersheds larger than 160 acres must be computed using a unit hydrograph method. The unit hydrograph technique is used to transform rainfall excess to sub-basin runoff. Two techniques are used, depending on the size of the sub-basin, to develop synthetic unit hydrographs. These methods are the dimensionless and Snyder unit hydrographs. The dimensionless unit hydrograph can be used for all sub-basins where the total basin (i.e., total area of all sub-basins at the design point) is draining less than 2,000 acres (3.13 sq mi). Snyder's unit hydrograph can be used for all sub-basins where the total basin is draining greater than 160 acres (0.25 sq. mi.).

The hydrograph methodologies presented herein are employed in the HEC-1 or HEC-HMS computer program. HEC-1 or HEC-HMS (latest version) should be used for hydrologic modeling. All FEMA-accepted models will be allowed. All other models shall be converted. Check the FEMA website for a current listing of FEMA-accepted models.

5.3.1 Snyder's Unit Hydrograph

Snyder's unit hydrograph method may be used for drainage areas 160 acres or larger. This method, detailed in the USACE Engineering Manual *Flood-Hydrograph Analysis and Computations* (USACE 1959) and the Bureau of Reclamation's *Flood Hydrology Manual, A Water Resources Technical Publication* (DOI-BR 1989), utilizes Equations 5-8 through 5-14.

The regional coefficient for variations in slopes within the watershed (C_t) can be estimated if the lag time (t_p), stream length (L), and distance to the basin centroid of gravity (L_{ca}) are known. The peaking coefficient (C_p) 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. A value of 0.72 shall be used for C_p. However, other values of C_p may be used if the hydrograph is calibrated to historical records in the area.

$$t_p = C_t (L * L_{ca})^{0.33} / S^{1/2} \qquad \text{(Equation 5-8)}$$

$$t_r = t_p \div 5.5 \qquad \text{(Equation 5-9)}$$

$$q_p = C_p * 640 \div t_p \qquad \text{(Equation 5-10)}$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \qquad \text{(Equation 5-11)}$$

$$q_{pR} = C_p * 640 \div t_{pR} \qquad \text{(Equation 5-12)}$$

$$q_{pR} = q_p\, t_p \div t_{pR} \qquad \text{(Equation 5-13)}$$

$$Q_p = q_p\, A \qquad \text{(Equation 5-14)}$$

where:

t_p = Lag time from midpoint of unit rainfall duration (t_r) to peak of unit hydrograph in hours
C_t = Regional coefficient for variations in slopes within the watershed
L = River mileage from the given station to the upstream limits of the drainage area
L_{ca} = River mileage from the design point to the centroid of gravity of the drainage area
S = Slope of land over which runoff will flow (ft/ft)
t_r = Rainfall duration in hours
q_p = Peak rate of discharge of unit hydrograph for unit rainfall duration (t_r) in cubic feet per second per square mile
C_p = Peaking coefficient (0.72)
t_{pR} = Lag time from midpoint of unit rainfall duration (t_r) to peak of unit hydrograph in hours
t_R = Unit rainfall duration in hours other than standard unit (t_r) adopted in specific study
q_{pR} = Peak rate of discharge of unit hydrograph for unit rainfall duration (t_R) in cubic feet per second per square mile
Q_p = Peak rate of discharge of unit hydrograph in cubic feet per second
A = Drainage area in square miles

5.3.2 Urbanization Curves

To account for the effects of urbanization, another method was developed by the USACE to adjust for lag time (t_p). 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 Dallas-Fort Worth area are found in Figures 5-4 and 5-5. Urbanization curves are calculated using Equations 5-15 (USACE 1986) and 5-16 (US-SCS 1975).

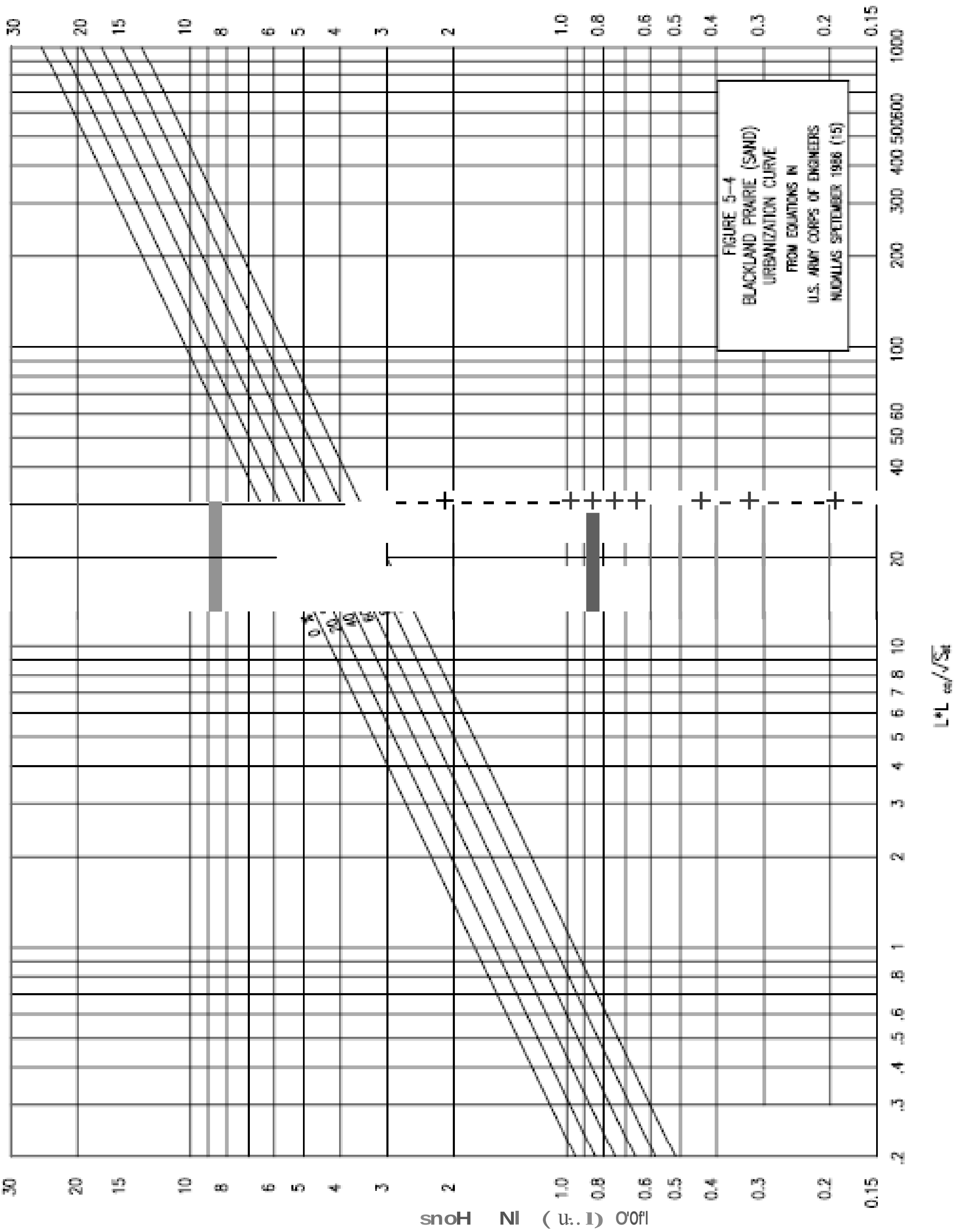
$$t_p = 10\text{^}[0.3833*\log_{10}(L*L_{ca}/(S_{st})\text{^}0.5)+(\log_{10}(I_p))-BW*(\%Urb/100)] \qquad \text{(Equation 5-15)}$$

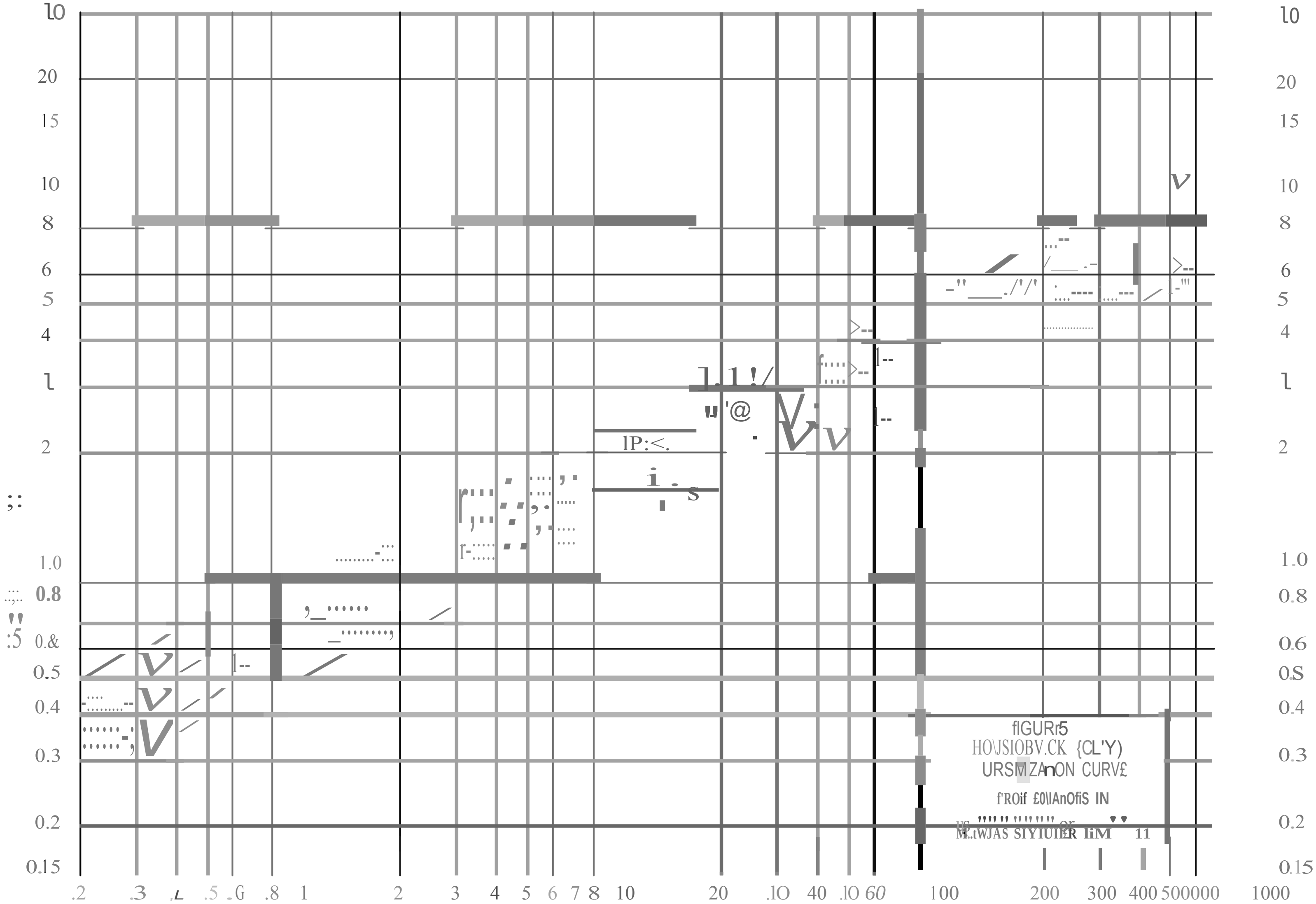
$$S_{st} = (el_{85\%} - el_{15\%})/(0.7*L) \qquad \text{(Equation 5-16)}$$

where:

t_p = Lag time from midpoint of unit rainfall duration (t_r) to peak of unit hydrograph in hours
L = River mileage from the given station to the upstream limits of the drainage area
L_{ca} = River mileage from the design point to the centroid of gravity of the drainage area
S_{st} = Weighted slope of the flow path in feet per mile
I_p = Calibration point, defined as t_p where ($L*L_{ca}/S_{st}$^.5)=1 and urbanization = 0%
BW = Bandwidth equal to the log of the width between each 20% urbanization line
%Urb = Percent urbanization that has occurred in the basin
$el_{85\%}$ = Elevation at a location 85% upstream of the given station
$el_{15\%}$ = Elevation at a location 15% upstream of the given station.

Figure 5-4. Ulbaniza1ion Cwve for Sand




fIGURr5
HO\JSIOBV.CK {CL'Y)
URSMZAnON CURV£
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M..tWJAS SIYIUIER liM 11
30 20 15 10 8 6 5 4 3 2 1.0 0.8 0.6 0.5 0.4 0.3 0.2 0.15
.2 .3 .4 .5 .6 .8 1 2 3 4 5 6 7 8 10 20 30 40 50 60 100 200 300 400 500 1000

For the Dallas-Fort Worth area, the calibration point (Ip) values used are 0.94 for clay and 1.76 for sand. The bandwidth (BW) value for both of the soil types is 0.266 (Chow 1988). For a study area that is composed of both sand and clay, a weighted average of lag time (t_p) can be calculated using Equation 5-17.

$$t_{p\ wght} = \%_{sand} * t_{p\ sand} + \%_{clay} * t_{p\ clay} \qquad \text{(Equation 5-17)}$$

where:

$t_{p\ wght}$ = Weighted average of lag time (t_p)

$\%_{sand}$ = Percent of drainage area composed of sandy soils

$t_{p\ sand}$ = Lag time from midpoint of unit rainfall duration (t_r) to peak of unit hydrograph in hours for portion of drainage area composed of sandy soils

$\%_{clay}$ = Percent of drainage area composed of clayey soils

$t_{p\ clay}$ = Lag time from midpoint of unit rainfall duration (t_r) to peak of unit hydrograph in hours for portion of drainage area composed of clayey soils

5.3.3 Soil Conservation Service Unit Hydrograph Method

The procedures for the SCS unit hydrograph method are outlined in *Urban Hydrology for Small Watersheds* Technical Release No. 55 (US SCS 1975) and in numerous hydrology textbooks. The SCS 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 ultimate watershed conditions. Table 5-6 contains runoff curve numbers to be used for the City for various land uses. The Soil Survey of Dallas County, Texas contains a listing of applicable soil types (US-SCS 1981). For areas that are not included in City zoning, the runoff coefficient (C) for the Rational Method must be a minimum of 0.60, and the curve number must be a minimum of 73, 82, 88 and 90 for soil types A, B, C and D, respectively.

5.4 Hydrologic Computer Programs

The USACE HEC-1 and HEC-HMS programs may be used to assist the designer when using the unit hydrograph methods. The procedures outlined in the users manuals for the HEC-1 and HEC-HMS programs must be consulted when using this method for definitions and values for the various modeling parameters. Hydrologic data obtained from the USACE in the NUDALLAS format must be converted to the HEC-1/HEC-HMS format.

The designer may also utilize other hydrologic models accepted by FEMA with approval of the Public Works Director. The procedures outlined in the models' associated user manual must be followed when using these programs.

Table 5-6. Runoff Curve Numbers

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

6.0 STREET FLOW

6.1 Street Flow Limitations

Streets may be used to convey storm water runoff for the 25-year and 100-year frequency storms in accordance with the water spread limitations listed in Table 6-1.

Table 6-1. Water Spread Limits For Roadways

Street Classification	25-Year Permissible Water Spread
Type A - Major Type B - Major Type C – Secondary Major	One 12-foot traffic lane must remain open in each direction
Type D - Collector	One 12-foot traffic lane must remain open
Type E – Residential Type F - Estate	6-inch depth of flow at curb; one lane must remain open

The permissible water spread limits are based on the 25-year storm frequency, but consideration must also be given to street conveyance of the 100-year flood. All streets must be capable of conveying the 100-year flood within the curb, as shown in Figure 6-1. The criteria for utilizing the street curb to convey the major storm runoff may require increasing the capacity of the enclosed drainage system beyond that required for the 25-year storm. Allowable depths of flow across street intersections for 25-year frequency storms are established as indicated in Table 6-2.

The standard street crown shall not be altered for the purposes of obtaining additional hydraulic capacity. Where additional hydraulic capacity is required, the proposed street gradient must be increased or curb inlets and storm sewers installed to remove a portion of the flow.

Figure 6-1. Street Cross-Section

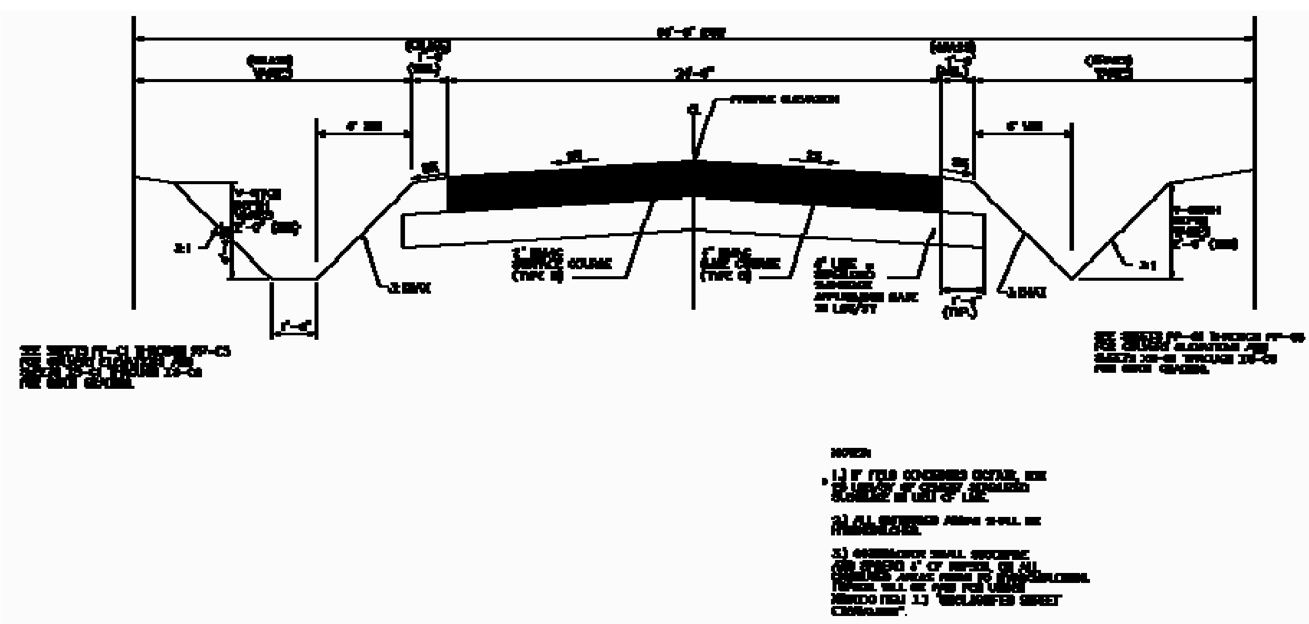


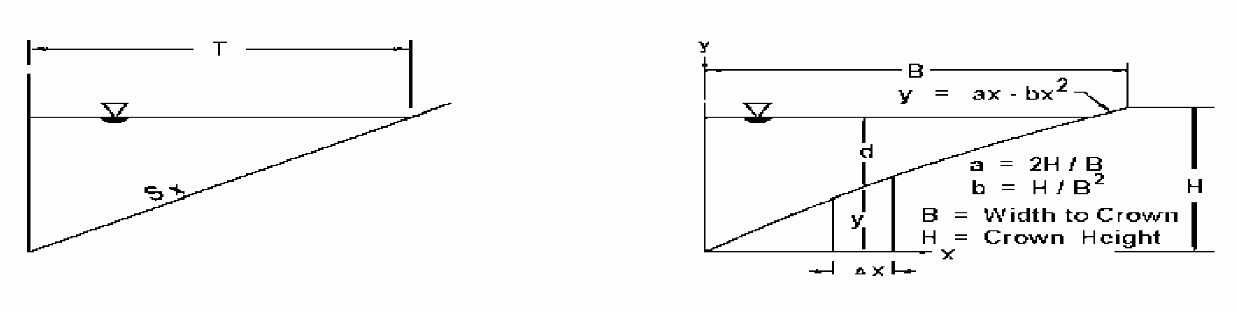
Table 6-2. Allowable Intersection Flows

Street Intersection		25-Year Cross Flow
Type A - Major Type B - Major Type C – Secondary Major		None
Type D - Collector		None
Type E – Residential Type F - Estate	Inlet(s) at Intersection (Grade < 3%)	None
	Inlet(s) at Intersection (Grade ≥ 3%)	No more than 10% of 25-year flow
	No Inlets at Intersection	Flow in valley gutter less than 4" in depth

6.2 Street Flow Calculations

Evaluation of street flow is based upon open channel hydraulics theory, with the Manning 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; straight and parabolic. The City has five standard street widths, 27, 30, 44, 48, and 66 feet. These street flow calculations are dependent on the shape of the street. The street types and typical widths are given in Table 6-3.

