City of Gatesville
Spur Capital of Texas

Drainage Criteria Manual

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CITY OF GATESVILLE
DRAINAGE CRITERIA MANUAL

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SECTION 1 – DRAINAGE POLICY

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SECTION 1 – DRAINAGE POLICY

1.1.0 GENERAL

This Manual represents the application of accepted principles of storm water drainage engineering and is a working supplement to basic information obtainable from standard drainage handbooks and other publications on drainage design. The policy statements of this section provide the underlying principles by which all drainage facilities shall be designed. The application of the policy is facilitated by the technical criteria contained in the remainder of the manual.

1.2.0 CITY OF GATESVILLE DRAINAGE POLICY

1.2.1 Application

The City's drainage policy shall govern the planning and design of drainage infrastructure within the Corporate Limits of the City and within all areas subject to its extra territorial jurisdiction, as required. Definitions, formulae, criteria, procedures and data in this manual have been developed to support this policy. If any condition requiring some additional measure of protection is identified during design or construction, the Engineer shall make provisions within the design.

1.2.2 General

A. Storm water runoff peak flow rates for the 25-yr and 100-yr frequency storms shall not cause increased adverse inundation of any building or roadway surface.
B. Street curbs, gutters, inlets and storm sewers shall be designed to intercept, contain and transport all runoff from the 25-yr frequency storm, without overtopping the curb.
C. In addition to B above, the public drainage system shall be designed to convey those flows from greater than the 25-yr frequency storm up to and including the 100-yr frequency storm within defined public rights-of-way or drainage easements.
D. When storm water detention is provided, storm water runoff peak flow rates shall not be increased at any point of discharge for the 25-yr storm and 100-yr storm frequency events.

1.2.3 Drainage Flow in Streets

No concentrated point discharges directly into streets will be allowed unless approved by the City Engineer.

No lowering of the standard height of street crown shall be allowed for the purposes of obtaining additional hydraulic capacity.

1.2.4 Street Cross Flow

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. Cross-flow is allowed only in case of super elevation of a curve or overflow from the higher gutter on a street with cross fall. When runoff is allowed to cross from one curb line to the opposing curb line, the depth of flow shall not exceed six (6) inches of depth at any point within the street. This policy prohibits the use of concrete valley gutters at points other than intersections.

At points of concentration other than intersections, cross-flows shall be contained within underground storm conduit. The crown of the street shall not be removed to allow cross-flow.

1.2.5 Allowable Flow of Water Through Intersections

As the storm water flow approaches a street intersection, inlets shall be required if the depth of flow exceeds six (6) inches at any portion of the street intersection. Concrete valley gutters shall be used to convey storm water flow through intersections. In the case of T intersections designed as sump conditions, the Design Engineer shall demonstrate that the depth of storm water will not exceed six (6) inches at any point within the intersection. Inlets in such cases shall not be installed within the curb radius of the intersection.

1.2.6 Drainage System

A. Construction plans for proposed reinforced concrete box culverts, bridges and related structures may be adaptations of the current Texas Department of Transportation (TxDOT) Standards.

B. For bridges and culverts in residential streets, runoff from the 100-yr frequency flow shall not produce a headwater elevation at the roadway greater than either six (6) IN above the roadway crown elevation or any top of upstream curb elevation, whichever is lower.

C. For bridges and culverts in streets other than a residential street, runoff from the 100-yr frequency storm shall not produce a headwater elevation at the roadway greater than three (3) IN above the roadway crown elevation or three (3) IN above any top of upstream curb elevation, whichever is lower.

D. All drainage facilities (including but not limited to headwalls, open channels, storm sewers, area inlets, and detention, retention and water quality controls and their appurtenances) shall comply with the following requirements, unless otherwise noted in this section.

1. Storm sewer inlets and gutter transitions shall be designed to avoid future driveways and to avoid conflicts with standard water and wastewater service locations. No utilities shall be allowed to cross under or through a storm sewer inlet or culvert.

2. Drainage channels and detention ponds that are to be maintained by the public (City) shall be contained within drainage easements. Adequate room for access shall be provided for drainage channels and detention ponds. Ramps no steeper than five (5) FT horizontal to one (1) FT vertical shall be provided at appropriate locations to allow access to drainage channels and detention ponds. The minimum bottom width for any channel with vegetative side slopes shall be four (4) FT.

3. Detention ponds shall be designed with adequate area around the perimeter for access and maintenance. The said area shall be a minimum of seven (7) FT wide for ponds with depths of five (5) FT or less (back slopes included) and a minimum of fifteen (15) FT wide for ponds over five (5) FT deep or with back slopes in excess of five (5) FT high. The said area shall not slope more than five (5) percent.

4. Rip-rap for slope protection or velocity dissipation shall be formed concrete dissipaters. Mortared rock or stone shall not be allowed.

5. Storm drains between lots (crossing blocks) shall be avoided as much as possible. When unavoidable, such drains shall be underground storm drains laid along an alignment that retains the conduit within the dedicated drainage easement. Storm drains along rear of residential lots (through back yards) shall be avoided. Easements shall be a minimum of fifteen (15) FT in width.

6. All bends, wyes and pipe size changes in storm sewers shall be prefabricated or shall occur at manholes and junction boxes.

7. Bedding of storm sewer shall be to the top of pipe.

8. Storm drains shall be reinforced concrete pipe (RCP), ASTM C76, minimum class III minimum eighteen (18) IN diameter. The Design Engineer shall provide load analysis to the Planning and Development Department as appropriate to demonstrate that class of pipe used is sufficient for the loading conditions. Higher strength pipes shall be used where loadings warrant such. Storm drains shall have a minimum of two (2) FT of cover in unpaved areas and a minimum of one and five tenths (1.5) FT of cover from sub-grade in paved areas.

9. The use of High Density Polyethylene (HDPE) shall be allowed only if approved by the City Engineer. Its use shall be limited to unpaved areas outside of City streets. All cross street storm drainage conduit shall be reinforced concrete pipe (RCP). All outfall structures shall be constructed of reinforced concrete and the connection with the outfall structure shall be accomplished using RCP. A transition fitting from HDPE to RCP shall be made upstream of the outfall structure.

10. Junction boxes and manholes shall be reinforced concrete. Junction boxes in lieu of manholes shall be provided where any pipe opening exceeds thirty-seven (37) inches and where the distance from the outside surfaces of any two (2) pipes entering a manhole is less than one (1) foot, measured along the inside of the manhole.

11. Prefabricated wyes, mitered angle fittings and pipe size reducers shall be allowed in lieu of junction boxes and manholes.

12. Channels
    a. Concrete Channels
    Concrete channels shall be of sufficient cross section and slope as to fully contain design flows and facilitate self cleaning. Outfalls shall enter major collector drainage ways and major streams at grade or be designed and constructed with adequate concrete aprons, energy dissipaters or similar features to prevent erosion.
    b. Vegetated Channels
    Vegetated channels shall have sufficient grade but with velocities that will not be so great as to create erosion. Side slopes shall not be steeper than three (3) FT horizontal to one (1) FT vertical for channels four (4) FT or less in depth and no steeper than four (4) FT horizontal to one (1) FT vertical in all other channels to allow for future growth and to promote slope stability. All slopes shall be hydro-mulched, sodded or seeded with approved grass, grass mixtures or ground cover suitable to the area and season in which they are applied. Seeded side slopes and bottoms shall be lined with erosion protection matting. All earthen channels must have vegetation established prior to acceptance by the City of Gatesville.
    c. Major streams or natural channels shall not be modified without consent of applicable state and federal agencies and authorization from the City Engineer.

13. Discharge from storm sewer outfalls shall not cause channel, bluff, or stream bank erosion. If the storm drain discharges to an open drainage facility (as determined by the City Engineer), the applicant must show acceptable non-erosive conveyance to that drainage facility, appropriate energy dissipation at the outfall and a stable headwall. No outfalls shall be allowed to discharge on the slope of the receiving channel.
14. If the development is located such that there is considerable drainage from potentially developable upstream areas, the developer may request participation by the City for the cost of over sizing of elements of the overall drainage system. The City shall consider these requests on a case by case basis.

1.2.7 Computations

A. Computations to support all drainage designs shall be submitted to the City Engineer for review. The computations shall be in such form as to allow for timely and consistent review and also to be made a part of the permanent city record for future reference. Computation shall include the impact of the proposed development to the downstream properties adjacent to the drainage resulting from the 100-yr event. All computations submitted shall be certified by a Professional Engineer licensed in the State of Texas. The Design Engineer shall provide the report to the City in both hard copy and a scanned electronic pdf file with the proper seal, signature and date.

B. **Determination of Runoff**
Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method shall be an acceptable means of computing runoff for drainage areas of 200 ACRES or less when designing streets, storm drainage systems, channels and culverts. When the drainage area exceeds 200 ACRES in size, the National Resource Conservation Service (NRCS) (formerly the Soil Conservation Service) hydrologic methods (available in TR-20, TR-55 or HEC) should be used.

C. **Detention Pond Storage Determination**
A flow routing analysis using detailed hydrographs must be applied for all detention pond designs. The NRCS hydrologic methods (available in TR-20, TR-55, HEC-1, HEC RAS and the Hydrologic Engineering Center (HEC)) hydrologic methods may be used for areas of 200 ACRES or more. Use of the Modified Rational Method is limited to drainage areas less than 200 ACRES.

1.2.8 Stormwater Detention

Pre-developed peak flows generated from the 25-yr frequency storm shall not be increased. The peak flows from the 25-yr storm shall be detained in onsite stormwater detention basins with release rates equal to, or less than the flows generated from the site for the 25-yr storm event when the site was in its existing (natural) state. Detention ponds must also be designed such that the 100-yr storm will not overtop the structure. The Engineer shall design an emergency spillway system that will safely discharge the 100-yr storm without damage to the downstream property.

The City Engineer shall have the authority to waive the requirement for onsite detention, provided that at least one (1) of the following conditions are met:

1. The development is eligible to financially participate in an approved Regional Stormwater Management Program (Facility). Under this provision, the applicant shall demonstrate that the peak, post-developed runoff generated from the 100-yr storm can be conveyed downstream to the Regional Facility and not impact adversely any downstream properties. An adverse impact shall be:

   a. any impact which causes an inundation, or an increased inundation, of any building structure, roadway, or improvement.
   b. downstream erosion and/or sedimentation, or an increase in erosion and/or sedimentation.

2. The development is adjacent to a defined water course that has sufficient capacity to convey the site's post-developed peak discharge from the 100-yr storm event without creating an adverse impact on any other properties. The discharge in the water course shall be determined by using the 100-yr storm event with the post-developed site and the remainder of the watershed in an ultimate build-out state.
3. The development is located such that onsite detention may worsen downstream conditions of the watershed. In such cases, the Design Engineer shall demonstrate that conveyance or a combination of detention & conveyance will provide a safer downstream condition.

1.2.9 Flood Plain Management

A. City of Gatesville
In all cases where floodplain delineation is required, its determination shall be based on the projected ultimate development of all properties contributing to the point of consideration. It is the responsibility of the Engineer to determine the ultimate developed drainage condition is based on the most accurate information available.

For the purposes of this policy, any concentrated flow within a watershed that has a drainage area of three hundred twenty (320) ACRES or greater, unless previously defined by FEMA, shall be delineated as a floodplain.

All existing floodplains created by the base flood as computed with current, existing conditions, shall be deemed the Floodway (regulatory floodway) and shall be wholly contained within dedicated rights-of-way or easements. Encroachments are prohibited, including fill, new construction, substantial improvements and other development unless certification by a Professional Engineer is provided, demonstrating that encroachments shall not result in any increase in flood levels within the community during the occurrence of the base flood discharge.

All new construction and substantial improvements of buildings (structures) shall have the lowest floor (including basement) one foot above the base flood, based

upon the projected, ultimate development of all properties (without stormwater detention) contributing to the point of consideration.

All floodplains shall be computed utilizing the computer software and methodologies outlined in the Drainage Criteria Manual.

If land development activities are proposed which will result in flood hazard boundary delineations different from those depicted on the current Flood Insurance Rate Map (FIRM) issued by the Federal Emergency Management Agency (FEMA), the applicant for a development permit shall obtain a Conditional/Letter of Map Revision (CLOMR/LOMR) from FEMA.

All floodplain delineations for FIRM revisions shall be based upon field-surveyed cross-sections performed in accordance with the guidelines outlined in this Manual.

B. Federal Emergency Management Agency

1. The Federal Emergency Management Agency (FEMA) maintains Flood Insurance Rate Maps (FIRMs) that depict floodplain and floodway boundaries. The floodplain and floodway boundaries depicted on FIRMs are based on existing conditions of development in the contributing area.
2. FEMA reviews and approves or denies all revisions or amendments to FIRMs. FEMA revises or amends FIRMs by approval of a Letter of Map Amendment (LOMA) or Letter of Map Revision (LOMR). FEMA establishes the process and fees necessary for review of an application for LOMA or LOMR.
3. FEMA reviews the impact of proposed site developments and offers or denies conditional assurance that a FIRM may be changed by the proposed development. FEMA offers this assurance by a Conditional Letter of Map Amendment (CLOMA) or Conditional Letter of Map Revision (CLOMR). The CLOMA or CLOMR is a conditional statement that the FIRM may be changed if:

    a. the development is constructed as proposed in the CLOMA/CLOMR application, and if
    b. a complete LOMA/LOMR is submitted after construction of the proposed development.

C. Coordination of City of Gatesville and FEMA Floodplain Delineations

1. If the floodplain depicted on the FIRM is required to be changed, due to updated analysis of the floodplain under existing conditions, then the following requirements are applicable:

    a. Prior to recordation of a final plat, the applicant must provide to the City evidence of receipt by FEMA of an application for a LOMR.
    b. Prior to final acceptance of the construction of the subdivision or issuance of building permits, the applicant must provide to the City evidence of final acceptance by FEMA of the LOMR submitted under (a) above.

2. If the floodplain depicted on the FIRM is required to be changed, due to land development activities that alter existing conditions, then the following requirements are applicable:

   a. Prior to approval of final plat, the applicant must provide to the City evidence of receipt by FEMA of an application for a CLOMR.
   b. Prior to recordation of a final plat, the applicant must provide to the City evidence of approval of the CLOMR submitted under (a) above.
   c. If the final plat is approved before it is determined that a CLOMR is necessary or desired, then prior to release of subdivision construction plans, the applicant must provide to the City a letter of acknowledgement by FEMA of receipt of a complete application for a CLOMR.
   d. Prior to final acceptance of the construction of the subdivision or issuance of building permits, the applicant must provide to the City evidence of final acceptance by FEMA of the CLOMR submitted under (c) above, and a letter of acknowledgement by FEMA of a complete application for a LOMR.

3. The applicant shall bear the cost of Engineering services required to develop the application, respond to review comments, and obtain final approval of LOMRs and CLOMRs. The applicant shall bear the cost of any fees associated with review and disposition of LOMRs and CLOMRs that are established by FEMA.

1.2.10 Lot Grading

A. All retail, commercial and industrial site developments must provide a site grading and drainage plan that includes drainage computations, detention of runoff (if required) and a detailed site grading plan that does not adversely affect adjacent lots, property or downstream property.

B. Finished floor elevations shall be shown on all lots on the subdivision plat and construction plans. Finished floor elevations shall be a minimum of one (1) FT above the average top of curb elevation fronting the lot. The grading plan shall include arrows indicating the direction of runoff for each lot. Where practical, all lots shall be graded from rear to front at which point the drainage shall be intercepted by the street. If the minimum one-foot requirement cannot be met due to land slope, topography or existing trees, alternate grading plans may be utilized. In these instances, it shall be demonstrated to the satisfaction of the City Engineer that grading from front to rear would be more reasonably adaptable to the existing topography. All lots that fall into this second category shall be identified on the Final Plat by a listing table.

C. Finished floor elevations shall be shown for all lots adjacent to or encroaching upon the FEMA designated 100-yr flood plain. Finished floor elevations shall be a minimum of two (2) FT above the base flood elevations.

D. The applicant for a building permit for a residential lot that is included in the listing table described in (B) above and is graded from front to rear shall prepare a detailed site grading plan that includes elevations for all corners of the subject lot, the finished floor slab elevation, final contours, swales, and any modifications to side yard or rear yard fencing to facilitate removal of runoff from the subject lot. The site grading plan must be sealed, signed and dated by a professional engineer licensed in the State of Texas.

1.2.11 Erosion Control

Rock berms, silt fences, sedimentation basins, stabilized construction entrances/exits and similar recognized techniques shall be employed during and after construction to prevent point source sedimentation loading of downstream facilities. Such installations shall comply with current TCEQ requirements. Additional measures may be required during and after construction if during subsequent runoff events erosion or sediment damage is documented as a violation of TCEQ regulations or City Ordinance by City Staff.

1.3   DEFINITIONS

All terms and abbreviations used in the text are presented in the Glossary of this Manual.

SECTION 2 - DETERMINATION OF STORM RUNOFF

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SECTION 2 - DETERMINATION OF STORM RUNOFF

2.1.0 GENERAL

If continuous records of the amounts of runoff from urban areas were as readily available as records of precipitation, they would provide the best source of data on which to base the design of storm drainage and flood protection systems. Unfortunately, such records are available in very few areas in sufficient quantity to permit an accurate prediction of the stormwater runoff. The accepted practice, therefore, is to relate runoff to rainfall, thereby providing a means for predicting the amount of runoff to be expected from urban watersheds at given recurrence intervals.

Numerous methods of rainfall runoff computations are available on which the design of storm drainage systems may be based. The method chosen is dependent upon the Engineer's technical familiarity and the size of the area to be analyzed. Within the method chosen the Engineer will be responsible for making assumptions as to the development characteristics of the study area.

2.2.0 EFFECTS OF URBANIZATION

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall. The hydraulic efficiency of a drainage area is generally improved by urbanization which in effect reduces the storage capacity of a watershed. This reduction of a watershed's storage capacity is a direct result of the elimination of porous surfaces, small ponds, and holding areas. This comes about by the grading and paving of building sites, streets, drives, parking lots, and sidewalks and by construction of buildings and other facilities characteristic of urban development. The result of the improved hydraulic efficiency is illustrated graphically in Figure 2-1 in Appendix B of this Manual, which is a plot of the runoff rate versus time for the same storm with two different stages of watershed development.

2.2.1 Design Assumptions for Stormflow Analysis

    A. When analyzing an area for channel design purposes, urbanization of the full watershed without detention ponds shall be assumed (except as noted in (E). below). Zoning maps, future land use maps, and master plans should be used as aids in establishing the anticipated surface character of the ultimate development. The selection of design runoff coefficients and/or percent impervious cover factors are explained in the following discussions of runoff calculation.

    B. An exception to (A) above may be granted if the channel is immediately downstream of a regional detention pond and written approval is obtained from the City Engineer.

    C. In designing a storm sewer system within a residential subdivision, full development of adjoining and interior tracts without detention must be assumed.

    D. In designing a storm sewer system within a commercial or multifamily subdivision, 25-yr stormflows can, at the Engineer's discretion, reflect the flow reduction anticipated by future detention ponds. This applies exclusively to the flows generated by those properties contained within the subdivision. Provisions for conveyance of the 100-yr undetained flows within the right-of-way or drainage easements still apply (See Section 1.2.2B.).

    E. In the event the Engineer desires to incorporate the flow reduction benefits of existing upstream detention ponds, the following field investigations and hydrologic analysis will

be required: (Please note, that under no circumstances will the previously approved construction plans of the upstream ponds suffice as an adequate analysis. While the responsibility of the individual site or subdivision plans rests with the Engineer to record, any subsequent engineering analysis must assure that all the incorporated ponds work collectively.)

1. A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original Engineer's design must be accounted for. If a dual use for the detention pond exists, (e.g., storage of equipment) then this too should be accounted for.
2. A comprehensive hydrologic analysis which simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis but rather a network of hydrographs which considers incremental timing of discharge and potential coincidence of outlet peaks.

2.3.0 METHOD OF ANALYSIS

Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method and the Variable Rainfall Intensity Method are accepted as adequate for drainage areas totaling 200 ACRES or less. For larger drainage systems, the Soil Conservation Service hydrologic methods (available in TR-20, TR-55 and HEC-1 or the Tabular/Graphical methods) should be used. The method of analysis must remain consistent when drainage areas are combined and the method which applies to the largest combined drainage area should be used. Table 2-1 is to be used as a guide in determining some of the applicable methods for calculating storm runoff. The Engineer can use other methods but must have their acceptability approved by the City Engineer.

Table 2-1	
Storm Runoff Calculation Methods	
Contributing Area	**Runoff Methods**
Less than 200 ACRES	Rational or VRIM[1] SCS Tabular/Graphical[2]
200 ACRES-400 ACRES	SCS Tabular/Graphical[3] TR-20, HEC-1 or HEC-HMS
Greater than 400 ACRES	SCS TR-20, TR-55, HEC-1 or HEC-HMS
1. VRIM, Variable Rainfall Intensity Method in Section 2.4.5 2. SCS, Tabular/Graphical and TR-20, TR-55, Methods in Section 2.6.4 3. It is recommended that the hand calculated SCS Tabular Method not be used for areas greater than four hundred (400) ACRES due to the rigorous nature of the calculations and likelihood of error	

2.4.0 RATIONAL METHOD

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the Equation 2-1:

$$Q_p = CiA \qquad\qquad (Eq.2-1)$$

where,

*Q_p = The peak runoff, CFS

C = Coefficient of runoff representing the ratio of peak runoff rate "Q_p" to average rainfall intensity rate "i" for a specified area "A".

i = Average intensity of rainfall for a period of time equal to the time of concentration (t_c) for the drainage area to the point under consideration, IN/HR.

A = Area contributing runoff to the point of design, ACRE.

* = Actually, Q_p is in units of IN/HR/ACRE. Since this rate of IN/HR/ACRE differs from CFS by less than one (1) percent (1 IN/HR/ACRE = 1.008 CFS), the more common units of CFS are used.

The following basic assumptions are associated with the Rational Method:

A. The storm duration is equal to the time of concentration.
B. The computed peak rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.
C. The return period or frequency of the computed peak flow is the same as that for the design storm.
D. The necessary basin characteristics can be identified and the runoff coefficient does not vary during a storm.
E. Rainfall intensity is constant during the storm duration and spatially uniform for the area under analysis.

2.4.1 Runoff Coefficient (C)

The proportion of the total rainfall that will reach the drainage system depends on the imperviousness of the surface and the slope and ponding characteristics of the area. Impervious surfaces, such as asphalt pavements and roofs of buildings, will be subject to approximately one hundred (100) percent runoff (regardless of the slope). Onsite inspections and aerial photographs may prove valuable in estimating the nature of the surfaces within the drainage area.

The runoff coefficient "C" in the Rational Formula is also dependent on the characteristics of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases as the rainfall continues for an extended period of time. The soil infiltration rate is influenced by the presence of soil moisture (antecedent precipitation), the rainfall intensity, the proximity of the ground water table, the degree of soil compaction, the porosity of the subsoil, and ground slopes.

It should be noted that the runoff coefficient "C" is the variable of the Rational Method which is least susceptible to precise determination. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing and interception, all of which affect the time distribution and peak rate of runoff.

Table 2-2 presents recommended ranges for "C" values based on specific land use types.

2.4.2 Time of Concentration

The time of concentration is the time associated with the travel of runoff from an outer point which best represents the shape of the contributing areas. Runoff from a drainage area usually reaches a peak at the time when the entire area is contributing, in which case the time of concentration is the time for a drop of water to flow from the most remote point in the watershed to the point of interest. Runoff may reach a peak prior to the time the entire drainage area is contributing. Sound engineering judgment should be used to determine the time of concentration. The time of concentration to any point in a storm drainage system is a combination of the sheet flow (overland), the shallow concentrated flow and the channel flow, which includes storm sewers. The minimum time of concentration for any area shall be 10 MIN.

A. **Sheet Flow.** Sheet flow is shallow flow overland surfaces which usually occur in the headwaters of streams. The Engineer should realize that sheet flow occurs for only very short distances in urbanized conditions. Urbanized areas are assumed to have sheet flow of three hundred (300) FT or less. The following Equation 2-2 has been developed for sheet flow of less than three hundred (300) FT.

$$t_c = Ln/(42s^{0.5}) \hspace{3cm} \text{(Eq.2-2)}$$

where,

t_c = Time of concentration, MIN
L = Length of the reach, FT.
n = Manning's Roughness Coefficient (see Table 2-3)
s = Slope of the ground, FT/FT

B. **Shallow Concentrated Flow.** After a maximum of three hundred (300) FT sheet flow becomes shallow concentrated flow. The time of concentration for shallow concentrated flows can be computed from equation 2-3 which is as follows:

$$t_c = Ln/(60s^{0.5}) \hspace{3cm} \text{(Eq. 2-3)}$$

where,

t_c = Time of concentration, MIN
L = Length of the reach, FT.
n = Manning's Roughness Coefficient (see Table 2-3)
s = Slope of the ground, FT/FT

C. **Channel or Storm Sewer Flow.** The velocity in an open channel or a storm sewer not flowing full can be determined by using Manning's Equation. Channel velocities can also be determined by using backwater profiles. Usually, average flow velocity is determined assuming a bank-full condition. The details of using Manning's Equation and selecting Manning's Roughness Coefficient, "n" values for channels can be obtained from Section 6 of this Manual.

For full flow storm sewer conditions (pressure flow) the following equation should be applied:

$$V = Q/A \hspace{3cm} \text{(Eq. 2-4)}$$

Where,

V = Average velocity, FPS
Q = Design discharge, CFS
A = Cross-sectional area, FT^2

TABLE 2-2
RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS
METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Character of Surface	Runoff Coefficient (C)						
	Return Period						
	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
DEVELOPED							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass Areas (Lawns, Parks, etc.)							
Poor Condition*							
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair Condition**							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition**	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Flat, 0-2%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Average, 2-7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Steep, over 7%							
UNDEVELOPED							
Cultivated							
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61

TABLE 2-2 (Continued)
RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS
Runoff Coefficient (C)

Character of Surface	Return Period						
	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
UNDEVELOPED							
Pasture/Range							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0-7%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

* Grass cover less than fifty (5) percent of the area.

** Grass cover on fifty (50) to seventy-five (75) percent of the area.

*** Grass cover larger than seventy-five (75) percent of the area.

Source: 1. Rossmiller, R.L. "The Rational Formula Revisited."
2. City of Austin, Watershed Engineering Division

TABLE 2-3
MANNING'S ROUGHNESS COEFFICIENT "n" FOR OVERLAND FLOW AND SHALLOW CONCENTRATED FLOW

Manning's Roughness Coefficient "n"	Condition
0.016	Concrete (rough or smoothed finish)
0.02	Asphalt
0.1	0-50% vegetated ground cover, remaining bare soil or rock outcrops, minimum brush or tree cover
0.2	50-90% vegetated ground cover, remaining bare soil or rock outcrops, minimum- medium brush or tree cover
0.3	100% vegetated ground cover, medium- dense grasses (lawns, grassy fields etc.) medium brush or tree cover
0.6	100% vegetated ground cover with areas of heavy vegetation (parks, green- belts, riparian areas etc.) dense under- growth

Source: City of Austin, Watershed Engineering Division

2.4.3 Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in IN/HR, and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The design frequency of occurrence is a statistical variable which is established by design standards or chosen by the Engineer as a design parameter.

The selection of the frequency criteria is necessary before applying any hydrologic method. Storm drainage improvements in Gatesville must be designed to intercept and carry the runoff from a 25-yr frequency storm, with an auxiliary or overflow system capable of carrying a 100-yr frequency storm.

The rainfall intensity used in the Rational Method is read from the intensity-duration-frequency curves based on the selected design frequency and design duration.

The Gatesville intensity-duration-frequency curves are shown in Figure 2-2 in Appendix B of this Manual and as follows:

Intensity Duration Frequency Curve for Gatesville, TX
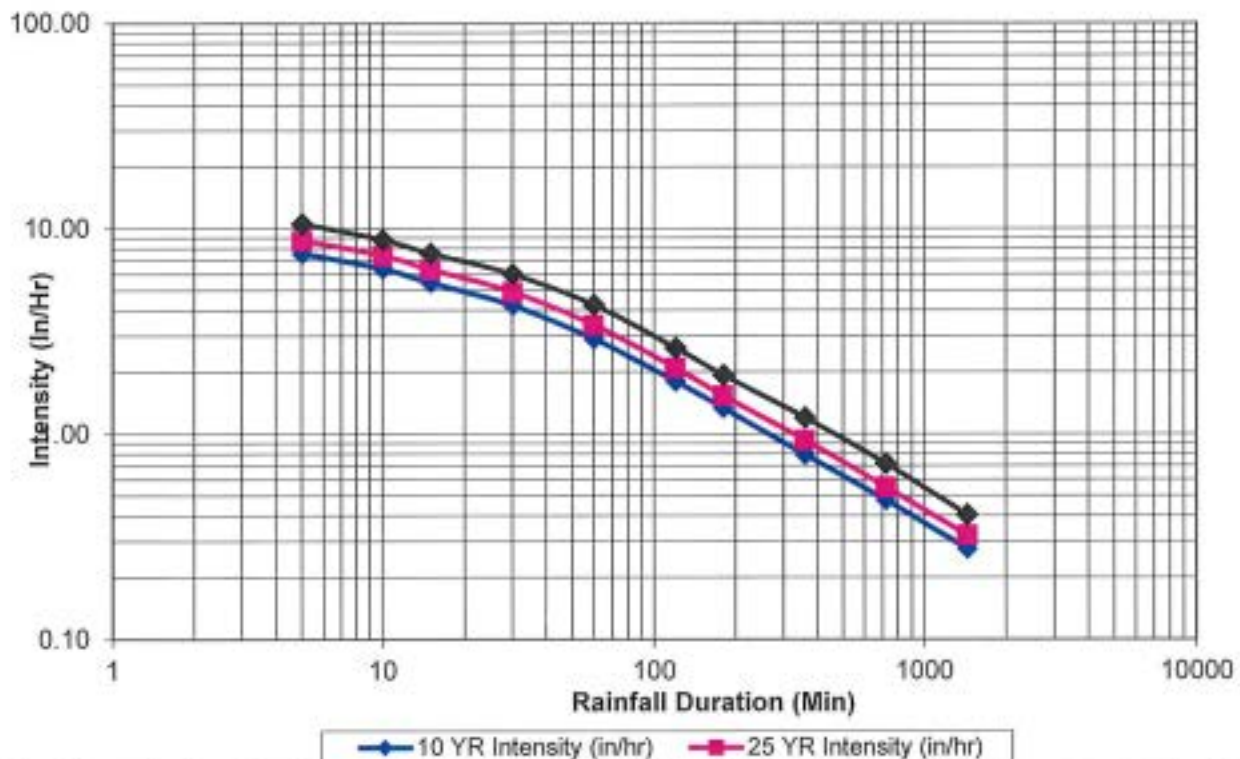
The intensity-duration-frequency curves and the intensity-duration equations are applicable for all design frequencies shown and for storm durations from 5 MIN to 3 HRS. They are required for use in determining peak flows using the Rational Method as well as other appropriate methods.

2.4.4 Drainage Area (A)

The size (ACRES) of the watershed needs to be determined for application of the Rational Method. The area may be determined through the use of maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. The drainage divide lines are determined by street layout, lot grading, structure

configuration and orientation, and many other features that are created by the urbanization process.

Example 2-1

Rational Method

An urbanized watershed is shown on the following figure. Three types of flow conditions exist between time of concentration and travel time in each reach is as follows:

Reach	Description of Flow	Slope (%)	Length (FT.)	Drainage Area (ACRE)	"n" Value	For reaches A-B and B-C, the time of concentration
A to B	Sheet flow (grass lawn)	4.5	300	3	0.300	
B to C	Shallow concentrated flow (gutter)	2.0	840	20	0.016	
C to D	Storm drain with inlets n=0.015 D=3	1.5	1,200	30		

can be calculated from Equations 2-2 and 2-3. The calculations are as follows:

$$t_{c\ (A-B)} = 300(0.3)/42(s)^{0.5}$$
$$= 2.14/(0.045)^{0.5}$$
$$= 10.1 \text{ MIN}$$

$$t_{c\ (B-C)} = 840(0.016)/60(s)^{0.5}$$
$$= 0.22/(0.02)^{0.5}$$
$$= 1.6 \text{ MIN}$$

The flow velocity in reach C-D needs to be calculated from Manning's Equation (see Eq. 5-2), using the assumption of full pipe flow, as follows:

$$V_{(C-D)} = (1.49/n)\ R^{0.67}s^{0.5}$$
$$= (1.49/n)(D/4)^{0.67}s^{0.5}$$
$$= (1.49/0.015)\ (3/4)^{0.67}(0.015)^{0.5}$$
$$= 10.0 \text{ FT/s}$$

The runoff coefficients (C) for the three (3) areas are given as follows for the 100-yr storm. The time of concentration (t_c) is calculated by dividing the length by the velocity.

Reach	Length (FT.)	Velocity (FPS)	t_c (MIN)	C	Area (ACRE)
A-B	300	—	10.1	0.41	3
B-C	840	—	1.6	0.85	20
C-D	1200	10.0	2.0	0.81	30
			13.7		53

The intensity (i) of the 100-yr storm (from Figure 2-2 in Appendix B of this Manual) for 13.7 MIN = 8.7 IN/HR.

The composite runoff coefficient (C) = ((0.41)(3) + (0.85)(20) + (0.81)(30))/53= 0.80

Thus the peak flow Q_p = CiA = (0.80)(8.7 IN/HR)(53 ACRE) = 368.9 CFS

2.4.5 Variable Rainfall Intensity Method

The Variable Rainfall Intensity Method is one of the methodologies which uses the peak flow (Q_p) calculated from the Rational Method to develop the hypothetical storm hydrographs. The detailed information on this method can be found in the Bibliograhpy (Item 2-5 of this Manual). The following example illustrates the application of the variable rainfall intensity method technique in constructing a 10-yr design storm hydrograph.

Example 2-2

Variable Rainfall Intensity Method

Given: A drainage area, when fully developed, will have the following characteristics:

> Drainage area = 100 ACRES, Runoff coefficient C = 0.45
> Design rainfall frequency: 10-yr
> Gatesville rainfall intensity-duration-frequency curves (Figure 2-2 in Appendix B of this Manual)
> Time of concentration = 40 MIN

Find: The 10-yr design storm and resulting flood hydrograph.

Solution: The solution is outlined in Table 2-6 which shows the development of the design 10-yr frequency storm and Table 2-7 which shows the computation of the design 10-yr flood hydrograph.

The computation procedures for Table 2-6 are explained as follows:

Column 1:	Duration (MIN) = Length of storm.
Column 2:	Rainfall Intensity read from Figure 2-2 in Appendix B of this manual corresponding to the duration time in Column 1.
Column 3:	Accumulated Depth (IN) = Total precipitation for storm of specified duration from Table 2-11.
Column 4:	Incremental Depth (IN) = Difference in total precipitation between specified duration and duration of 5 min less than specified duration (e.g., P35 MIN - P30 MIN).
Column 5:	Incremental Intensity (IN/HR) = Incremental Depth (IN) x (60 MIN/HR)/(5 MIN).

<div align="center">

Table 2-6
Development Of A 10-yr Frequency Storm

</div>

Duration (MIN) (1)	Intensity (IN/HR) (2)	Accumulated Depth (IN) (3)	Incremental Depth (IN) (4)	Incremental Intensity (IN/HR) (5)
5	8.64	0.034	0.034	0.41
10			0.036	0.43
15	6.16	0.108	0.038	0.46
20			0.040	0.48
25	5.00	0.19	0.040	0.48
30			0.050	0.60
35	4.30	0.29	0.050	0.60
40			0.060	0.72
45	3.73	0.41	0.060	0.72
50			0.070	0.84
55	3.33	0.56	0.080	0.96
60			0.090	1.08
65	3.00	0.76	0.110	1.32
70			0.130	1.56
75	2.74	1.07	0.180	2.16
80			0.240	2.88
85	2.50	1.67	0.360	4.32
90			0.720	8.64
95	2.32	2.89	0.500	6.00
100			0.290	3.48
105	2.17	3.38	0.200	2.40
110			0.150	1.80
115	2.05	3.65	0.120	1.44
120			0.100	1.20
125	1.94	3.83	0.080	0.96
130			0.080	0.96
135	1.85	3.98	0.070	0.84
140			0.060	0.72
145	1.77	4.09	0.050	0.60
150			0.050	0.60
155	1.69	4.19	0.050	0.60
160			0.040	0.48
165	1.62	4.27	0.040	0.48
170			0.040	0.48
175	1.56	4.34	0.030	0.36
180			0.030	0.36
185	1.50	4.38		0.36

Table 2-7 illustrates the computed 10-yr flood hydrograph for the drainage area described in Table 2-6. Referring to Table 2-7, the columns are identified and computed as follows:

Column 1: Time (MIN) = Time from the beginning of the storm.

Column 2: i (IN/HR) = Incremental intensities (from Table 2-6).

Column 3: Sum (i) = Summation of all incremental intensities to the specified time.

Column 4: "Sum" (i lagged) = Column 3 displaced a total time equal to the time of concentration for the area producing this hydrograph.

Column 5: (3) - (4) = Column 3 - Column 4.

Column 6: q_{tc}= Column 5 divided by the number of time increments in the time of concentration for the area producing this hydrograph. This column expresses the average intensity over a period of time equal to the time of concentration for the area producing this hydrograph, as measured at the specified chronological time.

Column 7: Q (CFS) = Column 6 x "C" x A (area producing this hydrograph). This column is for the rising limb calculation.

Column 8: Time Folded = Revised times and flows for falling limb of hydrograph; falling limb is mirror image of rising limb, but expanded to twice the length. Intermediate values can be linearly interpolated from neighboring values, since 5 MIN increments doubled to 10 MIN increments leave out intervening values.

The computations were stopped in Column 7 when the rising limb of the hydrograph reached its peak value. At this point, the time scale can be folded as shown in Column 8. Doubling the time increments for the falling limb serves to double the volume that would have been under that portion of the runoff hydrograph. The volume under the entire discharge hydrograph will be three (3) times that under the rising limb.

With this assumption, the volume of runoff expressed as a percentage from an area with a runoff coefficient of 0.45 becomes approximately sixty-seven and five tenths (67.5) percent rather than forty-five (45) percent of the rainfall. In this procedure the C value from the Rational Method formula represents the ratio of the peak runoff to the average rainfall intensity rate for a period equal to the time of concentration and not a simple runoff to rainfall ratio.

Table 2-7
Runoff Computations From A 100 ACRE
Area With A Time Of Concentration Of
40 MIN And C = 0.45

Time (MIN) (1)	I_{10} (IN/HR) (2)	Sum I_{10} (3)	Sum I_{10} (Lagged 40 MIN)	Time (3) - (4) (5)	Q40 (IN/HR) (6)	Q (CFS) (7)	Folded (8)
0							330
5	0.41	0.41		0.41	0.05	2.3	320
10	0.43	0.84		0.84	0.10	4.5	310
15	0.46	1.30		1.30	0.16	7.2	300
20	0.48	1.78		1.78	0.22	9.9	290
25	0.48	2.26		2.26	0.28	12.6	280
30	0.6	2.86		2.86	0.36	16.2	270
35	0.6	3.46		3.46	0.43	19.3	260
40	0.72	4.18		4.18	0.52	23.4	250
45	0.72	4.90	0.41	4.50	0.56	25.2	240
50	0.84	5.70	0.84	4.90	0.61	27.4	230
55	0.96	6.70	1.30	5.40	0.67	30.1	220
60	1.08	7.80	1.78	6.00	0.75	33.7	210
65	1.32	9.10	2.26	6.80	0.85	38.2	200
70	1.56	10.70	2.86	7.80	0.97	43.6	190
75	2.16	12.80	3.46	9.30	1.16	52.2	180
80	2.88	15.70	4.18	11.50	1.44	64.8	170
85	4.32	20.00	4.90	15.10	1.89	85.1	160
90	8.64	28.70	5.70	23.00	2.87	129.1	150
95	6.00	34.70	6.70	28.00	3.50	157.5	140
100	3.48	38.10	7.80	30.30	3.80	171.0	130
105	2.4	40.50	9.10	31.40	3.92	176.4	120
110	1.80	42.30	10.7	31.60	3.95	177.7	(peak)
115	1.44	43.80	12.8	31.00	3.87	174.1	

2.5.0 SOIL CONSERVATION SERVICE METHODS

The Soil Conservation Service (SCS) hydrologic methods have been widely used by engineers and hydrologists for analyses of small urban watersheds. These methods resulted from extensive analytical work using a wide range of statistical data concerning storm patterns, rainfall-runoff characteristics and many hydrologic observations in the United States. The SCS utilizes a twenty-four (24) HR storm duration, which is considered to be acceptable for the Gatesville area. It should be noted that if the SCS storms are applied, the Type III distribution should be used.