Figure 6-2. Typical Gutter Cross Sections



Straight **Parabolic**

Table 6-3. Street Types

Street Type	Width (ft)	Crown (in)
Straight	27	
	30	
	30	
	44	
Parabolic	27	
	30	
	44	
	48	
	66	

6.2.1 Flow Calculations for Straight Street Sections

The direct solution for gutter flow depth for a given flow in straight sections (triangular channel) is calculated using Equations 6-1 through 6-4.

$$Q = (0.56 * S_L^{1/2} * y^{8/3}) / (n * S_x) \quad \text{(Equation 6-1)}$$

$$y = [(Q * n * S_x) / (0.56 * S_L^{1/2})]^{3/8} \quad \text{(Equation 6-2)}$$

$$T = y / S_x \quad \text{(Equation 6-3)}$$

$$V = (2 * Q) / (T^2 * S_x) \quad \text{(Equation 6-4)}$$

where:

Q = Peak gutter discharge in cubic feet per second
S_L = Longitudinal slope of the gutter in feet per foot
y = Flow depth in the gutter in feet
n = Manning's coefficient of roughness, usually 0.016 for streets
S_x = Slope of the crown in feet per foot
V = Velocity of flow in feet per second
T = Spread of flow or ponding width in feet

6.2.2 Flow Calculations for Parabolic Street Sections

Flow from parabolic gutter sections to curb inlets on grade shall be computed with the following formulas:

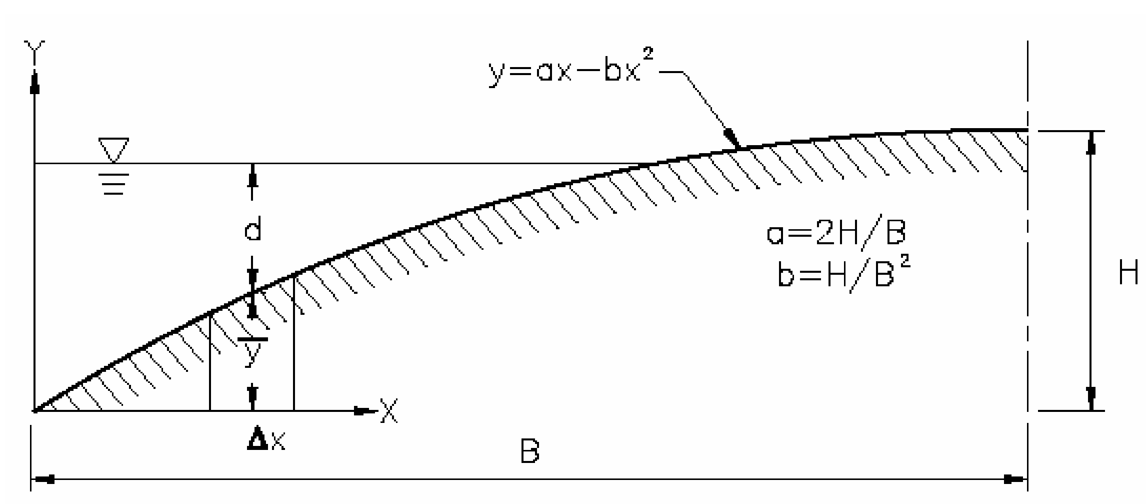
$$y = (a * x) - (b * x^2) \quad \text{(Equation 6-5)}$$

where:

a = $2H/B$
b = H/B^2
H = crown height (ft)
B = half width (ft)

The relationships between a, b, crown height, H, and half width, B, are shown in Figure 6.3.

Figure 6-3. Properties of a Parabolic Curve



To determine total gutter flow, divide the cross section into segments of equal width and compute the discharge for each segment by Manning's equation. The parabola can be approximated very closely by two foot chords. The total discharge will be the sum of the discharges in all segments.

The crown height (H) and half width (B) vary from one design to another. Since discharge is directly related to the configuration of the cross section, discharge-depth (or spread) relationships developed for one configuration are not applicable for roadways of other configurations. For this reason, the relationships must be developed for each roadway configuration. Refer to FHA Hydraulic Engineering Circular 22, Urban Drainage Design Manual, Second Edition for the step by step procedure for developing graphs for parabolic street sections.

6.2.3 Flow Calculations for Composite Street Sections or Local Depressions

Design of composite street sections requires consideration of the flow in the depressed section of the gutter, Q_W

$E_O = 1 / \{1 + [(S_W / S_X) / \{1 + [(S_W / S_X) / (T/W - 1)]^{2.67} - 1]\}$ *(Equation 6-6)*

$Q_W = Q - Q_S$ *(Equation 6-7)*

$Q = Q_S / (1 - E_O)$ *(Equation 6-8)*

where:

Q_W = flow rate in the depressed section of the gutter (ft^3/s)

Q_S = flow capacity of the gutter section above the depressed section (ft^3/s)
Calculated as $(0.56 * S_X^{1.67} S_L^{0.5} T_S^{2.67}) / n$

S_W = Cross slope of the depressed gutter (ft/ft) ($S_X + a/W$)

S_X = cross slope of road (ft/ft)

a = gutter depression (ft)

W = gutter depression width (ft)

T = Spread or width of flow (ft)

E_o = ration of depressed flow to total flow (Q_W/Q)

Determine the equivalent cross slope (S_e) for a depressed street section by using Equation 6-9:

$S_e = S_X + (a / W) * E_O$ *(Equation 6-9)*

where:

S_e = equivalent cross slope (ft/ft)

S_X = cross slope of road (ft/ft)

a = gutter depression (ft)

W = gutter depression width (ft)

E_o = ration of depressed flow to total flow

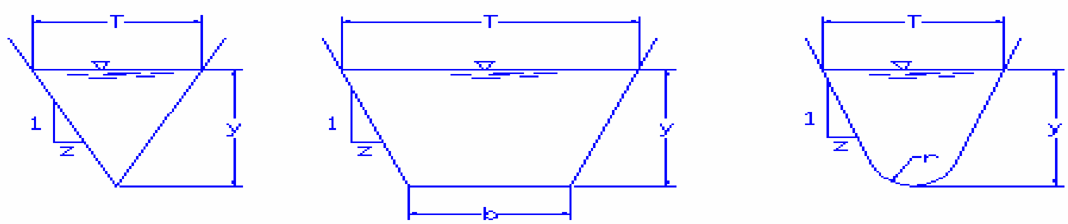
6.2.4 Computation Sheet for Straight and Compostie Street Sections

Data for gutter spread calculations must be submitted to the City either in the standard output format exported from a City approved modeling program or summarized using Computation Sheet 7-1 included in Appendix E. This data must be included in all preliminary and final drainage submittals.

6.3 Alley Flow Limitations

The flows created by the 100-year storm must be contained within the right of way of all paved alleys. Figure 6-3 shows the various alley cross-sections. Alley capacities must be checked at all alley turns and "T" intersections to determine if curbing is needed or if grades should be flattened. Curbing must be required for at least 10 feet on either side of an inlet in an alley and on the other side of the alley so that the top of the inlet is even with the high edge of the alley pavement. Alleys adjacent to a drainage channel are required to have curbs for the full length of the channel.

Figure 6-4. Typical Alley Cross Sections.



a. Triangular b. Trapezoidal c. Round Bottom

6.4 Alley Flow Calculations

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 using Equations 6-10 through 6-14.

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 Equations 6-15 through 6-18.

6.4.1 Triangular Cross Sections

$$y = (Q * n * (1 + z^2)^{1/3} / (0.936 * z^{5/3} * S^{1/2}))^{3/8} \quad \text{(Equation 6-10)}$$

$$T = 2 * z * y \quad \text{(Equation 6-11)}$$

$$A = z * y^2 \quad \text{(Equation 6-12)}$$

$$V = Q / A \quad \text{(Equation 6-13)}$$

$$Q = (1.486 / n)S^{1/2}(zy^2)\{zy/[2-(1+z^2)^{1/2}]\}^{2/3} \quad \text{(Equation 6-14)}$$

where:

y = Flow depth in feet
Q = Peak discharge in cubic feet per second
n = Manning's coefficient of roughness, usually 0.016 for streets and alleys
z = Inverse slope of the crown slope in feet per foot
S = Slope of the alley in feet per foot
T = Spread of flow or ponding width in feet
A = Cross-sectional area of flow in square feet
V = Velocity of flow in feet per second

6.4.2 Trapezoidal Cross Sections

$T = b+2zy$ *(Equation 6-15)*

$A = (b+zy)y$ *(Equation 6-16)*

$V = Q/A$ *(Equation 6-17)*

$Q = (1.486 / n) * S^{1/2} ((b+(z *y) * y)^{5/3})/ (b+(2 *y *(1+z^2)^{1/2})))$ *(Equation 6-18)*

where:

T = Spread of flow or ponding width in feet
b = Basal width of the trapezoid in feet
z = Inverse slope of the crown slope in feet per foot
y = Flow depth in feet
A = Cross-sectional area of flow in square feet
V = Velocity of flow in feet per second
Q = Peak discharge in cubic feet per second
n = Manning's coefficient of roughness, usually 0.016 for streets and alleys
S = Slope of the alley in feet per foot

The normal depth for a round-bottomed cross-section, as shown in Figure 6-3c, can be calculated if the top width is known. Equations 5-19 through 5-22 are used to calculate the top width area, and velocity using the Manning's equation.

6.4.3 Round Bottomed Cross Sections

$Q = [(1.486/ n) * S^{1/2} * ((T^2 /(4z)) - (r^2/z) * (1 - z * (\cot^{-1} z)))^{5/3}] / [((T/z * (1+z^2)^{½}) - (2 r /z) * (1 - z * (\cot^{-1} z)))^{2/3}]$ *(Equation 6-19)*

$y = ((T/2) - r * (1+z^2) + r) /z$ *(Equation 6-20)*

$A = (T /(4 z)) - (r^2/z) * (1 - z * (\cot^{-1} z))$ *(Equation 6-21)*

$V = Q / A$ *(Equation 6-22)*

where:

Q = Peak discharge in cubic feet per second
n = Manning's coefficient of roughness, usually 0.016 for streets and alleys
S = Slope of the alley in feet per foot
T = Spread of flow or ponding width in feet
z = Inverse slope of the crown slope in feet per foot
r = Radius of the curvature in the bottom of the alley in feet
y = Flow depth in feet
A = Cross-sectional area of flow in square feet
V = Velocity of flow in feet per second

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 that must be given attention during inlet design:

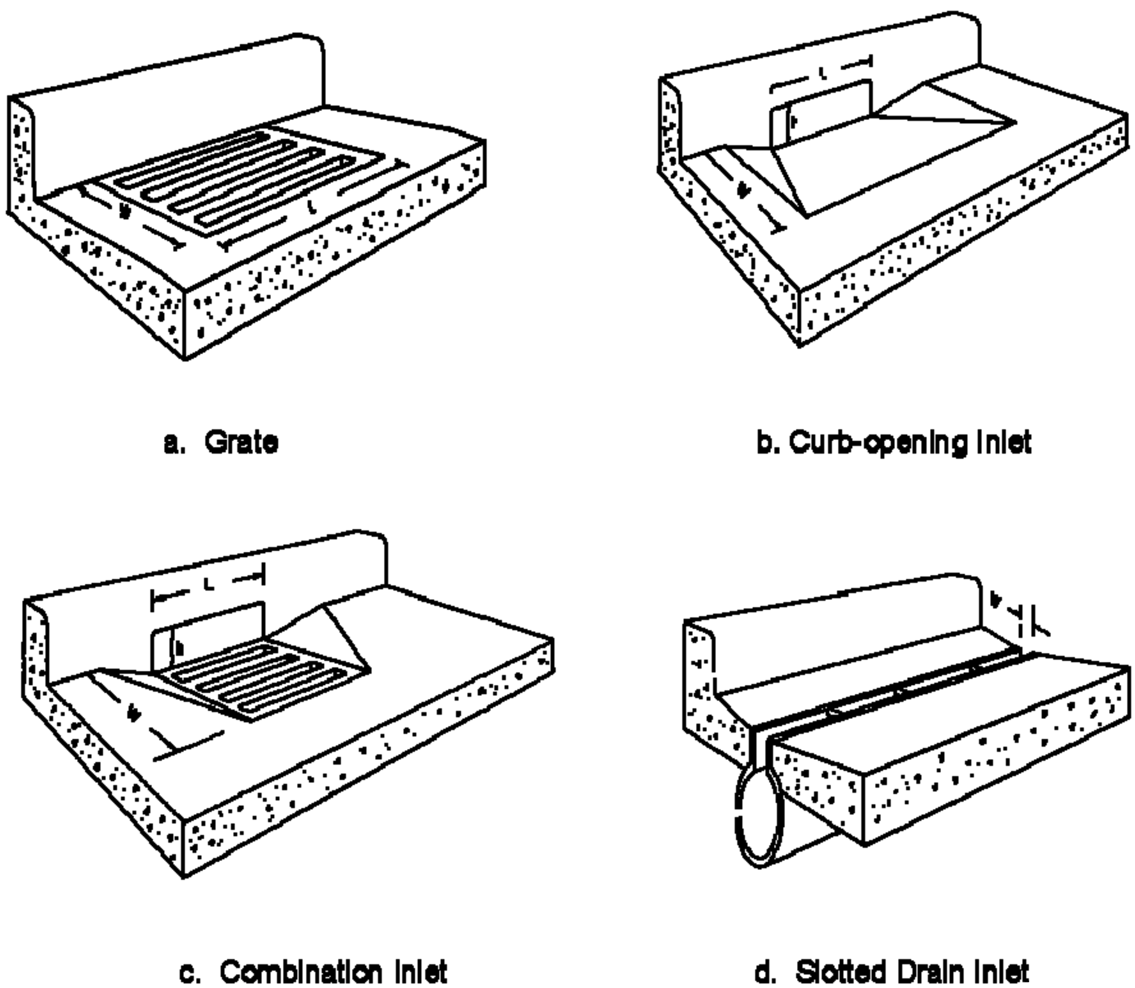
- Inlets must be located where the allowable street flow capacities are exceeded (specified in Section 6.1, Table 6-2), at low points (sumps or sags) and upstream of transitions between normal and super-elevated street sections.
- For storms of a 25-year frequency or less, water flowing in major streets must be intercepted by an inlet prior to super-elevated sections, to prevent water from flowing across the roadway.
- In super-elevated sections of divided major streets, inlets placed against the center medians must have no gutter depression. Interior gutter flow (flow along the median) must be intercepted at the point of super-elevation transition, to prevent pavement cross flow.
- At bridges with curbed approaches, gutter flow must be intercepted prior to flowing onto the bridge to prevent ice from developing during cold weather.
- The maximum approved vertical inlet opening is eight inches. Openings larger than six inches require approval of the Public Works Director and, if approved, must contain a bar or other form of restraint to prevent entry by a child.
- 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.
- The minimum length of curb inlet opening on grade is five (5) feet, the minimum length of curb inlet opening in a sag location in ten (10) feet.
- Not more than twenty (20) feet of curb inlet opening may be placed in one location.
- 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 with excess capacity as specified in this manual to compensate for clogging and must be approved by the Public Works Director.
- 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.
- Recessed inlets must not decrease the width of the sidewalk or interfere with utilities.
- Recessed inlets must also be depressed unless otherwise approved by the Public Works Director. The maximum allowable inlet depression for recessed inlets must be seven inches.
- Non-recessed, depressed inlets must have a maximum allowable inlet depression of five inches.
- 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 Public Works Director. If slotted drains are used, the inlet capacity must be the lesser of the calculated capacity from this manual or the manufacturer's design guidelines.
- Bypass flow should not exceed 0.5 cfs or 30% of the total gutter flow for inlets on grade.

7.2 Inlet Types and Descriptions

Storm water 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, as shown in Figure 7-1 (FHWA 2001):

- Grate inlets
- Curb-opening inlets and type y-inlets
- Combination inlets
- Slotted inlets

Figure 7-1. Inlet Types

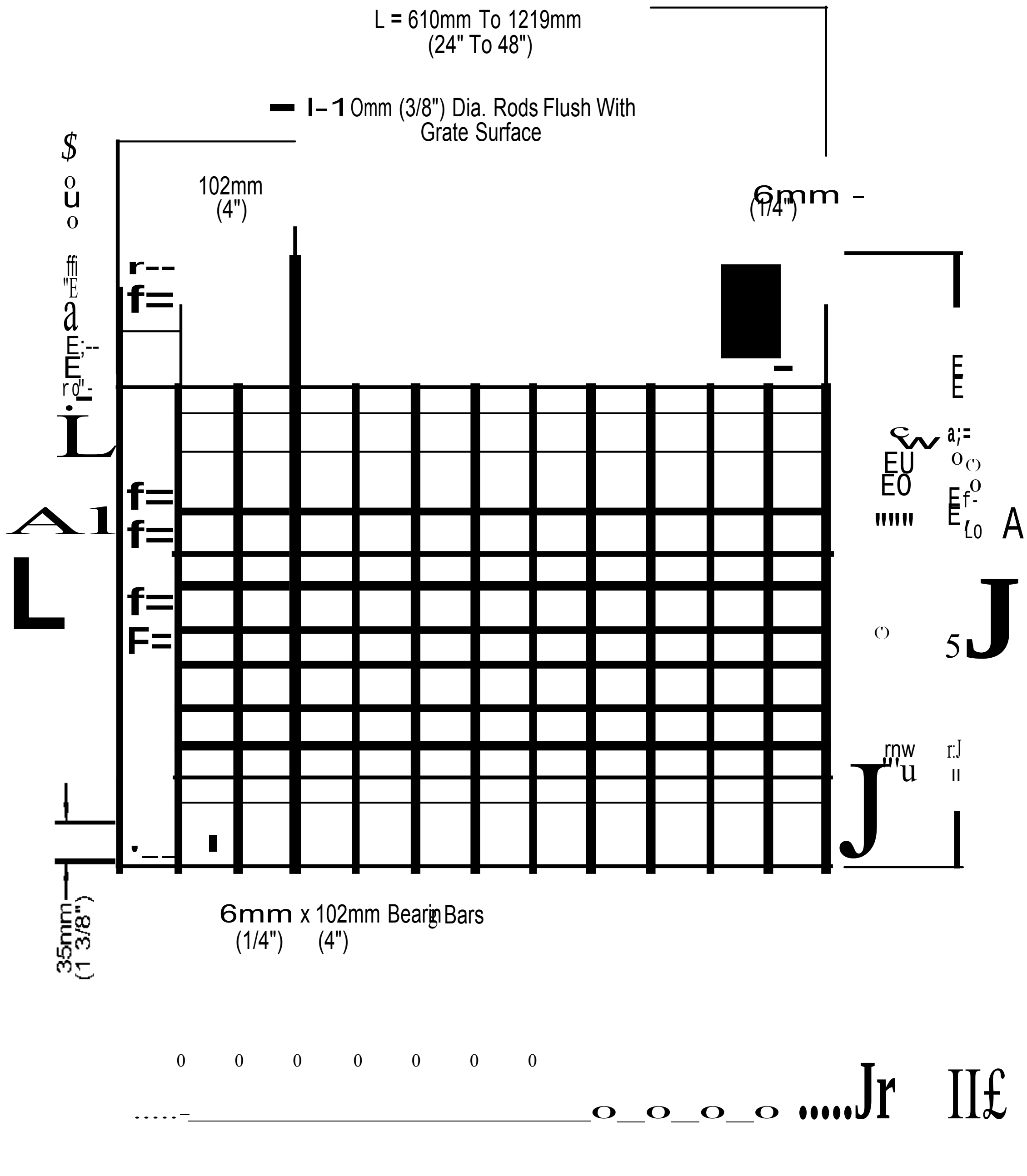


7.2.1 Grate Inlets

Although grate inlets may be designed to operate satisfactorily in a range of conditions, 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 Public Works Director. Figures 7-2 through 7-6 (FHWA 2001) provide examples of grates that are acceptable for use in the City.