The SCS methods can be applied to urban drainage areas of any size. A brief explanation of the runoff curve numbers, the tabular and graphical methods and the TR-20/TR-55 methods are introduced in this Section. The Supplemental Section 2.7.0 for the Soil Conservation Service hydrology includes the rainfall-runoff relationship and the dimensionless Unit Hydrograph. For detailed information, the user is referred to the following Soil Conservation Service publications. They are:

NEH-4:	"Hydrology," Section 4, National Engineering Handbook
TR-20:	Computer Program for Project Formulation, Hydrology
TR-55:	Urban Hydrology for Small Watersheds
TP-149:	A Method for Estimating Volume and Rate of Runoff in Small Watersheds

2.5.1 Left Blank Intentionally

2.5.2 Soil Conservation Service Runoff Curve Numbers

The Soil Conservation Service (SCS) has developed an index, the runoff curve number, to represent the combined hydrologic effect of soil type, land use, agricultural land treatment class, hydrologic condition, and antecedent soil moisture. These watershed factors have the most significant impact in estimating the volume of runoff, and can be assessed from soil surveys, site investigations and land use maps.

The curve number is an indication of the runoff producing potential of the drainage area for a given antecedent soil moisture condition, and it ranges in value from zero (0) to one hundred (100). The SCS runoff curve numbers are grouped into three (3) antecedent soil moisture conditions: Antecedent Moisture Condition I, Antecedent Moisture Condition II and Antecedent Moisture Condition III. Values of runoff curve numbers for all three (3) conditions may be computed following guidelines in "Hydrology, Section 4," National Engineering Handbook. Antecedent Moisture Condition I is the dry soil condition and Antecedent Moisture Condition III is the wet soil condition. Antecedent Moisture Condition II is normally considered to be the average condition.

However, studies of hydrologic data indicate that Antecedent Moisture Condition II is not the average throughout Texas. Instead, investigations have shown that the average condition ranges from Antecedent Moisture Condition I in west Texas to between Antecedent Moisture Condition II and Antecedent Moisture Condition III in east Texas. The values given in Table 2-10 are for an Antecedent Moisture Condition II. If it is desired to change to an Antecedent Moisture Condition I or III, the adjustments given in TR-20/TR-55 or "Hydrology, Section 4," National Engineering Handbook should be used.

The SCS has classified more than four thousand (4,000) soils into four (4) hydrologic groups, identified by the letters A, B, C, and D, to represent watershed characteristics.

> **Group A:** (Low runoff potential). Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well-drained to excessively drained sands or gravels.
>
> **Group B:** Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse texture.
>
> **Group C:** Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soil with moderately fine to fine texture.

Group D: (High runoff potential). Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface and shallow soils over nearly impervious material.

The list of most soils in the United States along with their hydrologic soil classification is given in the TR-20/TR-55 publication. The minimum infiltration rates for the four (4) soil groups are:

Group	Minimum Infiltration Rate (IN/HR)
A	0.30 - 0.45
B	0.15 - 0.30
C	0.05 - 0.15
D	0.00 - 0.05

Table 2-13 lists the curve numbers for the four (4) soil groups under various land uses, land treatment and hydrologic conditions. In order to determine the soil classifications in the Gatesville area, the SCS Soil Survey of Coryell County, Texas should be used.

Table 2-10 SCS Runoff Curve Numbers for Urban Areas and Agricultural Lands						
Cover Description		**Curve Numbers for Hydrologic Soil Group**				
Cover type and Hydrologic Condition	**Average % Impervious Area[1]**	**A**	**B**	**C**		**D**
Fully developed urban areas (vegetation established)						
Open space (lawns, parks, golf courses, cemeteries, etc.) Poor condition (grass cover 50%) Fair condition (grass cover 50% to 75%) Good condition (grass cover 75%)		68 49 39	79 69 61	86 79 74		89 84 80
Impervious areas: Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98		98

Table 2-10 (Continued) SCS Runoff Curve Numbers for Urban Areas and Agricultural Lands					
Cover Description		**Curve Numbers for Hydrologic Soil Group**			
Cover type and Hydrologic Condition	**Average % Impervious Area[1]**	**A**	**B**	**C**	**D**
Streets and roads: Paved; curbs and storms sewers (excluding right-of-way)		98	98	98	98
Paved open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts: Commercial and	85	89	92	94	95
business 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 areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Agricultural lands					
Grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80

Table 2-10 (Continued)
SCS Runoff Curve Numbers for Urban Areas and Agricultural Lands

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover type and Hydrologic Condition	Average % Impervious Area[1]	A	B	C	D
Meadow-continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods—grass combination (orchard or tree farm).[4]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[5]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots		59	74	82	86

[1] The average percent impervious area shown was used to develop the composite curve numbers. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a curve number of ninety-eight (98) and pervious areas are considered equivalent to open space in good hydrologic condition.

[2] Poor: less than fifty (50) percent ground cover or heavily grazed with no mulch.
  Fair: zero (0) to seventy-five (75) percent ground cover and not heavily grazed.
  Good: greater than seventy-five (75) percent ground cover and lightly or only occasionally grazed.

[3] Poor: less than fifty (50) percent ground cover.
  Fair: 50 to seventy-five (75) percent ground cover.
  Good: greater than seventy-five (75) percent ground cover.

[4] Curve numbers shown were computed for areas with fifty (50) percent woods and fifty (50) percent grass (pasture) cover. Other combinations of conditions may be computed from the curve numbers for woods and pasture.

[5] Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
  Fair: Woods are grazed but not burned, and some forest litter covers the soil.
  Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Source: Soil Conservation Service. TR-55: Urban Hydrology for Small Watersheds

2.5.3 Time of Concentration

The procedures for estimating time of concentration for the SCS method are described in the SCS's Technical Release 55 (TR-55). Three (3) types of flow (sheet flow, shallow concentrated flow and channel flow) are considered.

In hydrograph analysis, the time of concentration is the time from the end of excess rainfall to the point of inflection on the falling limb of the hydrograph. The time of concentration determines the shape of the runoff hydrograph. Times of concentration are required for the existing and developed conditions to adequately model the impact of the development on stormwater runoff. In general, times of concentration for the developed condition should be calculated based on conservative assumptions concerning the increased hydraulic efficiency expected with an ultimate developed condition. For instance, while sheet flow for existing conditions is typically limited to three hundred (300) FT, sheet flow for developed conditions should be limited to one hundred fifty (150) FT.

2.5.4 Peak Flow Calculation

The SCS has presented several methods for computing runoff hydrographs for drainage areas. The Tabular, Graphical and TR-20/TR-55 methods are considered acceptable for the Gatesville area. The parameters required to calculate the hydrograph are the rainfall distribution, runoff curve numbers, time of concentration and drainage area.

A. **Tabular Method.** The Tabular Method can be used to develop composite flood hydrographs at any point within a watershed by dividing the watershed into subareas. The method is useful for watersheds where runoff hydrographs are needed from nonhomogeneous areas, i.e., the watershed can be divided into homogeneous subareas. It is especially applicable for estimating the effects of land use change in a portion of the watershed. It should be noted that the tables in the TR-55 publication for the Tabular Method are based on the SCS twenty-four (24) HR rainfall distributions. The Engineer should apply those tables corresponding to a Type III rainfall distribution which is acceptable for the Gatesville area.

The basic requirement for use of this method is the tabular discharge values for the different types of storm distributions. The tabular discharge values in CSM/IN (cubic feet of discharge per second per square mile of watershed per inch of runoff) are given in TR-55 for a range of times of concentration from one tenth (0.1) to two (2) HRS and reach travel times of zero (0) to three (3) HRS. The discharge values were developed from the TR-20 program by computing hydrographs for a one (1) MI^2 drainage area at selected times of concentration and routing them through stream reaches with the range of travel times indicated.

The other input needed to develop the composite flood hydrograph includes the total runoff volume (Q_v) and the drainage area (A_m). The Equation 2-6 is for calculating the flow at any time:

$$Q = q_t A_m Q_v \qquad \text{(Eq. 2-6)}$$

where,

q = Hydrograph ordinate at hydrograph time t, CFS
q_t = Individual value read from the tabular discharge tables, CSM/IN
A_m = Drainage area of individual subwatershed, MI^2
Q_v = Total runoff volume, IN

The composite flood hydrograph is obtained by submission of the individual subarea hydrographs at each time step. For measuring runoff from a nonhomogeneous watershed, the subdivision of the watershed into relatively homogeneous subareas is required. For additional information regarding the Tabular Method the SCS publication TR-55 should be consulted.

B. **Graphical Method.** As in the Tabular Method the Graphical Method is based on hydrograph analyses using the TR-20 computer program. The Graphical Method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular or TR-20 methods. The TR-55 lists in detail the limitations of the Graphical Method and the engineer should be well aware of these before proceeding. The input requirements for the Graphical Method are as follows:

1. t_c, HR
2. Drainage Area, MI^2
3. Type III rainfall distribution
4. 24-HR rainfall, IN
5. CN

The peak discharge equation for the graphical method is:

$$q_p = q_u A_m Q \qquad\qquad\qquad \text{(Eq. 2-7)}$$

where,

*q_p = peak discharge, CFS
q_u = unit peak discharge, CSM/IN
A_m = drainage area, MI^2
Q = runoff, IN

*Note the original SCS equation also has an F_p factor for pond and swamp conditions. This has been omitted since it is not applicable to the Coryell County region.

For additional information regarding the Graphical Method the SCS publication TR-55 should be consulted.

C. **TR-20 Method.** The TR-20 method is a computer program which develops runoff hydrographs for a watershed. The input information includes drainage area, time of concentration, SCS curve number, a specific rainfall distribution and the antecedent soil moisture condition.

The TR-20 program was developed by the SCS to assist in the hydrologic evaluation of flood events for use in analysis of water resource projects. Besides developing the runoff hydrograph from any synthetic or natural storm rainfall, the program provides the

capability to route, add, store, divert or divide hydrographs to convey floodwater from the headwaters to the watershed outlets.

The program uses the procedures described in the SCS's National Engineering Handbook in "Hydrology, Section 4" except for the reach routing procedures. The modified Attenuation-Kinematic routing method is used for reach routing. Uniform rainfall depth and distribution over time are assumed over a subarea, groups of subareas or the whole watershed.

2.6.0 SUPPLEMENTAL SECTION: SOIL CONSERVATION SERVICE HYDROLOGY

2.6.1 Rainfall-Runoff Relationship

The SCS has developed a rainfall-runoff relationship to calculate the total runoff volume for a single storm. Based on the relationship between rainfall, runoff and retention (the rain not converted to runoff), a mathematic equation for a storm without any initial abstraction can be expressed as:

$$F/S = Q/P \qquad \text{(Eq. S-1)}$$

where,

F = Actual retention after runoff begins, IN
S = Potential maximum retention after runoff begins ($S \geq F$), IN
Q = Actual runoff, IN
P = Rainfall ($P \geq Q$), IN

The retention, S, is a constant for a particular storm because it is the maximum that can occur under the existing conditions if the storm continues without limit. The retention F varies because it is the difference between P and Q at any point on the mass curve, or:

$$F = P - Q \qquad \text{(Eq. S-2)}$$

The actual runoff (Q) can be solved as:

$$Q = P^2/(P+S) \qquad \text{(Eq. S-3)}$$

which is a rainfall-runoff relationship in which the initial abstraction is zero.

If an initial abstraction (I_a) greater than zero is considered, the amount available for runoff is $P - I_a$ instead of P. By substituting ($P - I_a$) for P in Equation S-1, the following equation results. The new mathematic expression becomes:

$$F/S = Q/(P - I_a) \qquad \text{(Eq. S-4)}$$

where,

F = Actual retention after runoff begins, IN
S = Potential maximum retention after runoff begins, IN
Q = Actual runoff, IN
P = Rainfall, IN
I_a = Initial abstraction, IN

When F≤S, and Q≤(P - I$_a$). The total retention for a storm consists of I$_a$ and F. The total potential maximum retention (as P gets very large) consists of I$_a$ and S.

The actual runoff is:

$$Q = ((P-I_a)+S) \qquad \text{(Eq. S-5)}$$

The initial abstraction (I$_a$) is a function of land use, treatment and condition, interception, infiltration, depression storage, and antecedent soil moisture. An empirical analysis performed by the SCS found that the initial abstraction is estimated as:

$$I_a = 0.2S \qquad \text{(Eq. S-6)}$$

Thus, the runoff volume (Q) can be obtained from the volume of precipitation (P) and potential maximum retention (S) as follows:

$$Q = (P - 0.2\,S)^2/(P + 0.8S) \qquad \text{(Eq. S-7)}$$

Empirical studies indicate that S is a function of the curve number as follows:

$$S = (1000/CN)-10 \qquad \text{(Eq. S-8)}$$

Therefore, the runoff volume can be determined as a function of precipitation volume and curve number.

2.6.2 Soil Conservation Service Dimensionless Unit Hydrograph

To estimate the peak discharge and establish a runoff hydrograph in the SCS methods, the concept of a dimensionless unit hydrograph is applied. The SCS dimensionless unit hydrograph was derived from analysis of a large number of unit hydrographs developed using gage data from watersheds of a wide range in size and geographical location. The dimensionless unit hydrograph has ordinate values expressed in a dimensionless ratio q/q_p and abscissa values of t/T_p, where q_p is the peak discharge at time T_p and q is the discharge at time t. Figure 2-3 in Appendix B of this Manual shows the shape of the dimensionless unit hydrograph. At the same time, the mass curve is also illustrated in Figure 2-3 in Appendix B of this manual with coordinates of Q_a/Q vs t/T_p, in which Q_a is the accumulated volume at time t, and Q is the total volume. Table 2-11 lists dimensionless discharge ratios and mass curve ratios for dimensionless time ratios for use in calculating unit hydrographs and mass curves.

The curvilinear unit hydrograph can be approximated by an equivalent triangular unit hydrograph, as shown by dotted lines in Figure 2-3 in Appendix B of this Manual. The area under the rising limb (before time T_p) of the two (2) unit hydrographs are the same. The time base of the dimensionless unit hydrograph is five (5) times the time-to-peak (T_p), while the time base of the triangular unit hydrograph is only two and sixty-seven hundredths (2.67) times the time-to-peak (T_p). The transformation of curvilinear unit hydrograph to triangular unit hydrograph provides a solution for the peak flow.

Derivation of Peak Flow. The area under the triangular unit hydrograph on Figure 2-3 in Appendix B of this Manual equals the volume of direct runoff Q, which can be calculated by:

$$Q = q_p(T_p + T_r)/2 \qquad \text{(Eq.S-9)}$$

where,

$$Q \quad = \text{Direct runoff, IN}$$
$$T_p \quad = \text{Time to peak, HR}$$
$$T_r \quad = \text{Recession time, HR}$$
$$q_p \quad = \text{Peak discharge, IN/HR}$$

The runoff Q derived from this equation is the same as estimated by Equation S-7. By Equation S-9, the peak discharge q_p can be solved as:

$$q_p \quad = 2Q/(T_p + T_r) \qquad \text{(Eq.S-10)}$$

Let,

$$K \quad = 2/(1 + (T_r/T_p)) \qquad \text{(Eq. S-11)}$$

therefore,

$$q_p \quad = KQ/T_p \qquad \text{(Eq. S-12)}$$

where,

$$Q \quad = \text{Direct runoff, IN}$$
$$T_p \quad = \text{Time to peak, HR}$$
$$T_r \quad = \text{Recession time, HR}$$
$$q_p \quad = \text{Peak discharge, IN/HR}$$

In making the conversion from IN/HR to CFS and putting the equation in terms ordinarily used, including drainage area (A) in MI^2, and time (T) in HRS, Equation S-12 becomes the general equation:

$$q_p \quad = (645.33 \ KAQ)/T_p \qquad \text{(Eq. S-13)}$$

Where q_p is peak discharge in CFS and the conversion factor 645.33 is the rate required to discharge one (1) IN of excess rainfall from one (1) MI^2 in one (1) HR.

The relationship of the triangular unit hydrograph, shows that $T_r = 1.67 \ T_p$ and gives K = 0.75 by Equation S-11. Then substituting into Equation S-13 gives:

$$q_p \quad = 484AQ/T_p \qquad \text{(Eq. S-14)}$$

Since the volume under the rising side of the triangular unit hydrograph is equal to the volume under the rising side of the curvilinear dimensionless unit hydrograph in Figure 2-3 in Appendix B of this Manual, the constant 484, or peak rate factor, is valid for calculation of the peak discharge for the dimensionless unit hydrograph.

Table 2-11
Ratios for Soil Conservation Service Dimensionless Unit Hydrograph and mass Curve

Time Ratios (t/T$_p$)	Discharge Ratios (q/q$_p$)	Mass Curve Ratios (Q$_a$/Q)
0.0	0.000	0.001
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

SECTION 3 - STREET FLOW

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SECTION 3 - STREET FLOW

3.1.0 GENERAL

The location of inlets and permissible flow of water in streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property for the 25-yr and 100-yr frequency storms. Interference to traffic is regulated by design limits of the spread of water into traffic lanes, especially in regard to arterials. Flooding of surrounding property from streets is controlled by limiting curb buildup to the top of curb for a 25-yr storm which is designated as the design storm. Conveyance provisions for the 100-yr storm must also be made within defined right-of-way and easements.

3.1.1 Interference Due to Flow in Streets

Water which flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into the traffic lane. On streets where parking is not permitted, as with many arterial streets, flow widths exceeding one traffic lane become a traffic hazard. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases, it becomes impossible for vehicles to operate without moving through water in an inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed in the open lane. Eventually, if width and depth of flow become great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances and police cars be able to traverse the street by moving along the crown of the roadway.

3.1.2 Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or because of the crown slope of intersecting streets has a substantial effect on the street-carrying capacity. The manner in which ponded water affects traffic is essentially the same as for curb flow; that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of surprise to drivers of moving vehicles, producing erratic and dangerous responses.

3.1.3 Street Cross Flow

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. Cross-flow is allowed only in case of superelevation of a curve or overflow from the higher gutter on a street with cross fall. No more than six (6) IN depth of flow at any point in the street for the 25-yr storm shall be allowed to cross flow from the higher elevation to the lower elevation.

3.1.4 Allowable Flow of Water Through Intersections

As the storm water flow approaches an arterial street or tee intersection, an inlet is required if more than six (6) IN depth of flow for the 25-yr storm shall cross the intersection. For a cul-de-sac with a slope greater than seven (7) percent, no more than five (5) CFS for the 25-yr storm shall be allowed to enter the bulb of the cul-de-sac. In both situations the inlet cannot

be placed inside the curb return.

3.1.5 Valley Gutter
Concrete valley gutters are useful in diminishing the deterioration of pavements, at intersections. At the intersection of two (2) arterial streets, a valley gutter cannot be used. At the intersection of two (2) collector streets or local streets, a valley gutter shall be installed. At an intersection of two (2) different types of streets, the valley gutter may be used across the smaller street only.

3.2.0 PERMISSIBLE SPREAD OF WATER

The flow of water in gutters of various streets of different categories shall be limited by those values found on Table 3-1. These clear widths at the crown of the roadway or at the high point on a divided roadway are necessary to provide access for vehicles in the event of an emergency. Equation 3-1 may be used to determine the spread of gutter flow for a specific street width and flow depth.

$$\text{Spread} = (W/2) - [(W^2/4) - 30y_0W^2/(30 + W)]^{1/2} \qquad \text{(Eq. 3-1)}$$

where,

W = Street Width, FT
Y_0 = Water depth in the gutter, FT

3.3.0 DESIGN METHOD

3.3.1 Gutter Flow Velocities
To insure scouring velocities for low flows, the gutter shall have a minimum slope of 0.004 FT/FT (0.4 percent).

3.3.2 Straight Crowns
Flow in gutters on straight crown pavements is normally assumed to be uniform, with Manning's Equation being used to determine the flow. However, because the hydraulic radius assumption in the Manning's Equation is not able to adequately describe the hydraulic characteristics of the gutter cross section, modification of the equation is necessary to accurately compute the flow. The modified Manning's Equation is:

$$Q_0 \quad = 0.56(Z/n)S_0^{1/2}Y_0^{8/3} \qquad \text{(Eq. 3-2)}$$

where,

Q_0 = Gutter discharge, CFS
Z = Reciprocal of the crown slope, FT/FT
S_0 = Street or gutter slope, FT/FT
n = Manning's Roughness Coefficient
Y_0 = Depth of flow in gutter, FT

Table 3-1
Minimum Clear Widths for Roadway Design Due to Gutter Flow*

Roadway Type	Proposed Usage	Minimum Clear Width
1. Local Street	a. Residential	0
	b. Commercial/Industrial	0
2. Collector	a. Minor	8
	b. Commercial/Industrial	12
	c. Major 4 Lanes	24
	5 Lanes	24
	4 Lanes Divided	12 (each way)
	6 Lanes Divided	12 (each way)
3. Arterial	a. 4 Lanes, Undivided	24
	b. 3 Lanes, One way	12
	c. 4 Lanes, One way	24
	d. 4 Lanes, with continuous left turn lane	24
	e. 4 Lanes, Divided	12 (each way)
	f. 6 Lanes, Divided	12 (each way)
	g. 8 Lanes, Divided	24 (each way)

Source: City of Austin Department of Public Works and Transportation

The nomograph in Figure 3-1 in Appendix B of this Manual provides a direct solution for flow conditions in triangular channels. For a concrete pavement gutter, a Manning's Roughness Coefficient, "n" value equal to 0.016 is recommended. For gutters with small slope less than one (1) percent where sediment may accumulate, a Manning's Roughness Coefficient, "n" value of 0.02 is recommended.

3.3.3 Parabolic Crowns
Flows in the gutter of a parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face of curb to face of curb.

A. **Streets Without Curb Split.** Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad \text{(Eq. 3-3)}$$

where,

Q = Gutter flow, CFS
S_0 = Street grade, FT/FT
y_0 = Water depth in the gutter, FT
K_0, K_1, K_2 = Constant coefficients shown in Table 3-2 for different street widths

Street Width (FT)	Coefficients		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

Table 3-2
Coefficients for Equation 3-3, Streets Without Curb Split

Source: City of Austin, Watershed Engineering Division

B. **Streets With Curb Split - Higher Gutter.** The gutter flow equation for calculating the higher gutter flows is as follows:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3(CS) \qquad \text{(Eq. 3-4)}$$

where,

Q = Gutter flow, CFS
S_0 = Street grade, FT/FT
Y_0 = Water depth in the gutter, FT
CS = Curb split, FT
K_0, K_1, K_2, K_3 = Constant coefficients shown in Table 3-3 for different street widths

Table 3-3
Coefficients for Equation 3-4, Streets With Curb Split - Higher Gutter

Street Width (FT)	Coefficients				Curb Split Range (FT)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0-0.6
36	2.89	0.50	2.99	-0.140	0.0-0.8
40	2.85	0.50	2.89	-0.084	0.0-0.8
44	2.84	0.50	2.83	-0.091	0.0-0.9
48	2.83	0.50	2.78	-0.095	0.0-1.0
60	2.85	0.50	2.74	-0.043	0.0-1.2

Source: City of Austin, Watershed Engineering Division

C. Streets with Curb Split - Lower Gutter. The gutter flow equation for the lower gutter is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3(CS) \qquad \text{(Eq. 3-5)}$$

where,

Q	= Gutter flow, CFS
S_0	= Street grade, FT/FT
y_0	= Water depth in the gutter, FT
CS	= Curb split, FT

K_0, K_1, K_2, K_3 = Constant coefficients shown in Table 3-4 for different street widths

Table 3-4
Coefficients for Equation 3-5, Streets With Curb Split - Lower Gutter

Street Width (FT)	Coefficients				Curb Split Range (FT)
	K_0	K_1	K_2	K_3	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

All the crown heights for different street widths are calculated by the following equation:

$$\text{Crown Height (FT)} = 0.5 + [(W - 30)/120] \qquad \text{(Eq. 3-6)}$$

where,

    W = street width, FT

D. Parabolic Crown Location.

The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high one fourth (1/4) point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the one fourth (1/4) point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 FT/FT of street width.

Example: Determination of Crown Location

Given: 0.4 FT Design split on 30-FT wide street.

Maximum curb split	= 0.02 x street width
	= 0.02 x 30 FT = 0.6 FT Maximum
Movement	= ¼ street width for 30 FT street
	= ¼ x 30 FT = 7.5 FT
Split Movement	= (Design split x W/Maximum Split x 4)
	= (0.4 x 30)/(0.6 x 4) = 5 FT

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections which have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section which has the crown one (1) IN above the top of curb.

SECTION 4-INLETS

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SECTION 4 - INLETS

4.1.0 GENERAL

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thus reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow, making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

A. Grated curb inlets are discouraged from use due to their increased tendency to clog and problems with replacement. In all instances where a curb inlet can be used in lieu of a grated curb inlet, it shall be required unless approval is given from the City Engineer.

B. Minimum transition for recessed inlets shall be ten (10) FT.

C. All curb inlets (whether in a sump or on grade) incorporate a standard five (5) IN depression. Unless otherwise approved in writing by City Engineer, all curb inlets shall be a minimum of ten (10) FT in length.

D. When recessed inlets are used, they shall not decrease the width of the sidewalk. Also, it should be noted that the use of recessed inlets must be approved by the City Engineer for all streets.

E. Design and location of inlets shall take into consideration pedestrian and bicycle traffic. In particular, grate inlets shall be designed to assure safe passage of bicycles.

F. Inlet design and location must be compatible with the criteria established in Section 3 of this Manual.

G. The use of slotted drains is discouraged except in instances where there is no alternative. If used, the manufacturer's design guidelines should be followed.

4.2.0 INLET CLASSIFICATIONS

Inlets are classified into two (2) major groups:

A. Inlets in sumps where flow contributes from two (2) or more sides (Type S)

B. Inlets on grade (Type G).

The following list references the various inlet types. (See Figures 4-1 through 4-7 in Appendix B of this Manual).

Inlets in Sumps
    (1) Curb Opening   Type S-1
    (2) Grate*   Type S-2
    (3) Combination (Grate and Curb Opening)*   Type S-3
    (4) Area Without Grate   Type S-4
Inlets on Grade
    (1) Curb Opening   Type G-1
    (2) Grate*   Type G-2
    (3) Combination (Grate and Curb Opening)*   Type G-3

Recessed inlets are identified by the suffix (R), i.e.: S-1(R).

* For the flow capacity through the grate inlets, the Engineer should check appropriate vendor catalog.

4.3.0 STORM INLET HYDRAULICS

4.3.1 Inlets In Sumps

Inlets in sumps are inlets at low points with gutter flow contributing from two (2) or more sides. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. Sump inlets should be designed using Figure 4-8 in Appendix B of this Manual, for an unsubmerged inlet. For submerged conditions, regardless of what depth of depression exists at the inlet, see Figure 4-9 in Appendix B of this manual.

A. Curb Opening Inlets (Type S-1) and Area Inlet Without Grate (Type S-4).

Unsubmerged curb opening inlets (Type S-1) and area inlets without grates (Type S-4) in a sump function as rectangular weirs with a coefficient of discharge of 3.0. Their capacity shall be based on the following Equation:

$$Q = 3.0h^{1.5}L \qquad \text{(Eq. 4-1)}$$
$$H = a + Y_0 \qquad \text{(Eq. 4-1A)}$$

where,

Q = Capacity of curb opening inlet or of area inlet, CFS
H = Head at the inlet, FT
L = Length of opening through which water enters the inlet, FT
a = Gutter depression, FT
Y_0 = Depth of flow at inlet opening, FT

Figure 4-8 in Appendix B of this Manual provides for direct solution of the above Equation 4-1.

Curb opening inlets and drop inlets in sumps have a tendency to collect debris at their entrances. For this reason, the calculated inlet capacity shall be reduced by ten (10) percent to allow for clogging.

B. Grate Inlets (Type S-2).

An area inlet with a grate (Type S-2) in a sump functions as an orifice with a coefficient of discharge of 0.60. Therefore, the orifice equation becomes:

$$Q = 4.82Ah^{0.5} \qquad \text{(Eq. 4-2)}$$

where,

Q = Capacity, CFS
h = Depth of flow at inlet, FT
A = Area of grate opening, FT^2

The curves shown in Figure 4-9 in Appendix B of this Manual provide for direct solution of the above equation.

Area inlets with grates in sumps have a tendency to clog from debris which becomes trapped by the inlet. For this reason, the calculated inlet capacity of a grate inlet shall

be reduced by fifty (50) percent to allow for clogging. Since the clogging problems require maintenance, grate inlets in sumps are discouraged.

C. **Combination Inlets (Type S-3).**
The capacity of a combination inlet Type S-3 consisting of a grate and curb opening in a sump shall be considered to be the sum of the capacities obtained from Figures 4-8 and 4-9 in Appendix B of this Manual. When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow.

Combination inlets in sumps have a tendency to clog and collect debris at their entrances. For this reason, the calculated inlet capacities shall be reduced by their respective percentages indicated previously (which are ten (10) percent for a curb opening and fifty (50) percent for grate inlets).

D. **Recessed Inlets in Sumps. (Type S-1(R), Type S-3(R))**
Recessed inlets can be either curb opening or combination types. The clogging factors shall remain the same for recessed or non-recessed inlets.

4.3.2 Inlets On Grade With Gutter Depression

A. **Curb Opening Inlets on Grade (Type G-1).**
The capacity of a depressed curb inlet should be determined by use of Figures 4-10 and 4-11 in Appendix B of this Manual. Because the inlet is on a slope and there is no grate to catch debris, the majority of the debris will be carried downstream; therefore, no reduction for clogging is necessary.

B. **Grate Inlets on Grade (Type G-2).**
The depression of the gutter at a grate inlet decreases the flow past the outside of the grate. The effect is the same as that caused by the depression of a curb inlet.
The bar arrangements for grate inlets greatly affect the efficiency of the inlet. In order to determine the capacity of a grate inlet on grade, the appropriate vendor catalog should be checked (see Bibliography, Item 4-3 of this Manual).
Grate inlets have a tendency to trap debris such as leaves and paper being carried by the gutter flows. This causes traffic problems from ponding water and requires maintenance. A reduction factor of thirty (30) percent to allow for clogging should be applied.

C. **Combination Inlets on Grade (Type G-3).**
Combination inlets (curb opening plus grate) have greater hydraulic capacity than curb opening inlets or grate inlets of the same length. Generally speaking, combination inlets are the most efficient of the three (3) types of inlets on grade presented in this manual. The basic difference between a combination inlet and a grate inlet is that the curb opening receives the carry-over flow that passes between the curb and the grate. The reduction factor for clogging of this type of inlet shall be zero (0) percent for the curb opening and thirty five (35) percent for the grate inlet.

D. **Recessed Inlets on Grade (Type G-1R, G-3R).**
Capacities for recessed inlets on grade shall be calculated as seventy-five hundredth (0.75) times the capacity for non-recessed inlets. The clogging factors shall remain the same for the various types of inlets.

4.3.3 Example 4-1

Given: Parabolic crown street width = 30 FT
       Cross Slope = 0 FT/FT
       Street Grade = 5 percent
       Q_a in one gutter = 12 CFS

Find: Capacity of a 10 FT curb inlet on grade (Type G-1) with a 5 IN gutter depression.

Step 1. From Equation 3-3 (Section 3 of this Manual) depth of flow in gutter is Y_0= 0.43 FT = 5.1 IN.

Step 2. Enter Figure 4-10 with Y_0 = 0.43 FT and a = 5 IN and find corresponding Q_a/L_a= 0.90

Step 3. Compute L_a = 12/0.90 = 13.33.

Step 4. Compute L/L_a = 10/13.33 = 0.75.

Step 5. Enter Figure 4-11 (in Appendix B of this Manual) with L/L_a = 0.75 and a/Y_0 = 0.98 and find corresponding Q/Q_a= 0.84.

Step 6. Determine Q from Q/Q_a
Q = 0.84 (12) = 10.1 CFS

Step 7. Determine Q_{pass}
Q_{pass} = 12-10.1 = 1.9 CFS

Step 8. The by-pass flow is 1.90 CFS

4.4.0 INLET SYSTEM LAYOUT

The following is intended to provide a general step by step procedure for the layout of an inlet system utilizing the information that has been provided in Chapters 3 and 4 of this Manual. This information is in no way a requirement for design and is provided solely for the benefit of the Engineer or Designer.

4.4.1 Preliminary Design Considerations

A. Prepare a drainage map of the entire area to be drained by proposed improvements. Contour maps serve as excellent drainage area maps when supplemented by field observation.

B. Outline the drainage area for each inlet in accordance with present and future street development. Show all existing underground utilities.

C. Make a tentative layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

D. Establish the design rainfall frequency.

E. Establish the minimum inlet time of concentration.

F. Establish the typical cross section of each street.

G. Establish the permissible spread of water on all streets within the drainage area.

H. Indicate each drainage area, the size of area, the direction of surface runoff by small arrows and the coefficient of runoff for the area.

4.4.2 Inlet System Design

Determining the size and location of inlets is largely a trial and error procedure. Based on criteria outlined in Sections 2, 3 and 4 of this Manual, the following steps will serve as a guide to the procedure to be used.

Step 1. Beginning at the upstream end of the project drainage basin, outline a trial subarea and calculate the runoff from it.

Step 2. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial subarea. If the calculated runoff is less than street capacity, increase the size of the trial subarea. Repeat this procedure until the calculated runoff

equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacities versus runoff entering the street downstream.

Step 3. Record the drainage area, time of concentration, runoff coefficient and calculated runoff for the subarea. This information shall be recorded on the plans or in tabular form similar to that shown in Table 4-1 convenient for review.

Step 4. If an inlet is to be used to remove water from the street, determine and record the inlet size, amount of intercepted flow and amount of flow carried over (bypassing the inlet).

Step 5. Continue the above procedure for other subareas until a complete system of inlets has been established. Remember to account for carry-over from one inlet to the next.

Step 6. After a complete system of inlets has been established, modification should be made to accommodate special situations such as point sources of large quantities of runoff, and variation of street alignments and grades.

Step 7. Record information as in Steps 3 and 4 above for all inlets.

Step 8. After the inlets have been located and sized the inlet pipes can be designed (see Section 5 of this Manual).

4.4.3 Inlet Flow Calculation Table

An example of a calculation table for inlet flow design is shown in Table 4-1 of this Manual.

The following is an explanation of each column in Table 4-1:

Column 1. Inlet number. All inlets are classified with a designated number.

Column 2. Drainage area number. List all numbers of the drainage areas which drain stormwater into inlet number in Column 1.

Column 3. The corresponding discharge from the drainage areas in Column 2.

Column 4. The carry-over flow (Q_{pass}) in this column is the quantity of water which has passed by the last preceding inlet to the inlet under consideration.

Column 5. The total runoff, Q_a, is the run-off from Column 3 plus the carry-over from preceding drainage areas.

Column 6. The slope, S, expressed in percentage, is obtained from established grade lines as shown on the plan-profile sheets, or from specified data.

Column 7. Gutter depression.

Column 8. The water depth, Y_0, in the gutter is expressed in FT." Y_0 " can be determined from Equation 3-1 or Figure 3-1 (in Appendix B of this Manual) for the straight crown streets and determined from Equations 3-3, 3-4 or 3-5 for the parabolic crown streets.

Column 9. The value of the ponded width is the product of the water depth (in Column 7) and the reciprocal of the cross slope (z) in the Equation 3-2. The ponding width must be kept within the maximum permissible ponded limit of the streets.

Column 10. The reduction factor for each inlet as specified in Section 4.3.0 of this Manual.

Column 11. Q_a/L_a is read from Figure 4-10 in Appendix B of this Manual by using the gutter depression and gutter flow depth.

Column 12. L_a is calculated from Q_a divided by the value in Column 11. L_a represents the length of an inlet for one-hundred (100) percent interception.

Column 13. Length of the inlet L.

Column 14. The ratio of L/L_a.

Column 15. The ratio of gutter depression (FT) to water depth in the gutter (FT).

Column 16. The ratio of Q/Q_a. The value is read from Figure 4-11 in Appendix B of this Manual.

Column 17. Q is the flow intercepted by the inlet of length L.

Column 18. The carry-over flow (Q_{pass}) is the result of $Q_a - Q$.

Column 19. This column is used to specify the inlet information.

Table 4-1 Inlet Flow Calculation Table

INLET FLOW CALCULATION TABLE

INLET NUMBER	DRAINAGE AREA NO.	Q (CFS)	Q_{pass} (CFS)	Q TOTAL (Q_a/CFS)	SLOPE (%)	G (IN)	Y_s (FT)	PONDED WIDTH (FT)	R.F. (%)	Q_a/L_a	L_a (FT)	LENGTH (FT)	L/L_a	A/Y_s	Q/Q_a	Q (CFS)	Q_{pass} (CFS)	REMARK
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

SECTION 5 - STORM DRAINS

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SECTION 5 - STORM DRAINS

5.1.0 GENERAL

The purpose of this section is to consider the hydraulic aspects of storm drains and their appurtenances in a storm drainage system. Hydraulically, storm drainage systems consist of conduits (open or enclosed) in which unsteady and non-uniform flow exists. The design storm shall be the 25-yr storm with provisions made for the 100-yr storm as noted in Section 3 of this Manual.

5.2.0 DESIGN GUIDELINES

The following rules are to be observed in the design of storm drain system components to be located in public right-of-way or public drainage easements in order to promote proper operation and to minimize maintenance of those systems:

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

B. Do not discharge the contents of a larger pipe into a smaller one even though the capacity of the smaller pipe may be greater due to a steeper slope.

C. For all pipe junctions other than a manhole, the angle of intersection between any two flow paths shall not be greater than sixty (60) degrees. This includes discharges into box culverts and channels.

E. No proposed pipe having a diameter greater than fifty (50) percent of the minimum dimension of an existing box culvert shall be allowed to discharge into that box culvert. Exceptions must be justified by structural engineering analysis.

G. Pipe shall be reinforced concrete. Concrete pipe shall be manufactured and installed in compliance with the City of Gatesville – Standards for Public Works Construction. The use of high density polyethylene pipe shall be only as approved by the City Engineer or Director of Public Works.

H. The 25-yr hydraulic grade line shall remain six (6) IN below the theoretical gutter flow line of inlets.

5.3.0 DESIGN PARAMETERS

5.3.1 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent deposition of solid material. The controlling velocity is near the bottom of the conduit and is considerably less than the mean velocity. Storm drains should be designed to have a minimum velocity of two and five tenths (2.5) FPS.

5.3.2 Maximum Velocities

Maximum velocities in conduits are important because of the possibility of excessive erosion of the storm drain pipe material. Table 5-1 lists the maximum velocities allowed.

Table 5-1 Maximum Velocity In Storm Drains	
Type	Maximum Permissible Velocity
Storm Drains (inlet laterals)	No limit
Storm Drains (trunk)	20 FPS

5.3.3 Minimum Diameter

Pipes that are to become an integral part of the public storm sewer system shall have a minimum diameter of eighteen (18) IN.

5.3.4 Roughness Coefficients

The coefficients of roughness listed in Table 5-2 are for use in Manning's Equation.

<table>
<tr><th colspan="2">Table5-2
Manning's Roughness Coefficients "n" For Storm Drains</th></tr>
<tr><th>Materials of Construction</th><th>Minimum Design Coefficient</th></tr>
<tr><td>Concrete</td><td>0.013</td></tr>
<tr><td>Corrugated-metal Pipe</td><td>0.024</td></tr>
<tr><td>Plain or Coated
    Paved Invert (Asphalt)</td><td>0.020</td></tr>
<tr><td>Plastic Pipe
    Smooth
    Perforated</td><td>0.010 0.020</td></tr>
</table>

5.4.0 FLOW IN STORM DRAINS

All storm drains shall be designed by the application of the Continuity Equation (Eq. S-1) and Manning's Equation (Eq. S-2) either through the appropriate charts and nomographs or by direct solution of the equations as follows:

5.4.1 Flow Equation Method

$$Q = AV$$
$$Q = (1.49/n)AR^{2/3}S^{1/2}$$
$$R = A/W_p$$

where,
  (Eq. 5-1)
  (Eq. 5-2)
  (Eq. 5-2A)

Q = Pipe flow, CFS
A = Cross-sectional area of flow, FT^2
V = Velocity of flow, FPS
n = Manning's Roughness Coefficient
R = Hydraulic radius, FT
S = Friction slope in pipe, FT/FT
W_p = Wetted perimeter, FT

5.4.2 Nomograph Method

Nomographs for determining flow properties in circular pipe, elliptical pipe and pipe-arches are given here as Figures 5-1 through 5-9 in Appendix B of this Manual. The nomographs are based upon a value of Manning's Roughness Coefficient, "n" equal to 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the following examples in this Section.

For values of Manning's Roughness Coefficient, "n" other than 0.012, the value of Q should be modified by using the following formula:

$$Q_c = 0.012 \, Q_n/n_c \qquad \qquad \text{(Eq. 5-2B)}$$

where,

Q_c = Flow based upon n_c, CFS
n_c = Value of "n" other than 0.012
Q_n = Flow from nomograph based on n = 0.012

This formula can be used in two (2) ways. If n_c = 0.015 and Q_c is unknown, use the known values to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of the known values, use the formula to convert Q_c (based on n_c) to Q_n (based on "n" = 0.012) first, and then use Q_n and the other known values to find the unknown variable on the nomograph.

Example 5-1:

Given: Slope = 0.005 FT/FT
      d = depth of flow = 1.8 FT
      D = diameter = 36 IN
      n = 0.018

Find: Discharge (Q).