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

(P-50 is this grate without lOmm (3/8") transverse rods)



SECTION A-A

Figure 7-3. P-30 Grate Inlet

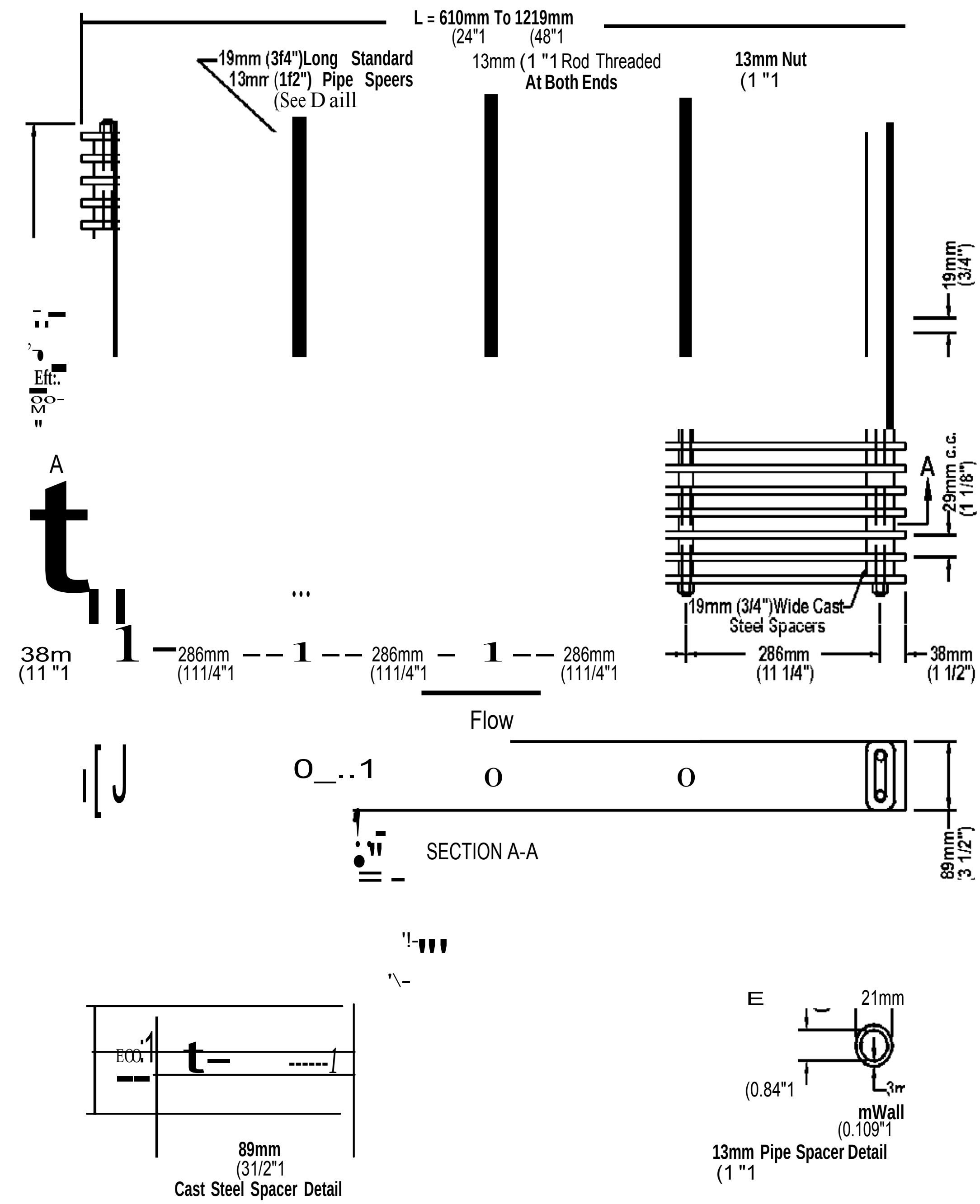
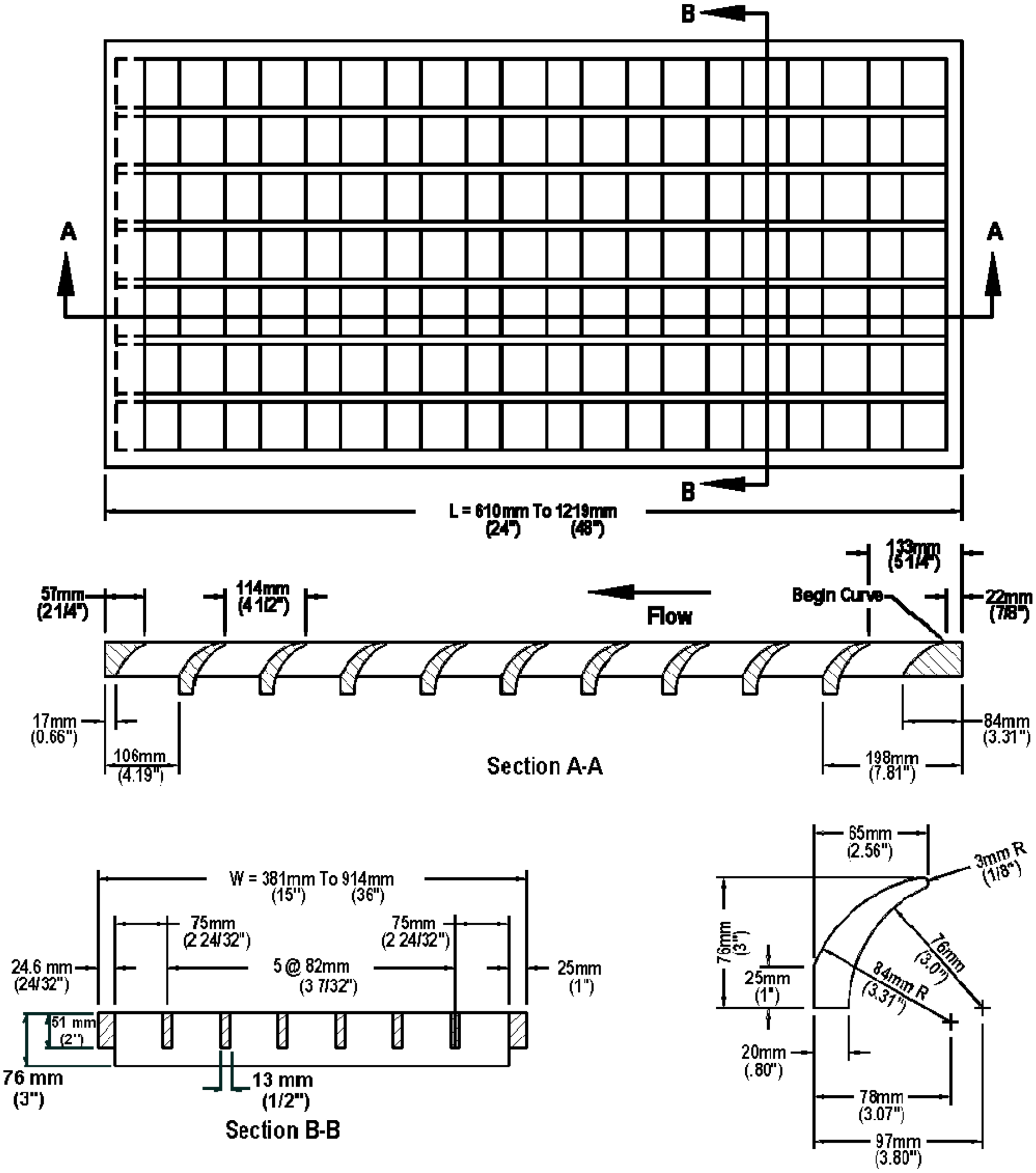
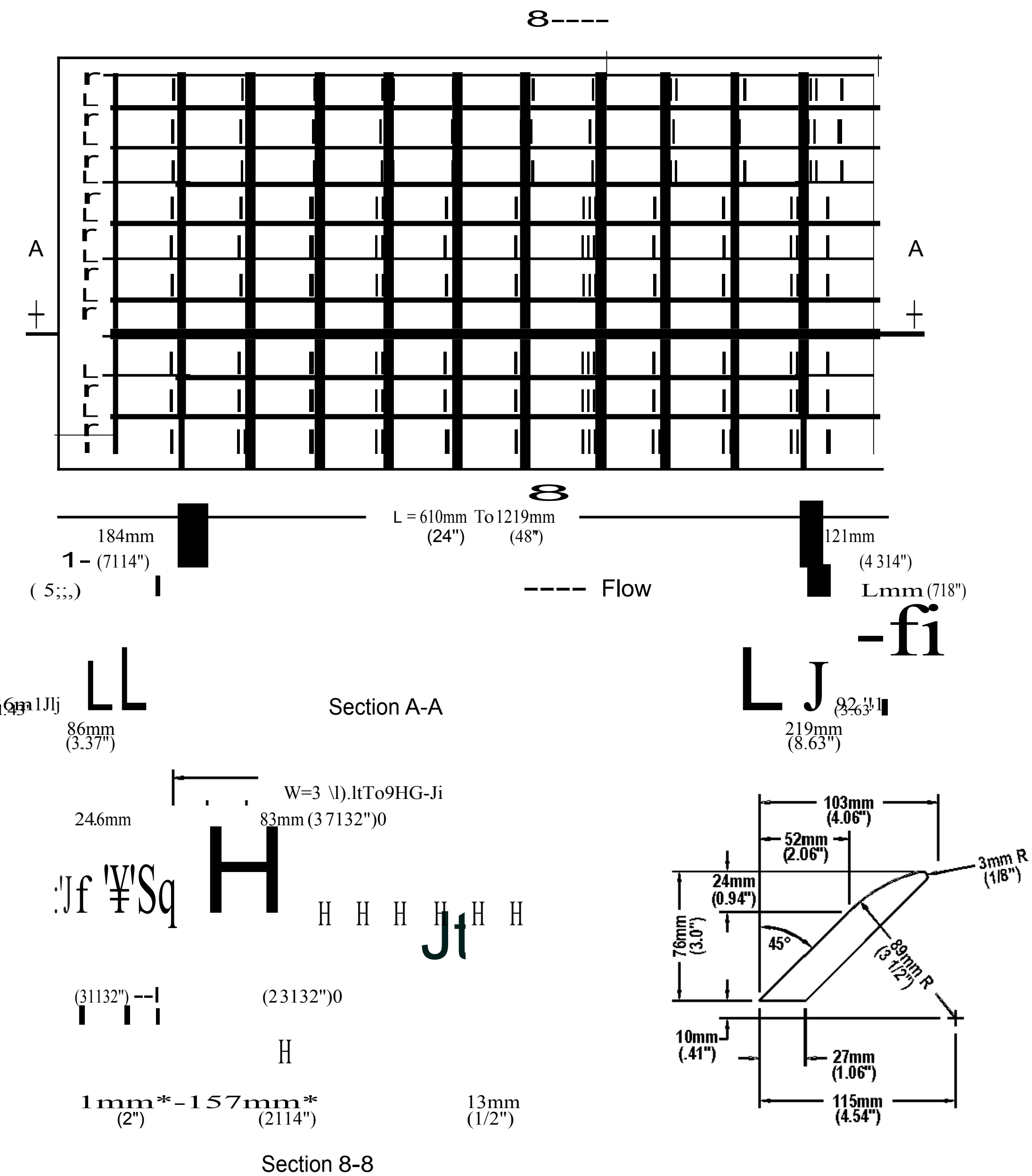


Figure 7-4. Curved Vane Grate Inlet



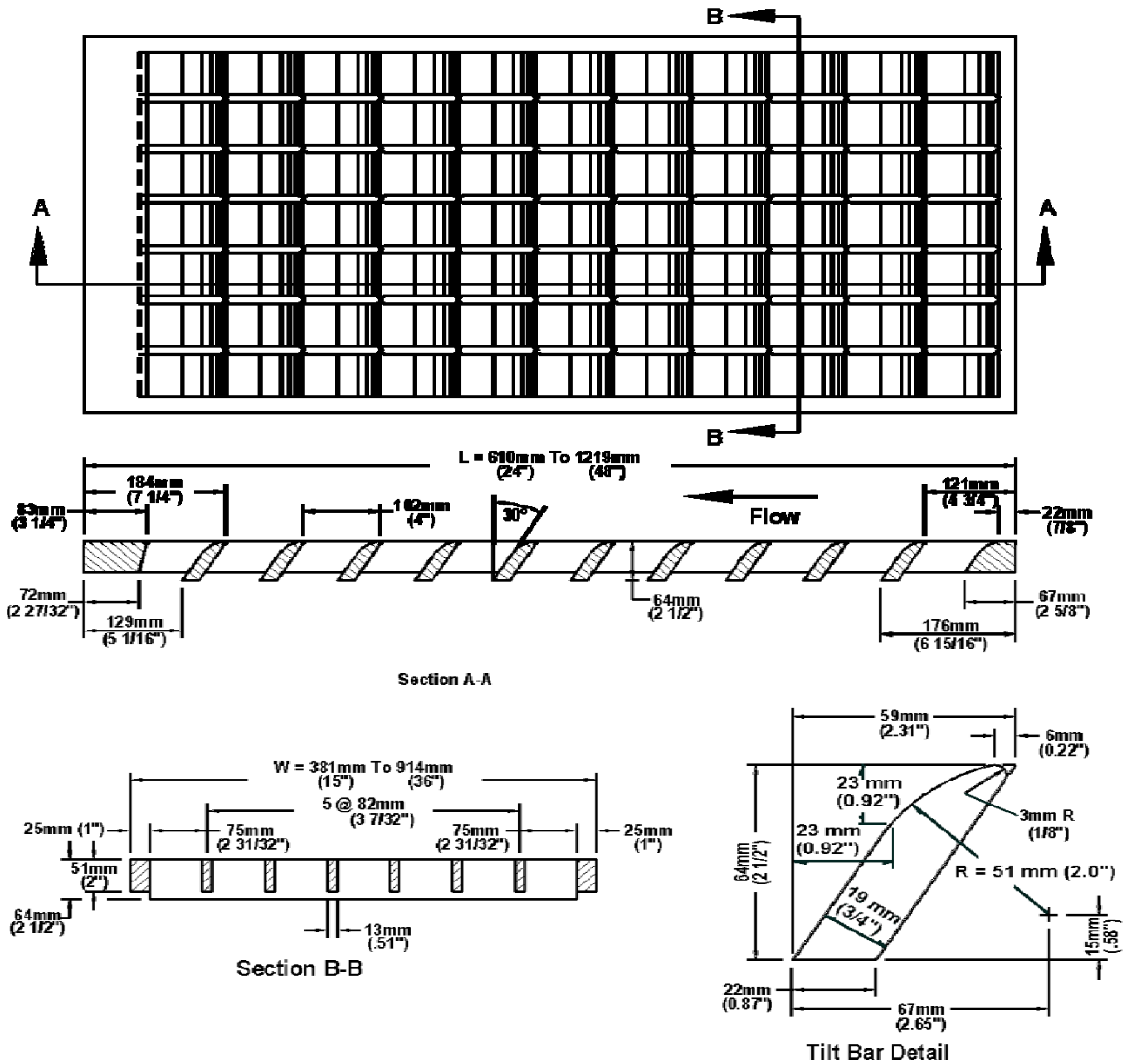
Fi ure 7-5. Transverse 45" Tilted Grate Inlet



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

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

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



7.2.2 Curb Inlets and Type-Y Inlets

Curb inlets are the most effective type of inlet on slopes flatter than 3%, in sump locations, and with flows that typically carry large amounts of debris. Similar to grate inlets, curb inlets 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 sump locations. Type-Y inlets are most often used in drainage of swales at sumps. Both curb and Type-Y inlets are bicycle-safe.

7.2.3 Combination Inlets

A combination inlet consists of both a grate inlet and a 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 is reaches the grate portion of the inlet. Combination inlets may only be used where space behind the curb prohibits the use of other inlet types.

7.2.4 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 since they are susceptible to clogging from debris. Slot inlets may only be used with the permission of the Public Works Director. If slot inlets are allowed, the inlet capacity must be calculated by both equations for a curb inlet found in Section 7.3.2.2 and the manufacturer's design guidelines. The lesser inlet capacity (the more conservative method) is required for design.

7.3 Inlet Capacity Calculations

Storm water inlets can be further classified into three groups: sump inlets, un-depressed inlets on grade, and depressed inlets on grade. Calculations of the capacity for each inlet type and group that pertains to it are discussed in this section. Many of the equations used for the calculation of inlet capacity come from directly or are modified forms of equations found in the FHWA Hydraulic Engineering Circular No. 22.

7.3.1 Inlets in a Sump

Sump locations are low points in the grade or terrain of an area. Inlets in a sump shall have sufficient capacity to capture all of the flow from the 100-year flood event. This includes the flow from the area that contributes directly to the inlets in a sump, as well as any bypass 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. In addition, no bypass 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.

7.3.1.1 Grate Inlets in a Sump

Grate inlets are subject to clogging by debris during storm events, and must 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 that 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 7.3.1.3 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 conditions. Capacity calculations for both conditions will be performed and the lesser of the two capacities will be the design capacity of the grate inlet. Because grate inlets in a sump are prone to clog, only 50 percent of the perimeter must be used for the weir calculations and 50 percent of the open area must be used for the orifice calculations.

The only grate types that are acceptable in a sump location are the P-50, P-30, and P-50 x 100 grates, as shown in Figures 7-2 and 7-3. Capacity of a grate inlet in a sump under weir conditions must be calculated using Equation 7-1.

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

where:

Q_i = Interception capacity of grate inlet under weir conditions in cubic feet per second

C_w = Weir coefficient of 3.0

P = Effecive perimeter of the grate inlet in feet (does not include length adjacent to curb face and assumes 50% clogging)

y = Head in feet at the inlet. Measured as the sum of the approach gutter flow depth and the inlet depression and derived in Section 6 or the adjusted head in feet required to accept the 100-year flood, whichever is greater.

Capacity of a grate inlet in a sump location under orifice flow conditions must be calculated by using Equation 7-2.

$Q_i = C_o * A * (2 * g * y)^{0.5}$ *(Equation 7-2)*

where:

Q_i = Interception capacity of grate inlet under orifice conditions in cubic feet per second
C_o = Orifice coefficient of 0.67
A = Effective area of grate inlet opening as calculated from Table 7-1
g = Acceleration due to gravity, 32.2 feet per second squared
y = Head in feet at the inlet. Measured as the the average depth across grate or the adjusted head in feet required to accept the 100-year flood, whichever is greater

Table 7-1. Grate Effective Area

Inlet	Area
Sag Grate P-50	L * W * 0.729 * 50%
Sag Grate P-30	(L * 0.948) * (W * 0.655) * 50%
Sag Grate 50x100	(L * 0.910) * (W * 0.729) * 50%

7.3.1.2 <u>Curb Inlets and Type-Y Inlets in a Sump</u>

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 are generally considered to function as rectangular broad-crested weirs. The capacity of an inlet in a sump should be calculated using Equation 7-3.

$Q_i = C_w * (L + 1.8 * W) * y^{3/2}$ *(Equation 7-3)*

where:

Q_i = Interception capacity of curb opening inlet or drop inlet (cfs)
C_w = Weir coefficient of 3.0 for straight street sections and 2.3 for locally depressed curb gutters and Type-Y inlets
L = Length in feet of curb opening or the portion of perimeter of inlet opening through which water enters the drop inlet
W = Width of the depression in feet (0 for non-depressed inlets)
y = Depth in feet at the inlet measured from the normal cross slope ($y = TS_X$) or the adjusted head in feet required to accept the 100-year flood, whichever is greater.

**Interception capacities for inlet lengths greater than twelve (12) feet should be calculated without a local depression regardless of gutter type.

Inlets should be located frequently enough along the street such that the inlet openings do not become submerged. 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 (orifice conditions) with a horizontal throat opening is derived from Equation 7-4.

$$Q_i = C_O * A * [2 * g * (y - (h / 2))]^{0.5} \qquad \text{(Equation 7-4)}$$

where:

Q_i = Interception capacity in cubic feet per second of curb opening inlet or drop inlet under submerged conditions

C_O = Orifice coefficient of 0.67

A = Area of the inlet opening in square feet

g = Acceleration due to gravity, 32.2 feet per second squared

y = Head in feet at the inlet. Measured as the sum of the approach gutter flow depth and the inlet depression and derived in Section 6 or the adjusted head in feet required to accept the 100-year flood, whichever is greater

h = Height of the curb opening in feet

7.3.1.3 Combination Inlets in a Sump

Combination inlets used in a sump must have a minimum of a five-foot "Sweeper" curb inlet on both sides of the grate. For weir flow conditions, the "Sweeper" curb inlet capacity, that portion of inlet that does not have a grate, must be calculated as if it was operating alone. The flow that bypasses the "Sweeper" inlets will then be used for sizing the grate inlet in the sump. The interception capacity of the equal length combination inlet is essentially equal to that of the grate operating alone. The capacity of the combination inlet for orifice conditions shall be computed by summing Equations 7-2 and half of the capacity of Equation 7-4. The only grate inlets that are allowed in a sump are the P-50, P-30, and P-50 x 100 grate inlets

7.3.1.4 Slot Inlets in a Sump

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

7.3.2 Inlets On Grade

Inlets on grade are to be placed to provide sufficient capacity to capture the flow from both the 25-year and 100-year flood events. Inlets on grade generally do not suffer diminished capacity due to floating debris. However, they do suffer 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.

Inlet efficiency is expressed as the ration of intercepted flow to total gutter flow and calculated by Equation 7-5.

$$E = Q_i / Q \qquad \text{(Equation 7-5)}$$

where:

E = Inlet Efficiency

Q_i = Interception capacity of curb opening inlet or drop inlet under submerged conditions (cfs)

Q = Total gutter flow (cfs)

Flow that is not intercepted by an inlet is called by pass flow and calculated by Equation 7-6.

$$Q_b = Q - Q_i \qquad \text{(Equation 7-6)}$$

where:

Q_b = By pass flow (cfs)

Q_i = Interception capacity of curb opening inlet or drop inlet under submerged conditions (cfs)

Q = Total gutter flow (cfs)

7.3.2.1 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 on grade inlets from floating debris than in a sump, it may still occur. Therefore, grate inlets are only to be used in areas where floating debris will not be a problem and with the approval of the Public Works Director.

Each grate inlet type has a splash over velocity (V_o), which is used to determine the amount of the flow that 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_o can be found in Table 7-2. If the velocity of the flow in the gutter is less than V_o, then essentially all of the frontal flow will be intercepted by the grate. If the velocity of flow is greater than V_o, then only a portion of the flow will be intercepted.

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_o) is derived using Equation 7-7. For non-depressed inlets, the calculation can be simplified using Equations 7-8 and 7-9.

Table 7-2. Splash Over Velocity (V_0) For Various Grate Inlets

Grate Type	Splash Over Velocity (V_o)
P-50	$2.218 + 4.031 * L - 0.65 * L^2 + 0.06 * L^3$
P-30	$1.762 + 3.117 * L - 0.45 * L^2 + 0.03 * L^3$
Curved Vane	$1.381 + 2.780 * L - 0.30 * L^2 + 0.02 * L^3$
45^O Tilt Bar	$0.988 + 2.625 * L - 0.36 * L^2 + 0.03 * L^3$
30^O Tilt Bar	$0.505 + 2.344 * L - 0.20 * L^2 + 0.01 * L^3$
P-50x100	$0.735 + 2.437 * L - 0.26 * L^2 + 0.02 * L^3$

$$E_o = 1 / \{1 + (S_W / S_X) / ([1 + (S_W / S_X) / ((T / W) - 1)]^{2.67} - 1)\} \quad \text{(Equation 7-7)}$$

$$E_o = 1 - (1 - W / T)^{2.67} \quad \text{(Equation 7-8)}$$

$$Q_S / Q = 1 - (Q_W / Q) = 1 - E_o \quad \text{(Equation 7-9)}$$

$$S_W = S_X + a / W \quad \text{(Equation 7-10)}$$

where:

E_o = Ratio of frontal flow or gutter flow to total flow
S_W = slope of gutter in feet per foot
S_X = Cross slope of the roadway in feet per foot
T = Spread width of the flow in the roadway in feet
W = Width of the depression in feet (if the grate width is less than the depressed gutter width see Equation 7-11)
Q_S = Side flow that does not flow in the gutter or depressed section and will flow into or along the side of the grate in cubic feet per second
Q_W = Flow in the gutter or depressed section in cubic feet per second
Q = Total flow in cubic feet per second
a = local depression in feet

The frontal flow to total gutter flow ration, E_O, for composite gutter sections assumes a grate width equal to the depressed gutter section width. If a grate having a width less than W is specified, the gutter flow ration, E_O, must be adjusted by Equation 7-11.

$$E'_o = E_o * (A'_W / A_W) \quad \text{(Equation 7-11)}$$

where:

E'_O = Adjusted frontal flow area ratio

A'_W = Gutter flow area in a width equal to the grate width (ft^2)

A_W = Flow area in depressed gutter witdh (ft_2)

E'_O should be used in Equations 7-14 and 7-15 instead of E_O for grate widths less than depressed gutter widths.

The ratio of flow intercepted by the grate to frontal flow (R_f) is given by Equation 7-12. R_f can not exceed 1.0.

$$R_f = 1 - K_c * (V - V_o) \quad \text{(Equation 7-12)}$$

where:

where:

R_f = Ratio of flow intercepted by the grate to frontal flow

K_c = 0.09, Head loss coefficient from frontal flow

V = Velocity of flow in the gutter in feet per second

V_o = Splash over velocity in feet per second determined from the Equations in Table 7-2.

The ratio of side flow intercepted to total flow (R_s) is derived from Equation 7-13.

$$R_S = 1 / [1+ \{(K_S * V^{1.8}) / (S_X * L^{2.3})\}] \quad \text{(Equation 7-13)}$$

where:

R_s = Ratio of side flow intercepted by the grate to total side flow

K_s = 0.15, Head loss coefficient from side flow

V = Velocity of flow in the gutter in feet per second

S_x = Cross slope of the roadway in feet per foot

L = Length in feet of the portion of perimeter of inlet opening parallel to flow direction and not adjcent to curb.

The efficiency (E) and total interception (Q_i) of a grate inlet on grade can be calculated using Equations 7-14 and 7-15, respectively.

$$E = R_f * E_O + R_S * (1 - E_O) \quad \text{(Equation 7-14)}$$

$$Q_i = E * Q = Q * [R_f * E_O + R_S * (1 - E_O)] \quad \text{(Equation 7-15)}$$

where:

E = Efficiency of grate inlet

R_f = Ratio of flow intercepted by the grate to frontal flow

E_o = Ratio of frontal flow or gutter flow to total flow (E'_o if width of grate is less than W)

R_s = Ratio of side flow intercepted by the grate to total side flow

Q_i = Total interception of grate inlet

Q = Total flow in cubic feet per second

7.3.2.2 Curb Inlets and Type-Y Inlets On Grade

Curb inlets on grade are classified into three groups:

- Curb inlets that have no depression and are not recessed
- Depressed curb inlets which are depressed but not recessed
- Recessed curb inlets that are both depressed and recessed

For curb inlets on grade, the depressed-recessed curb inlet is recommended due to its superior interception efficiency. In areas where there is insufficient space to construct a recessed inlet, or a recessed inlet would pose the possibility of a traffic hazard, other inlet types may be used with the permission of the Public Works Director. The other inlet types on grade are listed below, in order from the most desirable to the least, as follows:

1. Depressed, non-recessed inlets
2. Combination inlets, with Sweeper inlet
3. Combination inlets, without Sweeper inlet
4. Grate inlets
5. Non-depressed, non-recessed curb inlets
6. Slot inlets

Due to possible traffic hazards, depressed non-recessed inlets must not be used on Type A, B, or C major thoroughfares as described in Section 6 of this manual.

The calculation of the amount of flow intercepted by a curb inlet on grade varies based on the street section and inlet type. For straight street sections and non-depressed, non-recessed curb inlets the length of inlet required for total interception of gutter flow can be calculated by Equation 7-16.

$$L_T = K_C * Q^{0.42} * S_L^{0.3} * (1 / (n * S_X))^{0.6} \qquad \text{(Equation 7-16)}$$

where:

L_T = Length of inlet required to intercept total gutter flow (ft)
K_c = 0.6
Q = Total gutter flow (cfs)
S_L = Longitudinal slope of the roadway in feet per foot
n = Manning's roughness coefficient, usually = 0.016 for streets
S_x = Cross slope of the roadway in feet per foot

For composite street sections and locally depressed inlets the length of inlet required for total gutter flow interception can be calculated with Equation 7-16 by using an equivalent cross slope, S_e Equation 7-17, in place of S_X.

$$S_e = S_X + S'_W * E_O \qquad \text{(Equation 7-17)}$$

where:

S_e = Equivalent cross slope in cross sections with a depression in feet per foot
S_x = Cross slope of the roadway in feet per foot
S'_w = Cross slope of the gutter measured from the cross slope of pavement in feet per foot (a/W)
E_o = Ratio of frontal or gutter flow to total flow from Equation 7-7 or 7-8
a = Gutter depression in feet

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) to intercept the entire flow and is determined by the Equation 7-18.

$E = 1 - (1 - (L / L_T))^{1.8}$ *(Equation 7-18)*

where:

E = Efficiency of curb inlet

L = Actual length of inlet (ft)

L_T = Length of inlet required to intercept total gutter flow

The total amount of flow (Q_i) intercepted by a curb inlet on grade is calculated using Equation 7-19.

$Q_i = Q * E = Q * [1 - (1 - (L / L_T))^{1.8}]$ *(Equation 7-19)*

where:

Qi = Total flow interception of inlet in cubic feet per second

L_T = Length of inlet required to intercept total gutter flow in feet

Q = Total flow in cubic feet per second

E = Efficiency of curb inlet

L = Actual inlet length in feet

If no bypass is allowed, the inlet length is assigned a dimension of at least L_T. Use a nominal length available for standard curb opening inlets. If bypass is considered, round the curb opening inlet length down to the next available standard curb opening length and compute the bypass flow.

Determine bypass flow by using Equation 7-20:

$Q_b = Q * [1 - (L / L_T)]^{1.8}$ *(Equation 7-20)*

where:

Q_b = bypass flow (cfs)

Q = total flow in gutter (cfs)

L = design length of the curb opening inlet required to intercept the total flow (ft)

L_T = length of inlet required to intercept total gutter flow in feet

Bypass flow rates usually should not exceed about 0.5 cfs or about 30 percent of the total flow in gutter. Greater rates can be troublesome and cause a significant departure from the principles of the Rational Method Application. In all cases, you must accommodate any bypass rate at some other specified point in the storm drain system.

If the curb opening inlet is depressed and recessed, the intercepted flow shall be reduced by 20 percent, and the by pass flow shall be increased by the same amount.

7.3.2.3 Combination Inlets On Grade

Combination inlets may be used, with the permission of the Public Works Director, 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 must have a minimum of a five-foot “Sweeper” curb inlet on the upstream side, unless specified otherwise by the Public Works Director. The “Sweeper” curb inlet capacity, that portion of inlet that does not have a grate, must be calculated as if it was operating alone. The flow that bypasses the “Sweeper” inlets must then be used for the sizing of the combination inlet on grade. Curved vane grates are preferred for on grade applications; however, any of the grate inlets shown in Figures 7-2 through 7-6 can be used in combination inlets on grade with approval of the Public Works Director.

7.3.2.4 Slot Inlets On Grade

Slot inlets may only be used with the permission of the Public Works Director. If slot inlets are allowed, the inlet capacity must be calculated by equations for a curb inlet found in Section 7.3.2.2 and the manufacturer's design guidelines. The more conservative method of the two must be used.

7.4 Gutter Flow - Inlet Interception Computation Sheet

Hydraulic data for curb inlets and drop inlets must be submitted to the City either in the standard output format exported from FEMA-accepted modeling programs or summarized using Computation Sheet 7-1 included in Appendix E. Instructions for Computation Sheet 7-1 are also included in Appendix E.

8.0 STORM DRAIN DESIGN

8.1 Applicable Design Criteria

Storm drain systems may be needed where the water depth, water spread, and/or intersection cross flow limits specified in Section 6.1 of this manual are exceeded. For residential estate subdivisions and other areas where there are significant natural features, including trees, springs, natural channels, and other environmental or aquatic features which would work positively in the aesthetics of a development, the use of natural drainage ways is encouraged. Open channel designs are encouraged in lieu of an enclosed pipe system where feasible, preferably in natural, unlined channel form.

The following guidelines must be considered and met in storm drainage design:

- The minimum lateral storm drain pipe diameter must be 18 inches, except in sump areas, which must be at least 21 inches in diameter. The minimum pipe diameter for a trunk line pipe must be 24 inches.
- Pipe diameters must increase downstream, unless otherwise approved by the Public Works Director.
- At points of change in storm drain size, pipe crowns (soffits) must be set at the same elevation.
- Concrete pipe collars or manufactured transition pieces must be used at all pipe size changes on trunk lines.
- Laterals must be connected to trunk lines using manholes or manufactured wye connections. Special situations may require laterals to be connected to the trunk lines by a cut-in (punch-in), and such cut-ins must be approved by the Public Works Director.
- Profiles for storm drain laterals greater than 25 feet must be provided.
- Vertical curves in the conduit will not be permitted, and horizontal curves must meet manufacturer's requirements for offsetting of the joints.
- To prevent sedimentation in the system, the minimum velocity for the design storm must be 2.5 feet per second.
- The coefficients of roughness listed in Table 8-1 are for use in Manning's Equation.
- The maximum manhole or junction box spacing for storm drain systems is shown in Table 8-2. Junction boxes must also be located at:
    1. pickup points having three or more laterals;
    2. trunk line size changes for pipes with diameter differences greater than 24 inches;
    3. vertical alignment changes; and
    4. future collection points as determined by the Public Works Director.
    5. All 90 degree horizontal bends

Table 8-1. Roughness Coefficient (n) For Storm Drains

Materials of Construction	Material Type	Minimum Roughness Coefficient (n)
Concrete Pipe	All	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 Public Works Director

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

Pipe Diameter (in.)	Max. Spacing (ft.)
24	400
27 - 39	800
42 - 60	1,000
Larger than 60	1,200

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:

- Select pipe size and slope so 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.
- For all pipe junctions other than manholes and junction boxes, manufactured connections should be used, and the angle of intersection must 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 Public Works Director.
- Inlet laterals will normally connect only one inlet to the trunk line. Special circumstances requiring multiple inlets to be connected with a single lateral must be approved by the Public Works Director.
- Storm drain pipes must be reinforced concrete pipe, minimum Class III, or stronger as determined by the design engineer. The use of corrugated metal pipe is discouraged except in instances where there is no alternative. If corrugated metal pipe is used, the manufacturer's design guidelines should be followed. Concrete lining must be used with corrugated metal pipes with diameters of 36 inches or greater.

- Plastic or corrugated metal pipe can be used only if authorized by the Public Works Director.
- Corrugated metal and plastic pipe will not be allowed beneath pavement in public easements and rights-of-way.
- The cover over the crown of circular pipe should be at least three feet 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 Public Works Director.

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 must be consistent with the maximum allowable velocities for the receiving channel (refer to Section 9, Open Channels). 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 must be conveyed in improvements to the flowline of the channels. Erosion protection is required for disturbed banks of natural channels.

Table 8-3. Maximum Velocity In Storm Drains

Storm Drain Type	Maximum Velocity
Inlet Laterals (shorter than 30 feet)	No Limit
Inlet Laterals (longer than 30 feet)	15 fps
Trunk Lines	15 fps

8.3 Calculation of the Hydraulic Grade Line

The 25-year and 100-year frequency hydraulic grade lines (HGL) must be computed and plotted for all storm drain systems. The 25-year frequency hydraulic grade line must be calculated throughout the system and must be at least one foot below the gutter elevation at the entrance to the inlet. 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.

8.3.1 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 Public Works Director. The tailwater elevation must be the greater of the water surface of the receiving stream and the minimum outlet water surface (y_m) both in feet above mean sea level (ft msl).

The water surface of the receiving stream must be the 25-year water surface of the receiving stream for the 25-year conduit design, and the 100-year water surface of the receiving stream for the 100-year conduit design. The minimum water surface (y_m) is derived from Equation 8-1. The critical depth (y_c) is determined using the Froude equation, which is set equal to 1.0, as shown in Equation 8-2.

$$y_m = (D_o + y_c) / 2 + FL \quad \text{(Equation 8-1)}$$

$$1.0 = (Q / A) / (g * D)^{0.5} \quad \text{(Equation 8-2)}$$

where:

y_m = Minimum water surface elevation of the pipe in feet relative to mean sea level

D_o = Pipe outlet diameter in feet

y_c = Critical depth in feet of the channel for a given flow and geometric conditions

FL = Flow line of the pipe, lateral, trunk, or channel in feet relative to mean sea level

Q = Flow in the inlet pipe in cubic feet per second

A = Cross-sectional area of the flow (for circular pipe $A=1/8(\theta\text{-}\sin\theta)d_o^2$, where: $\theta=\pi+2(\arcsin[(y-d_o/2)/d_o/2)]$, and d_o =normal depth)

g = Acceleration due to gravity, 32.2 feet per second squared

D = Diameter of the inlet pipe in feet

For storm drains being connected to an existing downstream storm drain, it is necessary to analyze the hydraulics of the downstream drainage system. It is the responsibility of the engineer to evaluate all data necessary for the analysis. If the existing downstream storm drain system is undersized for the increased flow, detention will be required to predeveloped conditions. No flooding increases will be allowed at any point in the existing system.

The velocity at the outfall of the existing system should be considered in this evaluation as well. Velocity should not be increased above those values presented in Tables 9-1 and 9-2 without erosion protection being provided at the outfall. In the event velocities at the outfall exceed those in Tables 9-1 and 9-2 no increase will be allowed without erosion protection being provided at the outfall.

8.3.2 Friction Losses

Friction losses must be computed using Equations 8-3 through 8-5.

$$H_f = LS_f \quad \text{(Equation 8-3)}$$

$$S_f = V^2n^2/(2.22R^{1.33}) \quad \text{(Equation 8-4)}$$

$$R = A/P \quad \text{(Equation 8-5)}$$

where:

L = Length of Conduit in feet

n = Manning's roughness coefficient from Table 8-1

S_f = Friction slope of the flow in the closed conduit (ft/ft)

A = Cross-sectional area in square feet of the flow

P = Wetted perimeter in feet of the flow (for circular conduit in partial depth $P=1/2\theta d_o$)

V = Velocity of the flow in the conduit in feet per second

R = Hydraulic Radius (ft)

Equations 8-3 through 8-5 are derived from Manning's equation:

$$Q = (1.486 / n) * S^{0.5} * A^{(5/3)} / P^{(2/3)} \quad \text{(Equation 8-6)}$$

where:

Q = Flow in the conduit in cubic feet per second

S = Slope of the conduit in the direction of flow (ft/ft)

8.3.3 Minor Losses

Three types of minor losses must be considered in hydraulic calculations: losses at pipe junctions, losses due to bends, and losses at pipe transitions. The values for K_j specified in Table 8-4 shall be used for all minor loss computations with Equations 8-7 through 8-9.

Losses at junctions, structures, enlargements, and contractions:

$$h_j = V_2^2/2g - K_j * (V_1^2/2g) \qquad \text{(Equation 8-7)}$$

Losses at bends and obstructions:
when $V_1=V_2$

$$h_L = K_j * (V_1^2/2g) \qquad \text{(Equation 8-8)}$$

when $V_1>V_2$

$$h_L = V_2^2/2g - (1 - K_j) * (V_1^2/2g) \qquad \text{(Equation 8-9)}$$

The hydraulic grade for upstream inlets depends on the depth of flow in the downstream conduit. For partial flow depths in the downstream conduit, the hydraulic grade line shall be computed by inlet control values from Chart 1B: 'Headwater depth for concrete Pipe culverts with Inlet Control' taken from Hydraulic Design of Highway Culverts (FHWA, 2001) located in Appendix G. For full flow in the downstream conduit the hydraulic grade line should be the larger value from Chart 1B, or the pressure control Equation 8-10.

$$h_L = 1..5 * V_2^2/2g \qquad \text{(Equation 8-10)}$$

For hydraulic grade line calculations, the minimum hydraulic loss should be 0.00 feet.

8.4 Hydraulic Grade Line Computations

Hydraulic data for the drainage system must be submitted to the City using Computation Sheet 8-1 included in Appendix F. This data must be included in all preliminary and final drainage plan submittals.

In Appendix F Hydraulic Computations for Storm Drains, each row in computation Sheet 8-1 represents a section of conduit. The calculations begin at the outfall and work upstream with each section of conduit. The losses are summed to determine upstream HGL levels. The minimum hydraulic loss should be 0.00 feet.

Hydraulic *jumps* should be avioded or should occur at structures. A 0.00 feet value for hydraulic *jumps* shall be taken for hydraulic grade line calculations. Hydraulic gade line computations are required for partial flow as well as full flow conditions.

Table 8-4. Loss Coefficients

Location	Loss Coefficient (k_j)
JUNCTIONS	
45° to 60° branch [1]	0.75
90° branch [1]	0.50
2- 45° to 60° branches [1]	0.50
True Y	0.60
MANHOLES [2]	
Straight run	0.75
Straight run w/45° branch [3]	0.50
Straight run w/90° branch [3]	0.25
90° bend	0.00
ENLARGEMENTS	
$A_2 / A_1 = 1.4$	0.90
$A_2 / A_1 = 2.6$	0.65
$A_2 / A_1 = 4.0$	0.48
CONTRACTIONS	
$A_2 / A_1 = 0.7$	0.92
$A_2 / A_1 = 0.4$	0.75
$A_2 / A_1 = 0.3$	0.64
BENDS	
Conduit on curve for 90° bend [4]	
Curve radius = 1.0 diameter	0.50
Curve radius = 4.0 diameters	0.40
Curve radius = 14.0 diameters	0.25
Curve radius ≥ 20.0 diameters	0.00
Bends where the curve radius equals the diameter	
90° bend	0.50
60° bend	0.43
45° bend	0.35
22½° bend	0.20
OBSTRUCTIONS	
$A_{Obstruction} / A_{Conduit} = 0.1$	0.25
$A_{Obstruction} / A_{Conduit} = 0.2$	0.66
$A_{Obstruction} / A_{Conduit} = 0.3$	1.28
$A_{Obstruction} / A_{Conduit} = 0.4$	2.94
$A_{Obstruction} / A_{Conduit} = 0.5$	5.55
INLETS	
At upstream end of conduit [5]	1.50

[1] When $Q_{Branch} < 0.05\ Q_{Main}$, then $k_j = 1.00$ may be used for calculation of hydraulic grade on main.
[2] Specified values for k_j for manholes may also be used for analysis of existing inlets.
[3] When $Q_{Branch} < 0.05\ Q_{Main}$, then $k_j = 0.75$ may be used for calculation of hydraulic grade on main.
[4] For bends other than 90°, adjust k_j values as $k_j = c\ k_j'$ (k_j' is from the table) where c = 0.85 for a 60° bend, c = 0.70 for a 45° bend, and c = 0.40 for a 22½° bend.
[5] Specified k_j is for pressure control calculation. Use the higher hydraulic grade based on pressure or inlet control.