First determine d/D = 1.8 FT/3.0 FT = 0.6. then enter Figure 5-1 (in Appendix B of this Manual) to read Q_n = 34 CFS. From the formula, Q_c = 34 (0.012/0.018) = 22.7 CFS.

Example 5-2:

Given: Slope = 0.005 FT/FT
      D = diameter = 36 IN
      Q = 22.7 CFS
      n = 0.018

Find: Velocity of flow (FPS).

First convert Q_c to Q_n so that nomograph can be used. Using the formula Q_n = 22.7 (0.018)/(0.012) = 34 CFS, enter Figure 5-1 (in Appendix B of this Manual) to determine d/D = 0.6. Now enter Figure 5-3 (in Appendix B of this Manual) to determine V = 7.5 FPS.

5.5.0 HYDRAULIC GRADIENT

In storm drain systems flowing full, all losses of energy are a function of resistance of flow in pipes or by interference with flow patterns at junctions. These losses must be accounted for by their accumulation along the system from its tailwater elevation at the outlet to its upstream inlet. The purpose of determining head losses is to include these values in a progressive calculation of the hydraulic gradient. In this way, it is possible to determine the hydraulic gradient line which will exist along the storm drain system. The hydraulic gradient line shall be computed and plotted for all sections of a storm drain system flowing full or under pressure flow. The determination of friction loss and minor loss are important for these calculations.

5.5.1 Friction Losses

Friction loss is the energy required to overcome the roughness of the pipe and is expressed as:

$$h_f = (29n^2/R^{1.33})(V^2/2g)L \qquad \text{(Eq. 5-3)}$$

where,

h_f	= Friction loss, FT
n	= Manning's Roughness Coefficient
L	= Length of pipe, FT
R	= Hydraulic radius, FT
V	= Velocity of flow, FPS
g	= Acceleration due to gravity, 32.2 FT/SEC²

In addition to Equation 5-3, Table 5-3 can be used to determine the friction slope and applied in Equation 5-4.

$$h_f = S_f L \qquad \text{(Eq.5-4)}$$
$$S_f = (Q/C)^2 \qquad \text{(Eq. 5-4a)}$$

where,

h_f	= Friction loss, FT
S_f	= Friction slope, FT
L	= Length of pipe, FT
C	= Full flow coefficient from Table 5-3
Q	= Discharge, CFS

Example 5-3:

Given: Discharge Q = 24 CFS, diameter D = 24 IN, the length of pipe L=300 FT and
n = 0.013

Find: The friction loss h_f

First, from Table 5-3 for D = 24 IN and n = 0.013, the full flow coefficient C = 226. Second, the friction slope $S_f = (Q/C)^2 = (24/226)2 = 0.011$
The friction loss $h_f= S_f L = 0.011 \times 300 = 3.3$ FT

Table 5-3
Full Flow Coefficient Values for Circular Concrete Pipe

D Pipe Diameter (IN)	A Area (FT²)	R Hydraulic Radius (FT)	Value of C* for			
			n = 0.010	n = 0.011	n = 0.012	n = 0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.250	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137.0	124.0	114.0	105.0
21	2.405	0.437	206.0	187.0	172.0	158.0
24	3.142	0.500	294.0	267.0	245.0	226.0
27	3.976	0.562	402.0	366.0	335.0	310.0
30	4.909	0.625	533.0	485.0	444.0	410.0
33	5.940	0.688	686.0	624.0	574.0	530.0
36	7.069	0.750	867.0	788.0	722.0	666.0
42	9.621	0.875	1308.0	1189.0	1090.0	1006.0
54	15.904	1.125	2557.0	2325.0	2131.0	1967.0
60	19.635	1.250	3385.0	3077.0	2821.0	2604.0
66	23.758	1.375	4364.0	3967.0	3636.0	3357.0
72	28.274	1.500	5504.0	5004.0	4587.0	4234.0
78	33.183	1.625	6815.0	6195.0	5679.0	5242.0
84	38.485	1.750	8304.0	7549.0	6920.0	6388.0
90	44.170	1.875	9985.0	9078.0	8321.0	7681.0
96	50.266	2.000	11850.0	10780.0	9878.0	9119.0
102	56.745	2.125	13940.0	12670.0	11620.0	10720.0
108	63.617	2.250	16230.0	14760.0	13530.0	12490.0
114	70.882	2.375	18750.0	17040.0	15620.0	14420.0
120	78.540	2.500	21500.0	19540.0	17920.0	16540.0
126	86.590	2.625	24480.0	22260.0	20400.0	18830.0
132	95.033	2.750	27720.0	25200.0	23100.0	21330.0
138	103.870	2.875	31210.0	28370.0	26010.0	24010.0
144	113.100	3.000	34960.0	31780.0	29130.0	26890.0

* $C = (1.486/n)AR^{0.667}$

Source: American Concrete Pipe Association.
Concrete Pipe Design Manual.

5.5.2 Minor Losses

From the point at which stormwater enters the drainage system at the inlet until it discharges at the outlet, it encounters a variety of hydraulic structures such as manholes, bends, enlargements, contractions and other transitions. These structures will cause head losses which are called "minor head losses."

The minor head losses are generally expressed in a form derived from the Bernoulli and Darcy-Weisbach Equations:

$$h = KV^2/2g \qquad \text{(Eq. 5-5)}$$

where,

h = Velocity head loss, FT
K = Coefficient for head loss
V = Velocity of flow, FPS
g = Acceleration due to gravity, 32.2 FT/SEC^2

The following are minor head losses of hydraulic structures commonly found in a storm drainage system.

A. **Junction Losses.** Equation 5-6 is used to determine the head loss at a junction of two (2) pipes, with the various conditions of the coefficient K_j given in Table 5-4.

$$h_j = (V_2{}^2 - K_j V_1{}^2)/2g \qquad \text{(Eq. 5-6)}$$

where,

V_1 = Velocity for inflowing pipe, FPS.
V_2 = Velocity for outflowing pipe, FPS.
K_j = Coefficient of loss in junction or structure
g = Acceleration due to gravity, 32.2 FT/SEC^2

The detailed design information for junction losses can be found in the Appendix B of this Manual, Item 5-10.

B. **Bend Losses.** The minor head loss at a bend results from a distortion of the velocity distribution, thereby causing additional shear stresses within the fluid. The bend loss is considered to be that in excess of the loss for an equal length of straight pipe. The equation to compute the bend loss is:

$$h_b = K_b V^2/2g \qquad \text{(Eq. 5-7)}$$

where,

K_b = Coefficient of loss in a bend
V = Velocity of flow, FPS
g = Acceleration due to gravity, 32.2 FT/SEC^2

The coefficient K_b varies with the angle of the bend. Table 5-4 and Figure 5-11 in Appendix B of this Manual show the different K_b coefficients used in bend losses.

<table>

Table 5-4
Junction or Structure Coefficient of Loss

Cases	Reference Figure	Description of Condition	Coefficient K_j
A	5-10	Manhole on Main Line with 45° Branch Lateral	0.50
B	5-10	Manhole on Main Line with 90° Branch Lateral	0.25
C	5-11	45° Wye Connection or cut-in	0.75
D	5-11	Inlet or Manhole at Beginning of Main Line or Lateral	1.25
E	5-11	Conduit on Curves for 90°* Curve radius = diameter Curve radius = (2 to 8) diameter Curve radius = (8 to 20) diameter	0.50 0.40 0.25
F	5-11	Bends where radius is equal to diameter 90° bend 60° bend 45° bend 22½° bend Manhole on line with 60° Lateral Manhole on line with 22½° Lateral	0.50 0.43 0.35 0.20 0.35 0.75

*Where bends other than 90 degrees are used, the 90 degree bend coefficient can be used with the following percentage factor applied:

60° Bend - 85%; 45° Bend - 70%; 22½° Bend - 40%

Source: City of Austin Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

</table>

C. **Transition Losses.** The head losses resulting from sudden and gradual changes in the cross-section or flow direction are included in this category. Four (4) transition losses are discussed here.

1. Sudden Enlargement. Table 5-5 shows the coefficients used in the different cases for head losses due to a sudden enlargement.
2. Gradual Enlargement. Table 5-6 shows the coefficients for calculating the head loss based on the angle of the cone transition.
3. Sudden Contraction. Table 5-7 illustrates the values of coefficients in determining the head loss due to a sudden contraction.
4. Gradual Contraction. The head losses due to a gradual contraction are determined by the following equation with a constant head loss coefficient.

$$h_{gc} = 0.04 \, V^2/2g \qquad \text{(Eq. 5-8)}$$

where,

V = velocity for smaller pipe, FPS

g = Acceleration due to gravity, 32.2 FT/SEC^2

	Table 5-5 **Values of K for Determining Loss of Head Due to Sudden** **Enlargement in Pipes, from the Formula H = K (V^2/2g)**										
d_2/d_1	**Velocity, V, FPS**										
	2	3	4	5	6	7	8	10	12	15	20
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84
	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86

V = velocity in smaller pipe

d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

Source: Brater, E.F. and H.W. King. <u>Handbook of Hydraulics</u>, 1976.

Table 5-6

Values of K for Determining Loss of Head Due to Gradual Enlargement in Pipes from the Formula H = K (V²/2g)

d_2/d_1	Angle of cone*													
	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	0.01	0.01	0.01	0.02	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.20	0.21	0.23
1.2	0.02	0.02	0.02	0.03	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.33	0.35	0.37
1.4	0.02	0.03	0.03	0.04	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.47	0.50	0.53
1.6	0.03	0.03	0.04	0.05	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.54	0.57	0.61
1.8	0.03	0.04	0.04	0.05	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.58	0.61	0.65
2.0	0.03	0.04	0.04	0.05	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.60	0.63	0.68
2.5	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.62	0.65	0.70
3.0	0.03	0.04	0.04	0.05	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.63	0.66	0.71
	0.03	0.04	0.04	0.06	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.64	0.67	0.72

* Angle of cone is twice the angle between the axis of the cone and its side.
V = velocity in smaller pipe.
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe.

Source: Brater, E.F. and H.W. King. Handbook of Hydraulics, 1976.

Table 5-7

Values of K for Determining Loss of Head Due to Sudden Contraction in Pipe From the Formula H = K (V²/2g)

d_2/d_1	Velocity, V, FPS										
	2	3	4	5	6	7	8	10	12	15	20
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.31
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43
	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44

V = velocity in smaller pipe
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

Source: Brater, E.F. and H.W. King. Handbook of Hydraulics, 1976.

5.5.3 Hydraulic Gradient Calculation Table

After computing the quantity of storm runoff entering each inlet, the storm drain system required to convey the runoff can be designed. The ground line profile is now used in conjunction with the previous runoff calculations. Table 5-8 can be used to keep track of the pipe design and corresponding hydraulic grade line calculations. Note that the computations begin at the

downstream discharge point and continue upstream through the pipe system.

The following is an explanation of each of the columns in Table 5-8:

Column 1. Design Point; this point is the first junction point* upstream.
* "Junction Point" refers to any inlet, manhole, bend, etc. that occurs which would cause a minor head loss.

Column 2. Junction point immediately downstream of design point.

Column 3. Distance between one (1) and two (2) in FT.

Column 4. Design discharge as determined in inlet calculations. (See Table 4-1).

Column 5. Size of pipe chosen to carry an amount equal to or greater than the design discharge (Figures 5-12 and 5-15 in Appendix B of this Manual can be used to determine this).

Column 6. Slope of frictional gradient (can be determined from Table 5-3 using ($S_f = (Q/C)^2$).

Column 7. Elevation of hydraulic gradient (h_g) at upstream end of pipe = elevation of downstream end + Column 6 times Column 3, or elevation at upstream end + d/D if pipe is not flowing under pressure flow conditions.

Column 8. Elevation of hydraulic gradient at downstream end of pipe (Note: at outfall point assume h_g is at top of pipe or above if actual tailwater elevation exists).

Column 9. Velocity of flow in incoming pipe at design point (use Q=AV for full flow and Figures 5-1 and 5-3 in Appendix B of this Manual for partial flow).

Column 10. Velocity of flow in outgoing pipe at design point.

Column 11. Velocity head loss for outgoing pipe at design point.

Column 12. Velocity head loss for incoming pipe at design point.

Column 13. Head loss coefficients at junction (see Figures 5-10 and 5-11 in Appendix B of this manual).

Column 14. Column 12 times Column 13.

Column 15. Column 11 - Column 14 (Note for bends and inlets or manholes at the beginning of a line, $V_1 = V_2$. The appropriate K_j value should be used in Column 14, and Column 14 = Column 15).

Column 16. Column 7 + Column 15.

Column 17. Invert elevation at design point for incoming pipe.

Column 18. Invert elevation at design point for outgoing pipe.

Table 5-8 Hydraulic Computations Storm Sewers

Manholes/ Inlets		DIST	DISCHARGE	PIPE SIZE	FRICTION GRADE	HYDR. GRADIENT		V_1 INFLOW	V_2 OUTFLOW	$\frac{V_1^2}{2g}$	$\frac{V_2^2}{2g}$	K_j	$\frac{K_jV_1^2}{2g}$	h	H.G.EL. DESIGN PT.	INV. IN	INV. OUT
						UP STREAM	DOWN STREAM										
FROM	TO	FT	CFS	IN	FT/FT	ELEV	ELEV	FPS	FPS	FT	FT	CON-STANT	FT	FT	ELEV	ELEV	ELEV
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18

5.6.0 MANHOLES

Manholes provide a very important access point for maintenance purposes. Due to equipment

restraints, every point within the storm drain must be a maximum of two hundred fifty (250) FT from an access point for drains thirty (30) IN in diameter or smaller. For storm drains greater than thirty (30) IN in diameter, manholes shall be placed so that there is a maximum distance of three hundred (300) FT to an access point. Inlets and storm drain outfalls may be considered as access points for maintenance purposes. Access points must be accessible in accordance with the requirements of Section 1.2.4D. of this Manual and must provide a maintenance path within the storm drain that has no more than one (1) horizontal bend, with that bend having a deflection of no more than forty-five (45) degrees in the direction of the maintenance path, and no vertical bend with a deflection of greater than five (5) degrees. Storm drain slope adjustments of less than five (5) degrees are not subject to this requirement.

Manholes shall also be located where two or more laterals intersect the main line within five (5) FT of each other (See Figure 5-12 in Appendix B of this Manual for examples of possible manhole locations). Manholes shall also be placed at locations where changes in pipe size occur.

5.7.0 DEPTH OF COVER
The design of storm drains for areas that will or could receive vehicular traffic or that will be subject to other loading must be supported by structural engineering calculations or references to structural engineering standards.

SECTION 6 - OPEN CHANNELS
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SECTION 6 - OPEN CHANNELS

6.1.0 GENERAL
Open channels for use in a major drainage system have significant advantages related to cost, capacity, multiple use for recreational and aesthetic purposes and potential for detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages and to increase the benefits.

The general classifications for open channels are:

A. Natural channels, which include all watercourses that have been carved by nature through erosion; and
B. New or altered channels, which are constructed or existing channels that have been significantly altered by human effort. New or altered channels can be lined with grass, concrete, mortared rocks or other materials. The channels should be designed for the 25-yr storm with provisions for the 100-yr storm within dedicated easements or right-of-way.

6.1.1 Natural Channels
The ideal natural channel has the following benefits:

A. Velocities are usually low, resulting in longer times of concentration and lower downstream peak flows.
B. Maintenance needs are usually low because the channel is somewhat stabilized.
C. The channel provides a desirable green belt and recreational area adding significant social benefits.

6.1.2 New or Altered Channels
Grass channels are the most desirable of the various types of new channels for the following reasons:

A. The grass can stabilize the body of the channel.
B. The grass consolidates the soil mass of the bed.
C. The grass controls the movement of soil particles along the channel bottom.

Concrete lined channels are designed to protect the channel body from the erosive potential of high velocities. In addition to concrete-lined channels, other methods to combat erosive velocities in channels may be available and should be submitted to the City Engineer for review.

6.1.3 Section 404 Permit
When a project to modify a natural channel is proposed, the Engineer should check the requirements of Section 404, Permits for Dredged or Fill Material, of the Clean Water Act. If required, a permit should be obtained from the U.S. Army Corps of Engineers by the Engineer.

6.2.0 OPEN CHANNEL HYDRAULICS
An open channel is a conduit in which water flows with a free surface. The classification of open channel flow is made according to the change in flow depth with respect to time and space.
Flow in an open channel is said to be "steady" if the depth of flow does not change or if it can be assumed to be constant during the time interval under consideration. The flow is "unsteady" if the depth changes with time.

Open channel flow is said to be "uniform" if the depth of flow is the same at every section of the channel. A uniform flow may theoretically be steady or unsteady, depending on whether or not the depth changes with time. The establishment of unsteady uniform flow requires that the water surface fluctuate with time while remaining parallel to the channel bottom. Since it is impossible for this condition to occur within a channel, steady uniform flow is the fundamental type of flow treated in open channel hydraulics.

Flow is "varied" if the depth of flow changes along the length of the channel. Varied flow may be either steady or unsteady. Since unsteady uniform flow is rare, the term "unsteady flow" is used to designate unsteady varied flow exclusively.

Varied flow may be further classified as either "rapidly" or "gradually" varied. The flow is rapidly varied if the depth changes abruptly over a comparatively short distance; otherwise, it is gradually varied. Rapidly varied flow is also known as a local phenomenon; an example of which is the hydraulic jump.

With these varying conditions, open channel hydraulics can be very complex, encompassing many different flow conditions from steady uniform flow to unsteady rapidly varied flow. Most of the problems in stormwater drainage involve uniform, gradually varied or rapidly varied flow situations. In this Section, the basic equation and computational procedures for uniform, gradually varied and rapidly varied flows are presented.

6.2.1 Uniform Flow
For a given channel condition of roughness, discharge and slope, there is only one (1) possible depth for maintaining a uniform flow. This depth is referred to as normal depth.

Manning's Equation is used to determine the normal depth for a given discharge.

$$Q = (1.49/n)AR^{2/3}S^{1/2} \qquad \text{(Eq. 6-1)}$$
$$R = A/P \qquad \text{(Eq. 6-1A)}$$

where,

Q = Total discharge, CFS
n = Manning's Roughness Coefficient
A = Cross-sectional area of channel, FT^2
R = Hydraulic radius of channel, FT
S = Slope of the frictional gradient, FT/FT
P = Wetted perimeter, FT

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in nature or to obtain in the laboratory. The Engineer must be aware of the fact that uniform flow computations provide only an approximation of what will occur but that such computations are usually adequate and useful and, therefore, necessary for planning.
The computation of normal depth for trapezoidal sections can be performed by using Figure 6-1 in Appendix B of this Manual.

6.2.2 Gradually Varied Flow
The most common example of gradually varied flow in urban drainage systems occurs in the backwater of bridge openings, culverts, storm sewer inlets and channel constrictions. Under these conditions, gradually varied flow will be created and the flow depth will be greater than

normal depth in the channel. Backwater techniques would need to be applied to determine the water surface profile.

Calculations of water surface profiles can be accomplished by using standard backwater methods or acceptable computer routines, which take into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions in open channels.

There are several acceptable methods for backwater calculations. The most common hand calculation method for prismatic channels and irregular-uniform channels is the Standard Step Method. The most widely used backwater analysis computer program is HEC-RAS, developed by the U.S. Army Corps of Engineers. This program can compute water surface profiles for natural and new channels.

6.2.3 Rapidly Varied Flow

Rapidly varied flow is characterized by abrupt changes in the water surface elevation for a constant flow. The change in elevation may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Some common causes of rapidly varied flow in urban drainage systems are side-spill weirs, weirs and spillways of detention basins.

6.3.0 MANNING'S ROUGHNESS COEFFICIENTS

6.3.1 Existing and Natural Channels

Because several primary factors affect the roughness coefficient, a procedure has been developed to estimate this value, "n." By this procedure, the value of "n" may be computed by:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m \qquad \text{(Eq. 6-2)}$$

where,

n_0 = Channel material coefficient from Table 6-1
n_1 = Channel surface roughness coefficient from Table 6-1
n_2 = Channel shape coefficient from Table 6-1
n_3 = Obstructions in channel coefficient from Table 6-1
n_4 = Vegetation in channel coefficient from Table 6-1
m = Degree of meandering of channel from Table 6-1

In selecting the value of n_1, the degree of irregularity is considered smooth for surfaces comparable to the best attainable for the materials involved; minor for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; moderate for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels; and severe for badly sloughed banks of natural streams, badly eroded or sloughed sides of canals or drainage channels, and unshaped, jagged and irregular surfaces of channels excavated in rock.

In selecting the value of n_2, the character of variations in size and shape of cross-section is considered gradual when the change in size or shape occurs gradually; alternating occasionally when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side; and alternating frequently when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

The selection of the value of n_3 is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders and fallen and lodged logs. One should recall that conditions considered in other steps must not be re-evaluated or double-counted in this selection. In judging the relative effect of obstructions, consider the following: the extent to which the obstructions occupy or reduce the average water area, the obstruction characteristics (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects) and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

Table 6-1
Computation of Composite Roughness Coefficient
For Excavated and Natural Channels
$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m$$

Channel Conditions		Values
n_0 Material Involved	Earth	0.020
	React	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
n_1 Degree of Irregularity	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
n_2 Relative Effect of Channel Cross-section	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.013
n_3 Relative Effect of Obstructions	Negligible	0.000
	Minor	0.013
	Appreciable	0.025
	Severe	0.050
n_4 Vegetation	Low	0.008
	Medium	0.018
	High Very	0.038
	High	0.075
m Degree of Meandering	Minor	1.000
	Appreciable	1.150
	Severe	1.300

Source: Chow, V.T. Open Channel Hydraulics, 1959.

In selecting the value of n_4, the degree of effect of vegetation is considered in the following way:

A. <u>Low</u> for conditions comparable to the following:

1. dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples, where the average depth of flow is two (2) to three (3) times the height of vegetation; and
2. supple seedling tree switches, such as willow, cottonwood or salt cedar where the average depth of flow is three (3) to four (4) times the height of the vegetation.

B. **Medium** for conditions comparable to the following:

1. turf grasses where the average depth of flow is one (1) to two (2) times the height of vegetation; and
2. stemmy grasses, weeds or tree seedlings with moderate cover where the average depth of flow is two (2) to three (3) times the height of vegetation and brush growths, moderately dense, similar to willows one (1) to two (2) years old, dormant season, along side slopes of a channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than two (2) FT.

C. **High** for conditions comparable to the following:

1. turf grasses where the average depth of flow is about equal to the height of vegetation,
2. dormant season – willow or cottonwood trees eight (8) to ten (10) years old, intergrown with some weeds and brush, where none of the vegetation is in foliage, where the hydraulic radius is greater than two (2) FT; and
3. growing season - bushy willows about one (1) year old intergrown with some weeds in full foliage along side slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than two (2) FT.

D. **Very high** for conditions comparable to the following:

1. turf grasses where the average depth of flow is less than five tenths (0.5) the height of vegetation,
2. growing season - bushy willows about one (1) year old, intergrown with weeds in full foliage along side slopes, or dense growth of cattails along channel bottom, with any value of hydraulic radius up to ten (10) or fifteen (15) FT, and
3. growing season-trees intergrown with weeds and brush, all in full foliage, with any value of hydraulic radius up to ten (10) or fifteen (15) FT.

In selecting the value of m, the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for ratios of one (1) to one and two tenths (1.2), appreciable for ratios of one and two tenths (1.2) to one and five tenths (1.5), and severe for ratios of one and five tenths (1.5) and greater.

In applying the above method for determining the Manning's Roughness Coefficient, "n" value, several things should be noted. The method does not consider the effect of suspended and bed loads. The values given in Table 6-2 were developed from a study of some forty (40) to fifty (50) cases of small and moderate channels. Therefore, the method is questionable when applied to large channels whose hydraulic radii exceed fifteen (15) FT. The method applies only to unlined natural streams, floodways, and drainage channels and shows a minimum value of two one hundredths (0.02) for the Manning's Roughness Coefficient, "n" value of such channels. The minimum value of "n" in general, however, may be as low as twelve one thousandths (0.012), in lined channels and as low as eight one thousandths (0.008), in artificial laboratory flumes.

6.3.2 New or Altered Channels

The Manning's Roughness Coefficients, "n" for new or altered channels are shown in Table 6-2.

6.4.0 DESIGN REQUIREMENTS

Channel design involves the determination of the channel cross-section required to

accommodate a given design discharge. The design requirements for open channels are discussed in the sections below and apply to channels or waterways that are proposed to be modified or constructed.

6.4.1 Grass-Lined Channels and Waterways

Key parameters in grass-lined channel or waterway design include permissible velocity, roughness coefficient, side slope, curvature, bottom width, and freeboard. The grass species selected shall be suitable for permanent application based upon the anticipated operation and maintenance of the channel or waterway.

    A. **Velocity.** The maximum permissible velocity for the 100-yr storm is six (6) FPS and includes all transitions to or from channels and waterways with similar or different materials. In all cases, the velocity for the 100-yr storm must be non-erosive. The minimum permissible velocity for the 2-yr storm is two (2) FPS.

    B. **Roughness Coefficient.** The roughness coefficients selected shall be based on the degree of retardance of vegetation. Table 6-2 provides minimum Manning's Roughness Coefficients, "n" for channel design. The roughness coefficient shall be adjusted to reflect the relationship between the depth of flow and the typical height of the design vegetation, especially for shallow depths of flow, as well as other factors affecting channel conveyance.

    C. **Slope.** The flow line slope of the channel shall be a minimum of two (2) percent unless the velocity for the 2-yr storm flow is greater than two (2) FPS, in which case the channel slope may be a minimum of one (1) percent. Compliance with this requirement must take into account the variation in channel flow due to distributed inflows to the channel. A reinforced concrete pilot channel must be used if the channel slope is less than one (1) percent. The pilot channel must be at least four (4) FT wide, two (2) IN deep, and be capable of withstanding vehicular loading. Any grass-lined portion of the channel bottom must have a slope of at least two (2) percent from that portion to the concrete-lined pilot channel. However, no open channel flow line slope may be less than five tenths (0.5) percent.

    D. **Side Slopes.** Side slopes shall be four (4) FT horizontal to one (1) FT vertical or flatter for channels equal to or over four (4) FT deep and three (3) FT horizontal to one (1) FT vertical or flatter for channels less than four (4) FT deep.

    E. **Curvature.** The center line curvature shall have a minimum radius of twice the top width of the 100-yr storm flow.

    F. **Bottom Width.** The minimum flat bottom width of the channel is four (4) FT.

    G. **Freeboard.** All grass-lined channels shall be designed to convey the 100-yr storm event. The freeboard for the channel shall be the velocity head for the 100-yr storm.

6.4.2 Concrete-Lined Channels

Concrete-lined channels may be needed in channel reaches where the velocities are excessive (See Section 6.4.1A. of this Manual) or where the channel characteristics require such use.

    A. **Velocity.** In concrete-lined channels the probability of achieving supercritical flow is greatly increased. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel in considering the 25-yr and 100-yr storms. Flow with a Froude number equal to one (1) is unstable and should be avoided. If supercritical flow does occur, then freeboard and superelevation must be determined. In addition, all channels carrying supercritical flow shall be continuously lined with reinforced concrete.

    B. **Roughness Coefficient.** Table 6-2 provides the Manning's Roughness Coefficients, "n"

for concrete-lined channels.

C. **Freeboard.** Adequate channel freeboard shall be provided for the 100-yr storm in reaches flowing at critical depth by Equation 6-3 or using the energy grade line, whichever is less.

$$H_{FB} = 2.0 + 0.025V \, (d)^{1/3} \qquad \text{(Eq. 6-3)}$$

where,

H_{FB} = Freeboard height, FT
V = Velocity, FPS
d = Depth of flow, FT

Freeboard shall be in addition to superelevation, standing waves and/or other water surface disturbances. Concrete sideslopes shall be extended to provide freeboard. Freeboard shall not be obtained by the construction of levees.

D. **Superelevation.** Superelevation of the water surface shall be determined at all horizontal curves which deviate more than forty-five (45) degrees off the projected centerline. An approximation of the superelevation at a channel bend can be obtained from the following equation:

$$H = V_2 T_w / g r_c \qquad \text{(Eq. 6-4)}$$

where

H = Superelevation, FT
V_2 = Flow velocity, FPS
T_w = Top width of channel, FT
r_c = Centerline radius of curvature, FT
g = Acceleration due to gravity, 32.2 FT/SEC^2

The freeboard shall be measured above the superelevation water surface.

E. **Side Slopes.** Since concrete lined channels do not require slope maintenance, the side slopes may be as steep as vertical with appropriate structural methods applied.

F. **Slope.** The flow line slope of the channel shall be no less than five tenths (0.5) percent and must also be sufficient to produce a velocity for the 2-yr storm flow of at least two (2) FPS. Compliance with this requirement must take into account the variation in channel flow due to distributed inflows to the channel.

6.4.3 OTHER CHANNELS

Channels composed of materials other than vegetation or concrete shall be designed so that sediment deposition does not occur for the 2-yr storm (except for channel drop structures and energy dissipators as approved by the City) and velocities for the 100-yr storm are not erosive, using methods as approved by the City Engineer.

6.5.0 CHANNEL DROP STRUCTURES

The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes. Two (2) commonly used drop structures are shown in Figure 6-2 in Appendix B of this Manual.

The flow velocities in the upstream and downstream channels of the drop structure need to satisfy the permissible velocities allowed for channels. The design parameters for the sloping

channel drop and the vertical channel drop are given below.

6.5.1 Sloping Channel Drop

A. **Approach Apron.** A minimum ten (10) FT long riprap apron should be constructed immediately upstream of the drop to protect against the increasing velocities and turbulence which result as the water approaches the sloping portion of the drop structure. The same riprap and bedding design should be used as specified for the portion of the drop structure immediately downstream of the drop.

B. **Chute.** The chute shall have roughened faces and a slope shall be no steeper than two (2) FT horizontal to one (1) FT vertical. The length, L, of the chute depends upon the hydraulic characteristics of the channel and drop. For a unit discharge, q, of thirty (30) CFS/FT, L would be about fifteen (15) FT, that is, about five tenths (0.5) of the q value. The L should not be less than ten (10) FT, even for low q values.

C. **Downstream Apron.** The length of the downstream apron shall be sized according to Table 6-3 and shall be constructed of reinforced concrete or riprap depending on structural requirements.

Table 6-2 Minimum Roughness Coefficients of New or Altered Channels	
Type of Channel and Description	**Manning's Roughness Coefficients "n"**
1. Grass lined	
a. Bermuda (with regular mowing)	.040
b. St. Augustine (with regular mowing)	.045
c. Native grasses and vegetation not	.060
2. Concrete	
a. Concrete lined (rough finish)	.020
b. Concrete lined (smooth finish-culverts)	.015
c. Concrete rip-rap (exposed rubble)	.025
3. Gabion	.035
4. Rock-cut	.025
Source: 1. Chow, V.T. Open Channel Hydraulics. 1959. 2. WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual. 1984.	

Table 6-3 Length of Downstream Apron	
Maximum Unit Discharge, q (CFS/FT)	**Length of Downstream Apron, L_8 (FT)**
0-14	10
15	15
20	20
25	20
30	25
Source: City of Austin, Watershed Engineering Division.	

6.5.2 Vertical Channel Drops

The design criteria for the vertical channel drop is based upon the height of the drop and the

normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel.

The steepest allowable sideslope for the riprap stilling basin is four (4) FT horizontal to one (1) FT vertical. The riprap should extend up the side slopes to a depth equal to one (1) FT above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is four (4) FT from the upper channel bottom to the lower channel bottom.

A description of the drop structure and the design procedure, going from upstream to downstream, is given below and shown on Figure 6-2 in Appendix B of this Manual.

   A. **Approach Channel**: The upstream and downstream channels will normally be grass-lined trapezoidal channels.
   B. **Approach Apron**: A minimum ten (10) FT long riprap apron is provided upstream of the drop to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop.
   C. **Chute Apron**: The riprap stilling basin is designed to force the hydraulic jump to occur within the basin and is designed for essentially zero (0) scour.

6.6.0 ENERGY DISSIPATORS

Energy dissipators are used to dissipate excessive kinetic energy in flowing water that could promote erosion. An effective energy dissipator must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipators direct the water into an obstruction that diverts the flow in many directions and in this manner dissipates the energy in the flow. Baffled outlets and baffled aprons are two (2) impact-type energy dissipators.

Other energy dissipators use the hydraulic jump to dissipate the excess head. In this type of structure, water flowing at a higher than critical velocity is forced into a hydraulic jump, and energy is dissipated in the resulting turbulence. Stilling basins are this type of dissipator, where energy is diffused as flow plunges into a pool of water.

Generally, the impact-type of energy dissipator is considered to be more efficient than the hydraulic jump-type. Also the impact-type energy dissipator results in smaller and more economical structures.

The design of energy dissipators is based on the empirical data resulting from a comprehensive series of model structure studies by the U.S. Bureau of Reclamation, as detailed in its book Hydraulic Design of Stilling Basins and Energy Dissipators, 1984. Two (2) impact-type energy dissipators are briefly explained here.

6.6.1 Baffled Apron (U.S. Bureau of Reclamation Type IX)

Baffled aprons are used to dissipate the energy in the flow at a drop. They require no initial tailwater to be effective, although channel bed scour is not as deep and is less extensive when the tailwater forms a pool into which the flow discharges. The chutes are constructed on a slope that is two (2) FT horizontal to one (1) FT vertical or flatter and extends below the channel bottom. Backfill is placed over one (1) or more bottom rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur, the scour creates a stilling pool at the downstream end of the

chute, stabilizing the scour pattern. The simplified hydraulic design of the baffled apron is shown in Figure 6-3 in Appendix B of this Manual.

The general rules of hydraulic design of a baffled apron are as follows:

A. **Design Discharge.** The chute should be designed for the full capacity expected to be passed through the structure. The maximum unit discharge may be as high as sixty (60) CFS/FT for the 100-yr storm.

B. **Chute Entrance.** The flow entering into the chute should be well distributed laterally across the width of the chute. The velocity should be well below the critical velocity, preferably the value shown in the curve D of Figure 6-3 in Appendix B of this Manual. The curve C in Figure 6-3 in Appendix B of this Manual is the critical velocity in a rectangular channel.

$$V_c = (gq)^{1/3} \qquad\qquad\qquad (Eq.\ 6\text{-}4A)$$

where,

g = Acceleration due to gravity, 32.2 FT/SEC^2
q = Discharge per FT of width, CFS

C. **Chute Design.** The chute is usually constructed on a two (2) FT horizontal to one (1) FT vertical slope. The upstream end of the chute floor should be joined to the horizontal floor by a curve to prevent excessive vertical contraction of the flow. The upstream face of the first row should be no more than one (1) FT (vertically) below the high point of the chute.

Based on the results of U.S. Bureau of Reclamation experiments, the greatest tendency to overtop the training walls occurs in the vicinity of the second and third rows of baffles. To prevent this overtopping, a partial baffle (one third (1/3) to two thirds (2/3) of the width of a full baffle) should be placed against the training walls in the first row. This will place a space of the same width adjacent to the walls in the second row. Alternate rows are then made identical (i.e., rows 1, 3, 5, 7, etc., are identical; rows 2, 4, 6, 8, etc., are identical). Four (4) rows of baffles are necessary to establish the expected flow pattern at the base of the chute.

The height of the training walls on the chute should be three (3) or more times the baffle height, measured normal to the chute floor. Several rows of baffle piers are usually constructed below the channel grade to establish full control of the flow. At least one (1) row of baffles should be buried in the backfill which is used to restore the original bottom topography.

D. **Heights and Spacing of Baffle Pier.** Baffle pier height, H, should be about eight tenths (0.8) D_c to nine tenths (0.9) D_c, as shown in Curve B in Figure 6-3 in Appendix B of this Manual. D_c is the critical depth in a rectangular channel and determined by:

$$D_c = (q^2/g)^{1/3} \qquad\qquad\qquad (Eq.\ 6\text{-}5)$$

where,

g = Acceleration due to gravity, 32.2 FT/SEC^2
q^2 = Discharge per FT of width, CFS

Baffle pier widths and spaces should be equal, up to 1.5 H but no less than H. The slope distance between rows of baffle piers should be 2H, twice the baffle height.

6.6.2 Baffled Outlet
Baffled outlets are used to dissipate the discharge energy from flow in a pipe. They are normally used at outlets from detention ponds or storm drainage systems. The baffles are intended to decrease the discharge velocities and subsequent erosion of the receiving system.

6.7.0 STRUCTURE AESTHETICS
The design of hydraulic structures in the urban environment requires an approach not encountered elsewhere because appearance must be an integral part of the design. The treatment of the exterior appearance should not be considered of minor importance.

Parks. Hydraulic structures should not detract from the pleasures enjoyed in an urban park. Furthermore, parks and green belts may later be developed in an urban area in which the structure will play a dominant environmental role.

Play Areas. An important consideration is that drainage structures often are an attraction for neighborhood children. It is almost impossible to make drainage works inaccessible to children, and therefore what is constructed should be made as safe as is reasonably possible. Hazards to children's safety should be avoided whenever possible.

Concrete Surface Treatment. The use of textured concrete presents a pleasing appearance and removes form marks. Exposed aggregate concrete is also attractive but may require special control of the aggregate used in the concrete.

Rails and Fences. The use of rails and fences along concrete walls provides a pleasing topping to an otherwise stark wall, yet provides a safety measure against the hazard of falling from an unprotected wall.

6.8.0 SUPPLEMENTAL SECTION

6.8.1 Alternative New Channel Design
The following is a description of the cross-sectional characteristics of an alternative channel design to be applied at the Engineer's discretion but is in no way a requirement.

    A. A pilot channel designed to carry the 25-yr storm shall be calculated with Manning's Roughness Coefficient, "n" values in accordance with Tables 6-2 and 6-3. This channel is designed to separate the more frequent 25-yr storm via an unobstructed pilot channel. Side slopes of the pilot channel shall not exceed three (3) FT horizontal to one (1) FT vertical slope gradient and shall have a bottom width of no less than four (4) FT. The remaining cross-sectional area is designed to convey the additional storm flows up to the 100-yr storm. This upper platform will accommodate vegetation with minimal maintenance requirements.

    B. The ultimate 100-yr floodplain shall be contained within overbanks on each side of the pilot channel. These overbanks shall be a minimum width of ten (10) FT and have a slope gradient not to exceed six (6) FT horizontal to one (1).FT vertical The overbanks shall be stabilized with the seeds of grasses, native wildflowers and native woody

species appropriate to riparian habitat and with blanket products. In calculating Manning's Roughness Coefficient, "n" values for the overbanks, reference must be made to Tables 6-2 and 6-3 with the following assumptions:

1. Heavily wooded and brushy overbanks; and
2. Bank irregularities, which can be reasonably expected from occasional, moderate erosion.

Figure 6-4 in Appendix B of this Manual depicts the conceptual idea of the alternative channel design.

SECTION 7 - CULVERTS

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SECTION 7 - CULVERTS

7.1.0 GENERAL
The function of a drainage culvert is to pass the design storm flow without causing excessive backwater or overtopping of the structure and without creating excessive downstream velocities. The Designer shall keep energy losses and discharge velocities within allowable limits when selecting a structure that will meet these requirements. The design storm flow shall be determined by the hydrologic methods as set forth in Section 2 of this Manual. The system shall accommodate the runoff from a 100-yr storm meeting the limitations for overflows at bridges and culverts set forth in Sections 1.2.4C. and 1.2.4D. of this Manual.

7.2.0 CULVERT HEADWALLS

7.2.1 General
The normal functions of properly designed headwalls and endwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight-parallel, flared or warped. They may or may not require aprons, as determined by site conditions. Headwalls should be aligned with the direction of the receiving flow when discharging into a waterway. Precast headwalls and endwalls may be used if all other criteria are satisfied; generally precast headwalls/endwalls are available for smaller culverts eighteen (18) IN and twenty-four (24) IN diameters.

7.2.2 Conditions at Entrance
The operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Therefore, design of culverts must involve consideration of energy head losses that may occur at the entrance. Entrance head losses may be determined by the following equation:

$$h_e = K_e(V_2^2 - V_1^2)/2g \qquad\qquad (Eq.7-1)$$

where,

h_e = Entrance head loss, FT
V_2 = Velocity of flow in culvert, FPS
V_1 = Velocity of flow approaching culvert, FPS
K_e = Entrance loss coefficient as shown in Table 7-1
g = Acceleration due to gravity, 32.2 FT/SEC^2

7.2.3 Type of Headwall
The common types of headwall entrances are shown in Figure 7-1 in Appendix B of this Manual, but are not limited to the designs shown there. The following guidelines can be used in the selection of the type of headwall. Approach velocities are measured immediately upstream of the headwall under normal operating conditions.