9.0 OPEN CHANNELS

9.1 Applicable Design Criteria

Wherever possible, channels should be maintained in a natural state along with the natural floodplain. There are numerous water quality, flood control, environmental, and aesthetic benefits from such an approach.

Where channel improvements are required they must be in accordance with the Storm Water Master Plan (SWMP). Any variation from the SWMP must be approved by the Public Works Director.

Natural or lined open channels shall be designed to convey the flood peak flows while at the same time be designed in such a way to minimize erosion and maintain the stability of the stream banks. Concrete lined channels are generally discouraged by the City and will only be approved when other alternatives are not feasible. Bioengineering techniques may be used in natural channels with side slopes steeper than 4:1. Construction of a low-flow channel, where possible, is another recommended option. Low-flow channels should be sized using the channel-forming discharge or the 2-yr storm. The design engineer is reminded that it may be extremely difficult to obtain the proper permits from the State and Federal authorities for concrete channel designs. In addition, developers are responsible for acquisition of all regulatory agency permits.

The developer must be responsible for the initial (one time) channel modifications, including the replacement of trees at the direction of the City and as required by State and Federal agencies. Initial (one time) clearing of debris, small trees, brush, and vines from floodways and floodplains of channels must 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. Dedication of floodways/floodplains as greenbelts does not necessarily preclude open space requirements as set forth in the applicable subdivision ordinances of the City.

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

Unlined improved channels that contain bends must be designed such that erosion at the bends is minimized. Erosion protection at bends must be determined based on the velocity along the outside of the channel bend (refer to Section 9.3). Unlined improved channels must have side slopes no steeper than 4:1 and lined channels must have side slopes no steeper than 2:1, unless authorized by the Public Works Director. A soil analysis must be performed to determine maximum slope that the soil at the channel improvement site will sustain without failure.

Roadside ditches must be designed to carry the runoff from the 25-year storm event below the roadway elevation. The 100-year storm event runoff must remain within the right-of-way.

9.2 Design Parameters

Design flows in natural and improved channels and through bridges, culverts or other structures associated with a particular channel must be based on the 100-year ultimate development storm event as defined in Section 5.2 of this manual and the SWMP. If the design flow for a given channel, bridge, or culvert is not provided in Section 5.2 of this manual or the SWMP, the procedures described in Section 5 must be used to determine the design flows.

Improved channels, lined or unlined, must normally have a trapezoidal cross section. The channel section should have adequate flow area to take care of 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.

Channels with built-up levees are required to comply with the freeboard requirements in Table 3-1 of Section 3.1. Lesser freeboard will be allowed under special circumstances with the approval of the Public Works Director. Provisions must be made to drain the interior area behind the levee into the channel after the channel flow has subsided. A geotechnical investigation is required for all levee designs.

9.3 Channel Velocities

In order to provide for the safety of the members of the community as well as protect property maximum and minimum channel velocities have been determined. Where possible, channels should have sufficient gradient to provide velocities that will be self-cleaning (greater than two feet per second) but will not be so great as to create erosion.

Appropriate energy dissipating structures may be used to control erosion due to high velocities at pipe system outfalls and steep grades and must be designed in accordance with accepted design practice such as outlined by the NRCS, USACE, the Bureau of Land Reclamation, or the Texas Department of Transportation. The design of energy dissipaters must be based on a geotechnical investigation of the site. Maximum velocities will be checked for both the 25-year and 100-year storm frequencies.

9.3.1 Lined Channels

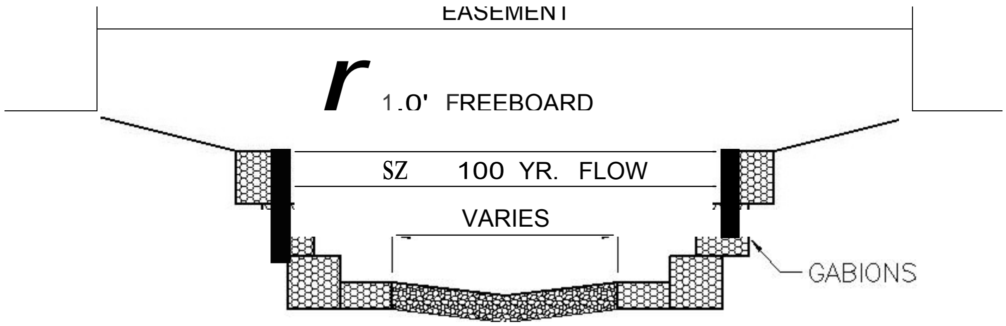
Due to their lining, lined channels can withstand higher velocities than unlined channels without suffering erosion. However, high velocities present a danger to the members of the community. Therefore, the velocities in lined channels must not exceed the maximum permissible velocities shown in Table 9-1.

Table 9-1. Maximum Velocity In Lined Open Channels

Channel Material	Maximum Flow Velocity, fps
Gabion Lined	12.0
Reinforced Concrete Lining	20.0
Rock Riprap (Placed Rock)	Use USACE Guidelines
Prefabricated Lining Products	Coordinate w/ Public Works Director; no more than 85% of Manufacturer's Recommended Velocity Limits

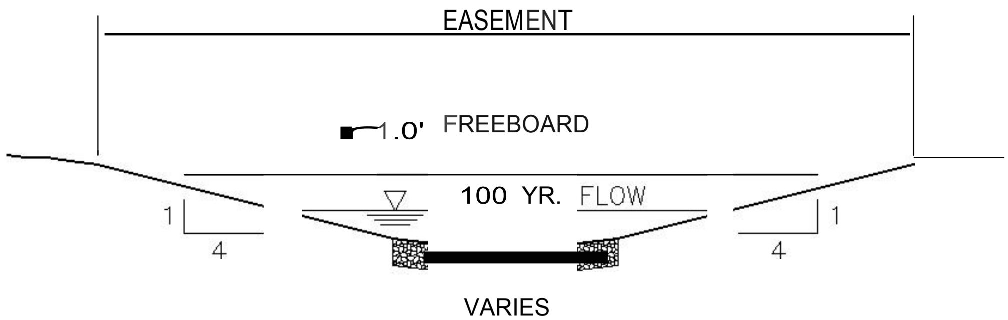
*Sheet flow into channels and streams must not exceed 6.0 feet per second

Figure 9-1. Freeboard Requirements and Channel Section Illustrations

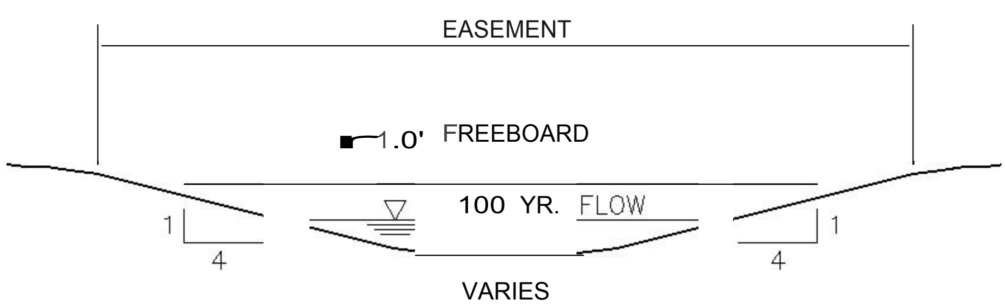


TYPICAL CHANNEL IMPROVEMENT WITH GABION LINING

NOT TO SCALE



TYPICAL IMPROVED CHANNEL WITH
ROCK PILOT CHANNEL LINING NOT
TO SCALE



TYPICAL IMPROVED UNLINED CHANNEL SECTION

NOT TO SCALE

9.3.2 Unlined Channels

Unlined channels must be designed for velocities that will not cause erosion in the channel. The maximum non-erodible velocity varies with the soil type in the channel bed. Table 9-2 shows the maximum permissible values for velocities in straight channels according to soil type. The reduction factors for winding channels are: 5% for slightly sinuous, 13% for moderately sinuous, and 22% for very sinuous.

Table 9-2. Maximum Velocity In Unlined Open Channels

Channel Material	Maximum Flow Velocity, fps
Fine sand collidal	2.5
Sandy loam, non-colloidal	2.5
Silt loam, non-colloidal	3.0
Alluvial silts, non-colloidal	3.5
Ordinary firm loam	3.5
Volcanic ash	3.5
Stiff clay, very colloidal	5.0
Alluvial silts, colloidal	5.0
Shales and hardpans	6.0
Fine gravel	5.0
Graded loam to cobbles when non-colloidal	5.0
Graded silts to cobbles when colloidal	5.5
Course gravel, non-colloidal	6.0
Cobbles and shingles	5.5

No improvements are allowed that will cause the velocity in a channel to exceed the values listed in Tables 9-1, 9-2, or 9-3. Velocities in bioengineered channels may vary from Tables 9-2 and 9-3 provided appropriate engineering data are provided to and approval is obtained from the Public Works Director. For an existing channel with a velocity greater than those listed in these tables, the existing velocity may be maintained, but no improvements must be made that increase the existing velocity.

9.3.3 Grass Lined Channels

Grass lined channels provide a greater retardance of flow and, therefore, result in a greater energy loss than soil-lined channels. However, grass provides stability to the body of the channel, and limits the erosion of the soil. Therefore, the permissible velocity of a grass-lined channel is greater than that of an unlined channel. Table 9-3 shows the values of permissible velocities for different grass lined channels with varying slopes based on USACE recommendations presented (USACE 1982).

Table 9-3. Maximum Velocity In Grass Lined Open Channels

Cover	Slope Range %	Permissible velocity, (fps) Erosion-resistant soils	Permissible velocity, (fps) Easily erodible soils
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass	0-5	7	5
	5-10	6	4
	>10	5	3
Native grass mixture	0-5	5	4
	5-10	4	3

9.4 Flow Conditions

The Froude Number provides a relationship between flow velocity and the hydraulic depth of flow, and gravitational action and must 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 using Equations 9-1 and 9-2.

$$Fr = V / (g * D)^{0.5} \qquad \text{(Equation 9-1)}$$

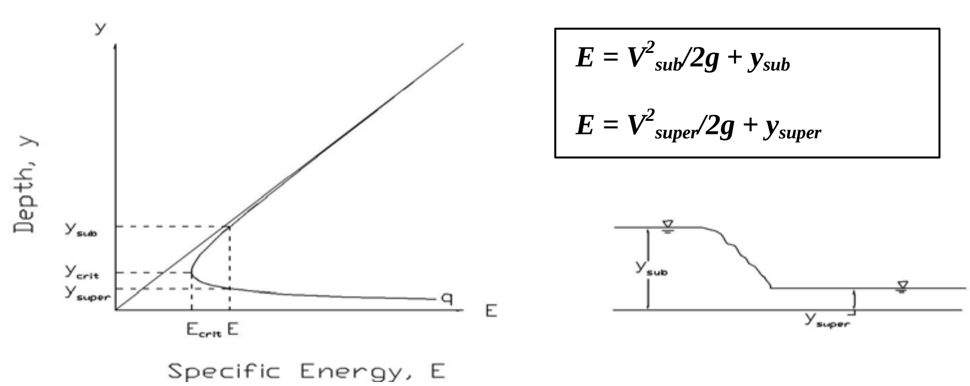
$$D = A / T \qquad \text{(Equation 9-2)}$$

where:

Fr = Froude Number
V = Velocity of flow in feet per second
g = Acceleration due to gravity, 32.2 feet per second squared
D = Hydraulic depth in feet
A = Cross-sectional area of the flow in square feet
T = Top width of the flow in feet

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 is equivalent. Figure 9-2 shows the relationship of specific energy to depth. If the normal depth of a channel 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.

Figure 9-2 Alternate Depths on the Specific Energy Curve



$$E = V^2_{sub}/2g + y_{sub}$$

$$E = V^2_{super}/2g + y_{super}$$

Subcritical flow conditions are recommended for all channel designs in the City, because supercritical flow tends to have a high velocity and a high potential for channel erosion. Supercritical flow conditions will not be allowed in unlined channels. Subcritical flow conditions may be achieved by using energy dissipaters in unlined channels in areas where the existing topography will not allow subcritical flow conditions to occur naturally. In all cases, the channel improvements must be designed to avoid the unstable transitional flow conditions that occur when the Froude Number is between 0.9 and 1.1.

9.5 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, or the water surface is superelevated. Therefore, additional freeboard must be provided to prevent the channel bank from being overtopped. The amount of superelevation[24] along the outside of the bend can be estimated using Equation 9-3 (USACE 1982).

$$_{\Delta}H = C^2 * (r_o^2 - r_i^2) / (2 * g * r_o^2 * r_i^2) \qquad \text{(Equation 9-3)}$$

where:

- $_{\Delta}H$ = Increase in water surface elevation in feet along the outside of the channel bend due to superelevation
- C = Circulation constant in square feet per second
- r_o = Outside radius of the channel bend in feet
- r_i = Inside radius of the channel bend in feet
- g = Acceleration due to gravity, 32.2 feet per second squared

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 using Equation 9-4 (C).

$$Q = C * [(y_a + ((V_a^2) / (2 * g)) - (C^2 / (2 * g * r_o * r_i)] * \ln (r_o / r_i) \qquad \text{(Equation 9-4)}$$

where:

- Q = Total flow in the channel in cubic feet per second
- C = Circulation constant in square feet per second
- y_a = Depth of flow in the approach to the bend in feet
- V_a = Average velocity in the approach to the bend in feet per second
- g = Acceleration due to gravity, 32.2 feet per second squared
- r_o = Outside radius of the channel bend in feet
- r_i = Inside radius of the channel bend in feet

The flow velocity along the outside of the bend (V_o) can then be approximated using Equation 9-5. V_o must not exceed the maximum values established in Section 9.3.

$$V_o = C / r_o \qquad \text{(Equation 9-5)}$$

where:

- V_o = Flow velocity along the outside of the bend in feet per second
- C = Circulation constant in square feet per second
- r_o = Outside radius of the channel bend in feet

9.6 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 must satisfy the permissible velocities allowed for channels (Section 9.3). The velocities must be checked for flows produced by the 25-, 50-, and 100-year frequency events.

An apron must be constructed immediately upstream and downstream of a drop structure to protect against the increasing velocities and turbulence. The upstream apron must extend at least ten feet upstream from the point where flow becomes supercritical and must include a toe constructed of concrete, gabion, riprap or other approved armoring material into the ground. The downstream apron must extend a minimum of twenty feet beyond the anticipated location of the jump and must include a concrete toe into the ground. The toe at each end must extend a minimum of 24 inches into the ground to minimize scour at the transition to natural ground.

The design of drop structures must 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 must be constructed of reinforced concrete, and the bottom and walls (if any) must have a minimum thickness of six inches. To facilitate maintenance, drop structures should be located in readily accessible locations, such as near bridges or culverts, as directed by the Public Works Director. Alternative materials may be used for drop structures with the approval of the Public Works Director.

9.6.1 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 (USACE1982, USACE 1997). In order to utilize a vertical drop structure, vehicular access must be provided to both the upstream and downstream ends of the structure.

9.6.2 Sloping Drop Structures

The location of the hydraulic jump should be determined based on the upstream and downstream flow depths and channel slopes (USACE 1982, USACE 1997). 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 slope no steeper than 6H:1V must be used to allow vehicular access from one end across the structure. Material must be adequate to accommodate vehicular access without degradation of the slope. If the slope of the drop structure is steeper than 6H:1V, vehicular access must be provided to both the upstream and downstream ends of the structure.

9.7 Maintenance Access Requirements

Access roads and/or ramps must be provided for all channels to allow vehicular access for maintenance. The location and design of access roads and ramps must be in accordance with the criteria contained in the Lancaster Storm Water Management Ordinance. Access roads must have a width of at least 12 feet and a cross slope no greater than two percent. Ramps on access roads must have a vertical grade no steeper than ten percent. Access location, intervals, and approaches must be approved by the Public Works Director.

9.8 Standard Step Backwater Computation Sheet

Water surface profiles for the design frequency storms must be computed for all channels and shown on all final drawings. The Standard Step Method for Backwater Calculations must be used to determine water surface profiles for steady uniform flow equal to the design discharge. Standard step backwater calculations must be submitted to the City either in the standard output format exported from a FEMA-accepted modeling program or summarized using Computation Sheet 9-1 included in Appendix H. Instructions for Computation Sheet 9-1 are also included in Appendix H.

Losses due to changes in velocity, drops, bridge openings, and other obstructions must be considered in the backwater computations, as described in the HEC-2 and HEC-RAS User's Manual.

Computation of the water surface profile must 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 9-6.

$$S_f = S_e = (n^2 * V^2) / (2.22 * R^{4/3}) \quad \text{(Equation 9-6)}$$

where:

S_f = Friction slope in ft/ft
n = Manning coefficient
V = mean velocity in ft/sec
R = Hydraulic radius in ft
S_e = Energy slope in ft/ft

10.0 BRIDGE AND CULVERT DESIGN

10.1 Applicable Design Criteria

Bridges must be designed to pass the 100-year frequency, fully developed watershed conditions, peak flow with one foot of clearance below the lowest part of the open span of the bridge, commonly called the low chord. Culverts must be designed to pass the 100-year frequency, fully developed watershed conditions, peak flow with one foot of freeboard below the lowest elevation of the roadway at the culvert.

- All bridge and culvert designs must contain the fully developed watershed condition 100-year frequency storm event within the right-of-way or drainage easement limits.
- Headwalls and necessary erosion protection must be provided at all culverts and must comply with the TxDOT standards.
- Proposed reinforced concrete box culverts, bridges, and related structures may be adaptations of the TxDOT standards.

10.2 Design Parameters

Where a proposed culvert will eventually become part of a planned storm drainage system, as shown in the SWMP, 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 the SWMP, the designer should proceed with the design from the nearest downstream control (i.e. bridge, culvert dam, etc.) and design the proposed drainage system improvements anticipating future system expansion due to fully developed watershed conditions.

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

- Channel transitions into and out of the bridge opening
- Overall length and height of bridge
- Cross-sectional opening of bridge
- Bridge alignment relative to general flow of main channel (i.e., is it a "skewed" crossing)
- Number of crossings ("dual" or multiple bridges)
- Other obstructions to flow (e.g., piers and abutments)
- Design flows for bridge opening to pass
- Any freeboard requirements for channel design
- Erosion protection at piers and abutments
- Scour analysis in the bridge design

All headwalls must be constructed of reinforced concrete. Wingwalls, if used, may be either straight-parallel, flared, or tapered. Approach and discharge aprons must 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. 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).

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)

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 must extend downstream to a point where non-erosive channel velocities are established in accordance with Table 9-1. The outlet protection must 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 must be based upon design guidelines set forth by the Texas Department of Transportation, the U.S. Department of Transportation, or other suitable material approved by the Public Works Director. Figures 10-1 through 10-6 are provided as design guides. Table 10-2 contains the culvert entrance loss coefficients (K_e) for use with Figures 10-2, 10-4, and 10-6.

10.5 Culvert Computation Sheet

Culvert design computations must be submitted to the City either in the standard output format exported from FEMA-accepted modeling programs or summarized using Computation Sheet 10-1 included in Appendix I. Instructions for Computation Sheet 10-1 are also included in Appendix I.

If bridge design parameters are used in the USACE HEC-2 Water Surface Profile program to design a bridge or culvert, the design parameters must be submitted to the City in the standard output format from the HEC-2 modeling software.

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

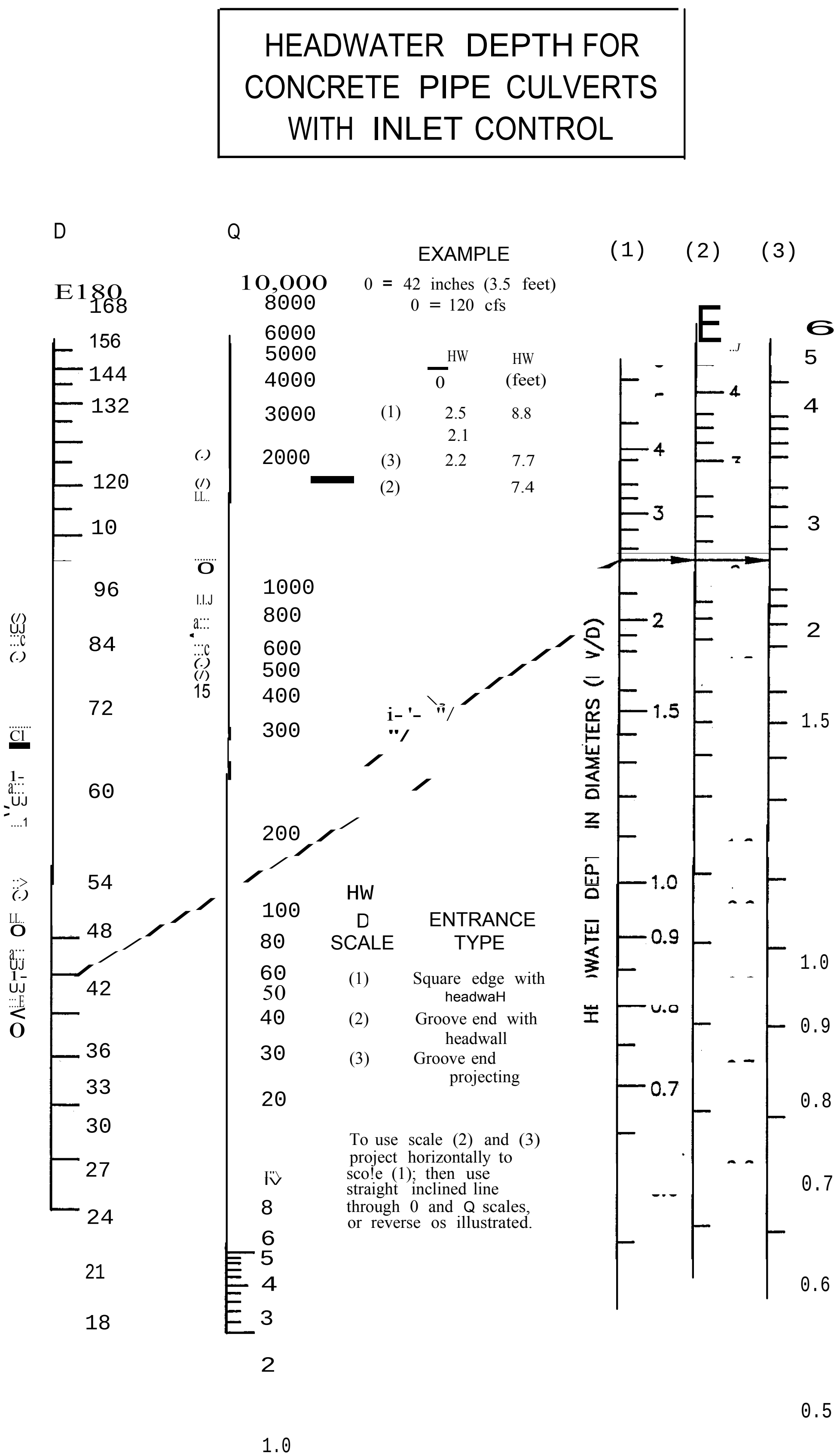


FIGURE 10-1

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

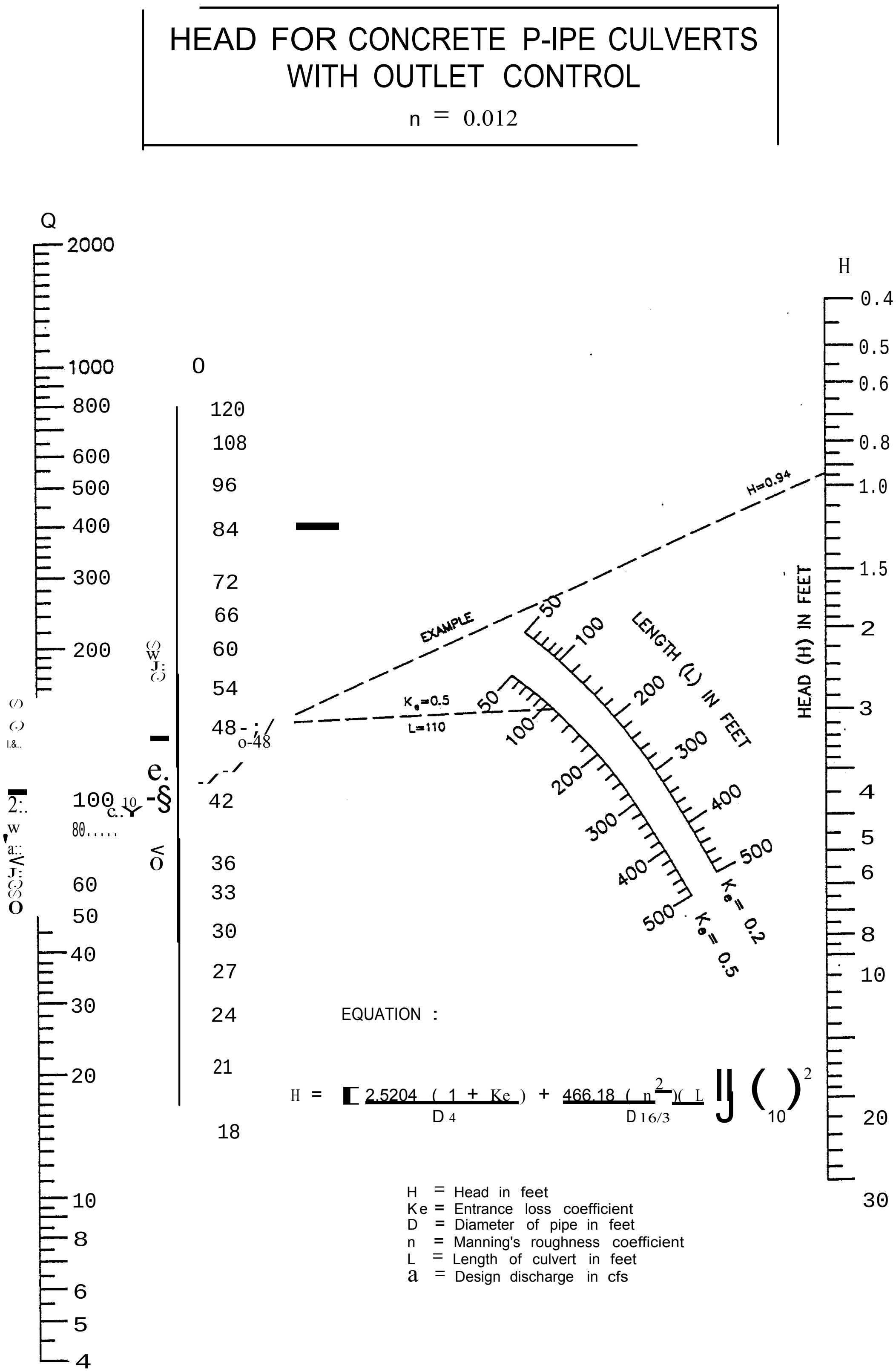


FIGURE 10-2

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

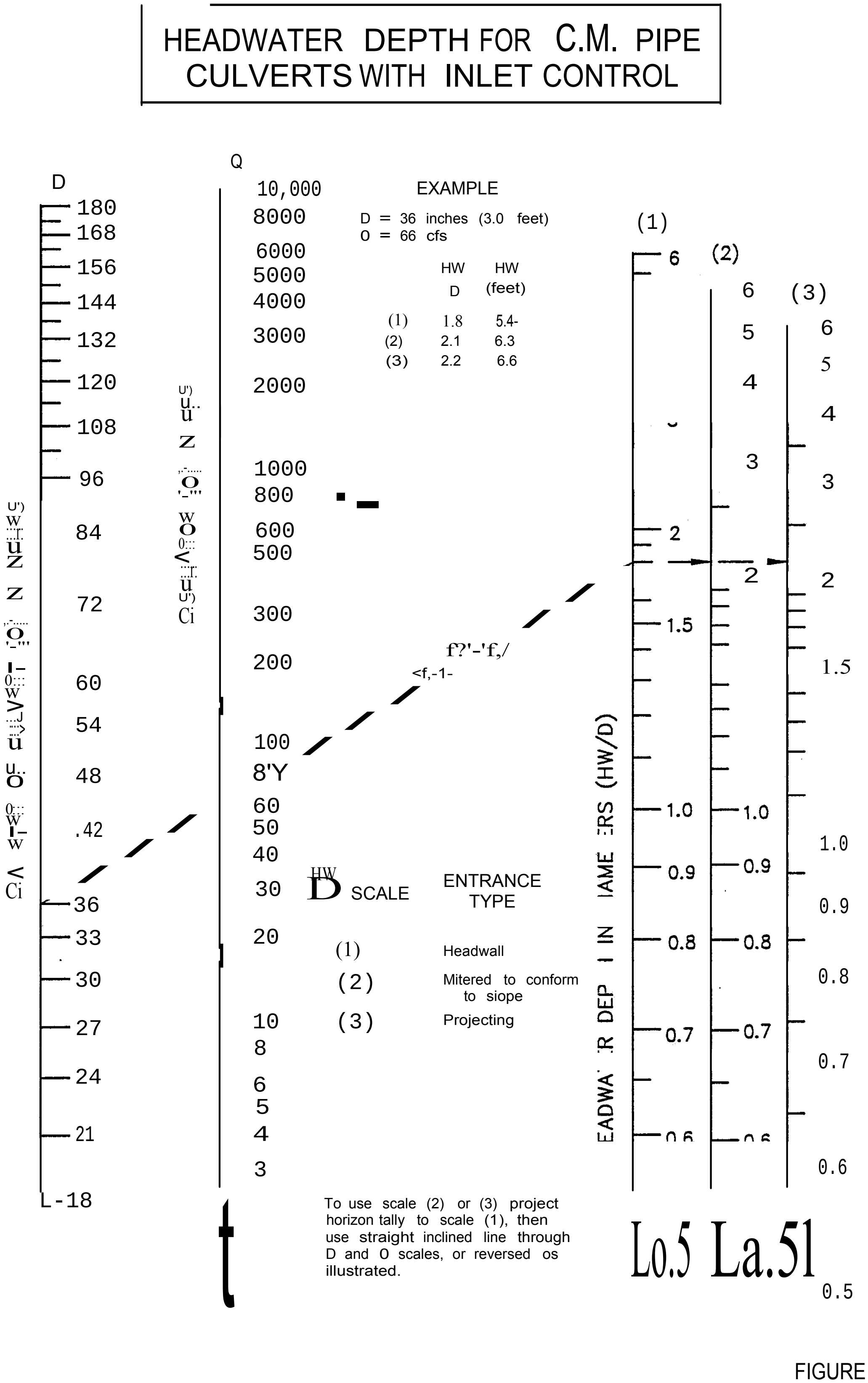


FIGURE 10-3

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

HEAD FOR C.M. PIPE CULVERTS
WITH OUTLET CONTROL
n = 0.024

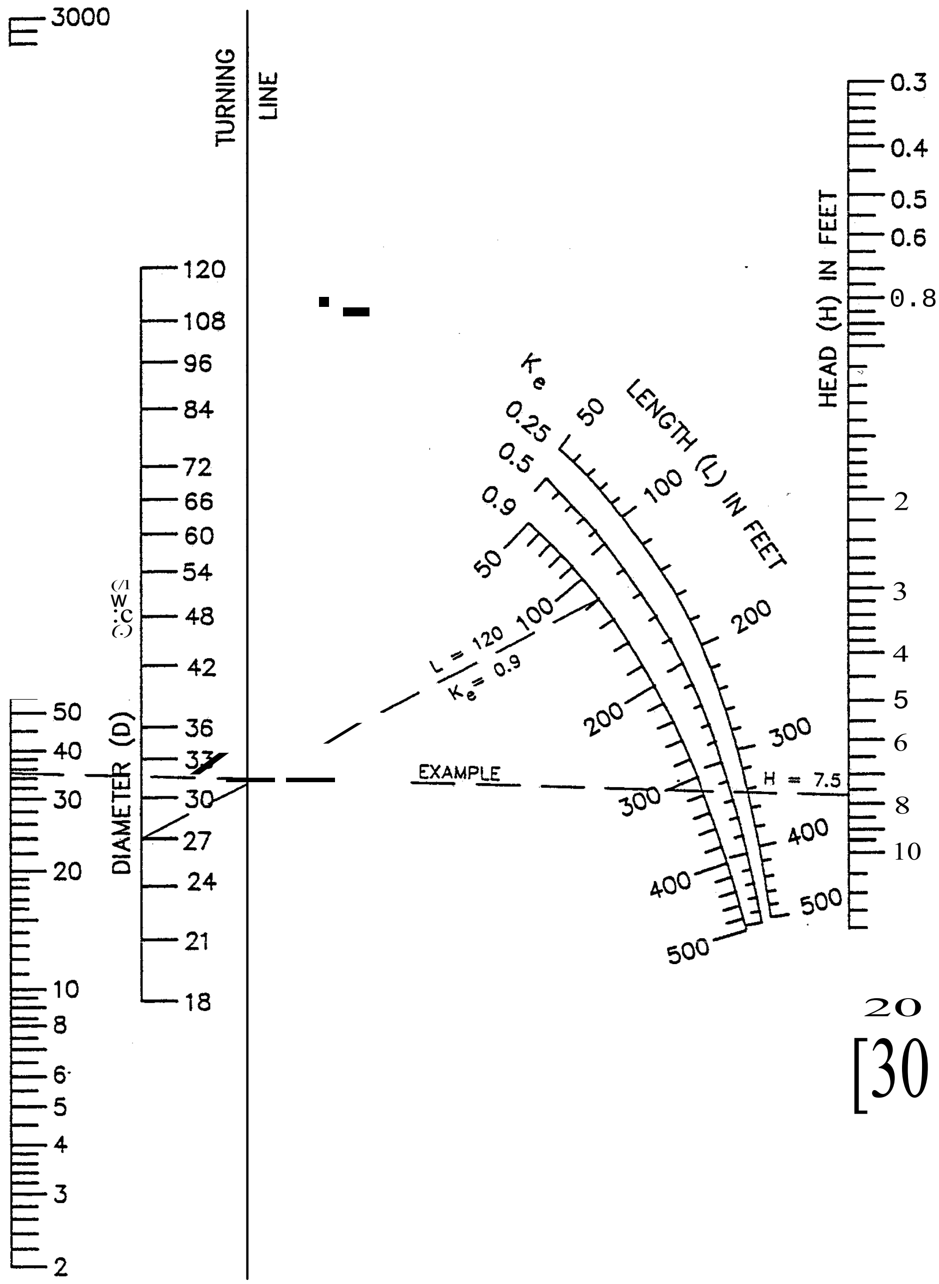


FIGURE 10-4

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

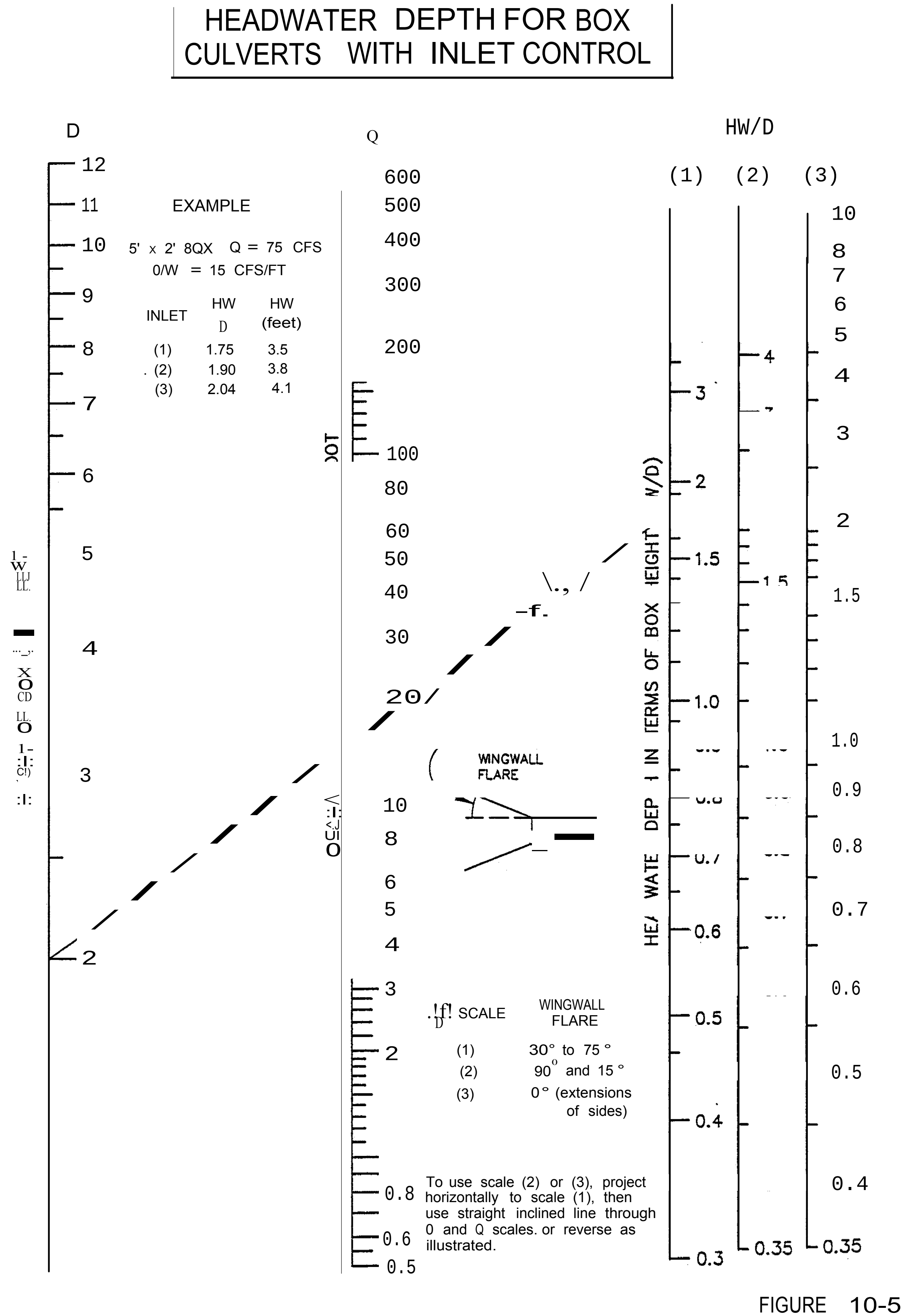


Figure 10-6. Head for Concrete Box Culverts with Outlet Control

HEAD FOR CONCRETE BOX CULVERTS WITH OUTLET CONTROL

n = 0.012

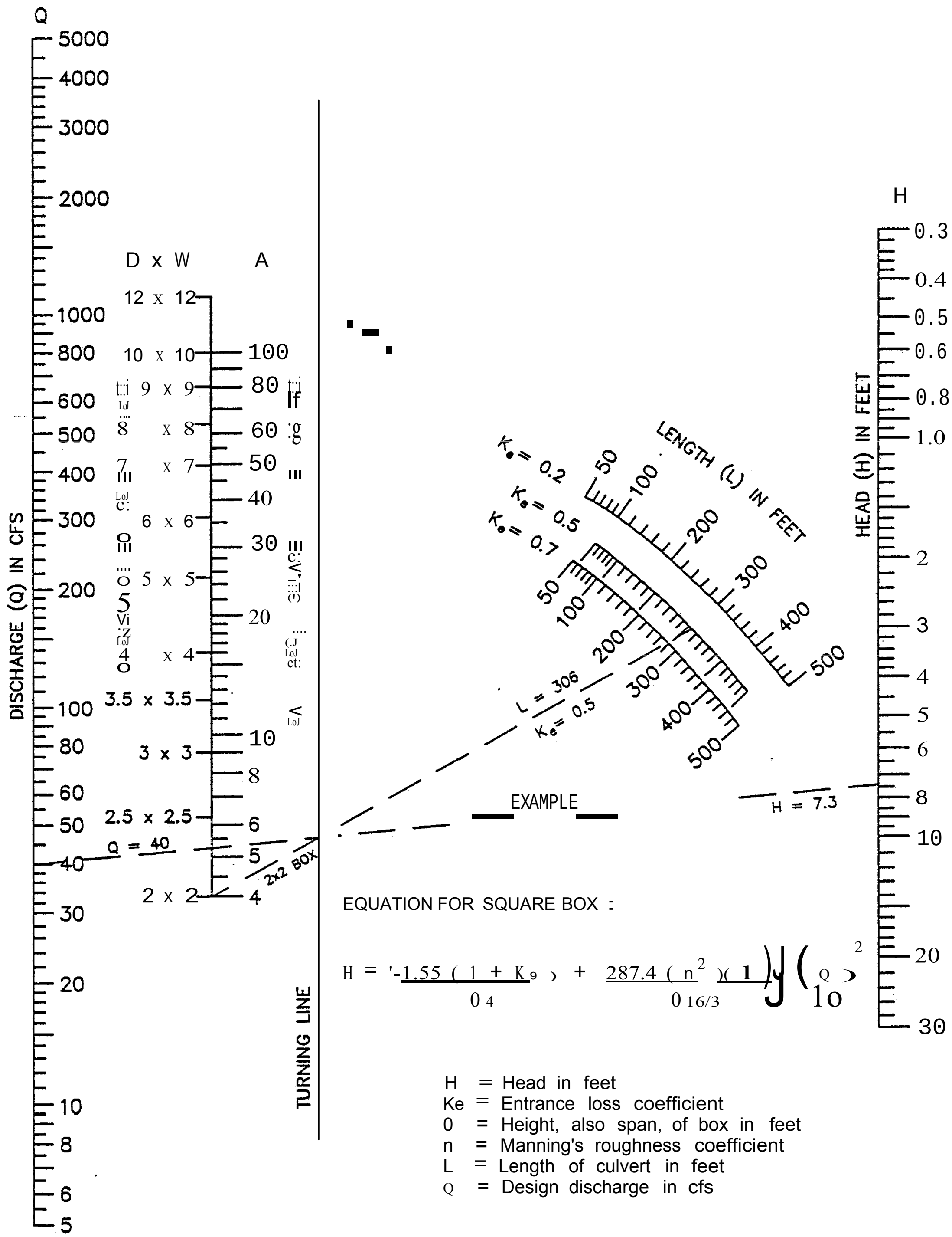


FIGURE 10-6

Table 10-2. Culvert Entrance Loss Coefficients (K_e)

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
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, 33.7° or 45° 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, 33.7° or 45° bevels	0.2
side or slope tapered inlet	0.2
Reinforced Concrete Box	
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
square-edged at crown	0.4
crown edge rounded to radius of 1/12 barrel dimension or bevelled top edge	0.2
wingwall at 10°-25° 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

11.0 DETENTION BASIN DESIGN

11.1 Applicable Design Criteria

Storm water detention basins are used to temporarily impound (detain) excess storm water, 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
- Prevention of erosive conditions in water courses
- Satisfaction of the requirements of downstream cities with detention requirements

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 and downstream to 10% rule of the detention site are required to evaluate the timing of inflow and outflow hydrographs from regional and on-site facilities.