Table 7-1
Values of Culvert Entrance Loss Coefficients in Pipes

Type of Entrance	Entrance Coefficient, K_e
Headwall (no wingwalls)	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall with 45° Wingwalls	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls	
Spaced 1.25D apart	0.30
Grooved edge	0.40
Square edge	0.25
Projecting Entrance (no headwall or wingwalls)	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin walls (CMP)	0.90
Sloping Entrance (no headwall or wingwalls)	
Mitered to conform to slope	0.70
Flared-end section	0.50
Headwall Parallel to Embankment (no wingwalls)	
Square edge on sides of opening	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel axis	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of culvert walls)	
Square edged at crown	0.70
RCP: Reinforced Concrete Pipe	
CMP: Corrugated Metal Pipe	
NOTE: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.	
Source: WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual, 1984	

A. Parallel Headwall.

1. Approach velocities are low, below six (6) FPS.
2. Backwater pools are permitted.

B. Flared Headwall.

1. Approach velocities are between six (6) and ten (10) FPS.
2. Ample right of way or easement is available.

The wings of flared walls should be located with respect to the direction of the approaching flow, not the culvert axis as in Figure 7-1 in Appendix B of this Manual.

C. Warped Headwall.

Approach velocities are between eight (8) and twenty (20) FPS. Warped headwalls are effective with aprons to accelerate flow through the culvert.

7.2.4 Debris Fins
For conditions where more than one (1) box culvert is required, the upstream face of the structure shall incorporate debris deflector fins to prevent debris buildup. For multiple-pipe situations installations of debris fins may be used but are not required.

The debris fin is an extension of the interior walls of a multiple-box culvert. The wall thickness shall be designed to satisfy structural requirements and reduce impact and turbulence to the flow.

A debris fin is constructed to the height of the culvert. A fin length of one and five tenths (1.5) times the height of the box culvert is required. Since the debris fins are subject to the same erosive forces as bridge piers, care must be taken in the design of the footing. A toewall at the upstream end of the debris fin and the apron is recommended.

Figure 7-2 in Appendix B of this Manual depicts the conceptual design for debris deflector fins. It should be noted that alternate types of wingwalls can be used other than the parallel design shown in Figure 7-2 in Appendix B of this Manual.

7.3.0 CULVERT DISCHARGE VELOCITIES

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

Velocity, FPS	Outlet Protection
Below 6	Riprap protection. (4 IN minimum thickness) or alternate approved material.
Above 6	Structurally reinforced apron, (6 IN minimum thickness) with toe wall.

The minimum apron length which provides transition from a culvert outlet to an open channel

shall be calculated from the following equation:

$$L = 0.2VD \qquad\qquad\qquad (Eq.\ 7\text{-}2)$$

where,

      L   = Apron length, FT
      V   = Culvert discharge velocity, FPS
      D   = Height of box culvert or diameter of pipe culvert, FT

7.4.0 SELECTION OF CULVERT SIZE AND FLOW CLASSIFICATION

Laboratory tests and field observations show that there are two (2) major types of culvert flow: (1) flow with inlet control; and (2) flow with outlet control. Under inlet control, the cross-sectional area of the barrel, the inlet configuration or geometry and the amount of headwater are the factors affecting capacity. Outlet control involves the additional consideration of the tailwater in the outlet channel and the slope, roughness and length of barrel. Under inlet control conditions, the slope of the culvert is steep enough so that the culvert does not flow full and the tailwater does not affect the flow.

7.4.1 Culvert Hydraulics

A. Inlet Control Condition.

Inlet control for culverts may occur in two (2) ways.

1. Unsubmerged: The headwater is not sufficient to submerge the top of the culvert opening and the culvert inlet slope is supercritical. The culvert inlet acts like a weir (Condition A, Figure 7-3 in Appendix B of this Manual).
2. Submerged: The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B, Figure 7-3 in Appendix B of this Manual).

The discharge capacity for several culvert materials, shapes, and inlet configurations under inlet control conditions are presented in Figures 7-5 to 7-10 in Appendix B of this Manual. These nomographs were developed empirically by the Bureau of Public Roads, the Federal Highway Administration and various pipe manufacturers. The nomographs are recommended for use in all inlet-control culvert calculations.

B. Outlet Control Condition.

There are three (3) types of outlet control culvert flow conditions:

1. The headwater submerges the culvert opening, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure 7-3 in Appendix B of this Manual).
2. The headwater submerges the culvert opening, the culvert outlet is not submerged by the tailwater (Condition B or C, Figure 7-3 in Appendix B of this Manual).
3. The headwater is insufficient to submerge the top of the culvert opening. The culvert slope is subcritical and the tailwater depth is lower than critical depth for the culvert (Condition D, Figure 7-3 in Appendix B of this Manual).

The capacity of a culvert for outlet control is calculated using Bernoulli's Equation, which is based on the conservation of energy principle. In the application of this equation, an energy balance is determined between the headwater at the culvert inlet and the tailwater at the culvert outlet. This balance is a function of inlet losses, friction losses and velocity head (See Figure 7-4 in Appendix B of this Manual).

Bernoulli's Equation:

$$d_1 + V_1^2/2g + LS_0 = TW + h_e + h_r + h_v \qquad \text{(Eq. 7-3)}$$

where,

d_1	= Approach Depth, FT
V_1	= Approaching Velocity, FPS
g	= Acceleration due to gravity, 32.2 FT/SEC^2
L	= Culvert length, FT
S_0	= Culvert slope, FT/FT
TW	= Tailwater depth, FT
h_e	= Head loss at the entrance, FT
h_r	= Heal loss due to friction, FT
h_v	= Velocity head, FT

The sum of the first two (2) terms on the left-hand side of Equation 7-3 is equal to the headwater (HW). That is:

$$HW = d_1 + V_1^2/2g \qquad \text{(Eq. 7-4)}$$

Substituting Equation 7-4 into Equation 7-3 and isolating the head losses on the right side results in the following equation:

$$HW + LS_0 - TW = h_e + h_r + h_v \qquad \text{(Eq. 7-5)}$$

From Figure 7-4 (in Appendix B of this Manual):

$$HW + LS_0 = H_L + TW \qquad \text{(Eq. 7-5a)}$$

Thus the total head loss can be determined from this relationship as shown in Equation 7-6:

$$H_L = HW + LS_0 - TW \qquad \text{(Eq.7-6)}$$

Substituting Equation 7-6 into Equation 7-5, the following results:

$$H_L = h_e + h_r + h_v \qquad \text{(Eq. 7-7)}$$

in which,

$$h_v = V^2/2g \qquad \text{(Eq. 7-8)}$$

For inlet losses, the governing equation is

Equation 7-1: $\quad h_e = K_e(V_2^2 - V_1^2)/2g$

From Equation 7-4, the headwater (HW) is above the actual depth by the velocity head of the approaching water. However, with water ponded at the entrance, this velocity head (V_1) is usually considered to be negligible, therefore,

$h_e \quad = K_e V^2/2g$ (Eq. 7-9)

where,

$K_e \quad$ = Entrance loss coefficient as shown in Table 7-1
$V \quad$ = Velocity of flow in culvert, FPS
$g \quad$ = Acceleration due to gravity, 32.2 FT/SEC^2

Friction loss is the energy required to overcome the roughness of the culvert material and is expressed as:

$h_f \quad = (29n^2L/R^{1.33})(V^2/2g)$ (Eq. 7-10)

where,

$n \quad$ = Manning's Roughness Coefficient
$L \quad$ = Length of culvert, FT
$R \quad$ = Hydraulic radius, FT
$V \quad$ = Velocity of flow in the culvert, FPS
$g \quad$ = Acceleration due to gravity, 32.2 FT/SEC^2

Combining Equations 7-7, 7-8, 7-9 and 7-10 and simplifying the terms results in the following equation:

$H \quad = (K_e + 1 + 29n^2L/R^{1.33})V^2/2g$ (Eq. 7-11)

Equation 7-11 can be used to calculate directly the capacity of the culvert flowing under outlet condition A or B in Figure 7-3 in Appendix B of this Manual. This is because conditions A and B have tailwater depths at or above the top of the culvert and conditions C and D have tailwater depths which are less than critical depth. The method for calculating headwater depth for conditions C and D is discussed in the following section.

C. Depths of Tailwater and Headwater.

In culverts flowing with outlet control, tailwater is an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience are needed to evaluate possible tailwater conditions during storms. A field inspection should be made to check on downstream controls and to determine water stages. Tailwater is often controlled by a downstream obstruction or by water stages in another stream.
An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's Equation in the channel with normal flow condition (see Section 6.2.1, Uniform Flow of this Manual). If the water surface in the outlet channel is established by

downstream controls, a backwater analysis is required (see Section 6.2.2, Gradually Varied Flow of this Manual).

The headwater depth can be calculated by the summation of head loss, tailwater depth and the elevation difference of the inlet and outlet, as shown in the following equation:

$$HW = H + h_0 - LS_0 \tag{Eq.7-12}$$

where,

HW = Vertical distance from flow line at the entrance to the pool surface, FT
H = Head loss, FT (use appropriate nomograph)
h_0 = Vertical distance from flow line at the outlet to the hydraulic grade line, FT (In this case h_0 equals TW, measured in FT above the flow line.)
L = Culvert length, FT
S_0 = Slope of barrel, FT/FT

Equation 7-12 has the same form shown in Equation 7-6, which was derived from Bernoulli's Equation. For a tailwater depth equal to or greater than the top of the culvert at the outlet (outlet control conditions A and B in Figure 7-3 in Appendix B of this Manual), h_0 can be set equal to TW and the headwater depth can be found by Equation 7-12. For tailwater elevation less than the top of the culvert at the outlet (outlet control conditions C and D in Figure 7-3 in Appendix B of this Manual), h_0 in Equation 7-12 will be assumed as:

$$h_0 = (d_c+D)/2 \text{ or TW, whichever value is greater} \tag{Eq. 7-13}$$

where,

d_c = Critical depth (d_c cannot exceed D), FT
D = Height of culvert opening, FT.

Headwater depth determined by Equations 7-12 and 7-13 becomes increasingly less accurate as the headwater computed by this method falls below the value of:

$$D + (1+K_e)V^2/2g.$$

A series of nomographs for various culvert materials and shapes have been developed by the Federal Highway Administration and the various pipe manufacturers. The nomographs presented herein include inlet control conditions (Figures 7-5 to 7-10 in Appendix B of this Manual) and outlet control conditions (Figures 7-11 to 7-17 in Appendix B of this Manual). The critical depth for pipes of different shapes are shown in Figures 7-16 to 7-22 in Appendix B of this Manual.

7.4.2 Design Procedures

The State Highway Department's THYSYS program can be used for culvert design in addition to help calculate the culvert size and related computations. Design procedures are as follows:

A. **Step 1.** List design data.

1. Design discharge Q, CFS
2. Approximate length L of culvert, FT
3. Slope of culvert, FT/FT

4. Allowable headwater depth, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert, FT
5. Allowable flow velocities in natural stream, FPS
6. Type of culvert for first trial selection, including material, cross-sectional shape and entrance type.

B. **Step 2.** Determine the first trial size culvert.
Since the procedure given is one of trial and error, the initial trial size can be determined by one of the following ways:

1. Make an arbitrary selection.
2. Use an approximating equation such as $Q/V = A$ assuming a V for the trial culvert.
3. Use inlet control nomographs for the culvert type selected (see Figures 7-5 to 7-10 in Appendix B of this Manual). If this method is used, an HW/D must be assumed. If any trial size is too large because of height restrictions or structure availability, multiple culverts may be used by dividing the discharge equally between the number of barrels used.

C. **Step 3.** Find headwater depth for trial size culvert assuming inlet control or outlet control.

1. Assuming Inlet Control
   a. Using the trial size from Step 2 above, find the headwater depth HW by use of the appropriate inlet control nomograph (Figures 7-5 to 7-10 in Appendix B of this manual). HW in this case is found by multiplying HW/D obtained from the nomograph by the height of the culvert D. Tailwater TW conditions are neglected in this determination.
   b. If HW is greater or less than the desired results, try another size until HW is acceptable for inlet control before computing HW for outlet control.
2. Assuming Outlet Control
   a. Determine the depth of tailwater (TW), FT, for the design flood condition at the outlet.
   b. For a TW elevation equal to or greater than the outlet soffit of the culvert, set h_o equal to the TW and find HW by Equation 7-12.
   c. For a tailwater elevation less than the outlet soffit of the culvert, find headwater HW by Equation 7-12 and Equation 7-13.
3. Compare the headwaters found in Step 3-1 and Step 3-2 above (Inlet Control and Outlet Control). The higher headwater governs and indicates the type of flow control for the given conditions and culvert size selected.

D. **Step 4.** If outlet control governs but the HW is too high select a larger culvert size and recalculate HW as instructed in Step 3-2 above. If the previously calculated inlet control governs, the smaller size is satisfactory as determined under Step 3-1 above.
E. **Step 5.** Compute the outlet velocity for the size selected and determine its compatibility with the criteria of Section 7.3.0 of this Manual. If the computed velocity is too high, go back to Step 2 above and select a larger culvert size.

1. If outlet control governs in Step 3-3 above, the outlet velocity equals Q/A_o, where A_o is the cross-sectional area of flow in the culvert at the outlet. If d_c or TW is less than the height of the culvert barrel, use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A

of the culvert barrel.

2. If inlet control governs in Step 3-3 above, outlet velocity can be assumed to equal mean velocity in open-channel flow in the barrel as computed by Manning's Equation for the rate of flow, barrel size, roughness and slope of culvert selected.

F. **Step 6.** Record final selection of culvert with size, type, required headwater and outlet velocity.

7.4.3 Instructions for Using Nomographs

A. **Inlet-Control Nomographs** (Figures 7-5 to 7-10 in Appendix B of this Manual).

1. To determine HW, given Q, and size and type of culvert:
   a. Connect with a straightedge the given culvert diameter or height D and the discharge Q, or Q/B for box culverts; mark intersection of straightedge on HW/D scale marked (1).
   b. If HW/D scale marked (1) represents entrance type used, read HW/D on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read HW/D.
   c. Compute HW by multiplying HW/D by D.
2. To determine discharge Q per barrel, given HW, and size and type of culvert.
   a. Compute HW/D for given conditions.
   b. Locate HW/D on scale for appropriate entrance type. If scale (2) or (3) are used, extend HW/D point horizontally to scale (1).
   c. Connect point on HW/D scale (1) as found in paragraph 2.(b) above and the size of culvert on the left scale. Read Q or Q/B on the discharge scale.
   d. If Q/B is read in (c) multiply by B (span of box culvert) to find Q.
3. To determine culvert size, given Q, allowable HW and type culvert.
   a. Using a trial size, compute HW/D.
   b. Locate HW/D on scale for appropriate entrance type. If scale (2) or (3) is used, extend HW/D point horizontally to scale (1).
   c. Connect point on HW/D scale (1) as found in paragraph 3.(b) above to given discharge and read diameter, height or size of culvert required for HW/D value.
   d. If D is not that originally assumed, repeat procedure with a new D.

B. **Outlet-Control Nomographs** (Figures 7-11 to 7-17 in Appendix B of this Manual). Outlet control nomographs can be used to solve Equation 7-11 for head H when the culvert barrel flows full for its entire length. They are also used to determine H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for HW, since they give only H in the equation $HW = H + h_0 - LS_0$.

1. To determine H for a given culvert and discharge Q:
   a. Select appropriate nomograph for type of culvert selected. Find K_e for entrance type from Table 7-1.
   b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales, follow the three (3) steps provided below.

   Step 1. If the "n" value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper K_e and locate the starting point at the

given culvert length. If a K_e curve is not shown for the selected K_e, see Step 2 below. If the "n" value for the culvert selected differs from that of the nomograph, see Step 3 below.

Step 2. For then of the nomograph and a K_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two (2) chart scales in proportion to the K_e values.

Step 3. For a different roughness coefficient n_1 than that of the chart "n," use the length scales shown with an adjusted length L_1, calculated by the following equation:

$$L_1 \quad =L(n_1/n)^2 \qquad\qquad \text{(Eq. 7-14)}$$

   c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line." See subsection 7.4.3A.2. below for size considerations for rectangular box culvert.

   d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in FT on the head H scale. For values beyond the limit of the chart scales, find H by solving Equation 7-13.

2. To use the box culvert nomograph (Figure 7-13 in Appendix B of this Manual) for full flow for other than square boxes:

   a. Compute cross-sectional area of the rectangular box.

   b. Connect proper point (see subsection 7.4.3A.1. above) on length scale to barrel area and mark point on turning line.

   c. The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 0.5D. For other box proportions use Equation 7-11 for more accurate results.

   d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in FT on the head H scale.

7.4.4 Example 7-1

The following example problem utilizes computation Tabie 7-2 for a culvert rating curve calculation.

Given: Culvert size   = 48 IN RCP
        length L   =110 FT
        n value   =0.012
        Inlet elevation   = 720.0 FT
        outlet elevation   = 718.8 FT
        slope S_0 =0.010
        Entrance condition (square edge), K_e = 0.50
        Maximum elevation for embankment   =732.0 FT.

Find: Culvert rating curve

Table 7-2 is used to take the computations for the culvert design.

        Step 1. List the elevations for headwater depths in Column 1. Then put headwater depth and ratio of headwater depth to culvert height (or pipe

diameter) in Column 2 and Column 3.

Step 2. Based on the inlet control conditions, the ratio of HW/D is used to find the flows Q which are put in Column 4. In this example, Item (B) in Figure 7-5 in Appendix B of this Manual is utilized.

Step 3. For outlet control conditions, the flow rate Q in Column 4 is used to determine the head loss H in Column 5. In this example, Figure 7-12 in Appendix B of this Manual is utilized.

Step 4. If the tailwater rating curve is available, the tailwater (TW) depth can be entered in Column 6. If the tailwater rating curve is not available, an estimate of the tailwater can be used.

Step 5. If the tailwater depth is less than the diameter of the culvert, Columns 7 and 8 should be calculated. If TW is larger than D, the TW value is entered in Column 9 for h_0.

Step 6. The critical depth (d_c) is found from Figures 7-18 to 7-22 in Appendix B of this manual, and then used to compile Column 8.

Step 7. The headwater depth (HW) now can be computed from Equation 7-12.

Step 8. Compare the two (2) headwater depth values from Column 2 and Column 10. The controlling headwater depth and type of control are recorded in Column 11 and Column 12, respectively. The calculated elevation is written in Column 14.

Step 9. The rating curve for the culvert can be plotted from the values in Column 4 and Column 13.

To size a culvert crossing, the same table can be used, with some variation in the basic data. First a design Q value is selected and the maximum allowable headwater is determined. An inlet type i.e., headwall is selected and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control then outlet control. If the controlling headwater exceeds the maximum allowable headwater, the input data is modified and the procedure repeated until the desired results are achieved.

7.5.0 HYDRAULIC CONSIDERATIONS IN BRIDGE DESIGN

7.5.1 General
Section 1.2.4C. and Section 1.2.4D. of this Manual state the City's position concerning storm water overtopping bridge structures. The current policy for overtopping of residential streets is a maximum of six (6) IN for the 100-yr frequency storm, and for any street other than residential, the allowable maximum is three (3) IN for the 100-yr frequency storm.

Several hydraulic parameters should be considered in bridge design. Among these considerations should be, but should not be limited to, the following:

A. Channel transitions into and out of the bridge opening.
B. Overall length and height of bridge.
C. Cross-sectional opening of bridge.
D. Location of the bridge opening relative to the main channel.
E. Bridge alignment relative to general flow of main channel i.e., "skewed" crossing.
F. Number of crossings or bridge openings.
G. Other obstructions to flow i.e., piers, abutments, deck width and clearances.
H. Design flows for bridge opening to pass.
I. Any freeboard requirements for channel design.

7.5.2 Types of Flow for Bridge Design

Three (3) types of flow caused by bridge construction on a flood plain are shown in Figure 7-23 in Appendix B of this Manual. The three (3) flow types are described below:

A. **Type I Flow.**

Referring to Item A of Figure 7-23 in Appendix B of this Manual, it can be observed that normal water surface is above critical depth at all points. This has been labeled Type I, or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between cross-sections 1 and 4.

B. **Type IIA Flow.**

There are at least two (2) variations of Type II flow which will be described here as Types IIA and IIB. For Type IIA flow, Item B of Figure 7-23 in Appendix B of this Manual, normal water surface in the unconstricted channel again remains above critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at cross-section 4).

C. **Type IIB Flow.**

The water surface for Type IIB flow, Item C of Figure 7-23 in Appendix B of this Manual, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction and then returns to normal. The return to normal depth can be rather abrupt as in Item C of Figure 7-23 in Appendix B of this Manual, taking place in the form of a poor hydraulic jump, since normal water surface in the stream is above critical depth.

D. **Type III Flow.**

In Type III Flow, Item D of Figure 7-23 in Appendix B of this Manual, the normal water surface is below critical depth at all points and the flow throughout is supercritical. This is an unusual case requiring a steep gradient but such conditions do exist, particularly in mountainous regions. Theoretically, backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, as indicated on Item D of Figure 7-23 in Appendix B of this Manual.

A more thorough and complete discussion of these parameters and preliminary design procedures are presented in Chapters 1 and 11 of Hydraulics of Bridge Waterways by U.S. Department of Transportation Federal Highway Administration, Second Edition, September, 1973.

7.5.3 Modeling Hydraulic Conditions

The most commonly used backwater program for modeling hydraulic conditions at existing or proposed bridge crossings is the U.S. Army Corps of Engineers HEC-RAS Water Surface Profiles Program. The normal and special bridge routines contained within the program are widely used. A thorough discussion of the application of these bridge routines are presented in the HEC-RAS User's Manual. HEC-RAS software is available from the U.S. Army Corps of Engineers Hydrologic Engineering Center.

Table 7-2 Calculation Table for Culvert Design

CALCULATION TABLE FOR CULVERT DESIGN

PROJECT:

TYPE =   So =   n =
INLET =   L =   Q FULL =
K_a =   LSo =   V FULL =

Total Losses

$H = h_e + h_f + h_v$

(1) $HW = H + ho - Lso$

(2) For TW < D; $ho = (d_c + D)/2$ or TW (Whichever is greater)

      For TW > D; $ho = TW$

(3) For Box Culvert; $d_c = 0.315 (D/8)^{2/3}$ less than or equal to D

| | INLET CONTROL | | | OUTLET CONTROL EQUATIONS | | | | | | | | Calcu-lated |
| | | | | | | TW < D | | TW > D | | Control- | | ated |
Elev.	HW	HW/D	Q	H	TW	dc	ho=(d_c+D)/2	ho	HW	led HW	Control	Elev.
1	2	3	4	5	6	7	8	9	10	11	12	13

Example Table 7-2 Calculation Table for Culvert Design

PROJECT: CALCULATION TABLE FOR CULVERT DESIGN
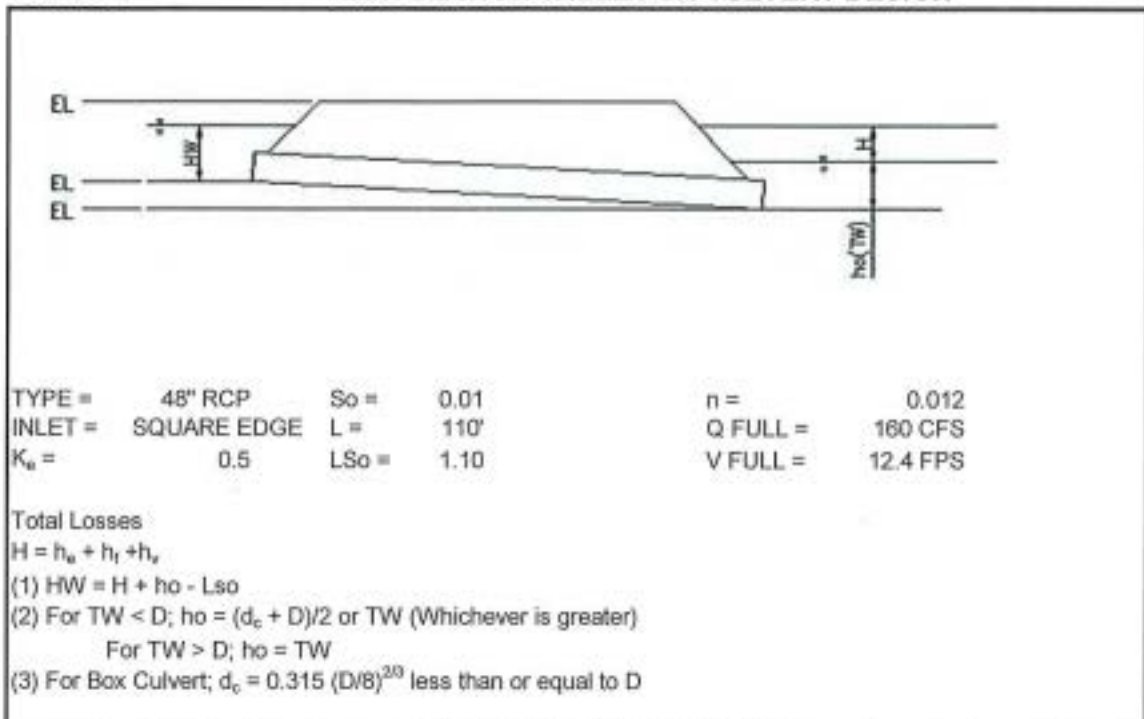
TYPE =	48" RCP	S_o =	0.01		n =	0.012
INLET =	SQUARE EDGE	L =	110'		Q FULL =	160 CFS
K_e =	0.5	LS_o =	1.10		V FULL =	12.4 FPS

Total Losses

$H = h_e + h_f + h_v$

(1) $HW = H + h_o - LS_o$

(2) For TW < D; $h_o = (d_c + D)/2$ or TW (Whichever is greater)

       For TW > D; h_o = TW

(3) For Box Culvert; $d_c = 0.315 (D/8)^{20}$ less than or equal to D

	INLET CONTROL			OUTLET CONTROL EQUATIONS									Calcul-ated
						TW < D		TW > D			Control-led HW	Control	
Elev.	HW	HW/D	Q	H	TW	dc	$h_o=(d_o+D)/2$	ho	HW			Elev.
1	2	3	4	5	6	7	8	9	10	11	12	13
720	0	0	0									720
724	4	1	70	0.94		2.55	3.28		3.12	4	INLET	724
725	5	1.25	95	1.72		2.9	3.45		4.07	5	INLET	725
726	6	1.5	115	2.6		3.2	3.6		5.1	6	INLET	726
728	8	2.0	150	4.5		3.6	3.8		7.2	8	INLET	728
730	10	2.5	170	5.8		3.85	3.93		8.63	10	INLET	730

SECTION 8 - STORMWATER MANAGEMENT

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SECTION 8 - STORMWATER MANAGEMENT

8.1.0 GENERAL

Stormwater Management (SWM) programs aimed at controlling increased urban runoff generated by development are a top priority in urban planning. More frequent flooding, increased rates and volumes of runoff, increased stream channel erosion and degradation, increased sedimentation and increased water pollution are all problems intensified by development. SWM facilities such as detention, retention, extended detention, infiltration, and sedimentation ponds have proven to significantly reduce downstream flooding, reduce sediment and pollutant loads, and provide debris removal which can benefit water quality.

The basic concept of SWM for peak rates of runoff is to provide for a temporary storage of stormwater runoff. Runoff is then released at a controlled rate which cannot exceed the capacities of the existing downstream drainage systems, or the predeveloped peak runoff rate of the site, whichever is less.

The solid lined hydrograph shown in Figure 8-1 in Appendix B of this Manual represents a storm runoff event without SWM, while the dashed line hydrograph depicts the same event with SWM. The peak flow of the undetained hydrograph could exceed the capacity of the downstream conveyance system and thereby cause surcharging and flooding problems. With the introduction of the SWM facility, the solid lined hydrograph is spread over a longer time period and its peak is reduced. The area between the two (2) curves to the left of their intersection represents the volume of runoff, temporarily stored or detained in the SWM facility.

The City approaches the control of excess flows through the application of both onsite and regional SWM. Essentially, the distinction between the two approaches is that onsite is generally limited to site specific criteria, while regional incorporates a basin wide hydrologic analysis.

8.2.0 REGIONAL STORMWATER MANAGEMENT PROGRAM

8.2.1 General

The Regional Stormwater Management Program (RSMP) provides for the planning, design and construction of regional drainage improvements, using fees paid by the owners of those developments. The RSMP uses a watershed-wide approach to analyze potential flooding problems, identify appropriate mitigation measures, and select site locations and design criteria for regional drainage improvements. These improvements include detention and retention ponds, waterway enlargement and channelization, and improved conveyance structures. The RSMP allows developers to participate in the program (in lieu of constructing onsite controls) if the resulting use of regional drainage improvements will produce no identifiable adverse impact to other properties due to increased runoff from the proposed development.

The fees charged for participation in the RSMP are non-refundable and are based upon the size of the development, and the proposed land use. The fees are deposited in a dedicated fund.

The benefits afforded by the RSMP include the following:

A. A higher level of confidence in the hydrologic analysis is obtained because each pond's interrelationship within a given basin can be readily determined. This is accomplished by establishing a hydrologic data base watershed master plan of the entire basin, and then using this to determine the most hydrologically efficient location for SWM facilities. This procedure takes into consideration the interrelated nature of tributary subareas within a watershed.

B. Adequate maintenance is more likely due to the City's vested interest and responsibility in the RSMP.

C. The cost of construction and the total land required can be considerably less than that needed for comparable onsite SWM

D. The expanded land area required for regional ponds lends itself to other uses (e.g., parks, nature areas, organized sports, etc.).

8.3.0 STORMWATER MANAGEMENT PONDS

8.3.1 General

Stormwater Management (SWM) ponds may be of two basic types:

A. Onsite
B. Regional.

In general, onsite ponds are those which are located off-channel and provide stormwater management for a particular project or development. Regional ponds are designed to provide stormwater management in conjunction with other improvements on a watershed-wide basis. SWM ponds may be further classified as retention or detention ponds and may incorporate water quality Best Management Practices (BMPs) such as sedimentation, infiltration, or filtration. The performance and safety criteria in this Section apply to all ponds which provide management of peak rates of stormwater runoff regardless of type.

8.3.2 Performance Criteria for Onsite SWM Ponds

A. Onsite SWM ponds are further classified as either small or large, as follows:

ONSITE SWM POND CLASS	DRAINAGE AREA, ACRES
Small	<25
Large	25-64

For design purposes, any pond with a drainage area larger than sixty-four (64) ACRES shall be classified as regional pond.

B. Onsite SWM ponds shall be designed to reduce post-development peak rates of discharge to existing pre-development peak rates of discharge for the 2-yr, 10-yr, and 25-yr storm events at each point of discharge from the project or development site. For the post-development hydrologic analysis, any off-site areas which drain to the pond shall be assumed to remain in the existing developed condition.

8.3.3 Performance Criteria for Regional SWM Ponds

A. Regional SWM ponds are classified as small and large based on the following criteria:

REGIONAL POND CLASS	IMPOUNDED VOLUME, ACRE-FT
Small	0-150
Large	>150

Any regional pond with a height of dam over fifteen (15) FT shall be classified as a large regional pond.

B. Performance criteria for regional ponds shall be determined by the Engineering and Development Services Department on a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by the Engineer.

8.3.4 Safety Criteria for SWM Ponds

All ponds shall meet or exceed all specified safety criteria. Use of these criteria shall in no way relieve the Engineer of the responsibility for the adequacy and safety of all aspects of the design of the SWM pond.

A. The spillway, embankment, and appurtenant structures shall be designed to safely pass the design storm hydrograph with the freeboard shown in the table below. All contributing drainage areas, including onsite and offsite areas, shall be assumed to be fully developed. Any orifice with a dimension smaller than or equal to twelve (12) IN shall be assumed to be fully blocked. For all spillways (especially enclosed conduits), the ability to adequately convey the design flows must take into account any submergence of the outlet, any existing or potential obstructions in the system and the capacity of the downstream system. For these reasons, enclosed conduit spillways connecting directly to other enclosed conduit systems are discouraged. If used, they must be justified by a rigorous analysis of all enclosed conduit systems connected to the spillway.

DETENTION POND CLASS	DESIGN STORM EVENT	FREEBOARD TO TOP OF EMBANKMENT, FT

Onsite:	Small	25-yr	0
	Large	25-yr	1.0
Regional:	Small	100-yr	2.0
	Large	100-yr	*

*Design storm event and required freeboard for large regional ponds shall be determined by a dam break analysis based on the principles outlined in Title 30, Part 1, Chapter 299 of the Texas Administrative Code. The dam break analysis shall be submitted to the Engineering and Development Services Department for approval.

B. If an embankment is classified as a dam pursuant to Title 30, Part 1, Chapter 299 of the Texas Administrative Code, all design criteria found in Title 30, Part 1, Chapter 299 of the Texas Administrative Code must be met, as evidenced by certification by a Professional Engineer licensed in the State of Texas.

C. **Intentionally left blank**.

D. The minimum embankment top width of earthen embankments shall be as follows:

TOTAL HEIGHT OF EMBANKMENT, FT	MINIMUM TOP WIDTH, FT
0-5	7
5-15	15
15 +	*

*To be determined on a case by case basis by the City Engineer.

E. The constructed height of an earthen embankment shall be equal to the design height plus the amount necessary to ensure that the design height will be maintained once all settlement has taken place. This amount shall in no case be less than five (5) percent of the total fill height. All earthen embankments shall be compacted to ninety-five (95) percent of maximum density, using standard proctor.

F. Earthen embankment side slopes shall be no steeper than four (4) FT horizontal to one (1) FT vertical. Slopes must be designed to resist erosion to be stable in all conditions, and to be easily maintained. Earthen side slopes for regional facilities shall be designed on the basis of appropriate geotechnical analyses.

G. Detailed hydraulic design calculations shall be provided for all SWM ponds. Stage-discharge rating data shall be presented in tabular form with all discharge components, such as orifice, weir, and outlet conduit flows, clearly indicated. Stage-storage table shall also be provided. In all cases, the effects of tailwater or other outlet control considerations should be included in the rating table calculations.

H. When designing ponds in series (i.e., when the discharge of one (1) becomes the inflow of another), the Engineer must submit a hydrologic analysis which demonstrates the system's adequacy.

This analysis must incorporate the construction of hydrographs for all inflow and outflow components.

I. No outlet structures from detention, filtration and/or sedimentation ponds, parking detention or other concentrating structures shall be designed to discharge concentrated flow directly onto arterial or collector streets. Such discharges shall be conveyed by a closed conduit to the nearest existing storm sewer. If there is no existing storm sewer within three hundred (300) FT, the outlet design shall provide for a change in the discharge pattern from concentrated flow back to sheet flow, following as near as possible the direction of the gutter.

J. Storm runoff may be detained within parking lots. However, the Engineer should be aware of the inconvenience to both pedestrians and traffic. The location of ponding areas in a parking lot should be planned so that this condition is minimized. Stormwater ponding depths (for the 100-yr storm) in parking lots are limited to an average of eight (8) IN with a maximum of twelve (12) IN. Maximum depths shall be permitted only in overflow parking areas not typically used in daily business.

K. All pipes discharging into a public storm sewer system shall have a minimum diameter of eighteen (18) IN and shall be constructed of reinforced concrete. In all cases, ease of maintenance and/or repair must be assured.

L. All concentrated flows into a SWM pond shall be collected and conveyed into the pond in such a way as to prevent erosion of the side slopes. All outfalls into the pond shall be designed to be stable and non-erosive.

8.3.5 Outlet Structure Design
There are two basic types of outlet control structures: those incorporating orifice flow and those incorporating weir flow. Rectangular and V-notch weirs are the most common types.

Generally, if the crest thickness is more than sixty (60) percent of the nappe thickness, the weir should be considered broad-crested. The coefficients for sharp-crested and broad-crested weirs vary. The respective weir and orifice flow equations are as follows:

A. **Rectangular Weir Flow Equation** (See Figure 8-2 in Appendix B of this Manual)

$$Q = CLH^{3/2}$$
(Eq. 8-1)

where,

Q = Weir discharge, CFS
C = Weir Coefficient
L = horizontal length, FT
H = Head on weir, FT

B. **V - notch Weir Flow Equation** (See Figure 8-2 in Appendix B of

this Manual)

$$Q = C_v \tan(O/2)H^{2.5}$$ (Eq. 8-2)

where,

Q = Weir Flow, CFS
C_v = Weir Coefficient
O = Angle of the weir notch at the apex, degrees
H = Head on Weir, FT

C. Orifice flow equation (See Figure 8-2 in Appendix B of this Manual)

$$Q = C_oA(2gH)^{0.5}$$ (Eq. 8-3)

where,

Q = Orifice Flow, CFS
C_o = Orifice Coefficient (use 0.6)
A = Orifice Area, FT^2
g = Gravitation constant, 32.2 FT/SEC^2
H = Head on orifice measured from centerline, FT

Analytical methods and equations for other types of structures shall be approved by the SWMD prior to use.

In all cases, the effects of tailwater or other outlet control considerations should be included in the rating table calculations.

8.4.0 DETENTION POND STORAGE DETERMINATION

A flow routing analysis using detailed hydrographs must be applied for all detention pond designs. The Soil Conservation Service (SCS) hydrologic methods (available in TR-20, TR-55, and HEC-1) and the Hydrologic Engineering Center (HEC) hydrologic methods may be used. The Engineer may use other methods but must have their acceptability approved by the City Engineer.

APPENDIX A

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APPENDIX A

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APPENDIX B. FIGURE AND DIAGRAMS

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FIGURES FROM SECTION 6

FIGURES FROM SECTION 7

FIGURES FROM SECTION 8

Figure 2-1 Effects of Urbanization on Flood Hydrograph
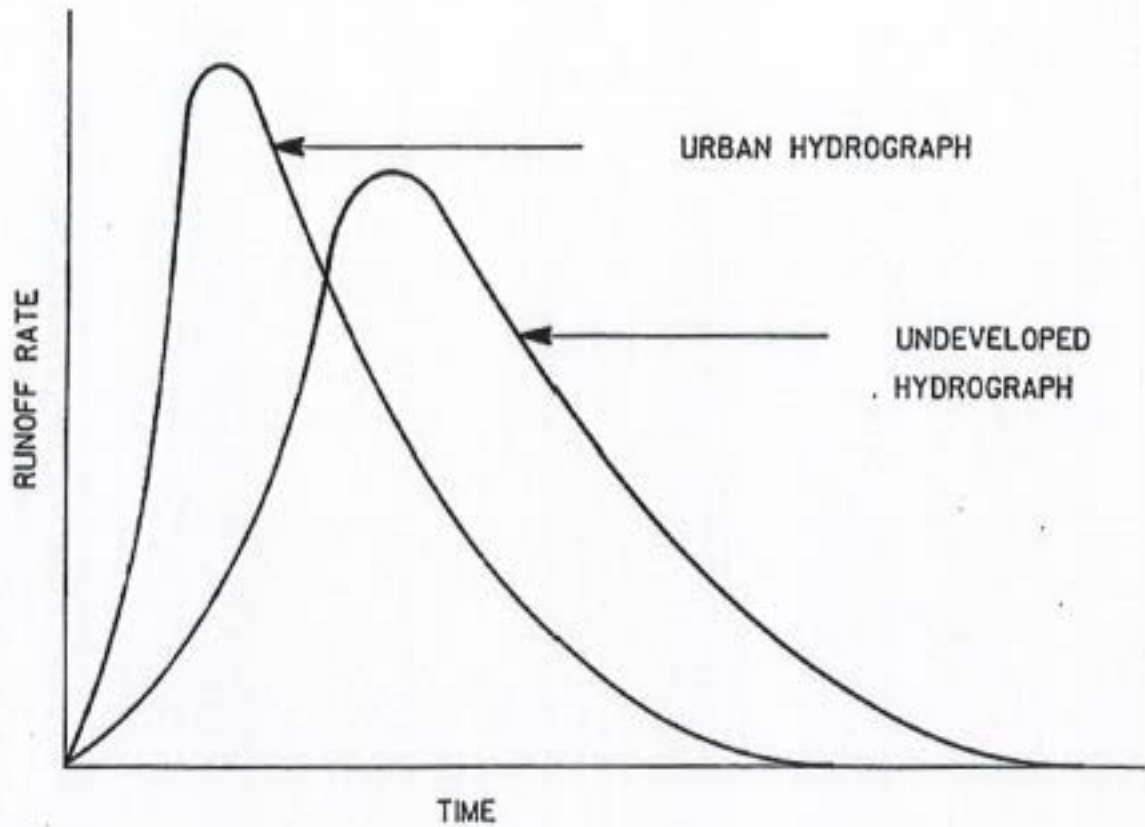
Source: City of Austin, Watershed Management Division

Figure 2-2 Intensity Duration Frequency Curve for Gatesville, TX
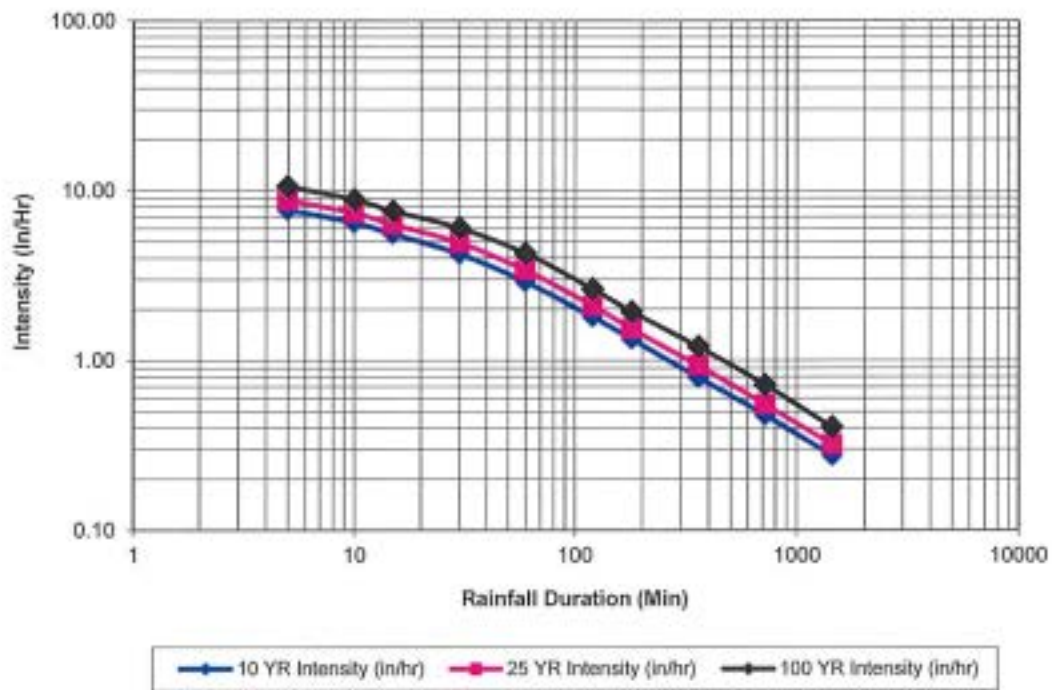
Figure 2-3 Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph
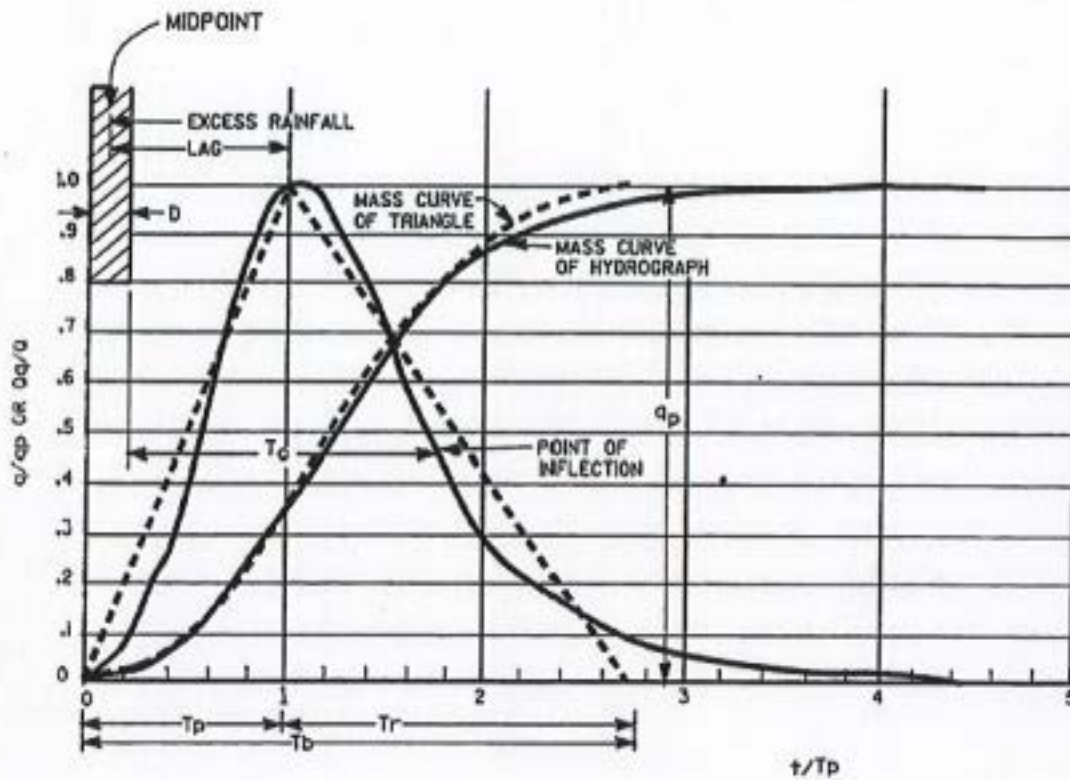
Source: U.S. Soil Conservation Service, "Hydrology Section 4," National Engineering Handbook. (NEH-4), 1972.