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

- The 100-year frequency storm for ultimate development watershed conditions must be used to determine the volume of detention storage required. In addition, the outlet structure must be designed such that the peak discharge is not increased for the 1-year storm as well as the 100-year storm event. This is intended to reduce the erosion that can result from the more frequent storm events.
- The design discharge must be determined as described in Section 5 of this manual.
- A non-erodible emergency spillway or overflow area must be provided at the maximum 100-year pool level.
- Where the outflow structure conveys flow through the embankment in a conduit, the conduit must be reinforced concrete and 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 must be provided for all conduits that discharge through the embankment.
- The outflow structure must discharge flows into the natural stream or unlined channels at a non-erosive rate in accordance with Section 9 of this manual.
- Detention basins to be excavated must 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.
- Earthen embankments used to impound a required detention volume must have a minimum top-width of 12 feet, must contain a non-permeable core, and must be based on a geotechnical investigation for the site. The geotechnical investigation must be performed by a licensed engineer and must include, at a minimum, the type of material on-site, water content, liquid limit, plasticity index and desired compaction. Earthen embankments higher than six feet must conform to 31 Texas Administrative Code (TAC) Chapter 299 and other applicable dam safety requirements.

- Security fencing with a minimum height of six feet 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 Public Works Director. The fence must be designed to allow access for maintenance as well as not to restrict storm water flow into or out of the detention basin.
- A maintenance ramp must be provided for vehicular access in detention basin design for periodic desilting and debris removal. The slope of the ramp must not exceed 6H:1V, and the minimum width must be 12 feet.
- Basins with permanent storage must include dewatering facilities to provide for maintenance.
- The design of detention facilities must include provisions for collecting and removing sediment deposited after collecting and releasing storm water.
- A non-erodible pilot channel must 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 adjacent to the pilot channel to prevent undermining of the pilot channel due to scour. This type of channel is not required for permanent pool basins (i.e. wet ponds).
- For permanent pool structures, the use of a flatter shelf at the permanent pool elevation and use of native aquatic vegetation on this shelf is recommended for providing wave erosion protection and water quality benefits. If a similar system is not used, the design engineer must provide an analysis which indicates no significant risk of erosion due to wave action.

11.2 Detention Basin Calculations

Detention basins without upstream detention areas and with drainage areas of 160 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 trapezoidal inflow hydrograph and the triangular outflow hydrograph. A range of storm durations is required to determine the critical duration for the detention basin.

Basins with drainage areas greater than 160 acres or situations 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.

11.3 Detention Pond Computation Sheet

Computations performed for detention pond sizing using the Unit Hydrograph Method must be submitted to the City either in the standard output format exported from a FEMA-accepted modeling program or summarized using Computation Sheet 11-1 included in Appendix J. Instructions for Computation Sheet 11-1 are also included in Appendix J.

12.0 SITE EROSION CONTROL DURING CONSTRUCTION

12.1 Applicable Best Management Practices

Areas where development activities or channel improvements occur must be protected from site erosion. Sediment carried by storm water runoff through these areas must be prevented from entering storm drain systems and natural watercourses. The developer is encouraged to utilize with the North Central Texas Council of Governments (NCTCOG) *Design Manual for Construction* for a list of Best Management Practices (BMPs) to control site erosion.

Some acceptable forms of site erosion control devices include silt fences, silt traps, compost/mulch berms, erosion control blankets, geonetting, and geotextiles.

The developer is responsible for maintenance of site erosion control devices until 70 percent of the native background vegetative cover in unpaved areas has been achieved as determined by the Public Works Director. Periodic maintenance must be performed by the developer to remove accumulated sediment that would otherwise inhibit the proper functioning of the erosion control devices. Barriers, if used, must be replaced as a part of the regular maintenance program.

12.2 TCEQ TPDES Requirements

The TCEQ issued a general permit for storm water discharges from construction sites in March 2003. Currently, construction projects which disturb one acre or more must comply with the TPDES construction general permit for storm water discharges from the site. The one-acre threshold would also be reached if the sum of individual components of an overall development will affect one acre or more. Prior to initiating any construction project, the permit requirements should be reviewed to determine the current requirements.

A Storm Water Pollution Prevention Plan (SWP3) must be prepared for affected sites in accordance with the requirements of the general permit. The SWP3 contains a description of the construction project, the construction schedule, proposed measures which will be implemented to minimize pollution of storm water runoff, and provisions for conducting inspections to determine the effectiveness of the erosion control measures.

The SWP3 is not submitted to TCEQ but must be retained onsite. The SWP3 must be signed and certified by the general contractor and any subcontractors with site control responsibilities.

To obtain coverage under the general permit for sites five acres or larger, a Notice of Intent (NOI) must be forwarded to TCEQ at the address on the form on the following pages. The NOI must either be postmarked 48 hours prior to the start of construction activity or submitted electronically through the TCEQ's STEERS system at least 24 hours prior to the start of construction activity. NOIs may be required to be prepared, signed and submitted by both the owner of the project and the general contractor.

In addition to retaining the SWP3 on-site, a copy of the NOI or a project description sign must be posted in a visible location. Necessary forms and associated instructions can be found on the TCEQ website at www.tceq.state.tx.us. The NOT is filed with the TCEQ once the site has been stabilized and vegetation has been re-established.

13.0 FLOODWAY/FLOODPLAIN DEVELOPMENT CRITERIA

All development within the floodplain must be approved by the Public Works Director. Listed below are the procedures that must be followed when developing within a floodplain, whether or not the floodplain has been designated as such by FEMA.

13.1 Floodplain Development

Development in the floodplain will be allowed provided a sealed engineering report is submitted to and approved by the City demonstrating the following criteria are met:

- Development within the 100-year floodplain must be in accordance with the requirements of the City of Lancaster Storm Water Management Ordinance.
- Construction of any structure (including fences) which would impede flow will not be allowed within the floodway and/or floodplain.
- Minimum finished floor elevations for proposed development areas within the floodplain fringe must be a minimum of one foot (1') above the 100-year frequency fully developed watershed flood elevation as defined in Section 5 of this manual.
- Except for special circumstances approved by the Public Works Director, in no event must floodplain or floodway modifications and/or reclamations increase the water surface elevation or adversely affect stream erosion potential upstream, downstream, or through the project site.
- Floodway realignments must be approved by the Public Works Director. If a floodway realignment is being requested by the developer, the developer or his engineer must submit all necessary data to the City of Lancaster for review prior to consideration.
- A Letter of Map Revision (LOMR) must be submitted to the City and FEMA, by the developer or his engineer, after floodplain construction has been completed adequately to reflect final grading and compaction within the floodplain.

If all above conditions are met, a Conditional Letter of Map Revision (CLOMR) report may not be required as determined by the Public Work Director. If it is determined by the Public Works Director that a CLOMR must be submitted, FEMA approval must be obtained before the final plat will be approved by the City. In addition, the developer must pay all fees required to obtain both the CLOMR and LOMR. Due to the typical review period required by FEMA, the developer is encouraged to submit floodplain development requests to the City as soon as possible to avoid delays in obtaining approval of the Floodplain Development Permit.

Subsection 13.1.1 contains the Sequence of Action for Reclamation in a FEMA designated Floodplain and Subsection 13.1.2 contains the Sequence of Action for Reclamation in a Floodplain not designated by FEMA.

13.1.1 Process for Reclamation in FEMA Designated Floodplains

Step 1 The Developer may submit a Preliminary Plat at this time. (Note: An approved Preliminary Plat expires after twelve (12) months unless the Final Plat has been submitted for approval.)

Step 2 The Developer submits the following:

1. Dual element Reclamation Plan with HEC-2 or HEC-RAS data for existing condition 100- year flows (FEMA) and HEC-2 or HEC-RAS data for ultimate developed 100-year flows (City) The report shall include all information necessary to determine if all floodplain development criteria have been met by the project and the submitting engineer's opinion on whether a CLOMR is or is not required by FEMA.

2. Development Permit Application to the City

Step 3 The City reviews and comments on the effects of the Reclamation Plan and determines if a CLOMR submittal to FEMA will be required.

Step 4 The Developer submits a revised Reclamation Plan to the City if required. Step 3 and Step 4 are repeated until City comments have been completely addressed to the City's satisfaction. If a CLOMR submittal to FEMA is not required, skip to Step 7.

Step 5 If required, a CLOMR submittal is sent to FEMA by the City for conditional approval. All fees required by FEMA for CLOMR submittal and approval must be paid by the developer.

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

Step 7 The City approves the Development Permit and Grading Plan and releases the fill activities.

Step 8 The Developer submits a Preliminary Plat to the City for City Council and Planning and Zoning Commission approval.

Step 9 The Developer submits a Final Plat to the City for City Council and Planning and Zoning Commission for approval. (Note: The developer may submit the Final Plat any time after the Preliminary Plat has been approved and before the Reclamation Plan has been submitted for staff review.) In those cases where the Final Plat is submitted prior to the completion of the Reclamation Plan activities, the Final Plat submittal must 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 must be paid by the developer.

Step 10 A LOMR submittal must be prepared by the developer or his engineer to reflect As-built (record) drawings of the reclamation project. This submittal must include HEC-2 or HEC-RAS data for the existing and fully developed conditions if modified based on as-built conditions and/or as required by FEMA, qualified geotechnical analysis certifying compaction of the fill to no less than 95 percent of the standard proctor densities of the material, and any other information required by FEMA.

Step 11 The City reviews and comments on the LOMR submittal.

Step 12 The Developer submits a revised LOMR submittal. Step 11 and Step 12 are repeated until City comments have been completely addressed to the City's satisfaction.

Step 13 The City will forward the LOMR submittal to FEMA for approval which will result in the revision of the special flood hazard area per the developers study.

Step 14 After the LOMR submittal has been forwarded to FEMA, the construction plans for the water, wastewater, street and storm water systems, if complete, will be reviewed and approved for construction. Construction may proceed while FEMA is reviewing the submittal. Building permits for non-residential buildings may be issued at this time.

Step 15 FEMA approves LOMR. (Note: If FEMA rejects the submittal for the LOMR, revisions to the Final Plat may be required to modify the drainage easements to reflect the adjustments required by FEMA.)

Step 16 Final Acceptance of the development. The Certificate of Occupancy may be issued for non-residential buildings and building permits for residential buildings may be issued at this time.

13.1.2 Process for Reclamation in Floodplain Not Designated by FEMA

Step 1 The Developer may submit a Preliminary Plat at this time. (Note: An approved Preliminary Plat expires after twelve (12) months unless the Final Plat has been submitted for approval.)

Step 2 The Developer submits the following:

1. Reclamation Plan and HEC-2 or HEC-RAS data for ultimate developed 100- year flows. The report shall include all information necessary to determine if all floodplain development criteria have been met by the project.
2. 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 3 and Step 4 are repeated until City comments have been completely addressed to the City's satisfaction.

Step 5 The City can then approve the Development Permit and Grading Plan and release the fill activities.

Step 6 The Developer submits a Preliminary Plat to the City for City Council and Planning and Zoning Commission approval.

Step 7 The Developer submits a Final Plat to the City for City Council and Planning and Zoning Commission approval. (Note: The developer may submit the Final Plat any time after the Preliminary Plat has been approved and before the Reclamation Plan has been submitted for staff review.) In those cases where the Final Plat is submitted prior to the completion of the Reclamation Plan activities, the Final Plat submittal must 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 fully developed conditions. This submittal must include HEC-RAS data for the existing and fully developed conditions, and qualified geotechnical analysis certifying compaction of the fill to no less than 95 percent of the standard proctor densities of the material.

Step 10 The construction plans for the water, wastewater, street, and storm water systems will be released for construction. Building permits for non-residential buildings may be issued at this time.

Step 11 Final Acceptance of the development. The Certificate of Occupancy may be issued for non-residential buildings and building permits for residential buildings may be issued at this time.

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APPENDIX B. LIST OF ABBREVIATIONS

Θ	Angle.
a	Gutter depression.
ac	Acres.
A	Area.
AMC	Antecedent moisture condition.
ASCE	American Society of Civil Engineers.
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.
CA	Product of runoff coefficient and drainage area used in Rational Method.
cfs	Cubic feet per second.
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).
h, H	Head.
hb	Headloss at a bend.
H_j	Headloss at a junction.
HL	Total headloss.
HW	Headwater depth.
I	Rainfall intensity.
K	Dimensionless coefficient used in the Ration Method to account for antecedent precipitation.
K_b	Headloss coefficient at a bend.
K_e	Entrance loss coefficient.
K_j	Headloss coefficient at a junction.
L	Length.
L_a	Length of curb inlet require 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.
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.
SWMP	Storm Water Master Plan.
T_c	Tme of concentration.
t_p	Hydrograph lag time from midpoint of rainfall duration, tr, to peak of unit hydrograph.
t_{pR}	Lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph.
tr	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

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.

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.

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 base flood for the City of Lancaster is defined as the 100-year frequency flood based on fully developed watershed conditions. The base flood elevation is the water surface elevation developed using the base flood as defined in part II of this manual. The City of Lancaster base flood elevation will not necessarily correspond FEMA base flood elevation.

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

Chute An inclined conduit or structure used for conveying water to a lower level.

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.

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 storm water 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 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.

Drainage System Drainage systems must include streets, alleys, storm drains, drainage channels, culverts, bridges, overflow swales and any other facility through which or over which storm water 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.

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.

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.

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 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 Storm Water Management.

Flood Management See Storm Water 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 flood based on ultimate watershed development conditions. The "FEMA floodplain" must refer to the area subject to flooding resulting from the 100-year flood for current watershed development conditions.

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 ultimate 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.

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 (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.

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, drains, etc. (c) The finished surface of a canal bed, road bed, top of an embankment, or bottom of excavation.

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 storm water runoff.
Histogram	Representation of statistical data by means of rectangles whose widths represent rainfall, runoff, etc. and whose height represents frequency.
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 drain 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 storm water.
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.
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.
Manning Coefficient (n)	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.
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.
100-year Event	Event (rainfall or flood) that statistically has a one percent chance of being equalled or exceeded in any given year.
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.
Peak Flow	The maximum rate of runoff during a given runoff event.
Percolation	To pass through a permeable substance such as ground water flowing through an aquifer.

Permeability The property of a material which permits movement of water through it when saturated and actuated by hydrostatic pressure.

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.

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 possiblein 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.

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 gaging 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.

Return Period See Recurrence Interval.

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.

Right-of-Way (ROW) 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.

Runoff Curve Number Index number used in the Soil Conservation Service unit hydrograph method as a measure of the tendency of rainfall to run off into streams rather than evaporate or infiltrate.

Scour The erosive action of running water in streams or channels in excavating and carrying away material from the bed and banks.

Sediment Material of soil and rock origin transported, carried, or deposited by flowing water.

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.

Spillway A waterway in or about a dam or other hydraulic structure for the escape of excess water.

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 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	
Type A – Major Type B – Major Type C – Secondary Major	Carries traffic from one urban area to another and serves the major activity centers of the urbanized areas. A major thoroughfare is used for longer urban trips and carries a high portion of the total traffic with a minimum of mileage.
Type D - Collector Street	Carries traffic from local streets to thoroughfares. Uses served would include medium and high density residential, limited commercial facilities, some small offices, and as direct access within industrial parks.
Type E – Residential Type F - Estate	Distributes traffic to and from residences. Short in length, non- continuous to discourage through traffic. Low-density residential areas.
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.
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.

Warped Headwall The wingwalls are tapered from vertical at the abutment to a stable bank slope at the end of the wall.

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.

APPENDIX D.
Not Used

APPENDIX E.
Computation Sheet 7-1: Gutter-Inlet Flow

BY ____________
DATE ____________
CK'D ____________
DATE

COMPUTATION SHEET 7 -1
GUTTER-INLET FLOW

CONTRACT OR FILE NO

Inlet	Drainage AreaiD	Drainage Area Size	RlDloiT Coefficient	Design Storm	Time to Inlet	Intensity	Q	Qb	Q,	n	S,	S,	a	W	S'w	Sw	T,	Q,	T/W	SwiS,	Eo	S,	QCALC	T	y	Cw	Lt	L	E	Qi	Qb	Byp • Tan: ;et	Intercepted Acrea2e
--	--	acres	constant	Y"	min	inlhr	'''	'''	'''	constant	ft!ft	ft!ft	ft	ft	ft!ft	ft!ft	ft	'''	ft!ft	ft!ft	--	ft!ft	'''	ft	ft	constant	ft	ft	--	'''	'''	--	Ac
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	30	31	32	33	34

Instructions for Computation Sheet 7-1

Column	Parameter	Instructions
1	Inlet Number	Inlet number of structure receiving inflow, which is the same as the primary contributing drainage area subbasin
2	Drainage Area ID	Drainage area subbasin unique identifier for contributing area
3	Drainage Area Size	Contributing drainage area subbasin size in acres
4	K * C	Adjusted runoff coefficient, anticedent moisture coefficient K is found in Table 5-1 runoff coefficient C is found in Table 5-2
5	Design Storm	Return period in years selected for gutter/inlet flow calculations
6	Time to Inlet (min)	Time of concentration to design point from Table 5-3 or hand calculations shown with flow paths
7	Intensity (in/hr)	Intensity corresponding to inlet time of concentration and design storm frequency being used
8	Q	Gutter flow (cfs) calculated as (K*C * Drainage Area Size * Intensity)
9	Q_b	Bypass flow from upstream inlet (cfs) from Column 32
10	Q_T	Total gutter flow (cfs) calculated as $Q + Q_b$
11	n	Mannings roughness coefficient for gutter (0.016)
12	S_L	Longitudinal slope of the roadway (ft/ft)
13	S_x	Cross slope of the roadway (ft/ft), for sump locations use 0.0015 ft/ft
14*	a	Depth of gutter depression or inlet local depression (ft)
15*	W	Gutter width over which local depression occurs (ft)
16*	S'_w	Gutter cross slope measured from the cross slope of the pavement (ft/ft) calculated as [a / W]
17*	S_w	Gutter cross slope calculated as ($S_x + S'_w$) (ft/ft)
18*	T_S	Width of spread in travel lane (T-W) (ft)
19*	Q_S	Flow capacity of the gutter above the depressed section (cfs)
20*	T/W	Spread over depressed width of gutter
21*	S_W/S_X	Gutter cross slope over roadway cross slope
22*	E_o	Ratio of flow in depressed gutter section to the total flow in the gutter For depressed inlets, $1/\{1+ (S_w / S_x)/([1+(S_w / S_x)/((T/W)-1)]^{2.67}-1)\}$
23*	S_e	Equivalent cross slope (ft/ft) Calculated as $S_e = S_x + S'_w * E_o$ Note: If W = 0, then $S_e = S_x$