Figure 3-1 Nomograph for Flow in Gutters
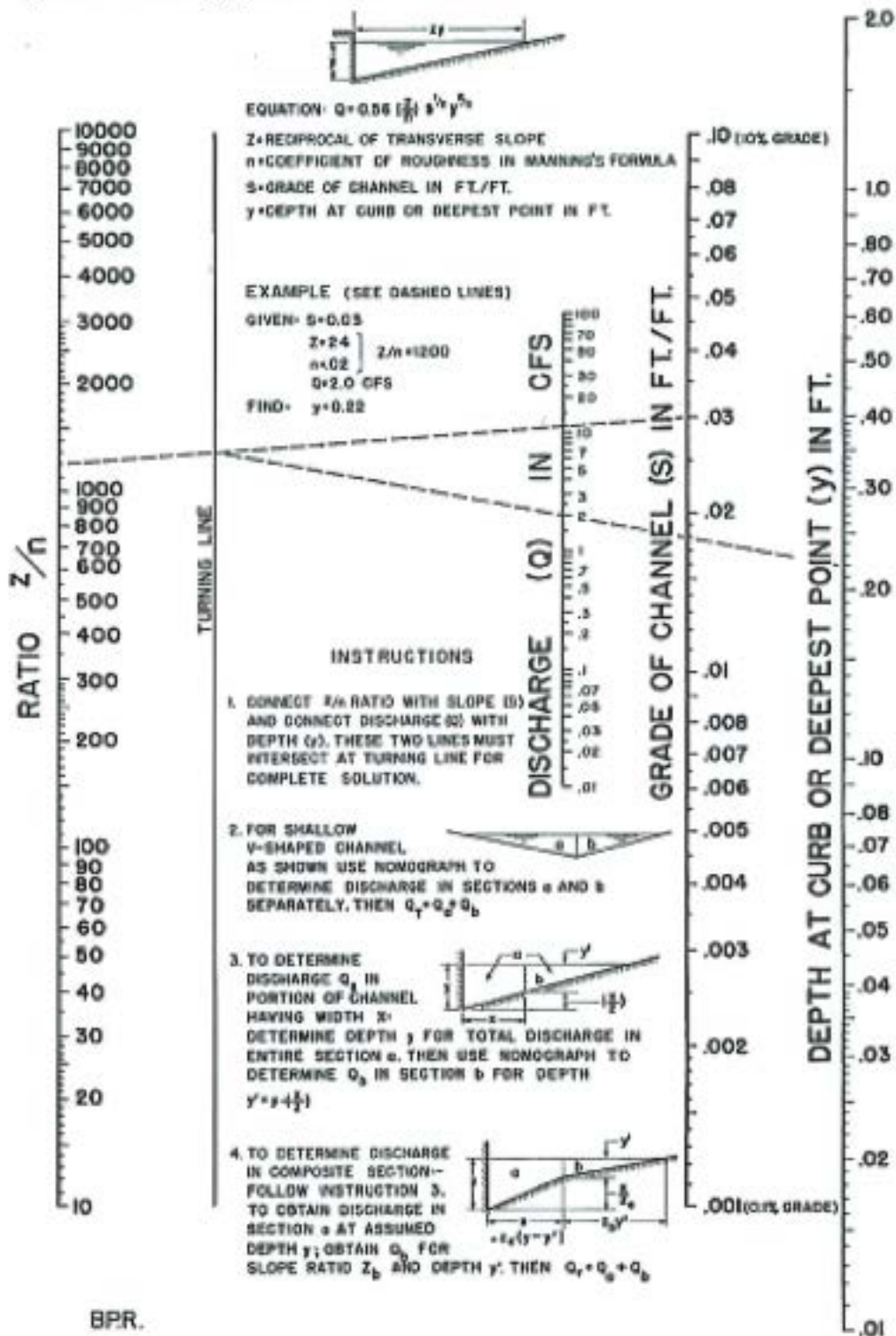
Source: Texas Highway Department, Bridge Division

Figure 4-1 Curb Opening Inlet in a Sump (Type S-1)
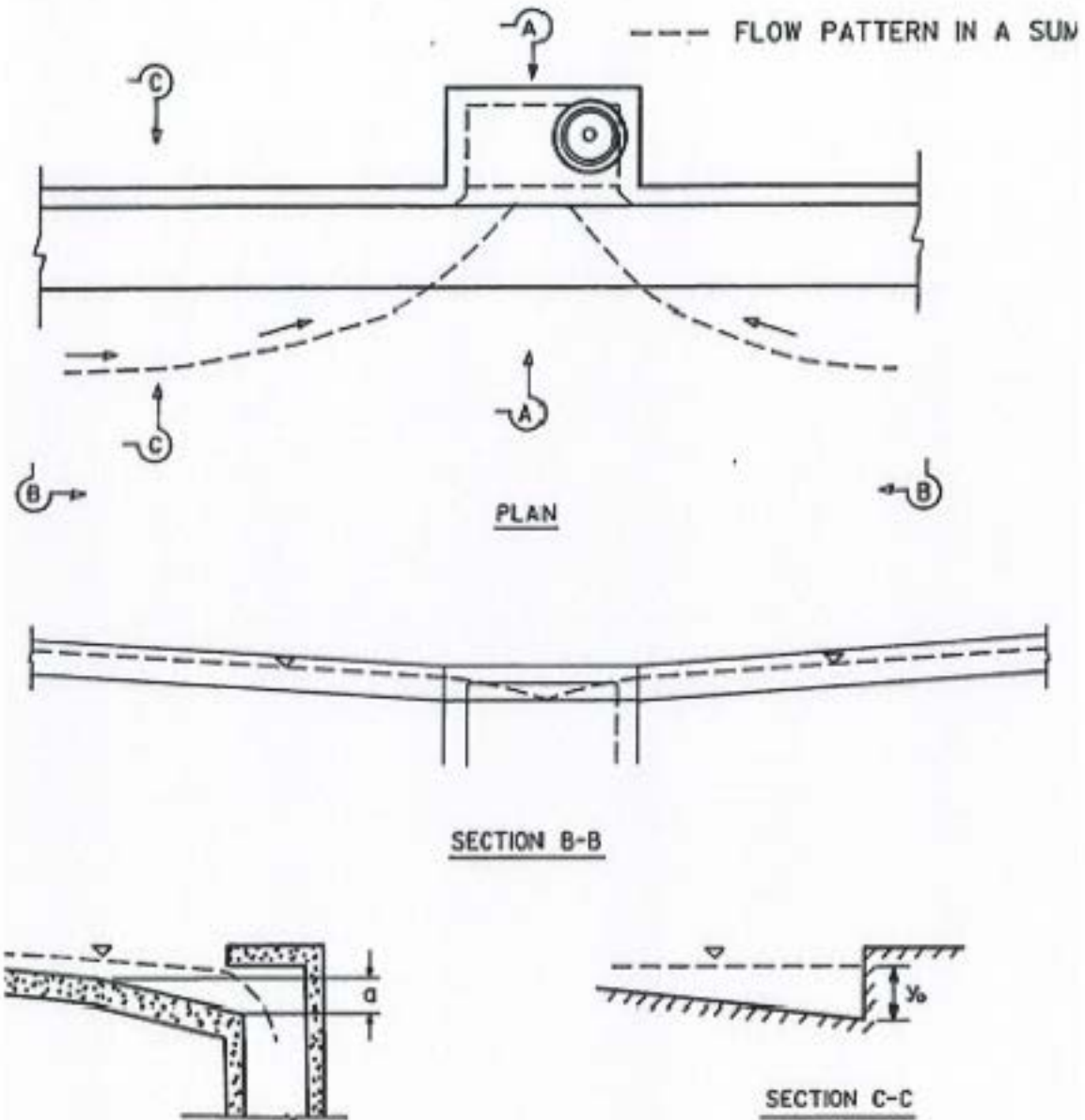
PLAN

SECTION B-B

SECTION A-A

SECTION C-C

y_0: WATER DEPTH IN THE APPROACH GUTTER

a: GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

Figure 4.2 Grate Inlet in a Sump (Type S-2)
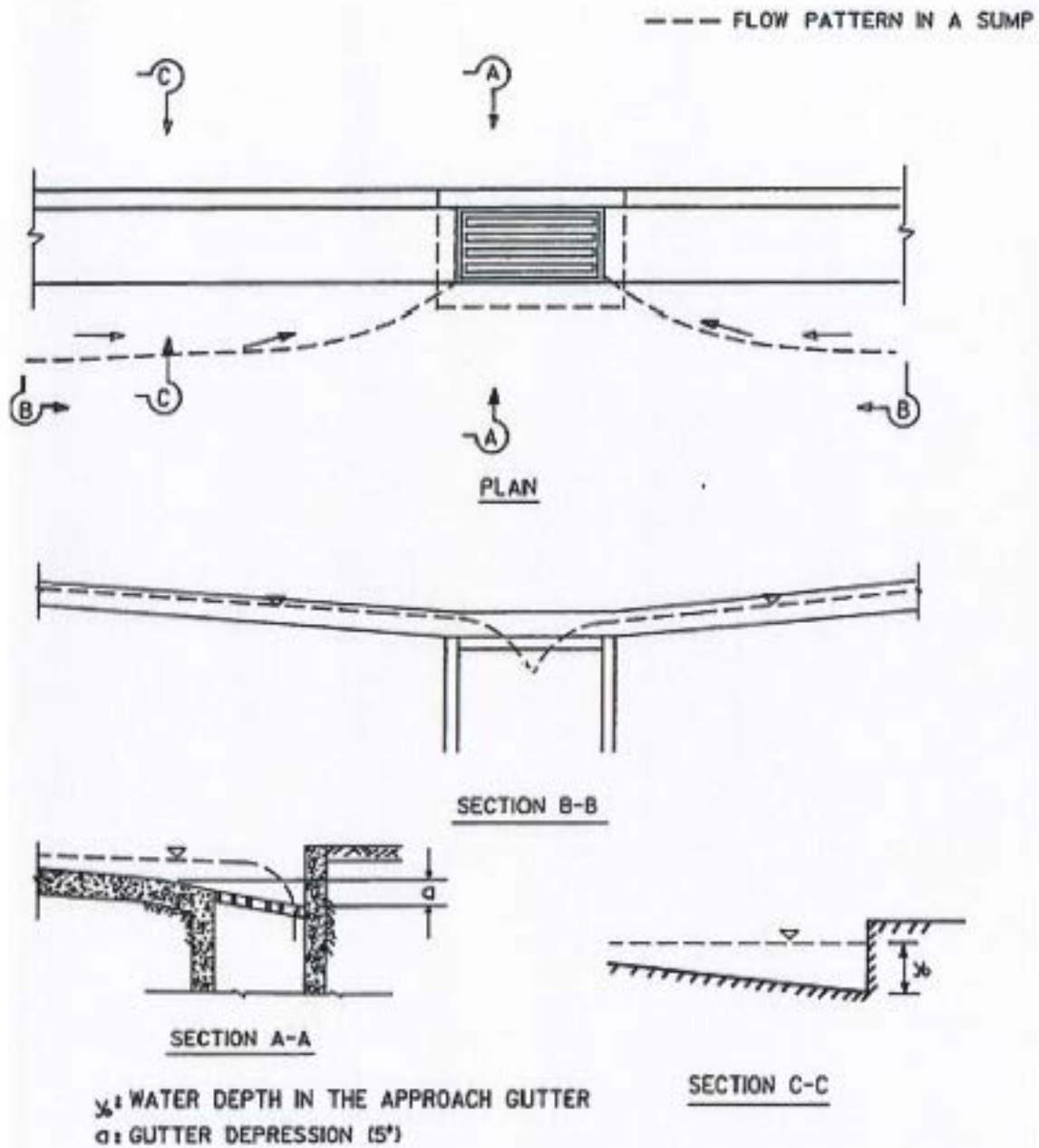
γ : WATER DEPTH IN THE APPROACH GUTTER
a : GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works.
Austin, Texas. January 1977.

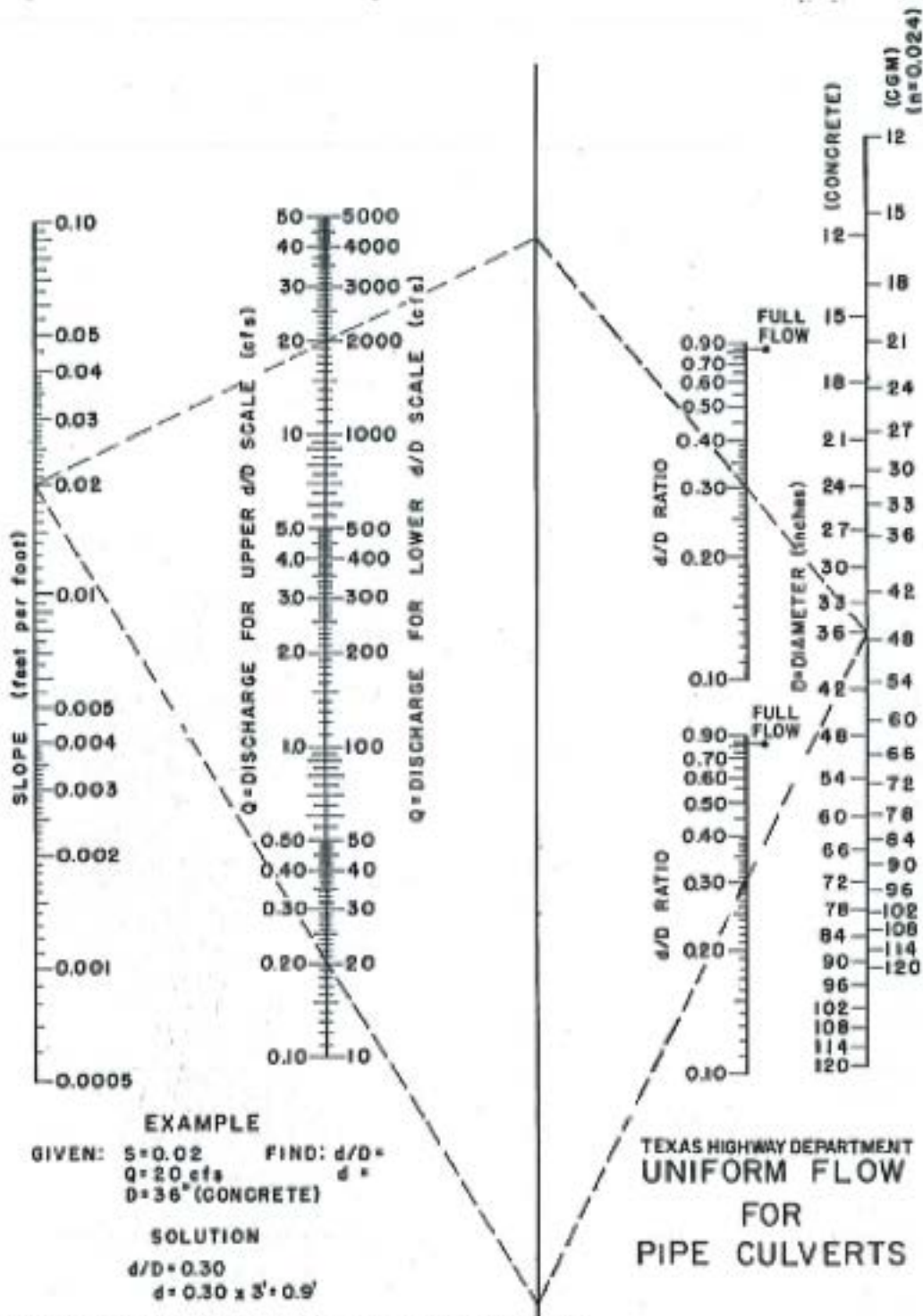# Figure 4-3 Combination Inlet in a Sump (Type S-3)
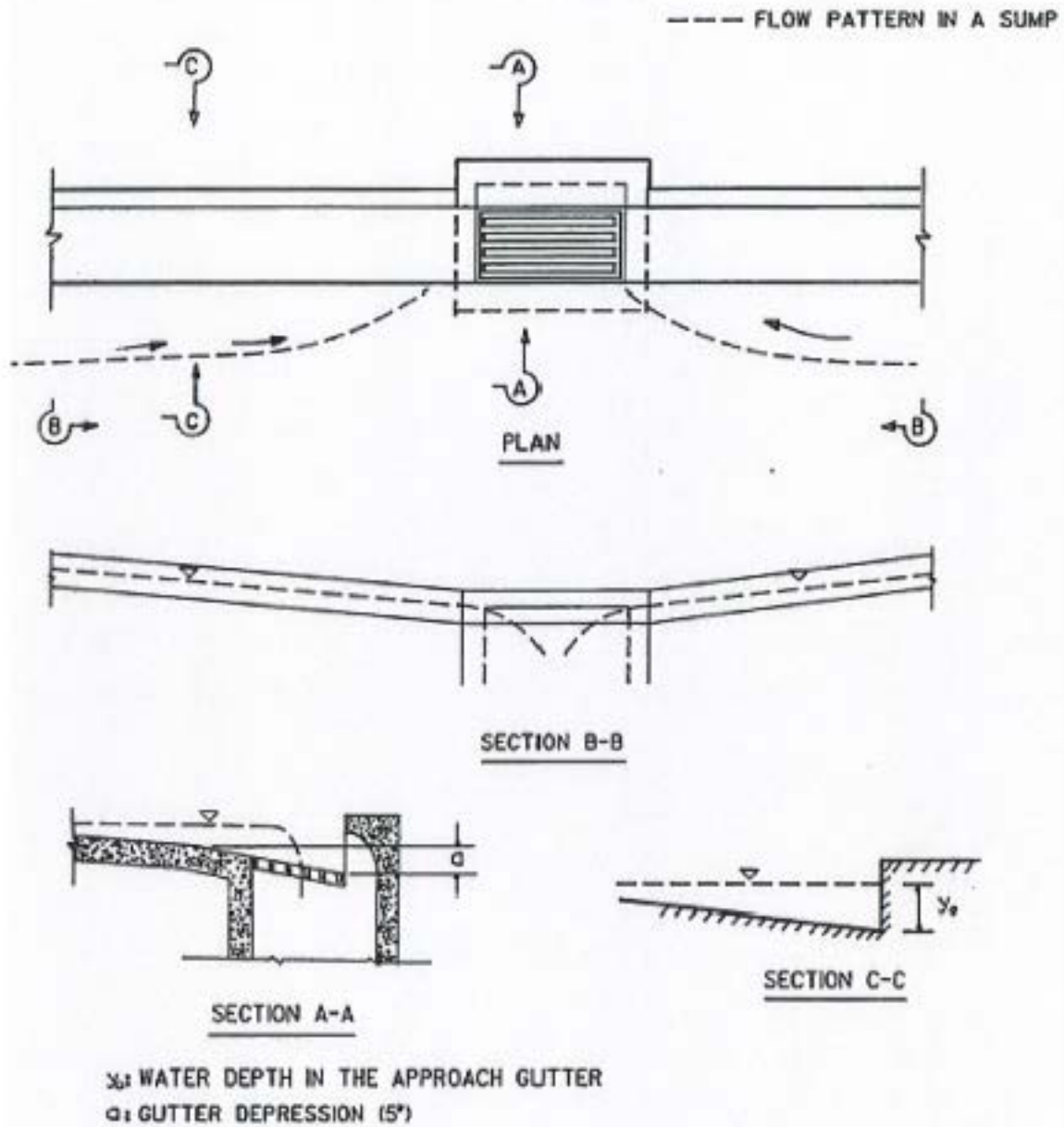
– – – – FLOW PATTERN IN A SUMP

PLAN

SECTION B-B

SECTION A-A

SECTION C-C

y_0: WATER DEPTH IN THE APPROACH GUTTER

a: GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works.
Austin, Texas. January 1977.

Figure 4-4 Area Inlet Without Grate (Type S-4)
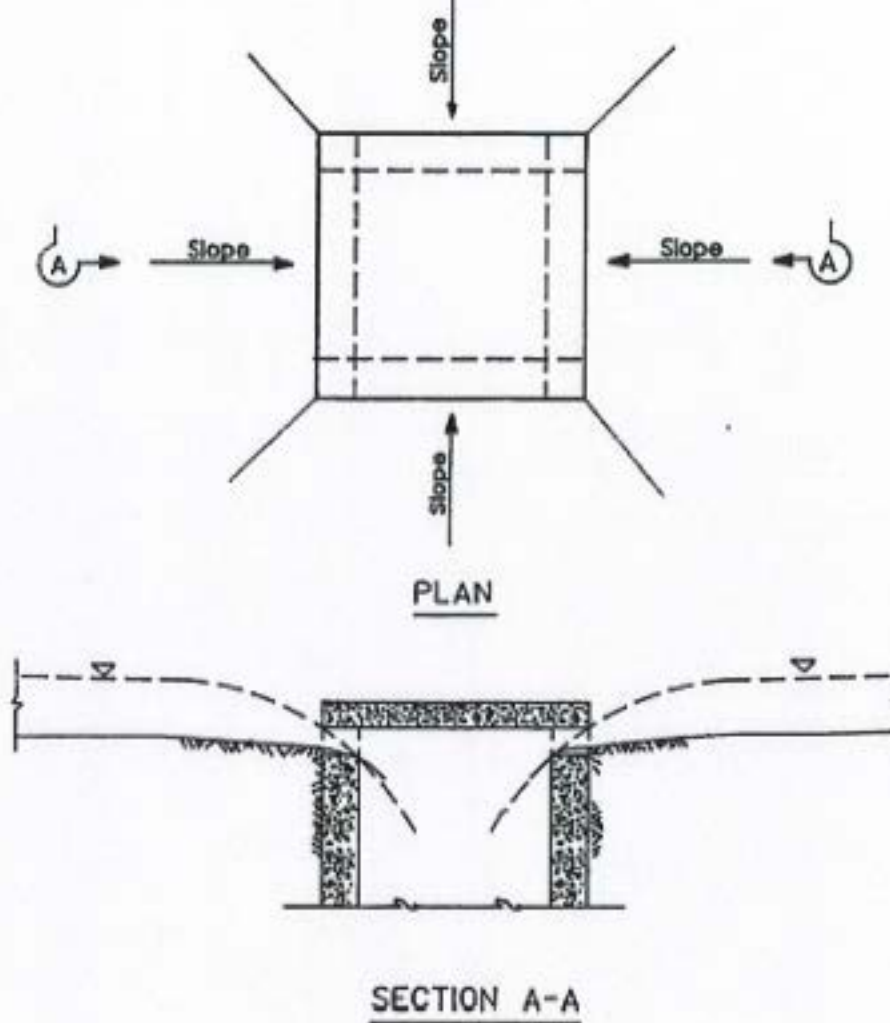
PLAN

SECTION A-A

Source: City of Austin, Drainage Criteria Manual, Department of Public Works. Austin, Texas. January 1977.

Figure 4-5 Curb Opening, Inlet on Grade (Type G-1)
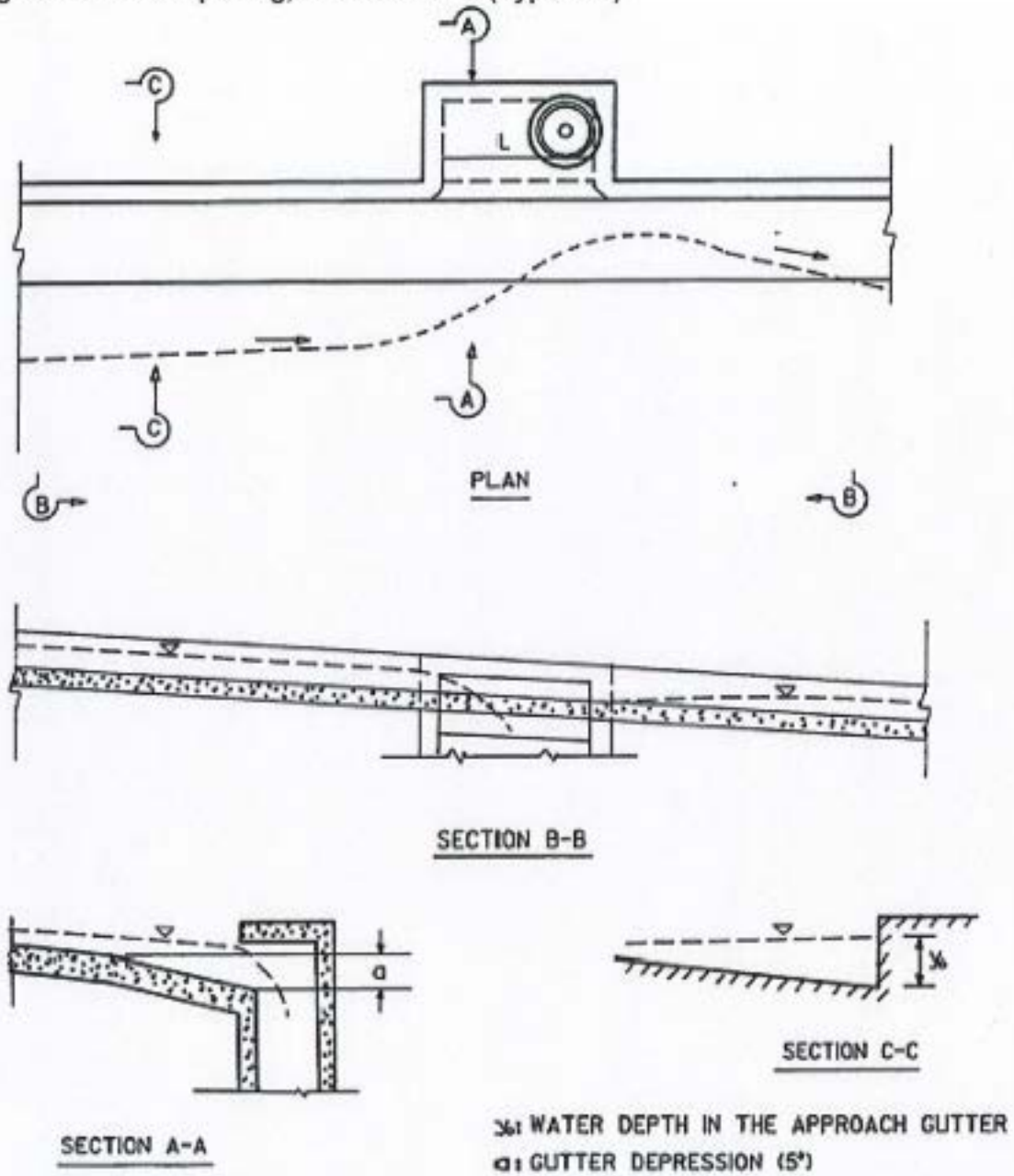
PLAN

SECTION B-B

SECTION A-A

SECTION C-C

𝑌ₒ: WATER DEPTH IN THE APPROACH GUTTER

a: GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

Figure 4-6 Grate, Inlet on Grade (Type G-2)
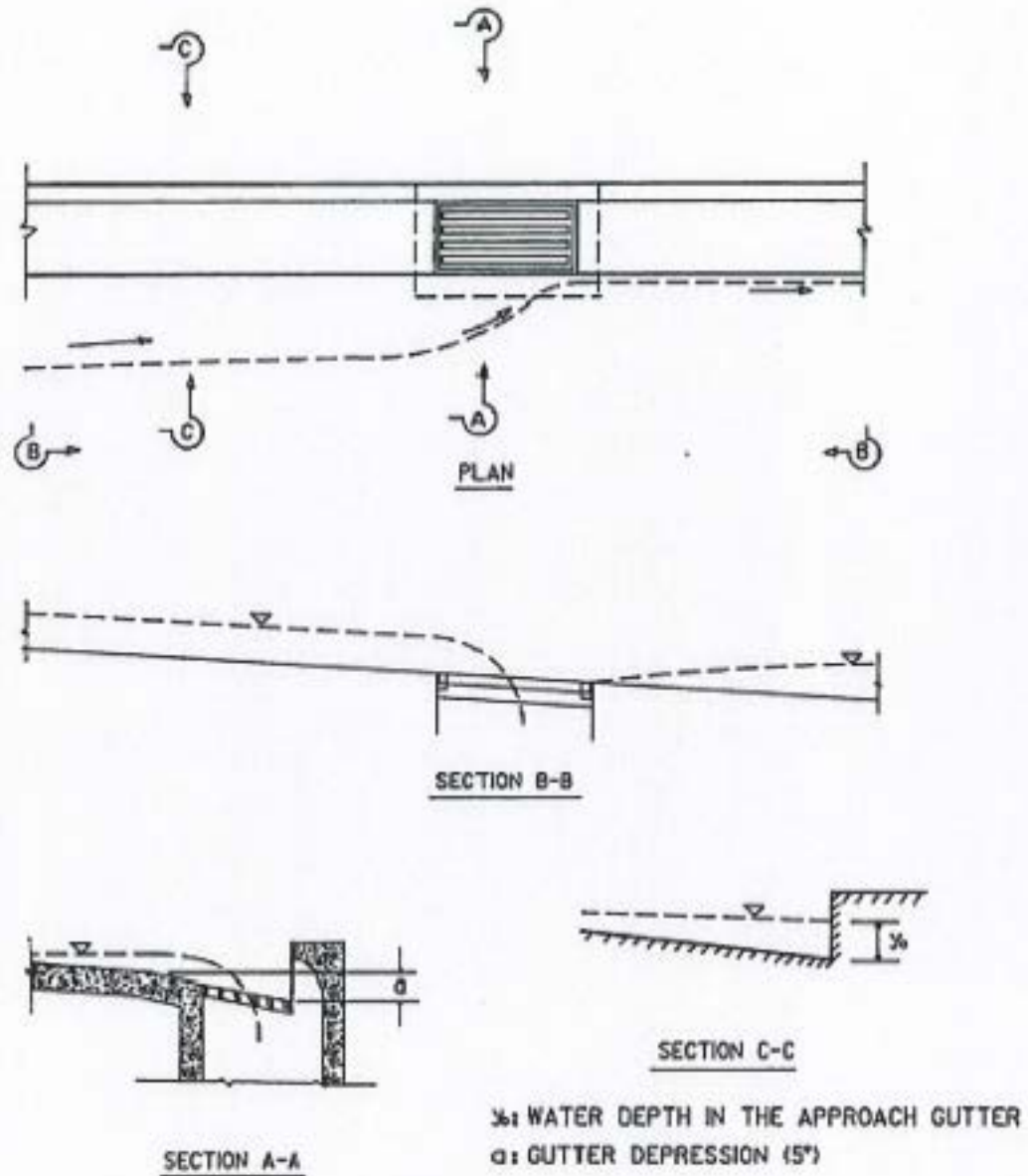
PLAN

SECTION B-B

SECTION A-A

SECTION C-C

$\%$: WATER DEPTH IN THE APPROACH GUTTER

a: GUTTER DEPRESSION (5")

Source: City of Austin, Drainage Criteria Manual, Department of Public Works, Austin, Texas, January 1977.

Figure 4-7 Combination Inlet on Grade (Type G-3)
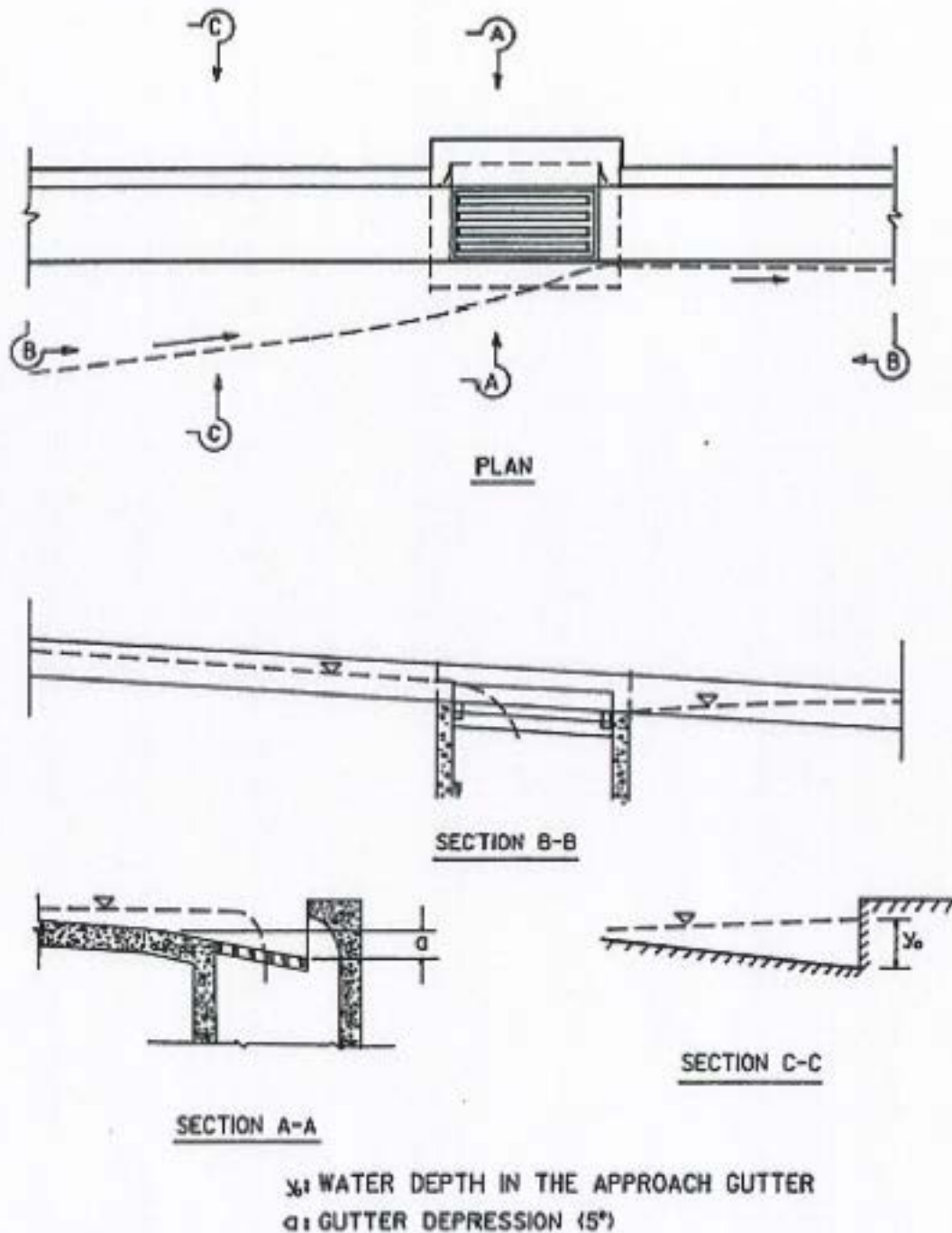
PLAN

SECTION B-B

SECTION A-A

SECTION C-C

y_0: WATER DEPTH IN THE APPROACH GUTTER
a: GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

Figure 4-8 Inlet Capacity for Type S-1 and S-3
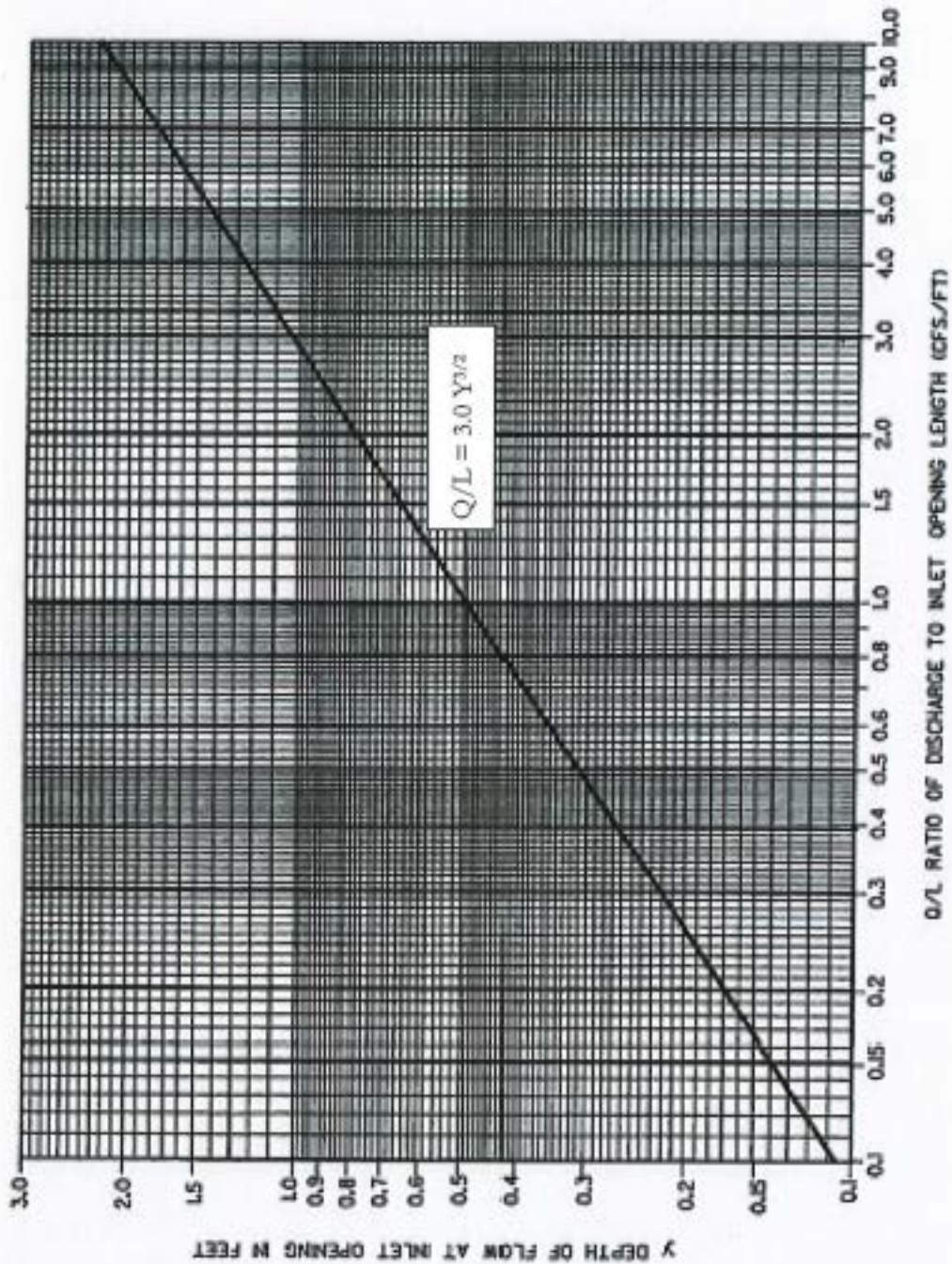
$$Q/L = 3.0 \, Y^{3/2}$$

Q/L RATIO OF DISCHARGE TO INLET OPENING LENGTH (CFS/FT)

y DEPTH OF FLOW AT INLET OPENING W FEET

Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

Figure 4-9 Inlet Capacity for Type S-2
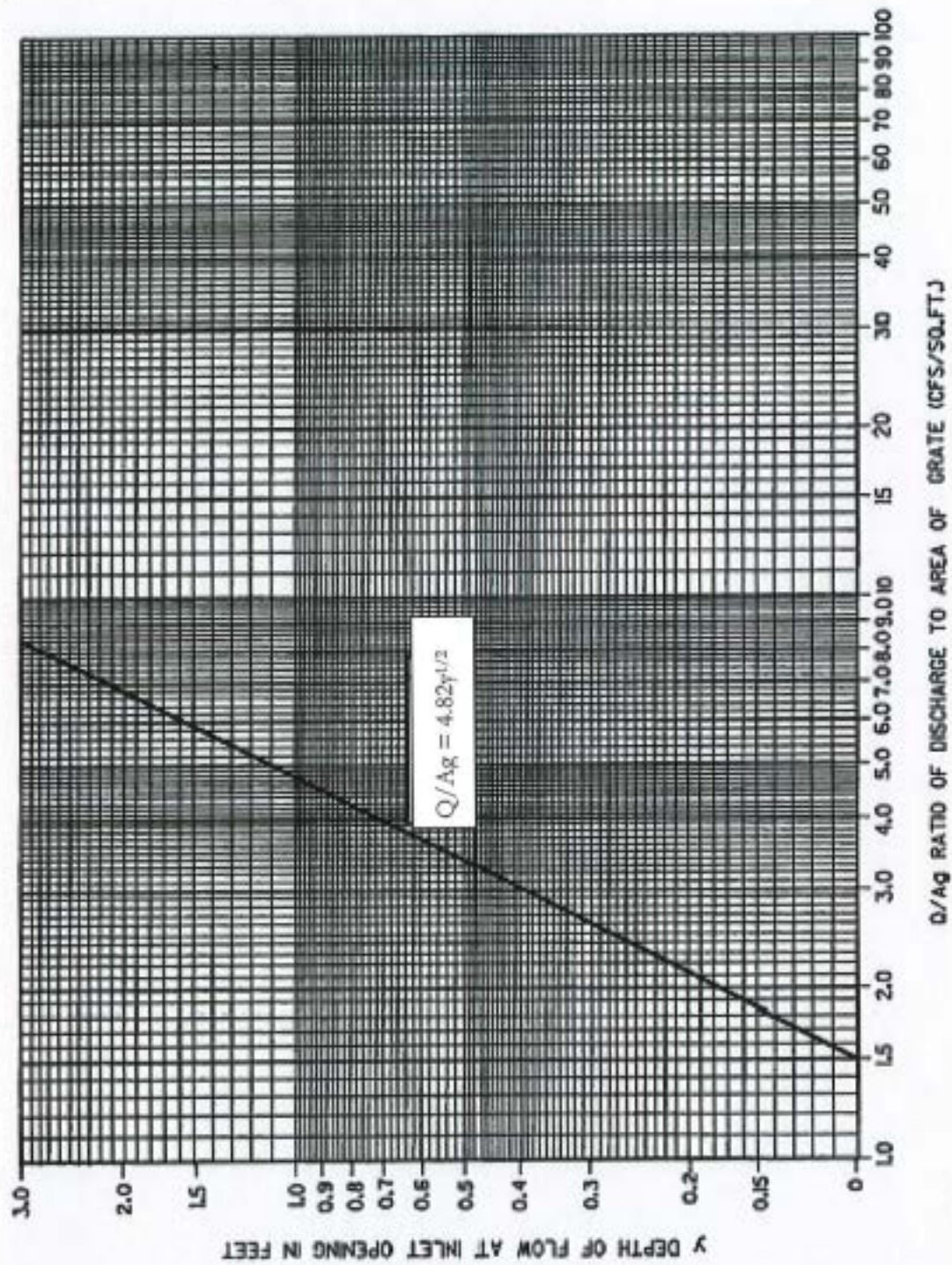
$$Q/A_g = 4.82y^{1/2}$$

y DEPTH OF FLOW AT INLET OPENING IN FEET

Q/Ag RATIO OF DISCHARGE TO AREA OF GRATE (CFS/SQ.FT.)

Figure 4-10 Capacity for Inlets on Grade
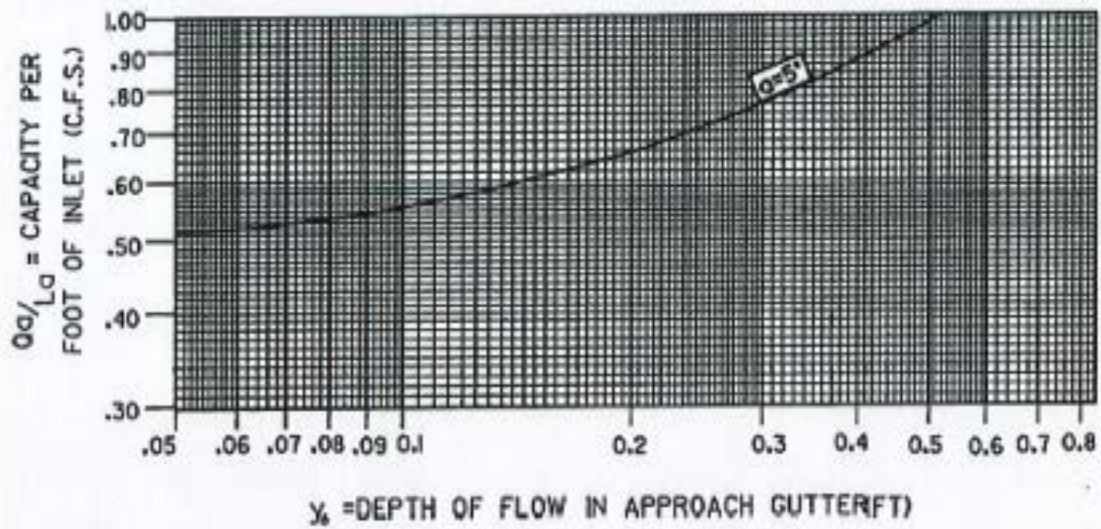
$$Qa/La = 0.7 \left[\frac{1}{H_1 - H_2}\right] \left[(H_1)^{\frac{5}{2}} - (H_2)^{\frac{5}{2}}\right]$$

$$H_1 = a + y_o$$

$$H_2 = a = \text{GUTTER DEPRESSION}$$

Figure 4-11 Ratio of Intercepted to Total Flow for Inlets on Grade
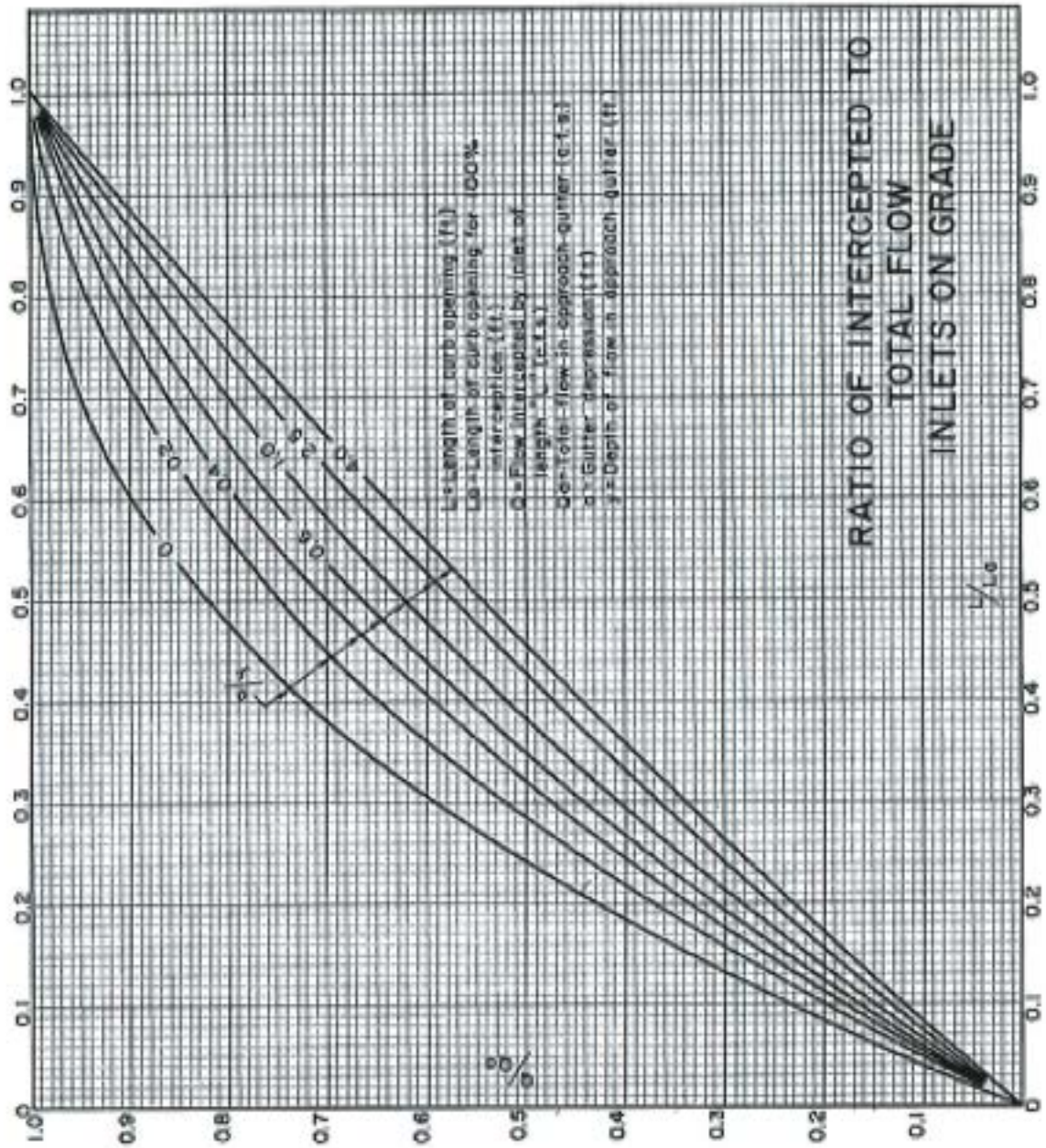
Source: Texas Highway Department, Bridge Division

FIGURES FROM SECTION 5

Figure 5-1 Uniform Flow for Pipe Culverts

TEXAS HIGHWAY DEPARTMENT
UNIFORM FLOW
FOR
PIPE CULVERTS

EXAMPLE

GIVEN: S=0.02   FIND: d/D=
        Q=20 cfs   d =
        D=36" (CONCRETE)

SOLUTION

d/D=0.30
d = 0.30 x 3' = 0.9'

Source: Texas Highway Department, Bridge Division

Figure 5-2 Critical Depth of Flow for Circular Conduits
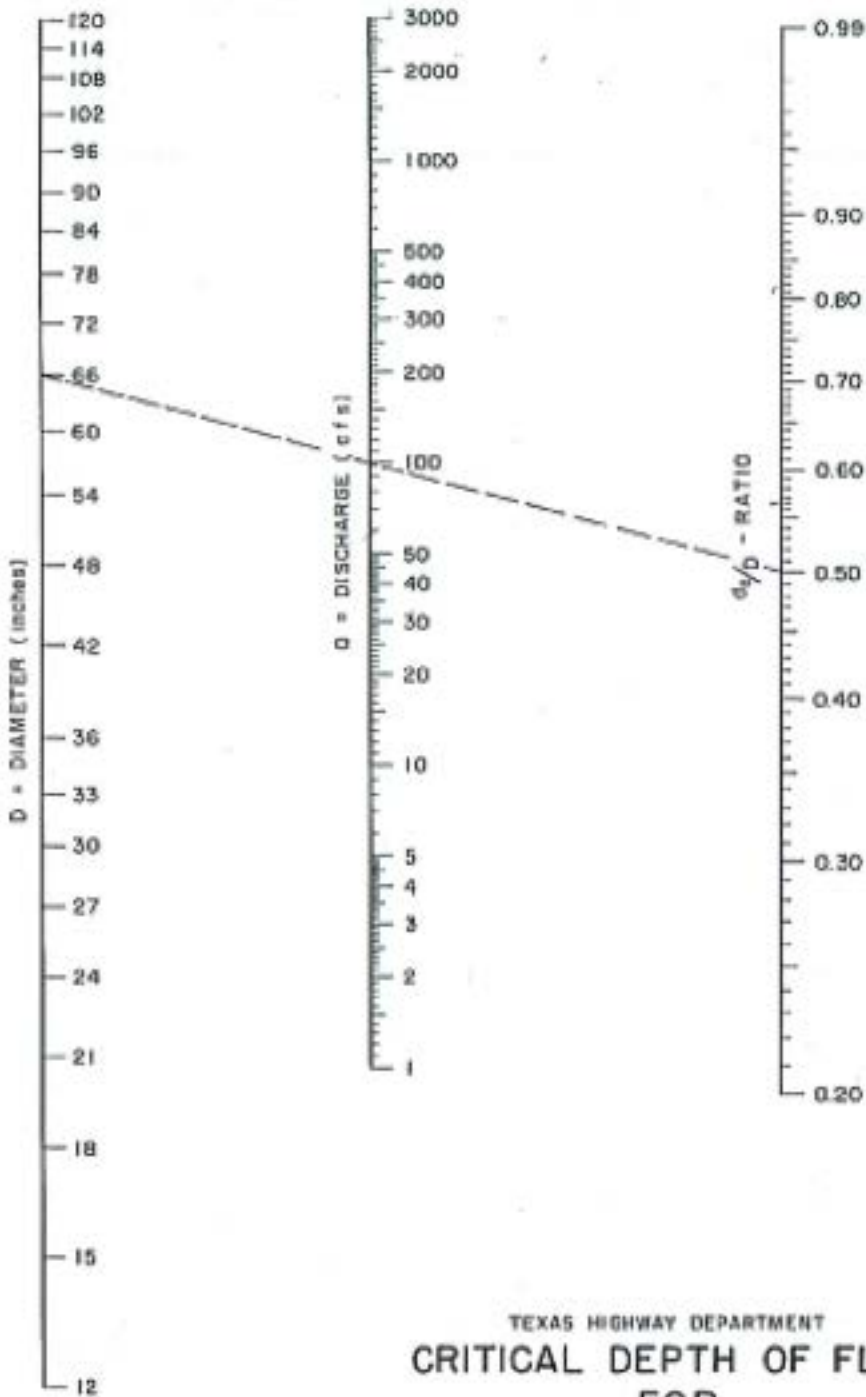
TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH OF FLOW
FOR
CIRCULAR CONDUITS

Source: Texas Highway Department, Bridge Division

Figure 5-3 Velocity in Pipe Conduits
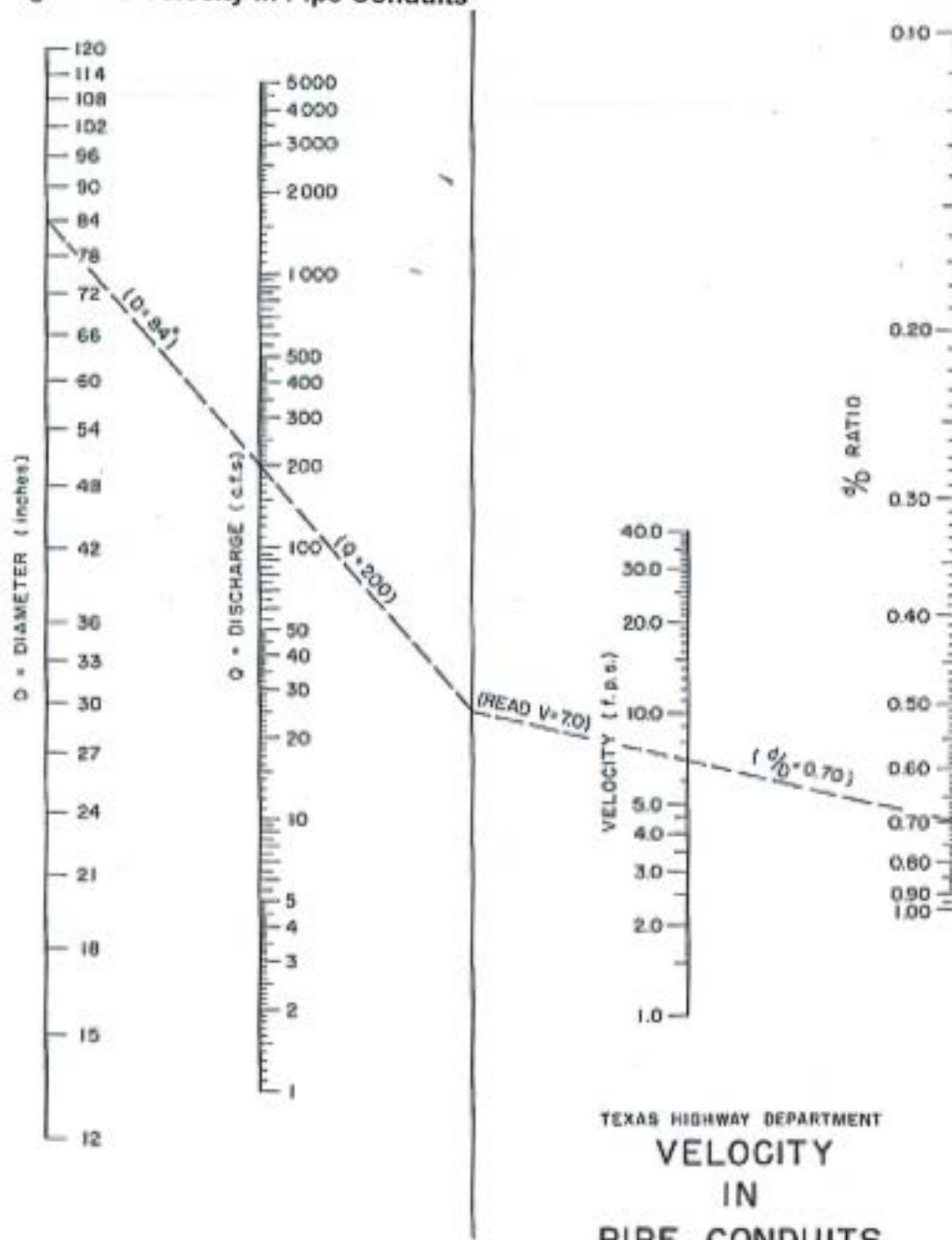
TEXAS HIGHWAY DEPARTMENT

VELOCITY IN PIPE CONDUITS

BASED ON
Q = VA

Source: Texas Highway Department, Bridge Division

Figure 5-4 Uniform Flow for Concrete Elliptical Pipe

IV - 41

SLOPE (FT/FT)
0.0005
0.001
0.002
0.003
0.004
0.005
0.01
0.02
0.03
0.04
0.05
0.06

Q
DISCHARGE (CFS)
2
3
4
5
10
20
30
40
50
100
200
300
400
500
1000
2000

TURNING LINE

d/RISE

d/RISE (14x23-38x60)
0.2
0.3
0.4
0.5
0.6
0.9 — 1.00 (Full Flow)

d/RISE (38x60-87x136)
0.2
0.3
0.4
0.5
.6
0.9 — 1.00 (Full Flow)

ELLIPTICAL PIPE SIZE (RISE X SPAN)
87x136
82x128
77x121
72x113
68x106
63x98
58x91
53x83
48x76
43x68
38x60

38x60
34x53
32x49
29x45
27x42
24x38
22x34
19x30
14x23

TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR CONCRETE ELLIPTICAL PIPE

Figure 23

Source: Texas Highway Department, Bridge Division

Figure 5-5 Critical Depth for Elliptical Pipe

CRITICAL DEPTH/RISE
0.99
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40

DISCHARGE (CFS)
2,000
1,000
500
400
300
200
100
50
40
30
20
10
5
4
3
2

ELLIPTICAL PIPE SIZE (RISE x SPAN)
87x136
83x128
77x121
72x113
68x106
63x98
58x91
53x83
48x76
43x68
38x60
34x53
32x49
29x45
27x42
24x38
22x34
19x30
14x23

Source: City of Austin. Drainage Criteria Manual. Department of Public Works.
Austin, Texas. January 1977.

Figure 5-6 Velocity in Elliptical Pipe
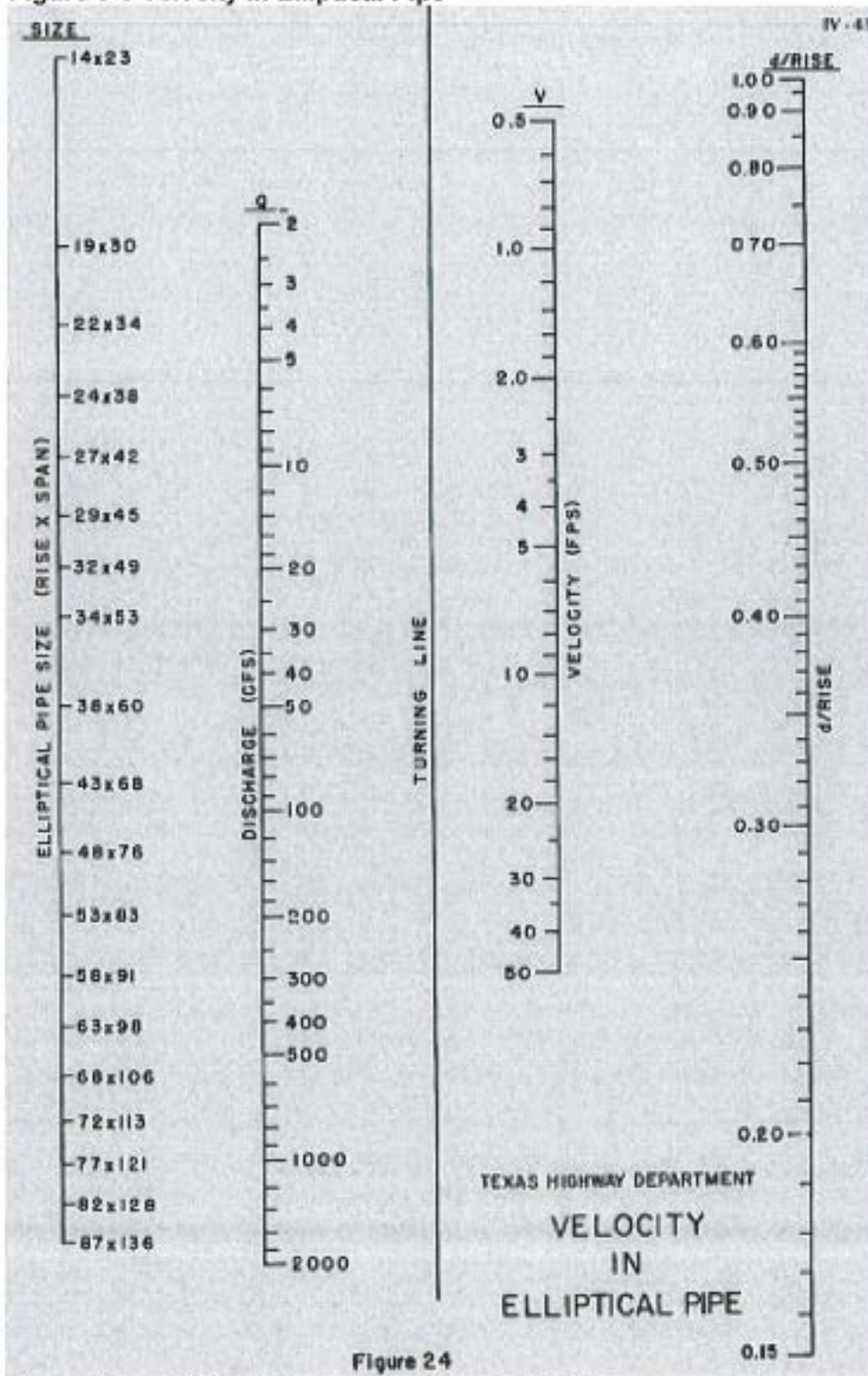
Figure 24

Source: Texas Highway Department, Bridge Division

Figure 5-7 Uniform Flow for Pipe Arch
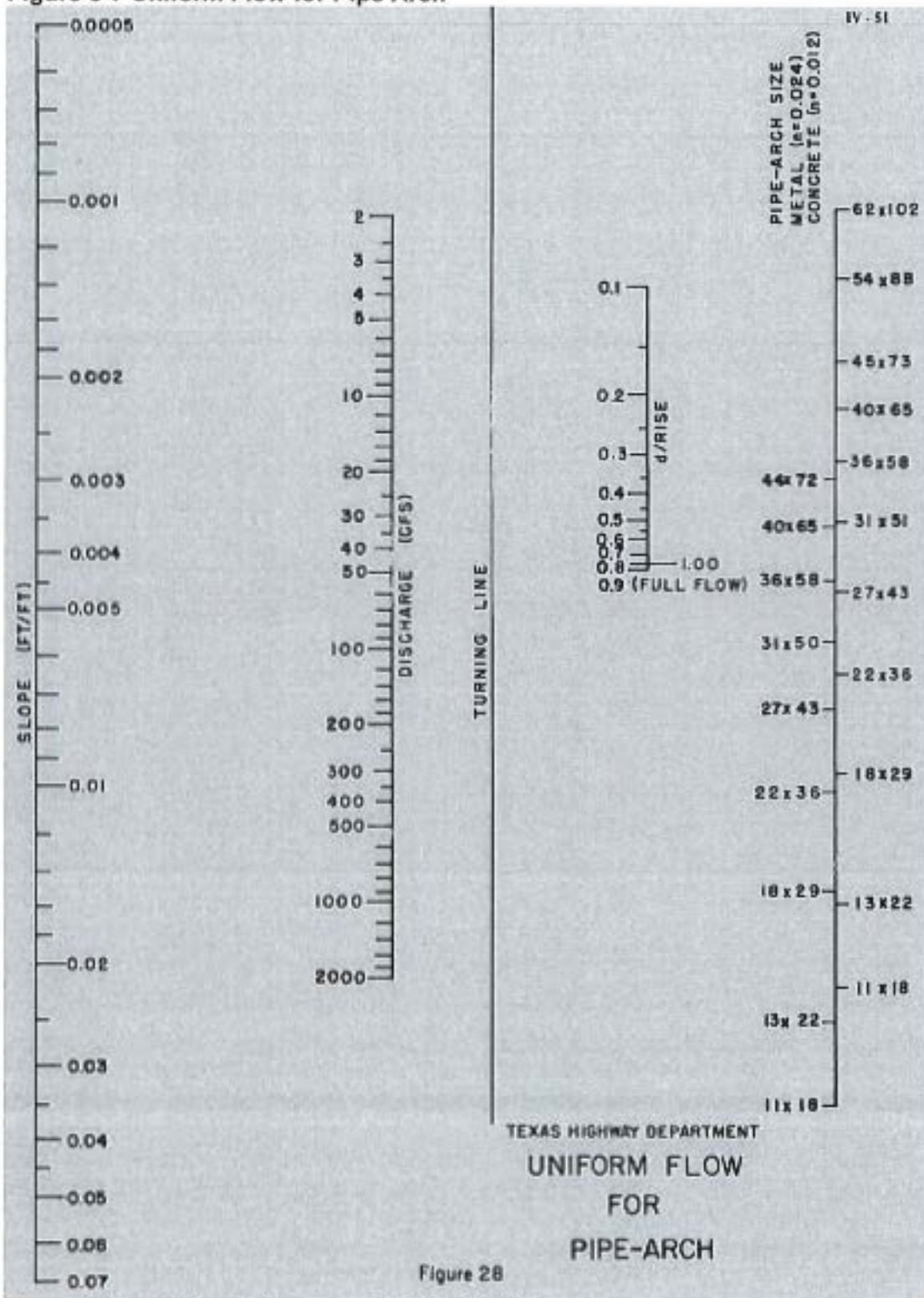
Source: Texas Highway Department, Bridge Division

Figure 5-8 Depth of Flow for Pipe-Arch
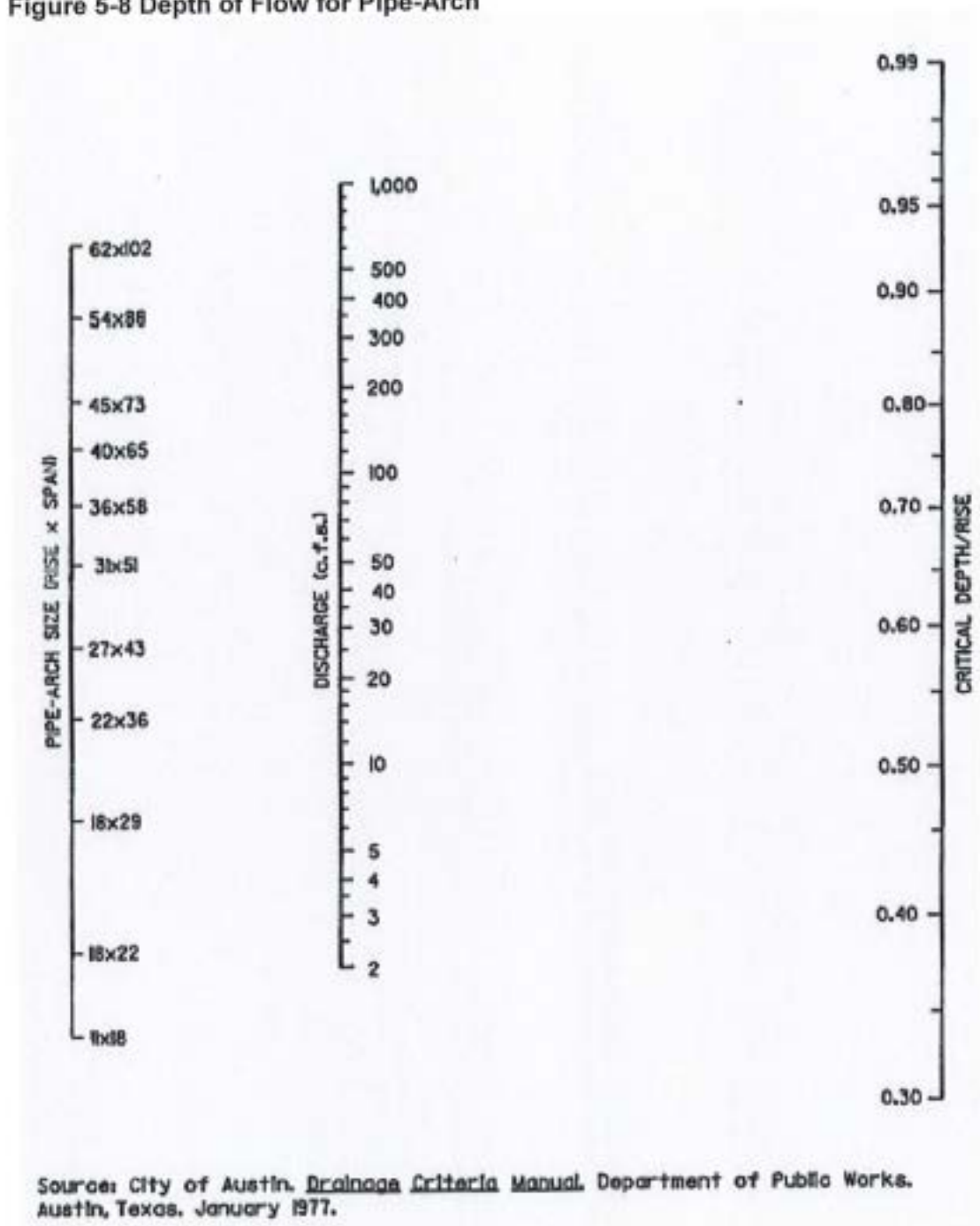
Figure 5-9 Velocity in Pipe-Arch

TEXAS HIGHWAY DEPARTMENT

VELOCITY IN PIPE-ARCH
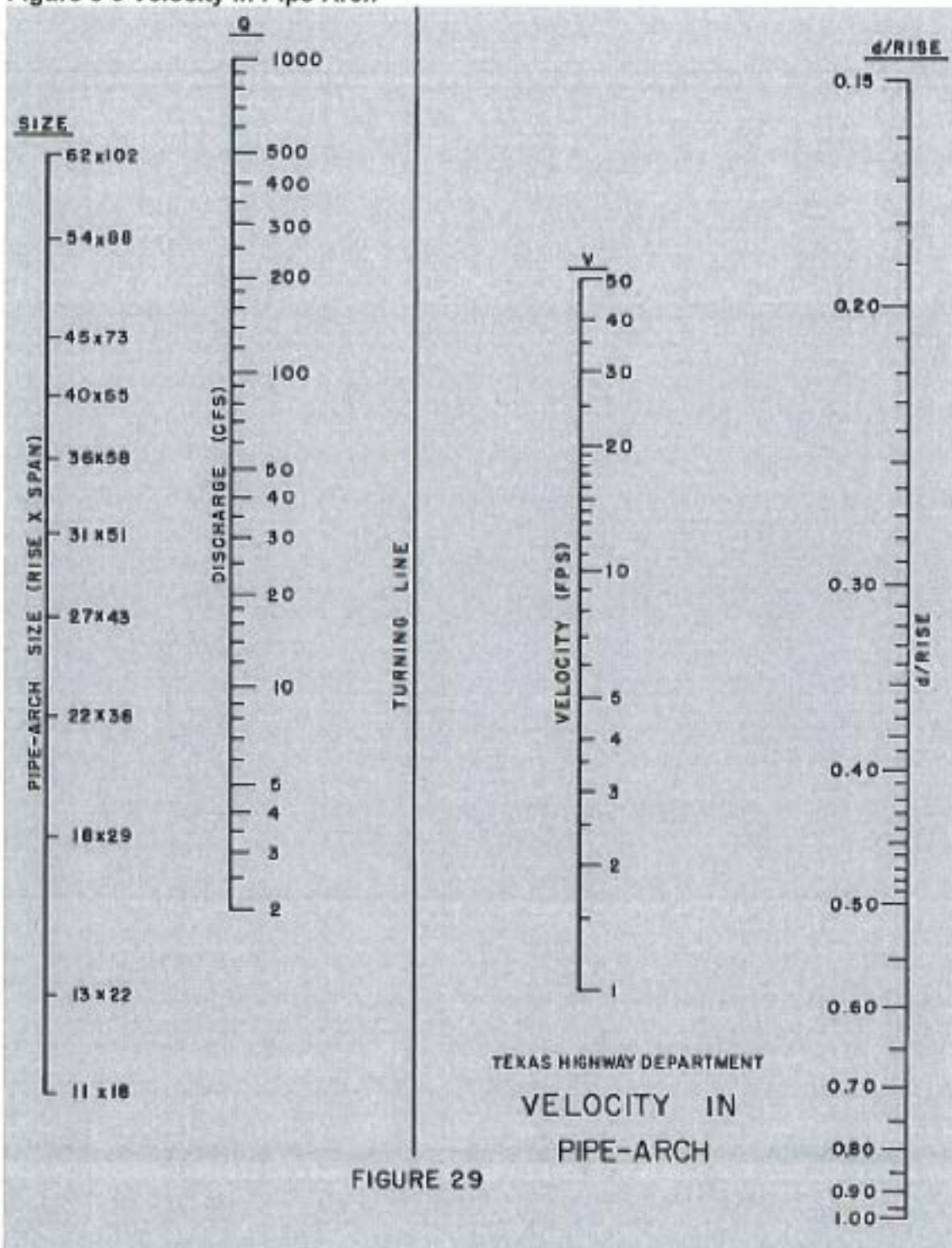
FIGURE 29

Source: Texas Highway Department, Bridge Division

Figure 5-10 Minor Head Losses Due to Turbulence at Structures
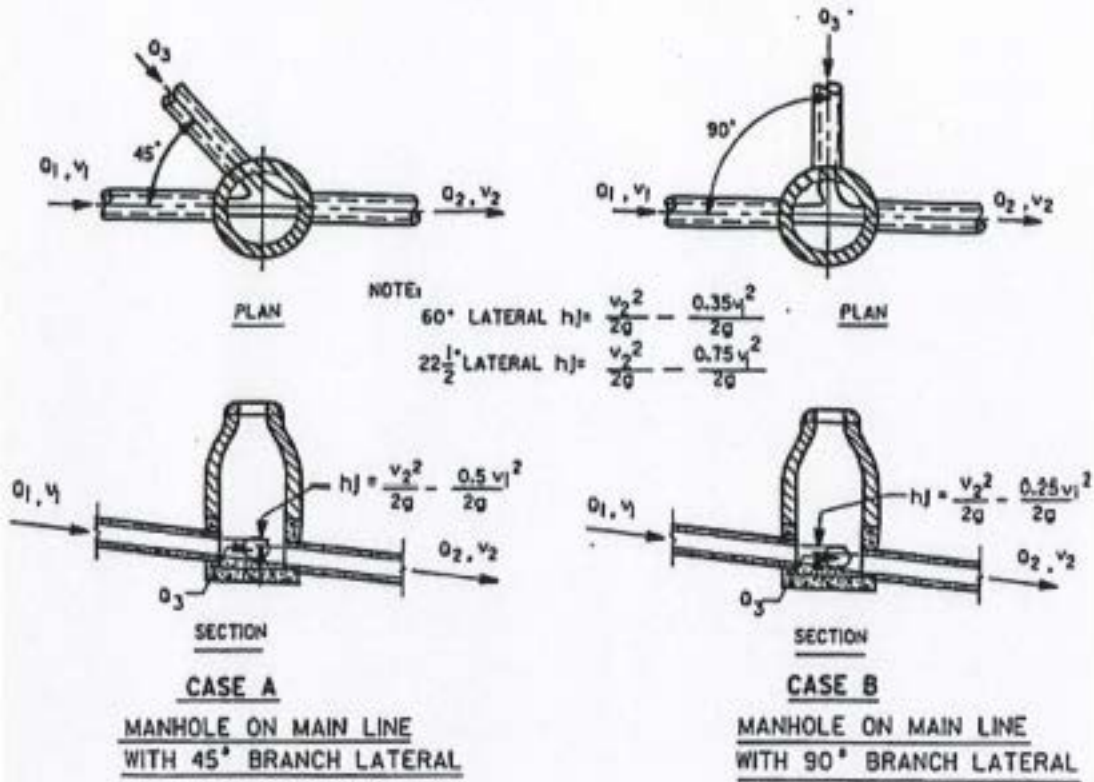
NOTE:

$$60° \text{ LATERAL } hJ = \frac{v_2^2}{2g} - \frac{0.35v_1^2}{2g}$$

$$22\tfrac{1}{2}° \text{ LATERAL } hJ = \frac{v_2^2}{2g} - \frac{0.75v_1^2}{2g}$$

$$hJ = \frac{v_2^2}{2g} - \frac{0.5v_1^2}{2g}$$

$$hJ = \frac{v_2^2}{2g} - \frac{0.25v_1^2}{2g}$$

CASE A

MANHOLE ON MAIN LINE
WITH 45° BRANCH LATERAL

CASE B

MANHOLE ON MAIN LINE
WITH 90° BRANCH LATERAL

Source: City of Austin. Drainage Criteria Manual. Department of Public Works.
Austin, Texas. January 1977.