Instructions for Computation Sheet 7-1 (continued)		
Column	**Parameter**	**Instructions**
24*	Q_{CALC}	For composite street sections this is the flow that corresponds to the assumed spread (T) in column 24 (cfs) calculated as: $Q_S/(1-E_O)$
25	T	Width of the spread or the ponding of the flow in the gutter (ft): For straight crowns, $T = y / S_x$ For composite street sections T is assumed and Q_{CALC} in column 24 is compared to the Q_T in column 10, T is adjusted until the Q's match
26	y	Depth of flow in the roadway at the curb (ft): For straight street sections, $y = T\ S_X$ For composite street sections, $y = [(Q * n * S_x) / (0.56 * S^{1/2})]^{3/8}$
27**	C_W	Wier coefficient for inlet required length in sag location $C_W = 2.3$ for composite street sections and locally depressed gutter sections or 3.0 for straight street sections
28	L_T	Total inlet length required for curb inlets (ft) For on grade inlets: $L_T = K_c * Q^{0.42} * S_L^{0.3} * (1 / (n * S_x))^{0.6}$ For sag inlets in wier conditions: $L_T = [Q / C_W * y^{1.5}] – 1.8*W$ For saginlets in orifice conditions: $L_T = Q / \{0.67*h [2g (y-(h/2))]^{0.5}\}$
29†	L	Length of the curb inlet (ft)
30	E	Efficiency of Curb inlet on grade when $L< L_T$ calculated as: $E = 1 - (1 - (L / L_T))^{1.8}$
31	Q_i	Intercepted flow: Inlets on-grade $Q_i = Q * E$ (cfs) Inlets in wier sag conditions $Q_i = C_w * (L + 1.8 * W) * y^{3/2}$ Inlets in orifice sag conditions $Q_i = C_O * Lh * [2 * g * (y - (h / 2))]^{0.5}$
32	Q_b	Bypass flow calculated as $Q_T - Q_i$ (cfs)
33	Bypass Target	Inlet where bypass flow is collected
34	Intercepted Area	Equivalent are captured by inletcalculated by : Q_i / (K*C*i)

* Fields required for composite gutter sections or locally depressed inlet calculations only

** Field required for sag gutter conditions only

† For sag conditions if $L< L_T$ spread must be recalculated for new y from the equation provided

APPENDIX F.
Computation Sheet 8-1: Hydraulic Gradeline Computations for Storm Drains

BY ______________

DATE ______________

CK'D ______________

DATE ______________ "n" = 0.013

COMPUTATION SHEET 8 - 1
HYDRAULIC GRADELINE COMPUTATIONS FOR STORM DRAINS

CONTRACT OR FILE NO. ______________

Manhole or Inlet		Distance between Manhole or Inlet	Drainage Area			Coefficient of Runoff "KC"	Incremental "KCA"	Total "KCA"	Time of Concentration			Design Frequency	Intensity	Discharge "Q"	Slope of Pipe	Selected Size of Pipe	Flow depth	Slope of Frictional Gradient Sf	Hydraulic Gradient Elevations		Head Loss at Change in Section							HGL at Design Point	Elev. of Invert		Notes
From	To		Inrement Area		Total Area				Inlet/System Time	Flow Time in Sewer	Total Time										V_1	V_2	$\frac{V_2^2}{2g}$	$\frac{V_1^2}{2g}$	K_j	$\frac{K_jV_1^2}{2g}$	h_j				
Design Point	Design Point		No.	Area															Up Stream	Down Stream	Inflow	Outflow							Upstream	Downstream	
		ft.		acres	acres				min.	min.	min.	years	in/hr	cfs	ft/ft	inches	ft	ft/ft	ft msl	ft msl	ft/sec	ft/sec	ft	ft	constant	ft	ft	ft msl	ft msl	ft msl	
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	30	31	32
SD LINE " "																															
SD LINE " " LATERALS																															

Instructions for Computation Sheet 8-1

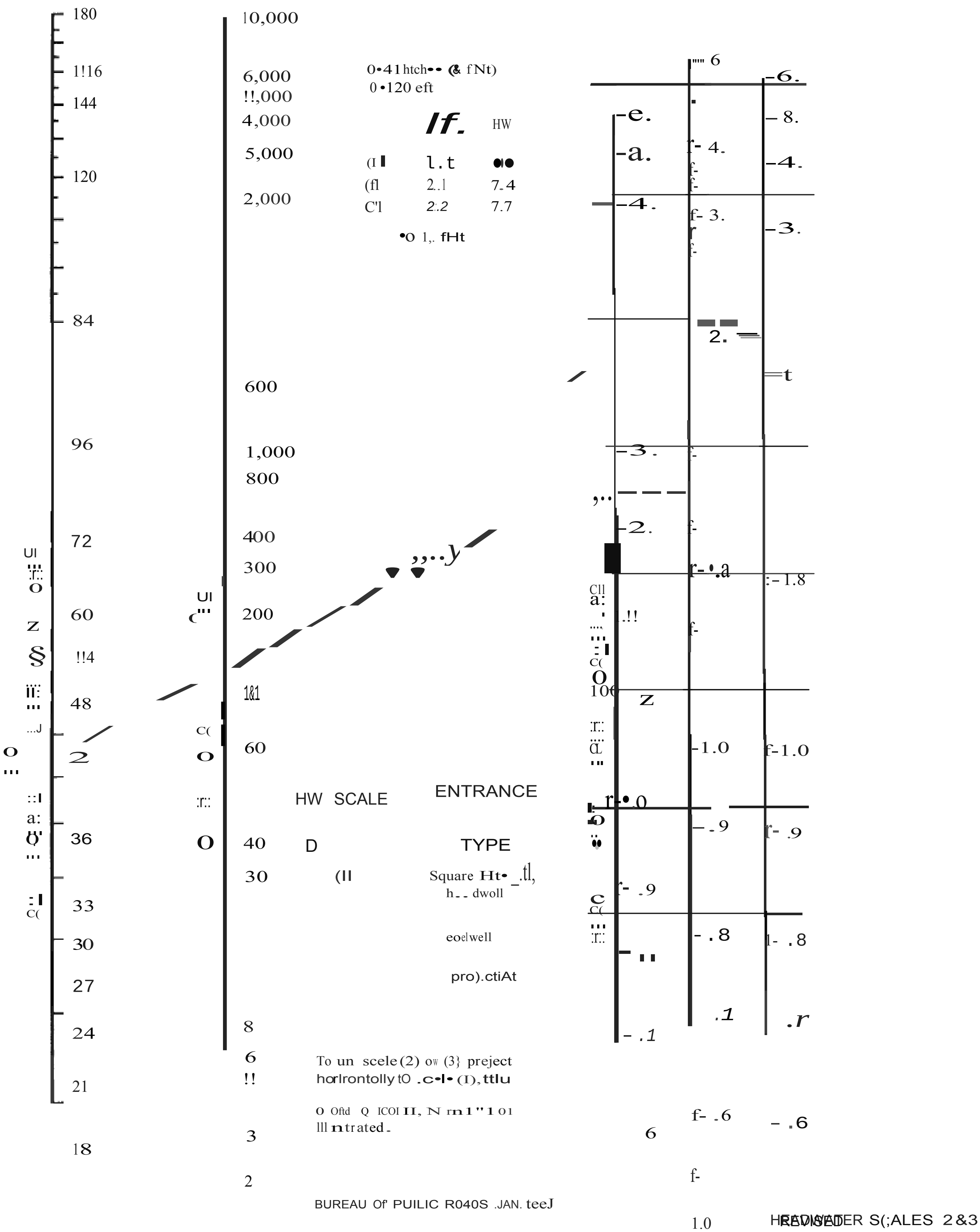
Column	Parameter	Instructions
1	From Design Point	Upstream design point
2	To	Junction point immediately downstream
3	Length (feet)	Conduit length between design points
4	Drainage Area Incremenetal Area, No.	Incremental drainage area number or name
5	Drainage Area Incremenetal Area, Area (acres)	Incremental drainage area in acres
6	Drainage Area Total Area (acres)	Sum of Column 5 (to that row)
7	Coefficient of Runoff “KC”	Weighted runoff coefficient for incremental area
8	Incremental “KCA”	Column 5 * Column 7
9	Total “KCA”	Sum of Column 8 (to that row)
10	Inlet/System Time of Concentration (T_c) (minutes)	For the most upstream most storm drain, value equals inlet T_c from Table 5-3 For all others, value equals (system T_c + flow time in sewer) from the previous row
11	Flow Time in Sewer (minutes)	Column 3 / (Column 23 * 60)
12	Total Time (minutes)	Column 10 + Column 11
13	Design Frequency (years)	Return period used for system design
14	Intensity (in/hr)	Rainfall intensity determined by using Column 10, Table 4-1 and Equation 4-1
15	Discharge Q (cfs)	Column 9 *Column 14
16	Slope of Pipe (ft/ft)	Proposed slope of the conduit, (Column 30 - Column 31) divided by Column 3
17	Selected Size of Pipe (ft)	Diameter of Pipe
18	Flow Depth (ft)	Depth of flow in Pipe (ft)
19	Slope of Frictional Gradient (ft/ft)	Friction Slope from Equation 8-4
20	Hydraulic Gradient Elevation, Upstream (ft msl)	Hydraulic Grade Line at upstream end of Conduit
21	Hydraulic Gradient Elevation, Downstream (ft msl)	Hydraulic Grade Line at dwonstream end of Conduit, Column 20+ Column 28

Instructions for Computation Sheet 8-1 (continued)

Column	Parameter	Instructions
22	V_1 inflow velocity (ft/s)	Velocity in upstream conduit, for upstream inlet set to 0.00
23	V_2 outflow velocity (ft/s)	Velocity in counduit based on full flow conditions
24	$V_2^2/2g$ (ft)	Downstream velocity head, velocity from Column 23
25	$V_1^2/2g$ (ft)	Upstream velocity head, velocity from Column 22
26	K_j (constant)	Loss coefficient from Table 8-4
27	$K_jV_1^2/2g$ (ft)	Adjusted upstream velocity head, Column 25 * Column 26
28	h_j (ft)	Headloss from Section 8.3.3 Equation 8-7, 8-8, 8-9 or 8-10)
29	HGL at Design Point (ft msl)	Hydraulic Grade Line into upstream end of Conduit, Column 20 + Column 28
30	Elev. Of Invert, Upstream (ft msl)	Invert elevation at upstream end of conduit
31	Elev. Of Invert, Downstream (ft msl)	Invert elevation at downstream end of conduit
32	Notes	Design notes

APPENDIX G.
Chart 1B: Headwater Depth for Concrete Pipe Culverts with Inlet Control

Chart 1B:
Headwater Depth for Concrete Pipe Culverts with Inlet Control



APPENDIX H.

Computation Sheet 9-1: Standard Step Backwater

Computation Sheet 9-1: Standard Step Backwater Calculations

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

________________________________ Date: ____________________ Checked By: ____________________

____

manning's n = __________

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)

Instructions for Computation Sheet 9-1

Column	Parameter	Instructions
1	Station	Cross-section identified by station number.
2	Invert Elevation (feet above Mean Sea Level - MSL)	Invert flowline elevation of channel or stream, (ft. m.s.l.).
3	Design (Q) (cubic feet per second)	Design discharge Q, (cfs)
4	Depth (Y)	Assumed depth of flow, Y (ft.).
5	Water Surface Elevation (W) (feet above MSL)	Water surface elevation, Col. 2 + Col. 4, (ft. m.s.l.).
6	Flow Area (A) (square feet)	Area of flow at depth Y, (ft.^{2}).
7	Flow Velocity (V) (feet per second)	Mean velocity, design discharge (Col. 3) divided by area (Col. 6), (ft./sec.).
8	Velocity Head ($V^2/2g$) (feet)	Velocity head, (Col. 6) V^2 / (2*g) (ft.).
9	Total Head Elevation at Sect 1 (H_1) (feet above MSL)	Elevation of the total head, Col. 5 + Col. 8, (ft.).
10	Wetted Perimeter (P) (feet)	Wetted perimeter of flow at depth Y, (ft.).
11	Hydraulic Radius (R) (feet)	Hydraulic radius, Col. 6 divided by Col. 10, (ft.).
12	$R^{4/3}$	Hydraulic radius to the 4/3 power.
13	Energy Slope (S_e) (feet per foot)	Energy slope at section, (ft./ft.), (See Eq. 9-6.)
14	Average Energy Slope (S_e) (feet per foot)	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. 9-6.)
15	Length (L) (feet)	Length between cross-sections, (ft.).
16	Head Loss (H_L) (feet)	Friction loss between cross-sections. Col. 14 times Col. 15.
17	Total Head Elevation at Sect 2 (H_2) (feet above MSL)	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.]

APPENDIX I.

Computation Sheet 10-1: Culvert Design

Computation Sheet 10-1: Culvert Design

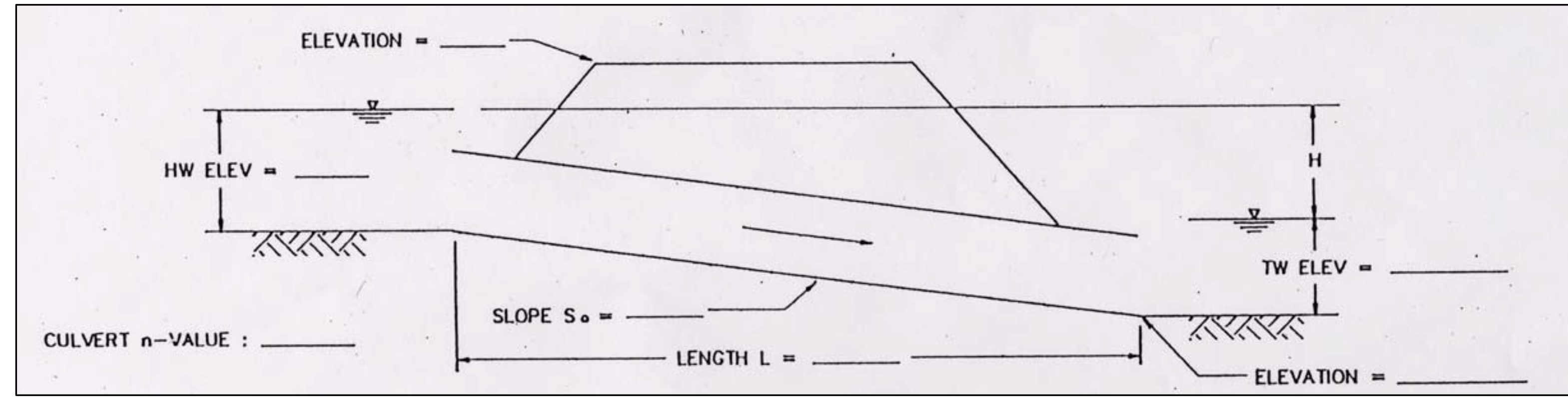
Project No.: ____________________

Date: ____________________

By: ____________________

Checked By: ____________________

Project Description:
____
____
____
____
____



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Culvert Description (Entrance type)	Flow (cfs)	Culvert Size	Headwater Computation										Controlling Headwater Depth	Outlet Velocity	Comments
			Inlet Control		Outlet Control HW= Ho - LSo										
			$\frac{HW}{D}$	HW	Ko	H	dc	$\frac{dc - D}{2}$	TW	ho	LSo	HW	.		

Instructions for Computation Sheet 10-1

Column	Parameter	Instructions
1	Culvert Description	Culvert description.
2	Flow (cfs)	Design discharge in cfs.
3	Culvert Size	Culvert size.
4	Inlet Control HW/D	Ratio of headwater depth to culvert height from Figure 10-1, 10-3 or 10-5.
5	Inlet Control HW	Headwater depth for inlet control conditions in feet, which is obtained by multiplying the culvert height times the value in Column 4.
6	K_o	Entrance loss coefficient, which is based on the configuration of the culvert entrance.
7	H	Head in feet between the upstream and downstream sides of the culvert, which is obtained from Figure 10-2, 10-4, or 10-6.
8	d_c	Critical depth in feet For circular conduits, $Q^2/g = A^3/b$ where: Q = The design discharge in cfs A = The flow area in sq. ft b = The water surface width in feet g = The acceleration due to gravity (32.2 ft/sec^2) For rectangular conduits, $d_c^3 = Q^2 / (gw^2)$ where: d_c = The critical depth in feet Q = The design discharge if cfs w = The width of the conduit in feet g = The acceleration due to gravity (32.2 ft/sec^2)
9	$(d_c\text{-}D)/2$	One half of the sum of the critical depth and the culvert height.
10	TW	The tailwater depth in feet at the culvert outlet.
11	h_o	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.
12	LS_o	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).

Instructions for Computation Sheet 10-1 (continued)

Column	Parameter	Instructions
13	HW	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.
14	Controlling Headwater Depth	Actual headwater depth in feet at the culvert, which is the greater of the values in Column 5 and Column 13.
15	Outlet Velocity	Outlet flow velocity in fps based on the depth of flow in the culvert.

APPENDIX J.

Computation Sheet 11-1: Detention Ponds

Modified Rational Method Detention Pond Design

Project Description: ____________________ Project No.: By: ____________________

____ Date: Checked By: ____________________

Step		K	C	Rainfall Intensity, I = b/(Tc+d)^e					A	Q peak
				b	d	e	Tc	I (in/hr)	(acres)	(cfs)
1	Exist. Cond.	1.25		95.78	12.00	0.757				
2	Prop. Cond.	1.25		95.78	12.00	0.757				

Step 3										
Time Step	Td Duration (min)	K	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		1.25								
2		1.25								
3		1.25								
4		1.25								
5		1.25								
6		1.25								
7		1.25								
8		1.25								
9		1.25								
10		1.25								
11		1.25								
12		1.25								

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=95.78, d=12, and e=0.757).

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

Computation Sheet 11-1

Instructions and Examples for Computation Sheet 11-1

GIVEN: A 53.91-acre site which is currently zoned as agricultural use, and is to be developed for Zone RS-60, Single-Family Residence District 6,000 ft^2 lot per unit subdivision with a Rational Method C of 0.55. The 44.27 acres are to be developed and 9.46 acres will become a park in which the proposed detention pond will be placed. The park will have 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. This is the maximum release rate from site after development.

Calculate Peak Discharge for Present Conditions (Agriculture)

Q	=	K*C*I*A
K	=	1.25
C	=	0.30
T_c	=	26 minutes
For 100-year		b = 95.776, d = 12, and e = 0.7566
i_{100}	=	$b/((T_c+d)^e) = 95.78 / ((26.00 + 12)^{0.7566}) = 6.11$ in./hr.
Q_{100}	=	1.25 * 0.30 * (6.11) * 53.91 = 123.52 cfs

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 (RS-60)

K	=	1.25
C	=	(C1 * A1 + C2 * A2) / (A1 + A2)
	=	(44.27 * 0.55 + 9.46 * 0.30) / (44.27 + 9.46)
	=	0.50
T_c	=	21 minutes
For 100-year		b = 95.776, d = 12, and e = 0.7566
i_{100}	=	$b/((T_c+d)^e) = 95.78 / ((21.00 + 12)^{0.7566}) = 6.80$ in./hr.
Q_{100}	=	1.25 * 0.50 * (6.80) * 53.91 = 229.12 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 = 6.80	Q = 1.25 * 0.50 * (6.80) * 53.91 = 229.12	cfs
30 minutes	i = 5.66	Q = 1.25 * 0.50 * (5.66) * 53.91 = 190.71	cfs
40 minutes	i = 4.82	Q = 1.25 * 0.50 * (4.82) * 53.91 = 162.40	cfs
50 minutes	i = 4.22	Q = 1.25 * 0.50 * (4.22) * 53.91 = 142.19	cfs
60 minutes	i = 3.77	Q = 1.25 * 0.50 * (3.77) * 53.91 = 127.03	cfs
70 minutes	i = 3.41	Q = 1.25 * 0.50 * (3.41) * 53.91 = 114.90	cfs
80 minutes	i = 3.13	Q = 1.25 * 0.50 * (3.13) * 53.91 = 105.46	cfs
90 minutes	i = 2.89	Q = 1.25 * 0.50 * (2.89) * 53.91 = 97.37	cfs
100 minutes	i = 2.70	Q = 1.25 * 0.55 * (2.70) * 53.91 = 90.97	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 concentration, (min) for that duration
Q = Flow for that T_c, (cfs)
T_c = Time of concentration of the basin
Q_0 = Original flow, pre-development conditions

For 21 Minute Storm

Inflow 21 * (229.12) * 60 sec/min	=	288,691 cf
Outflow (0.5) * (21+21) * (123.52) * 60 sec/min	=	155,635 cf
Storage (cf)	=	133,056 cf
Storage (acre-ft) = Storage (cf) / 43,560	=	3.05 acre-ft

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

Maximum volume required is 165,555 cubic feet or 3.80 acre-feet at the 60 min. storm duration, with a maximum release rate equivalent to the existing flow of 123.52 cfs.