Figure 5-11 Minor Head Losses Due to Turbulence at Structures

PLAN

SECTION

$h_J(\text{AT } \text{\textcentoblique} \text{ CONN.}) = \dfrac{v_2^2}{2g} - \dfrac{0.75\,v_1^2}{2g}$

CASE C
45° WYE CONNECTION
OR CUT-IN

PLAN

SECTION

$h_J = \dfrac{1.25\,v_2^2}{2g}$

CASE D
INLET OR MANHOLE AT
BEGINNING OF LINE

CASE E
CONDUIT ON 90° CURVES

NOTE : HEAD LOSS APPLIED AT P.C. FOR LENGTH OF CURVE

RADIUS = DIA. OF PIPE $h_J = 0.05\ \dfrac{v_2^2}{2g}$

RADIUS = (2-8) DIA. OF PIPE $h_J = 0.25\ \dfrac{v_2^2}{2g}$

RADIUS = (8-20) DIA. OF PIPE $h_J = 0.40\ \dfrac{v_2^2}{2g}$

RADIUS = GREATER THAN 20 DIA. OF PIPE $h_J = 0$

WHEN CURVES OTHER THAN 90° ARE USED, APPLY
THE FOLLOWING FACTORS TO 90° CURVES.
60° CURVES 85%
45° CURVE 70%
$22\frac{1}{2}$° CURVE 40%

CASE F
BENDS WHERE RADIUS IS
EQUAL TO DIAMETER OF PIPE

NOTE : HEAD LOSS APPLIED AT BEGINNING OF BEND

90° BEND $h_J = 0.50\ \dfrac{v_2^2}{2g}$

60° BEND $h_J = 0.43\ \dfrac{v_2^2}{2g}$

45° BEND $h_J = 0.35\ \dfrac{v_2^2}{2g}$

$22\frac{1}{2}$° BEND $h_J = 0.20\ \dfrac{v_2^2}{2g}$

Source: City of Austin, Drainage Criteria Manual, Department of Public Works,
Austin, Texas. January 1977.

Figure 5-12 Sample Stormsewer Layout
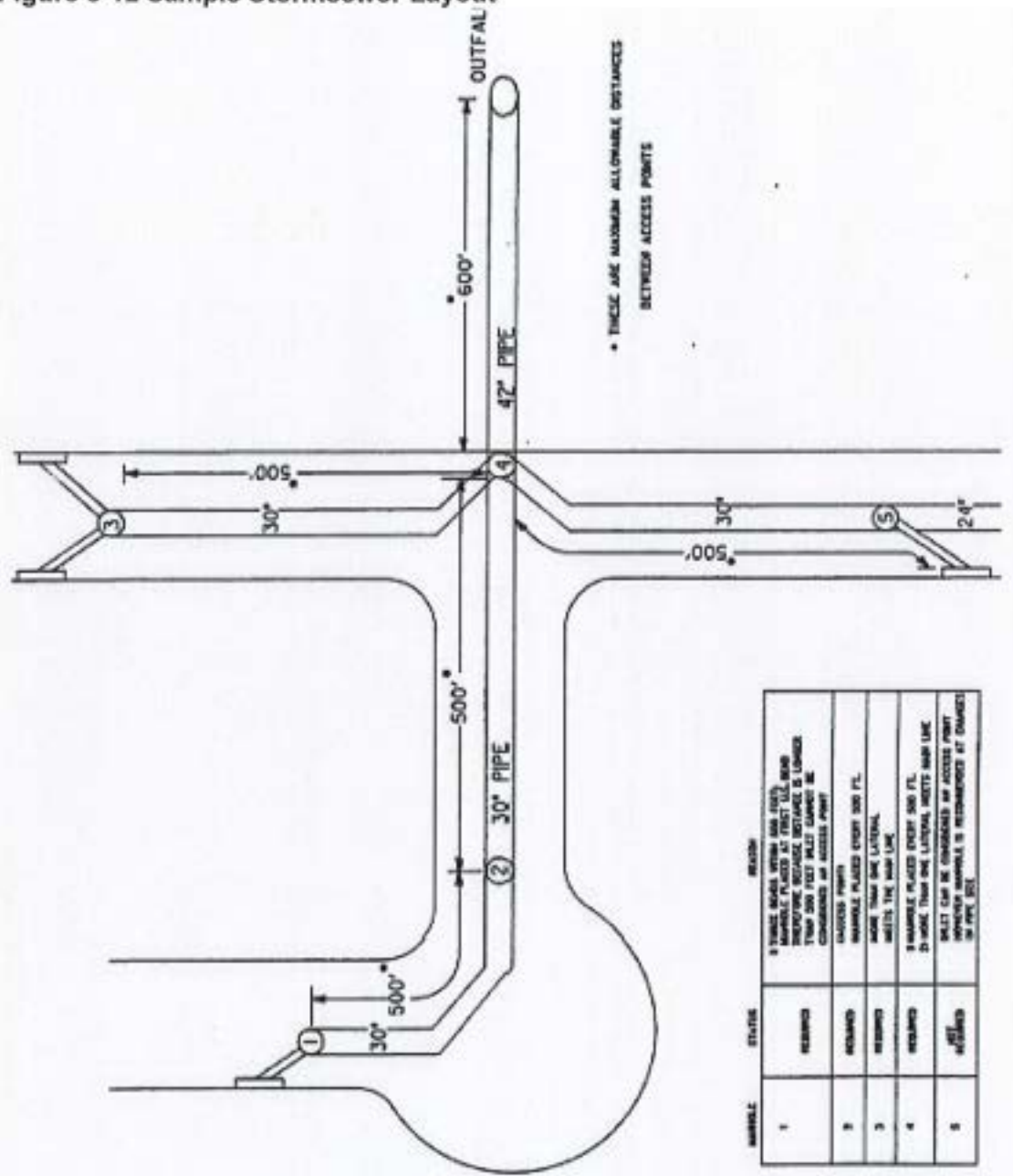
Source: City of Austin, Watershed Management Division

Figure 5-13 Flow for Circular Pipe Flowing Full (n=0.010)

BASED ON MANNING'S EQUATION n=0.010
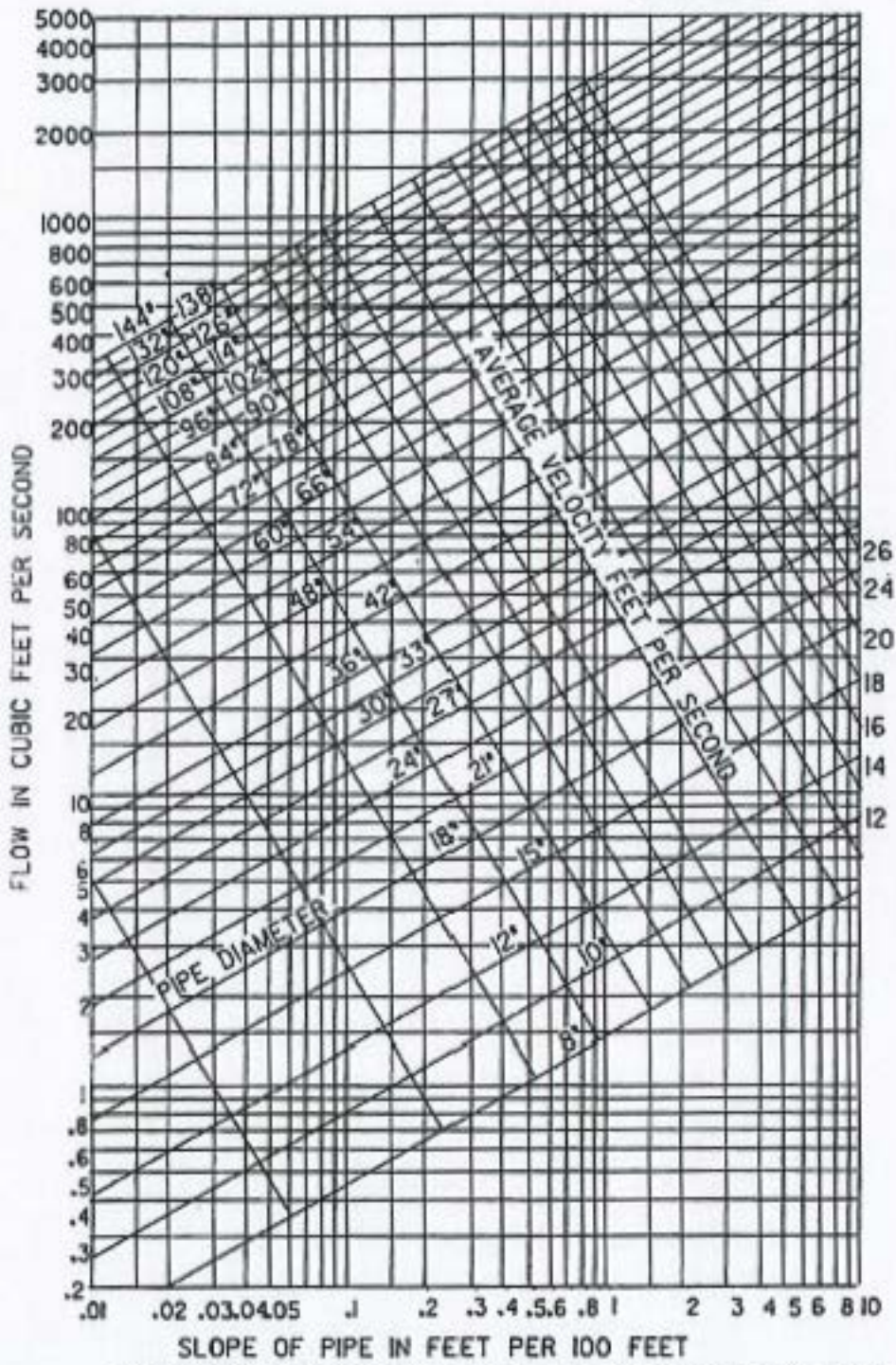
Source: Brater, E.F. and H.W. King, Handbook of Hydraulics, Sixth Edition, McGraw-Hill Book Company, N.Y., 1976

Figure 5-14 Flow for Circular Pipe Flowing Full (n=0.011)
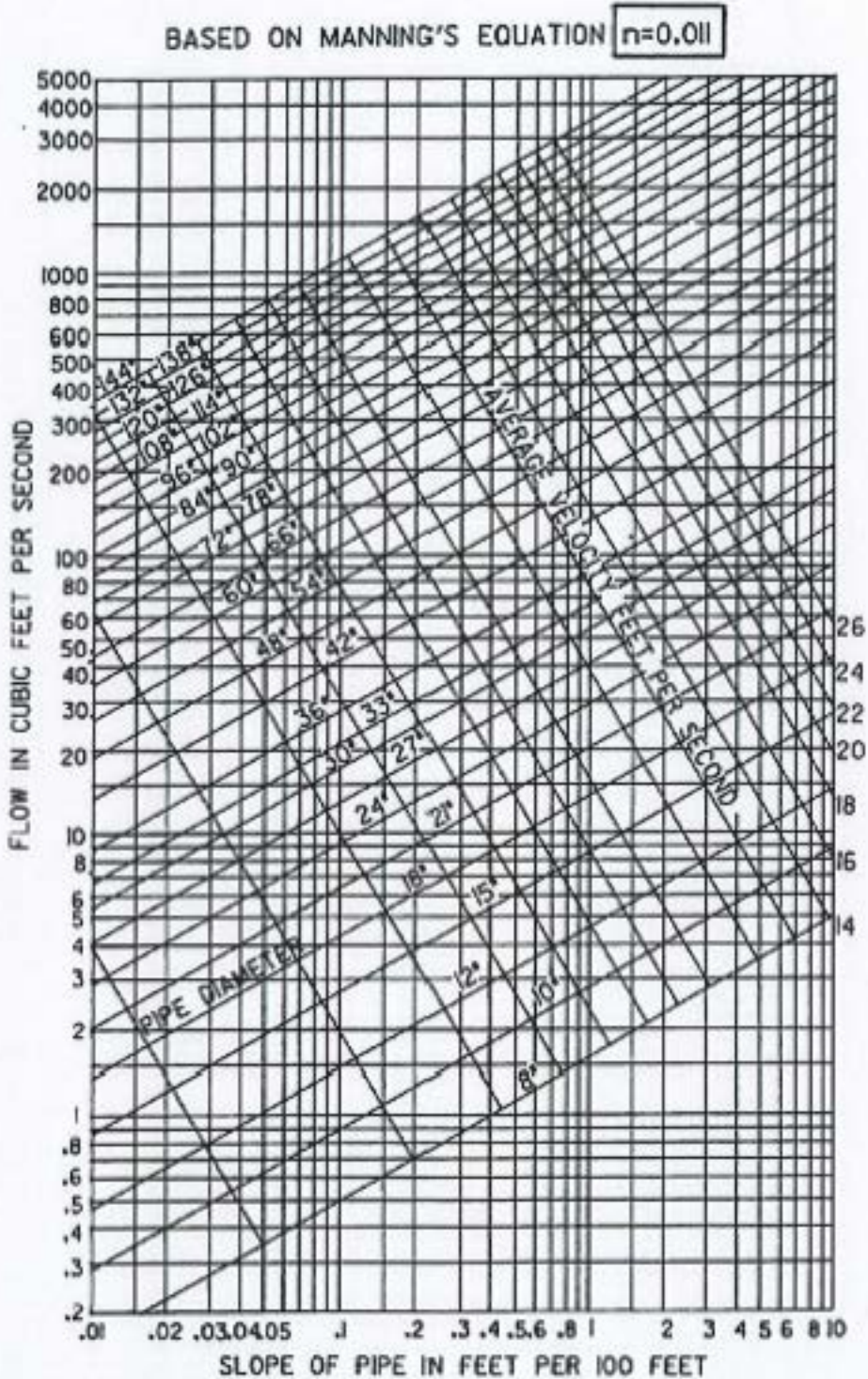
Figure 5-15 Flow for Circular Pipe Flowing Full (n=0.012)

BASED ON MANNING'S EQUATION n=0.012

Source: Brater, E.F. and H.W. King, Handbook of Hydraulics, Sixth Edition, McGraw-Hill Book Company, N.Y., 1976
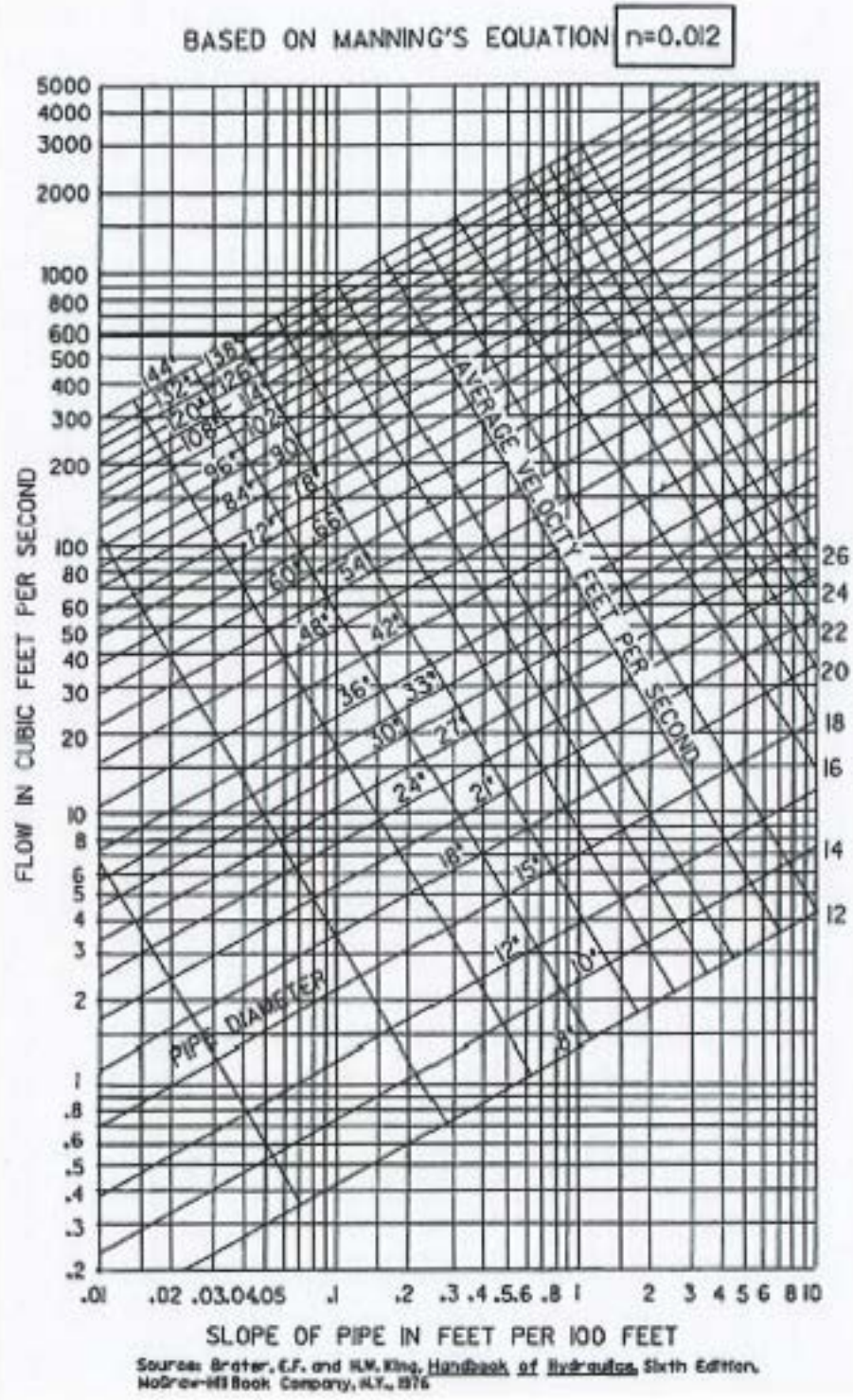
Figure 5-16 Flow for Circular Pipe Flowing Full (n=0.013)

BASED ON MANNING'S EQUATION n=0.013
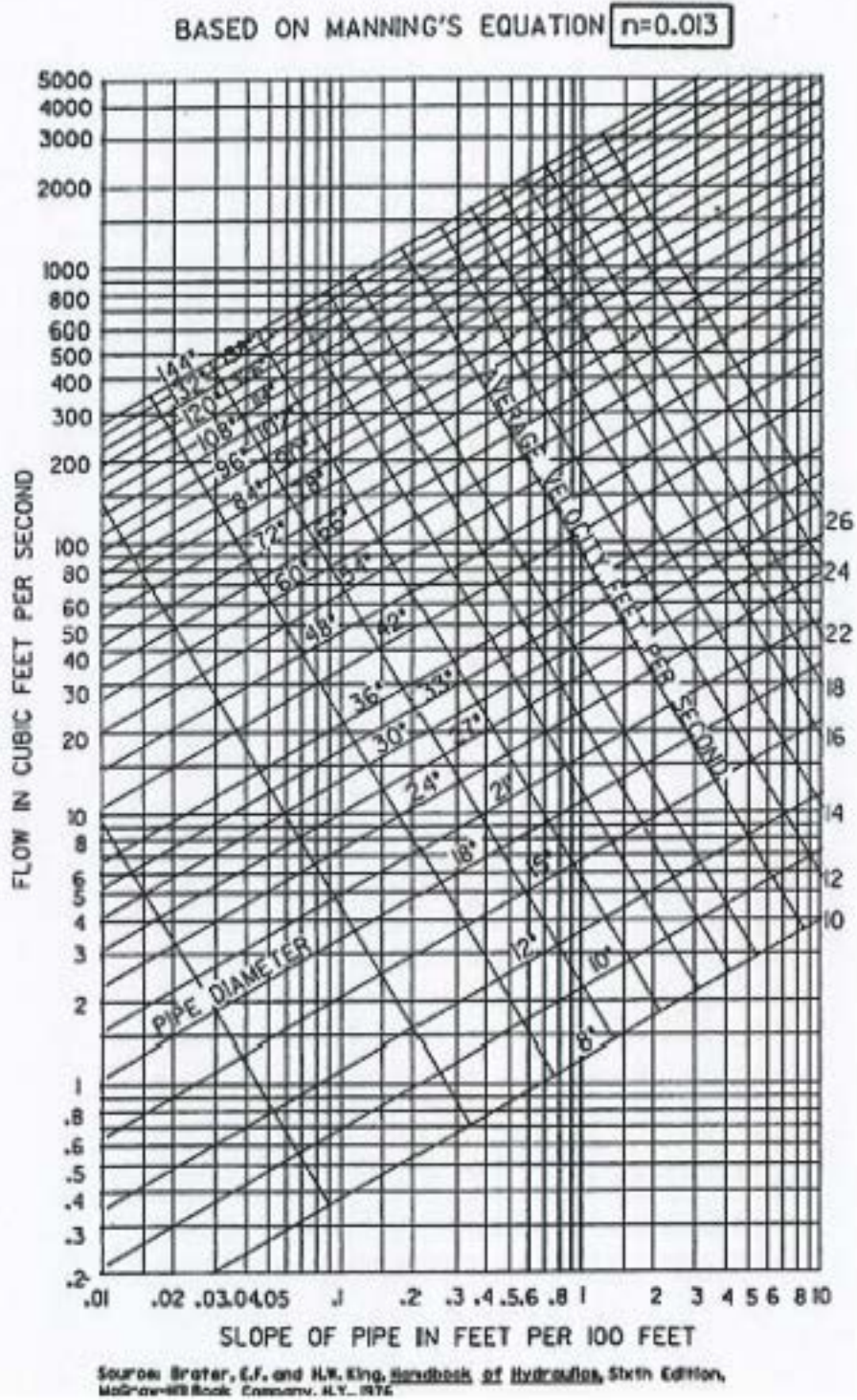
Source: Brater, E.F. and H.W. King, Handbook of Hydraulics, Sixth Edition, McGraw-Hill Book Company, N.Y. 1976

Figure 6-1 Uniform Flow for Trapezoidal Channels
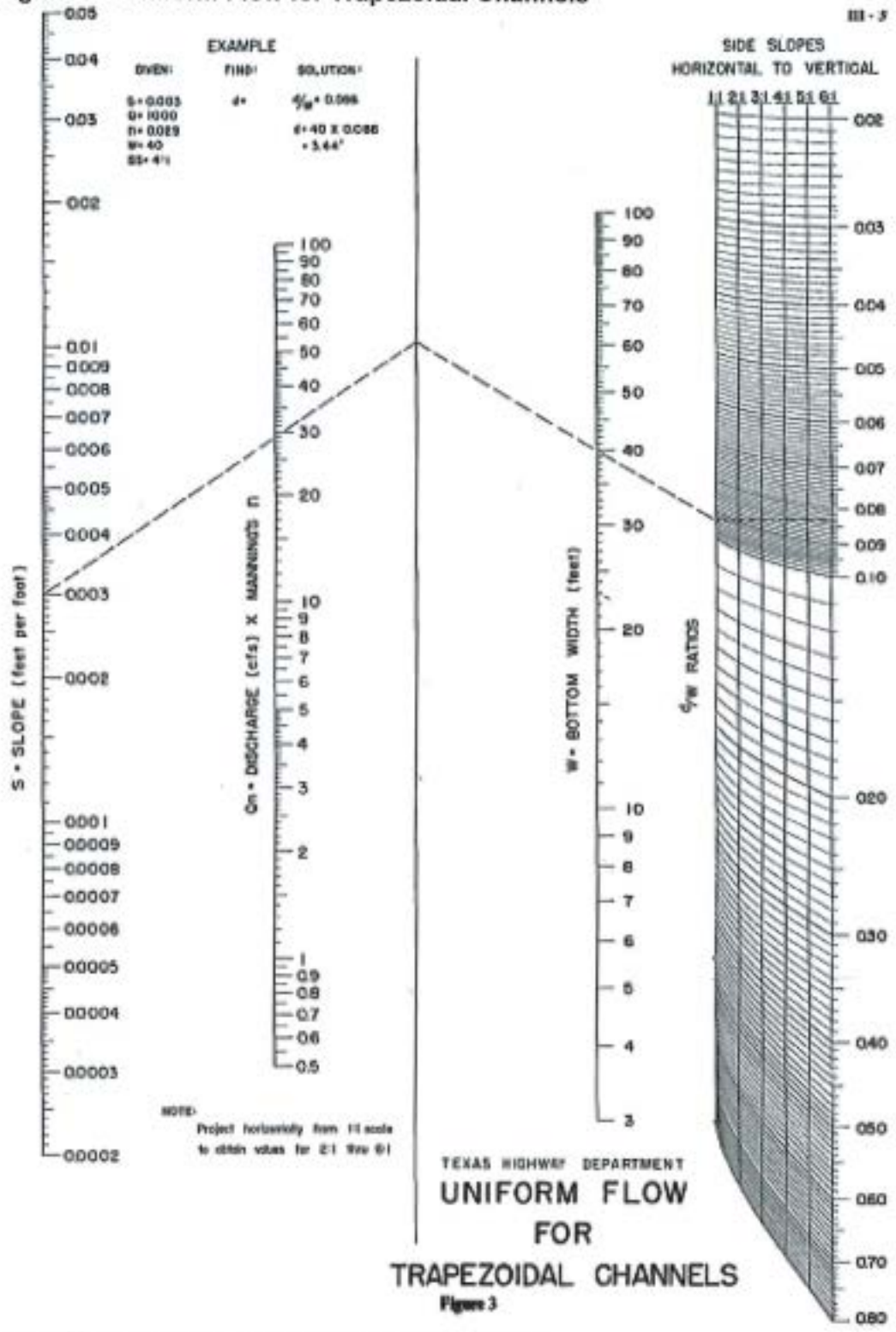
Source: Texas Highway Department, Bridge Division

Figure 6-2 Sloping and Vertical Channel Drops
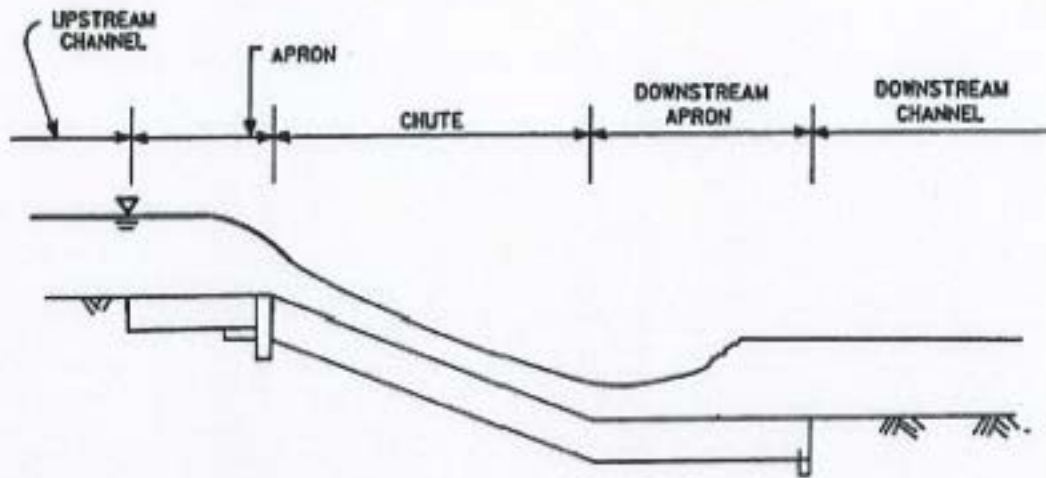
A. SLOPING CHANNEL DROP
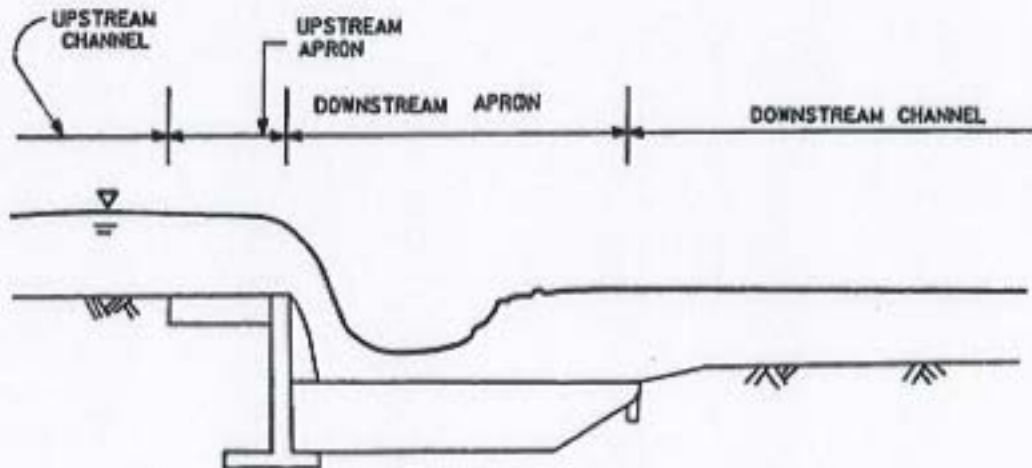
B. VERTICAL CHANNEL DROP

Source: U.S. Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Nomograph No. 25, Eigth Printing. Denver, May, 1984.

Figure 6-3 Baffled Apron and its Design Curve
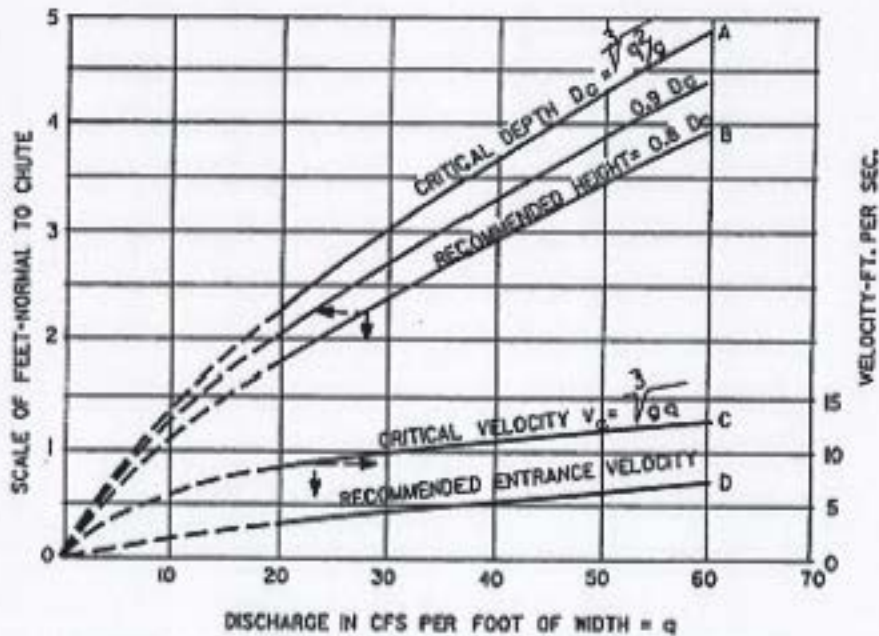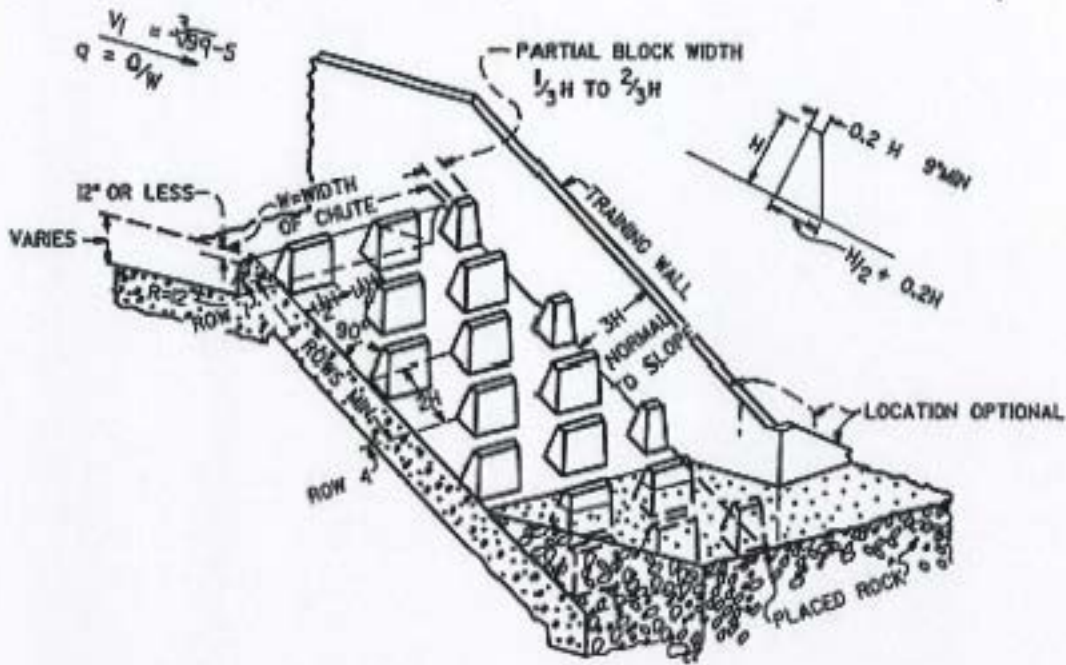
Source: U.S. Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Nomograph No. 25, Eigth Printing, Denver, May, 1984.

Figure 6-4 Conceptual Design of Alternative Channel

Source: City of Austin, Department of Environmental Protection
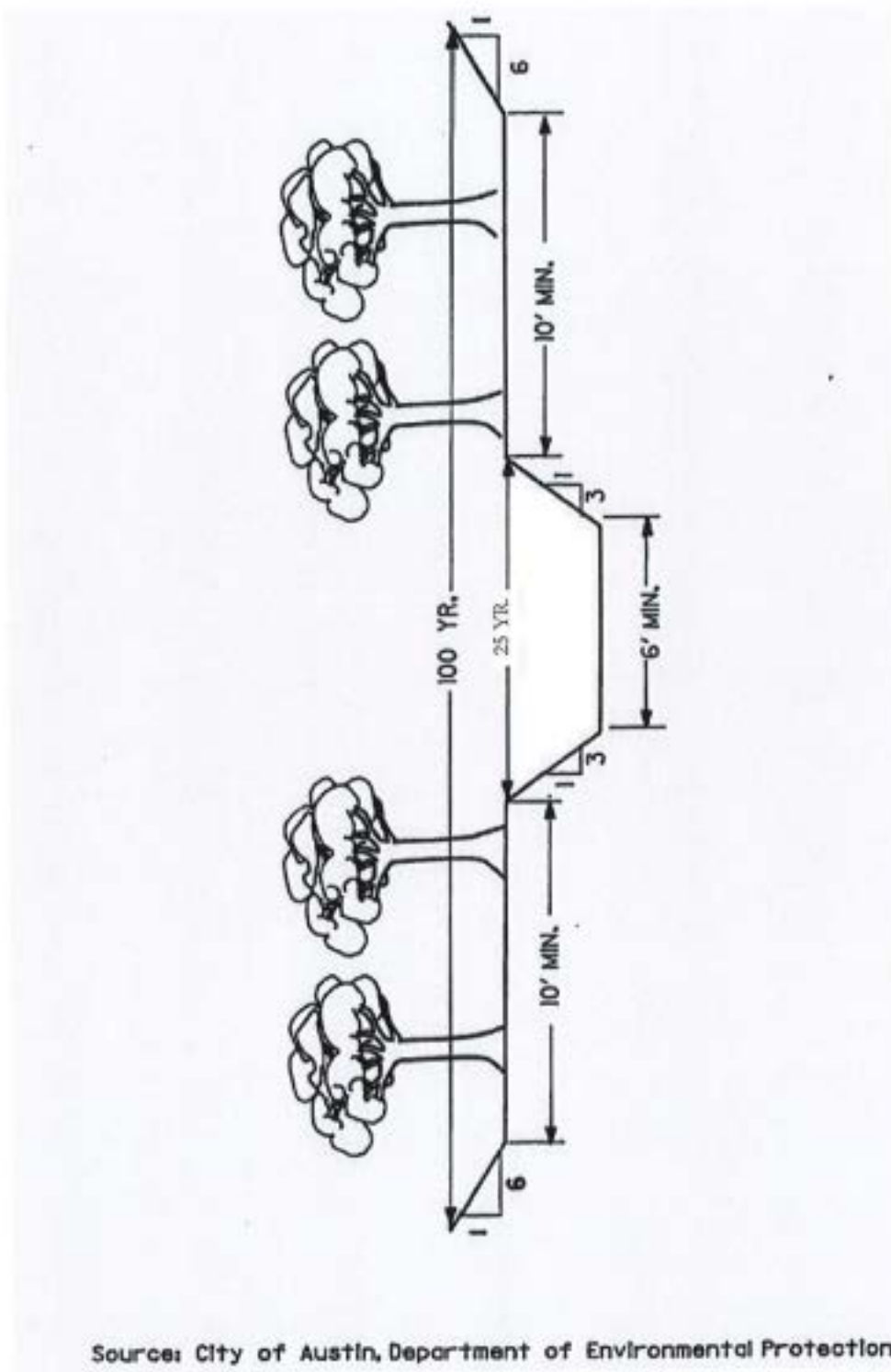
Figure 7-1 Headwall Entrance Type

flush headwall entrance

rounded-lip entrance with
radius of rounding=0.15D

R=0.15D

square-edged entrance

square-edged projecting
thickwall pipe

embankment

socket entrance with
parallel wingwalls

square-edged entrance
with parallel wingwalls

warped wall entrance

approaching flow

Wingwalls flared from
axis of stream

socket entrance with
45° wingwalls

square-edged entrance with
45° wingwalls

Source: City of Austin, Watershed Management Division

Figure 7-2 Conceptual Design of Debris Fins
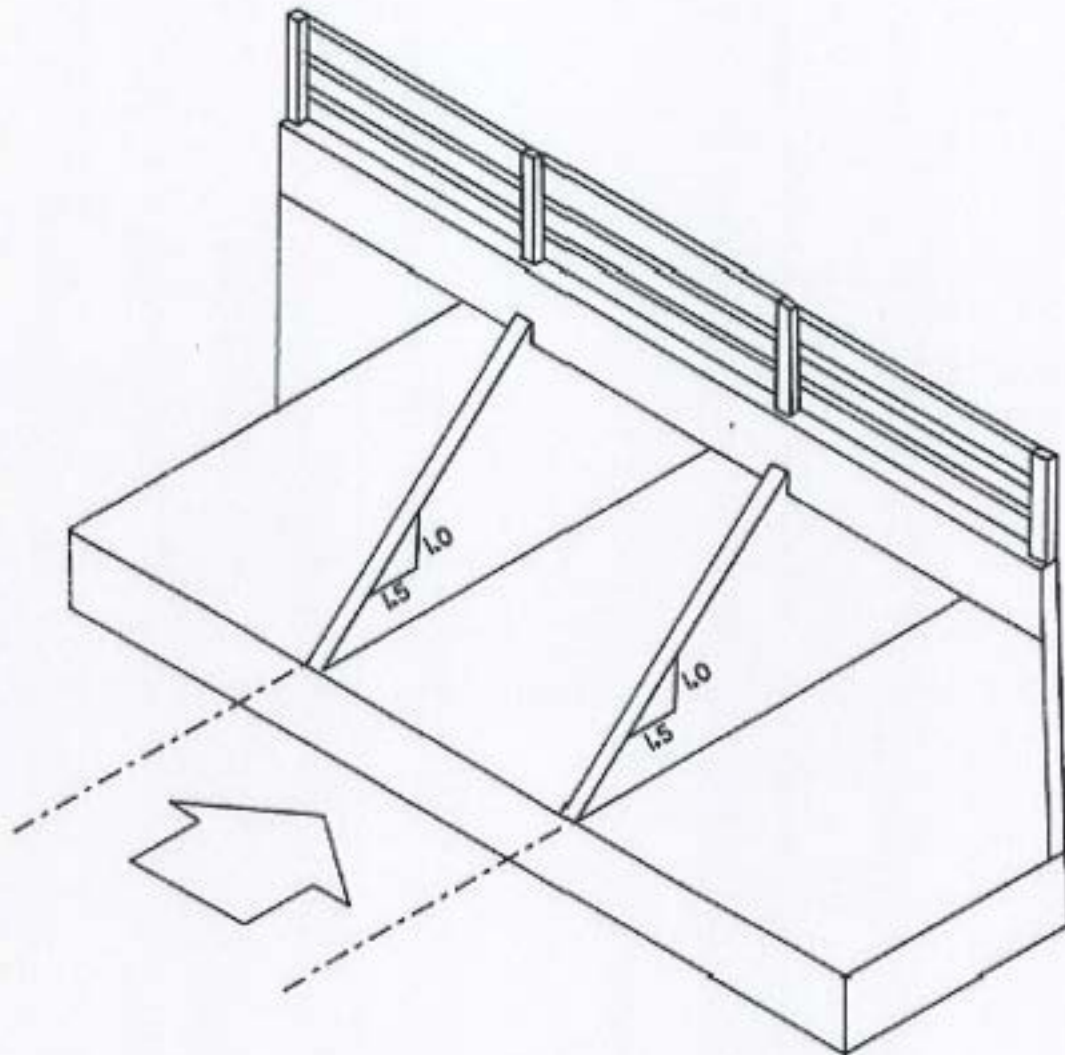
Source: City of Austin, Watershed Management Division

Figure 7-3 Inlet and Outlet Conditions for Culverts

HEADWATER (HW)

PROJECTING INLET END-UNSUBMERGED

WATER SURFACE

A

WATER SURFACE

B

HW

PROJECTING OR MITERED INLET
-SUBMERGED

WATER SURFACE
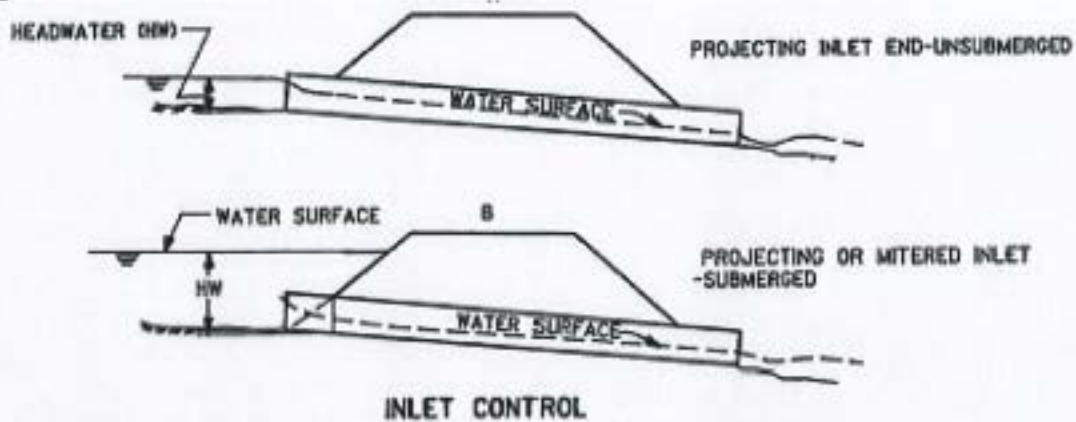
INLET CONTROL

INLET CONTROL IS ONE OF THE TWO MAJOR TYPES OF CULVERT FLOW. CONDITION A WITH AN UNSUBMERGED CULVERT INLET IS PREFERRED TO THE SUBMERGED END. SLOPE, ROUGHNESS AND LENGTH OF CULVERT BARREL ARE NOT A CONSIDERATION.

WATER SURFACE (WS)

A

H_L W.S.

HEADWATER (HW)

B

HW

H_L

W.S.

C

Hydraulic Grade Line A

HW

H_L

W.S.

D

HW

H_L

W.S.
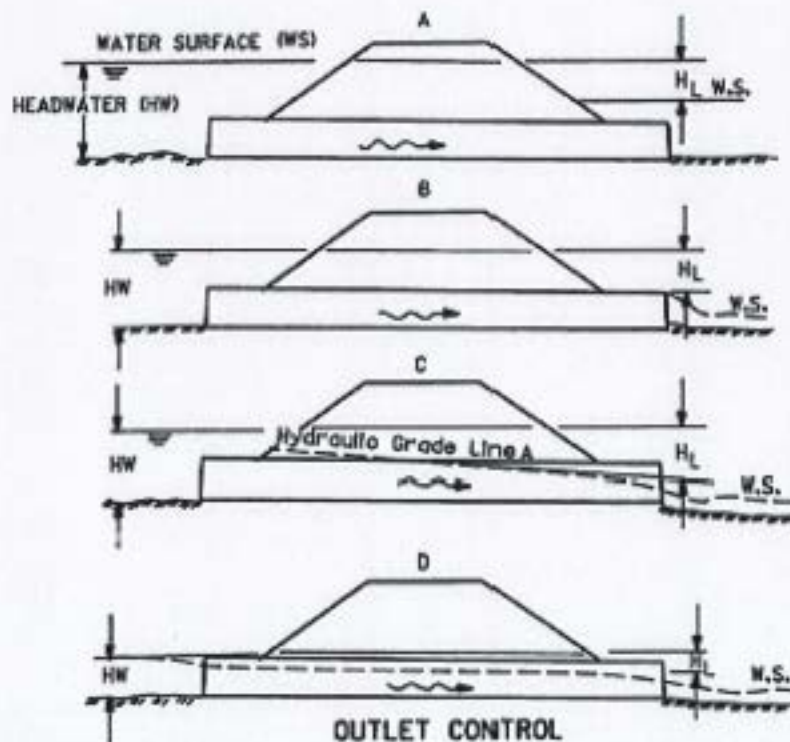
OUTLET CONTROL

OUTLET CONTROL INVOLVES THESE FACTORS: CROSS-SECTIONAL AREA OF BARREL, INLET "GEOMETRY", PONDING, SLOPE, ROUGHNESS, TAILWATER, AND LENGTH OF CULVERT BARREL.
Source: Boulder County, Storm Drainage Criteria Manual

Figure 7-4 Hydraulics of a Culvert Under Outlet Control Condition
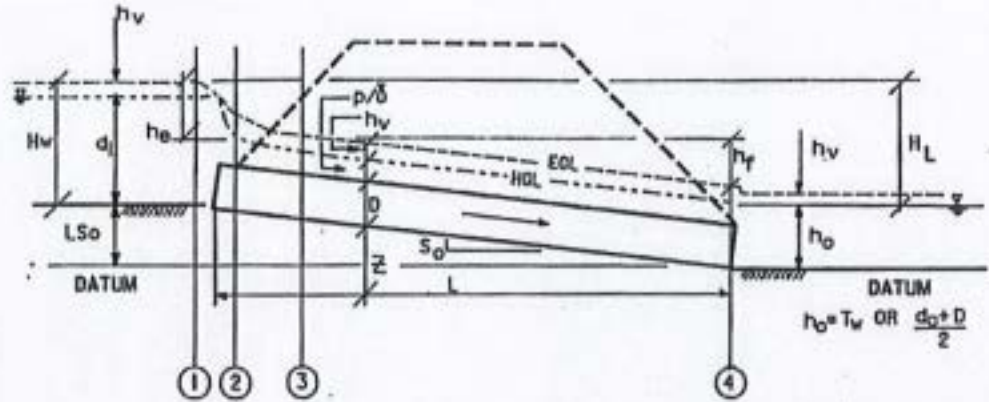
DEFINITION OF TERMS:

L = CULVERT LENGTH

S_0 = CULVERT SLOPE

H_w = HEADWATER DEPTH

h_v = VELOCITY HEAD

h_e = HEADLOSS AT THE ENTRANCE

z = DISTANCE FROM DATUM LINE

D = CULVERT DIAMETER OR RISE

P/δ = PRESSURE HEAD

HGL = HYDRAULIC GRADE LINE

EGL = ENERGY GRADE LINE

T_w = TAILWATER DEPTH

h_f = FRICTION LOSS IN CULVERT

d_1 = APPROACH DEPTH

Figure 7-5 Inlet Control Nomograph, Circular Pipe

(A) CORRUGATED STEEL PIPE

(B) REINFORCED CONCRETE PIPE

Figure 7-6 Inlet Control Nomograph, Box Culverts
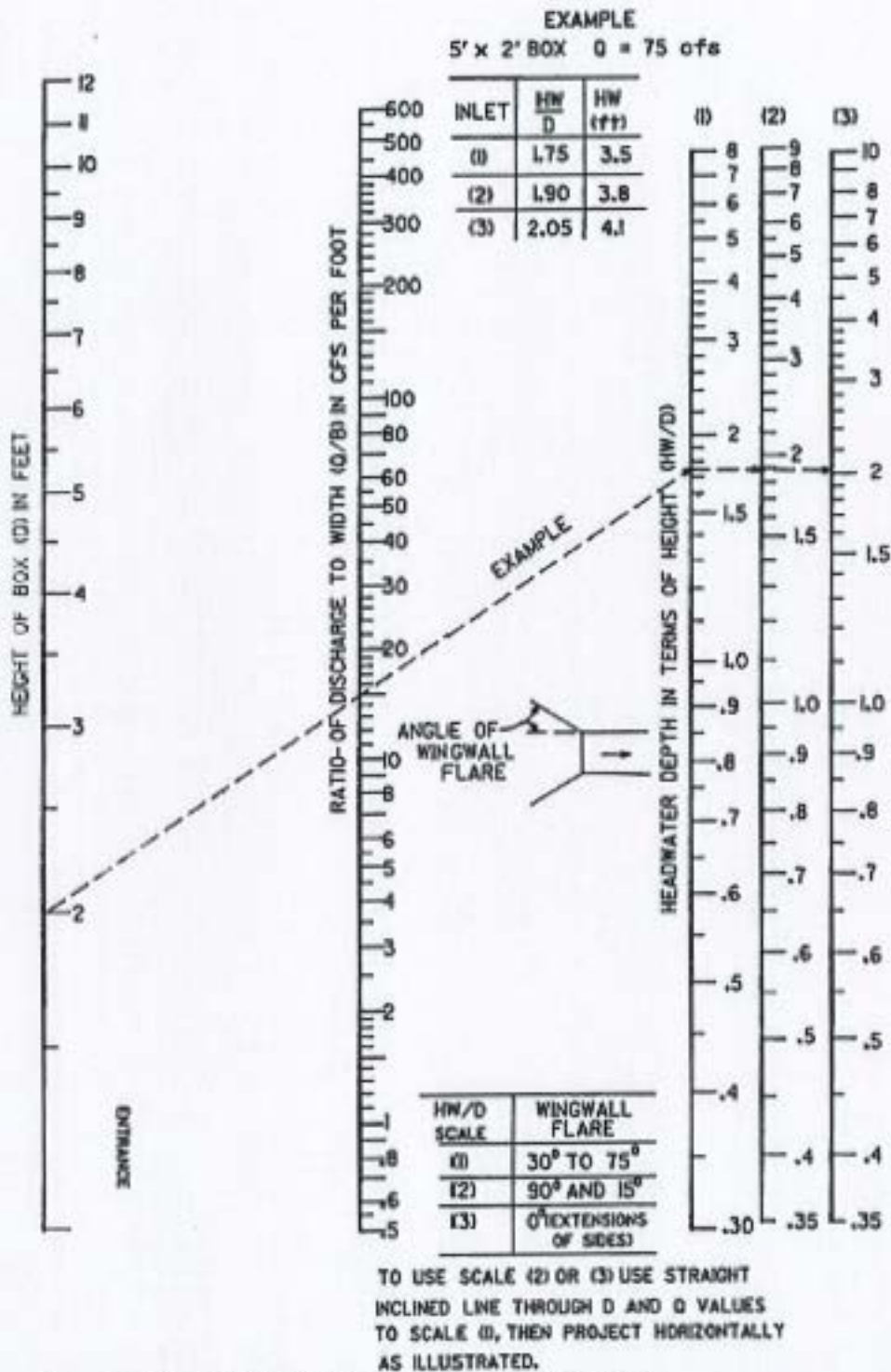
EXAMPLE
5' x 2' BOX   Q = 75 cfs

INLET	HW/D	HW (ft)
(1)	1.75	3.5
(2)	1.90	3.8
(3)	2.05	4.1

HW/D SCALE	WINGWALL FLARE
(1)	30° TO 75°
(2)	90° AND 15°
(3)	0° (EXTENSIONS OF SIDES)

TO USE SCALE (2) OR (3) USE STRAIGHT
INCLINED LINE THROUGH D AND Q VALUES
TO SCALE (1), THEN PROJECT HORIZONTALLY
AS ILLUSTRATED.

Source: "Hydraulic Charts for the Selection of Highway
Culverts", HEC-5, USDDT, FHA, Dec. 1965
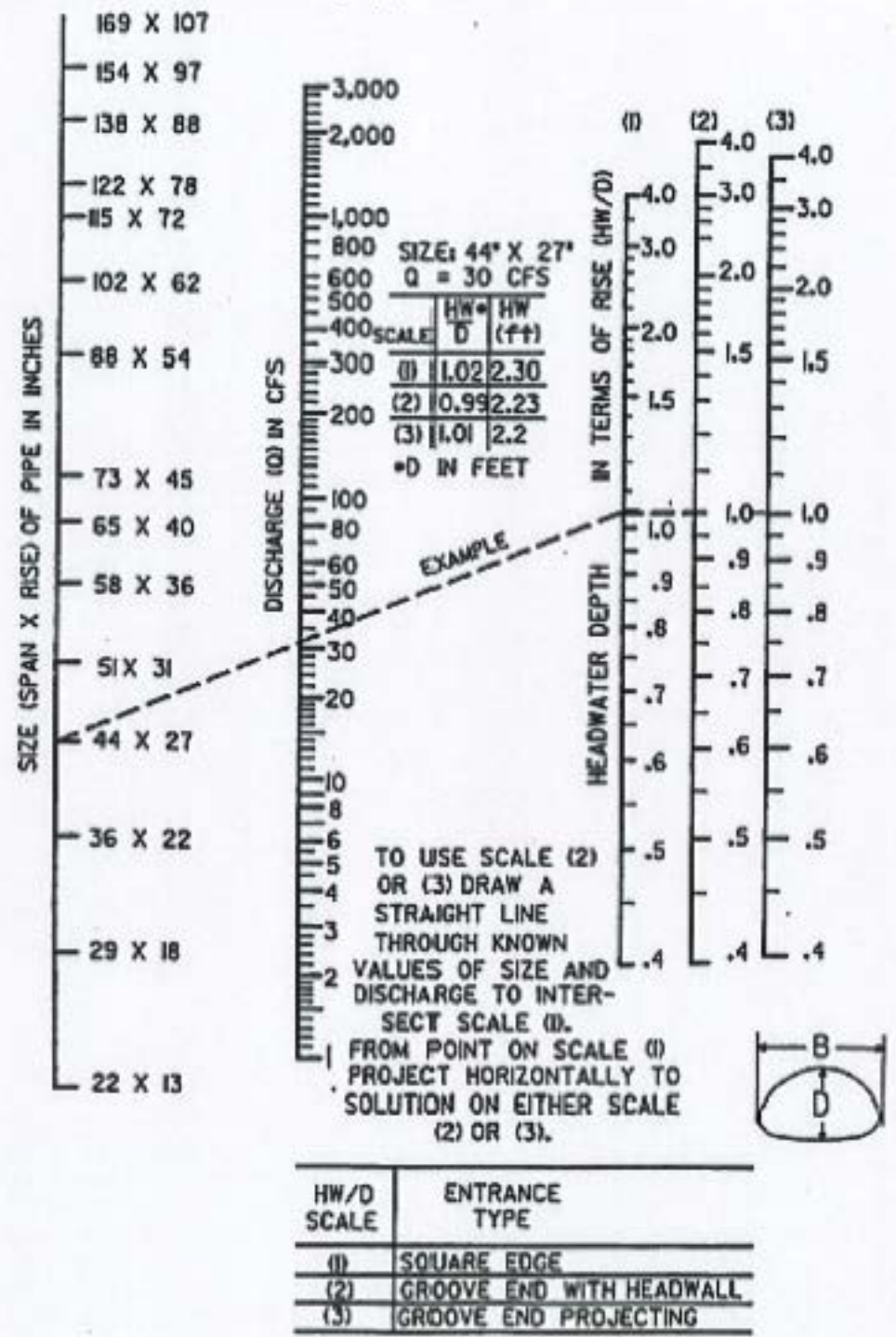
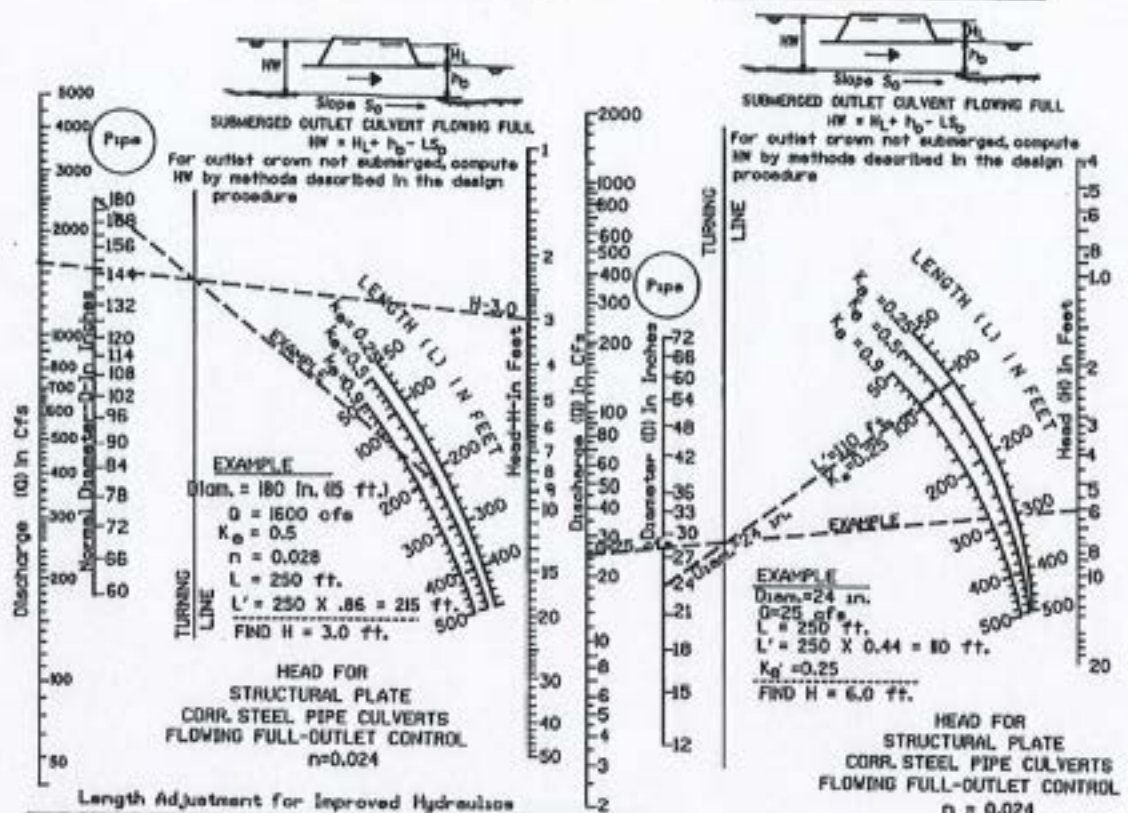## Figure 7-7 Inlet Control Nomograph, CSP Arch

16-IN. CORNER RADIUS

STRUCTURAL PLATE C.M. (SPAN X RISE) OF PIPE-ARCH

- 16'-7" X 10'-1"
- 15'-4" X 9'-3"
- 12'-10" X 8'-4"
- 11'-5" X 7'-3"
- 9'-6" X 6'-5"
- 8'-2" X 5'-9"
- 7'-0" X 3'-1"
- 6'-1" X 4'-7"

STANDARD C.M. SIZE (SPAN X RISE)

- 72" X 44"
- 65" X 40"
- 58" X 36"
- 50" X 31"
- 43" X 27"
- 36" X 22"
- 29" X 18"
- 25" X 16"
- 22" X 13"
- 18" X 11"

DISCHARGE (Q) IN CFS

- 5,000
- 4,000
- 3,000
- 2,000
- 1,000
- 800
- 600
- 500
- 400
- 300
- 200
- 100
- 80
- 60
- 50
- 40
- 30
- 20
- 10
- 8
- 6
- 5
- 4
- 3
- 2
- 1.0
- .8
- .6
- .5

EXAMPLE

SIZES 36" X 22"

Q = 20 CFS

Scale	HW*/D	HW (ft)
(1)	1.10	2.0
(2)	1.15	2.1
(3)	1.22	2.2

*D IN FEET

EXAMPLE

HEADWATER DEPTH IN TERMS OF RISE (HW/D)

Scale (1): 4, 3, 2, 1.5, 1.0, .9, .8, .7, .6, .5, .4, .35

Scale (2): 4, 3, 2, 1.5, 1.0, .9, .8, .7, .6, .5, .4, .35

Scale (3): 4, 3, 2, 1.5, 1.0, .9, .8, .7, .6, .5, .4, .35

HW/D SCALE	ENTRANCE TYPE
(1)	HEADWALL
(2)	MITERED TO CONFORM TO SLOPE
(3)	PROJECTING

TO USE SCALE (2) OR (3) PROJECT HORIZONTALLY TO SCALE (1), THEN USE STRAIGHT INCLINED LINE THROUGH D AND Q SCALES, OR REVERSE AS ILLUSTRATED
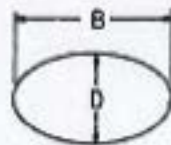
• ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

Source: 'Hydraulic Charts for the Selection of Highway Culverts', HEC-5, USDOT, FHA, Dec. 1965

Figure 7-8 Inlet Control Nomograph, RCP Arch

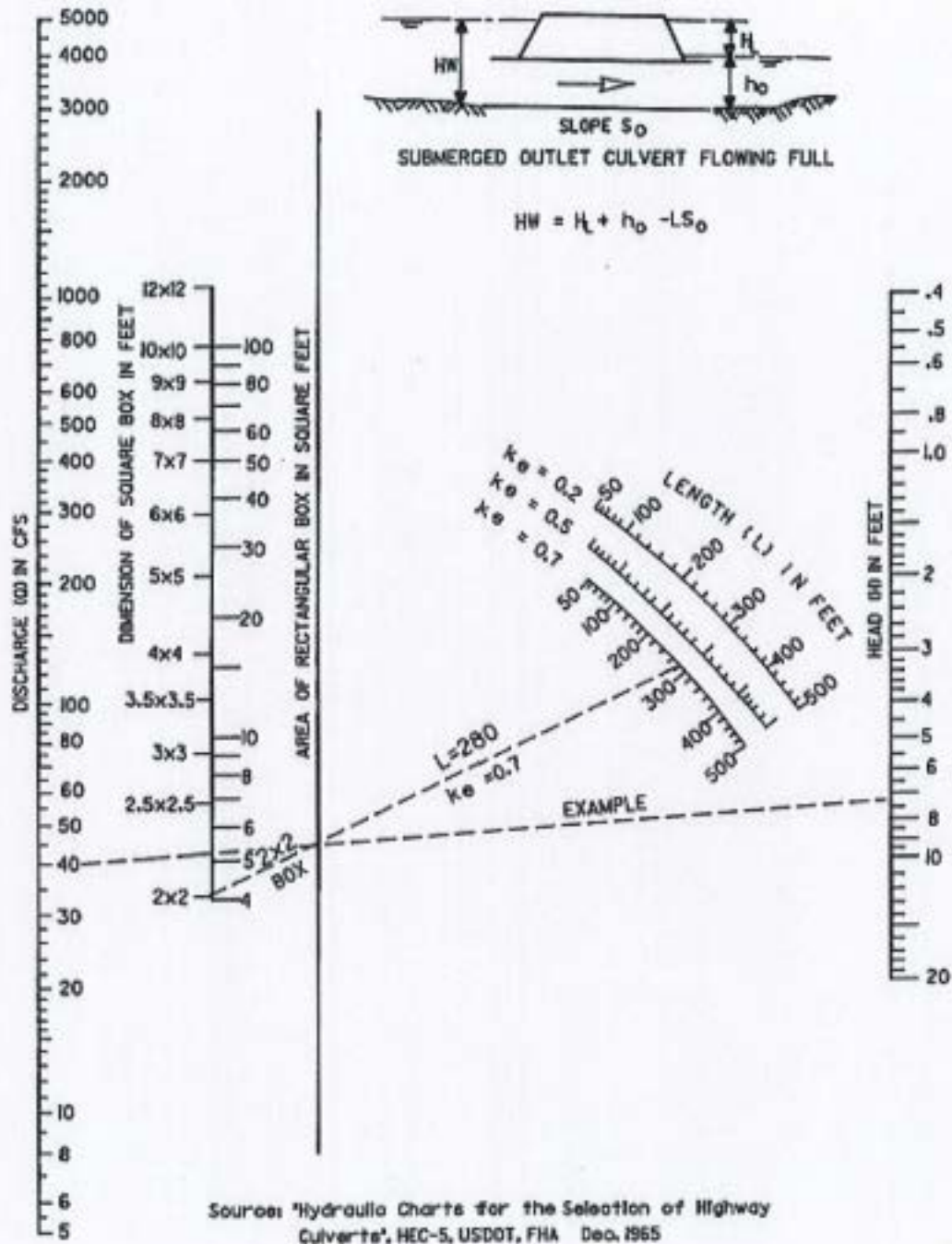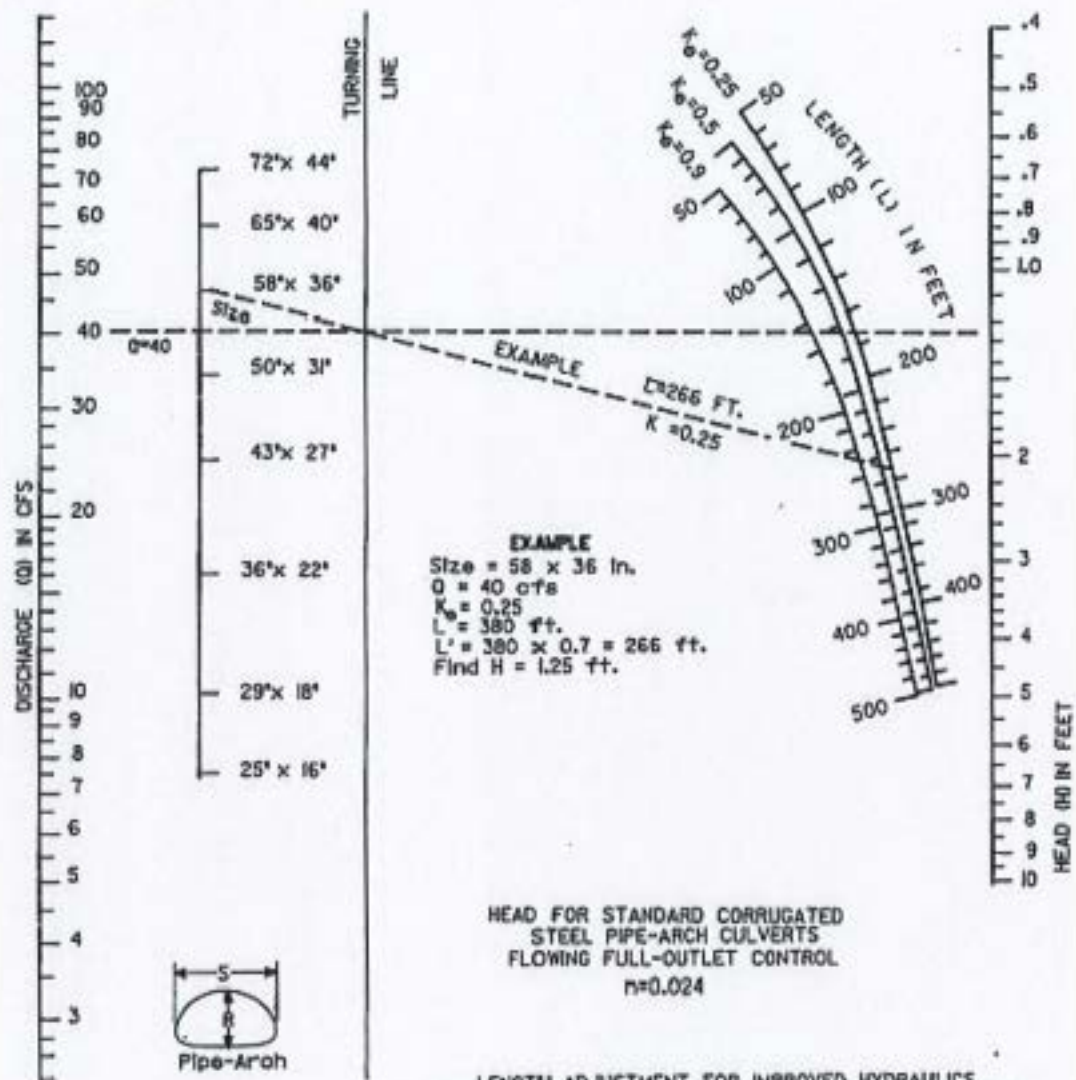SIZE: 44" X 27"
Q = 30 CFS

SCALE	HW*D	HW (ft)
(1)	1.02	2.30
(2)	0.99	2.23
(3)	1.01	2.2

*D IN FEET

TO USE SCALE (2) OR (3) DRAW A STRAIGHT LINE THROUGH KNOWN VALUES OF SIZE AND DISCHARGE TO INTER-SECT SCALE (1). FROM POINT ON SCALE (1) PROJECT HORIZONTALLY TO SOLUTION ON EITHER SCALE (2) OR (3).

HW/D SCALE	ENTRANCE TYPE
(1)	SQUARE EDGE
(2)	GROOVE END WITH HEADWALL
(3)	GROOVE END PROJECTING

Source: 'Concrete Pipe Design Manual', ACPA 1970
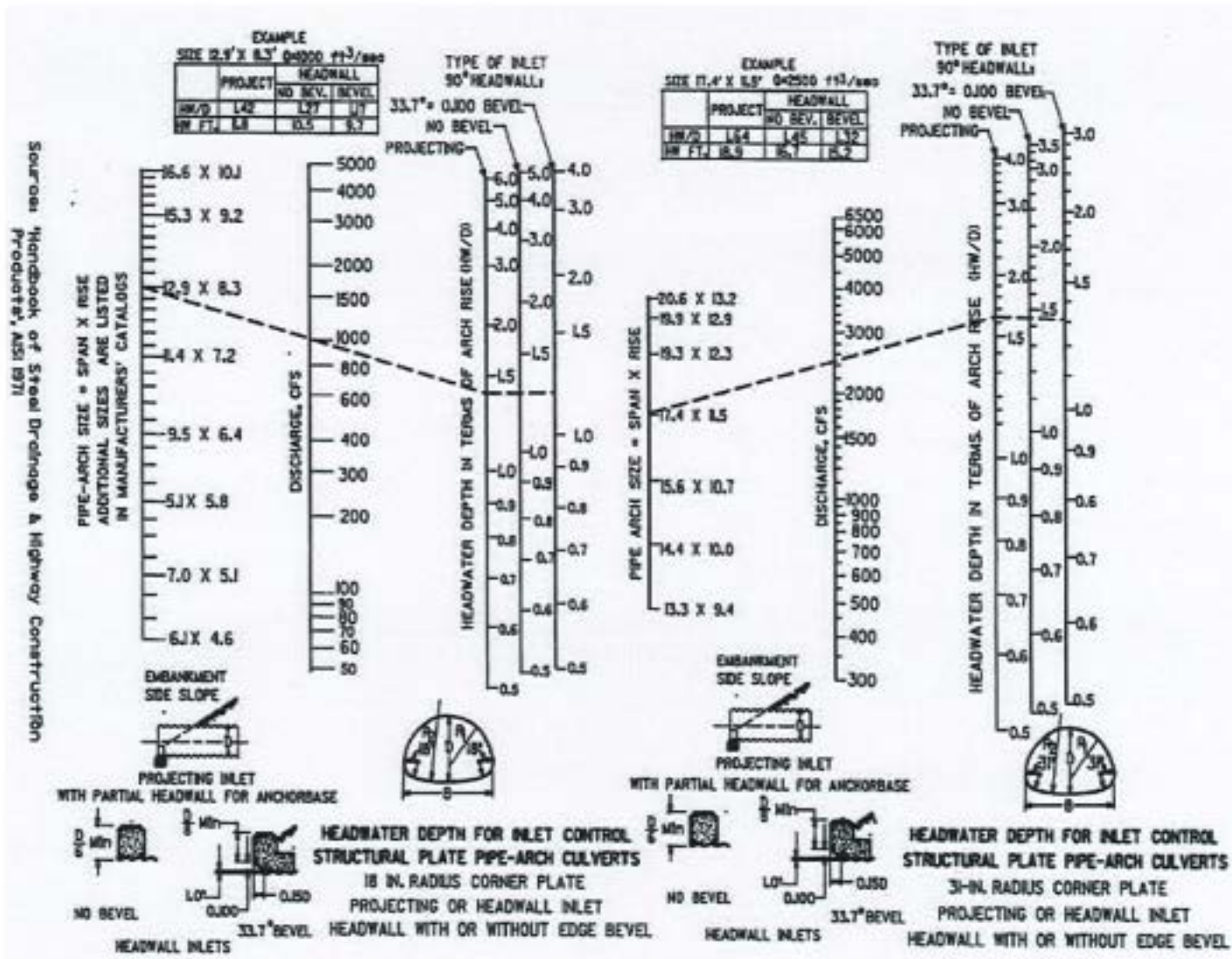
Figure 7-9 Inlet Control Nomograph, SSP Arch

Figure 7-10 Inlet Control Nomograph, RCP Elipse

EXAMPLE

SIZE: 76" x 48"
Q = 300 CFS

HW/D	HW (ft)	D
(1)	2.8	11.2
(2)	2.2	8.8
(3)	2.3	9.2

TO USE SCALE (2) OR (3)
DRAW A STRAIGHT LINE
THROUGH KNOW VALUES
OF SIZE AND DISCHARGE
TO INTERSECT SCALE (1)
FROM POINT ON SCALE (1)
PROJECT HORIZONTIALLY TO
SOLUTION ON EITHER SCALE
(2) OR (3).

HW/D SCALE	ENTRANCE TYPE
(1)	SQUARE EDGE WITH HEADWALL
(2)	GROOVE END WITH HEADWALL
(3)	GROOVE END PROJECTING
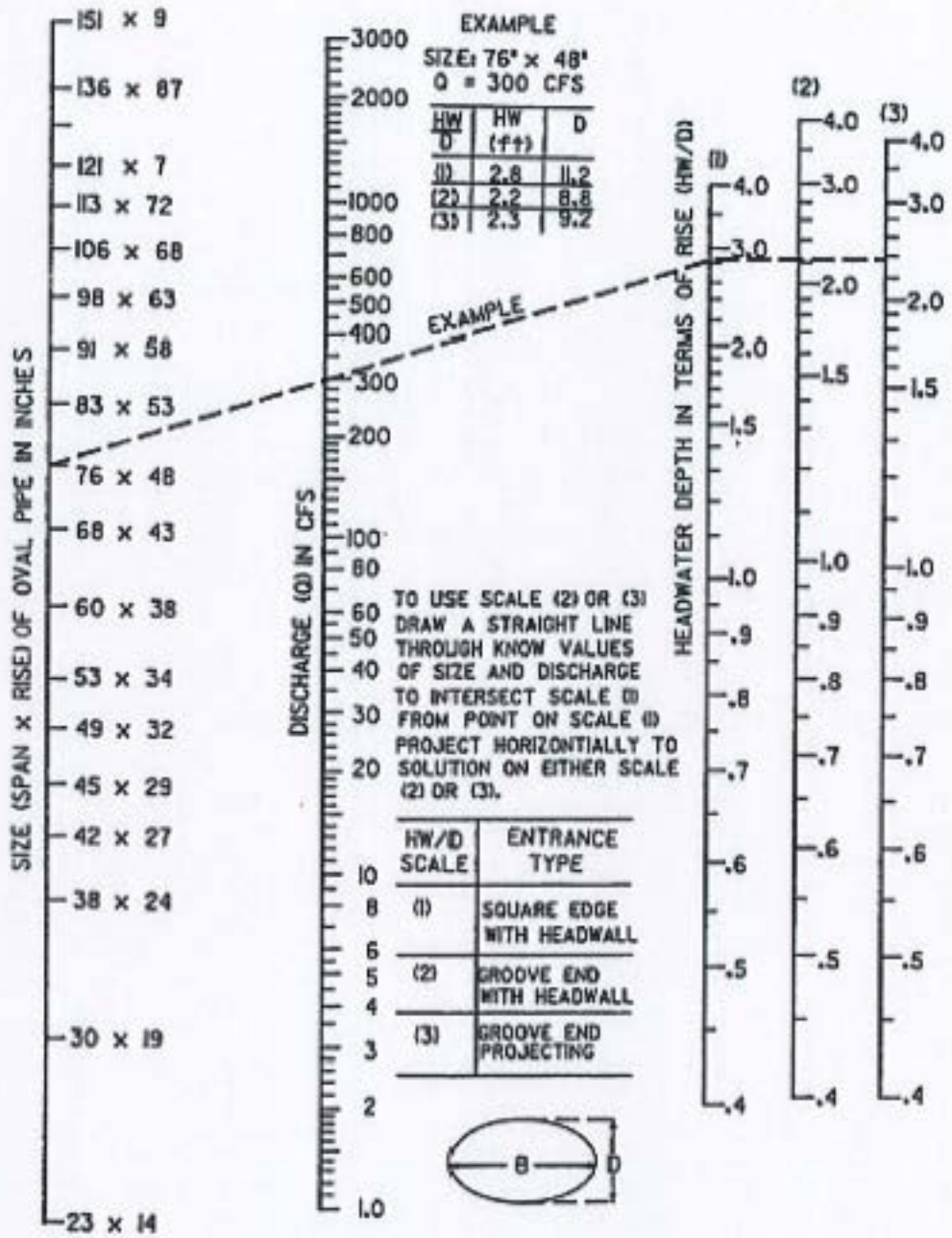
Source: "Hydraulic Charts for the Selection of Highway Culverts", HEC-5, USDOT, FHA Dec. 1965

INLET CONTROL NOMOGRAPH

Figure 7-11 Outlet Control Nomograph, Circular CSP

Figure 7-12 Outlet Control Nomograph, Circular RCP
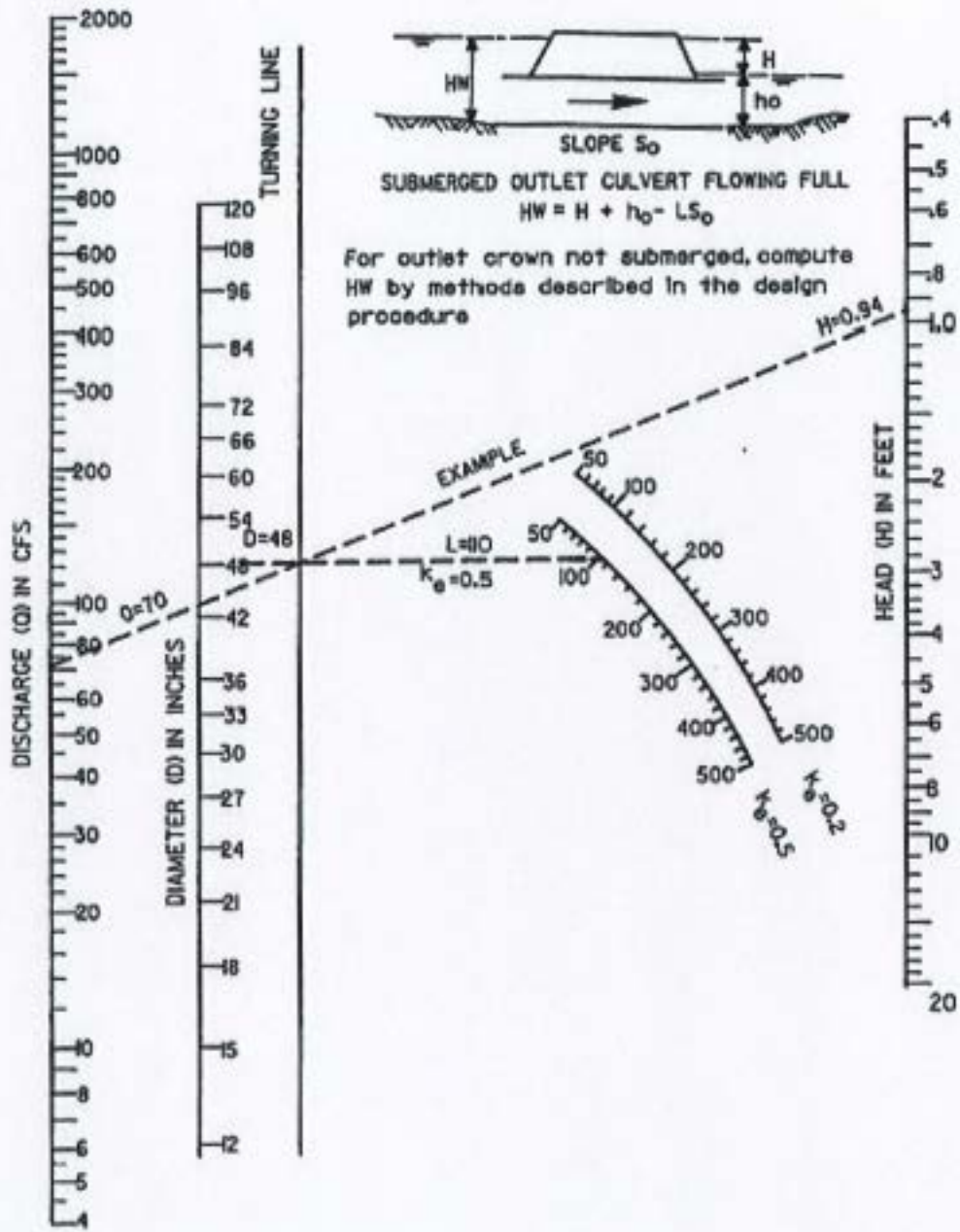
SLOPE S_0

SUBMERGED OUTLET CULVERT FLOWING FULL

$$HW = H + h_0 - LS_0$$

For outlet crown not submerged, compute HW by methods described in the design procedure

EXAMPLE

D=48

L=110

$k_e = 0.5$

H=0.94

Figure 7-13 Outlet Control Nomograph, Box Culverts

SUBMERGED OUTLET CULVERT FLOWING FULL

$$HW = H_L + h_o - LS_o$$

Figure 7-14 Outlet Control Nomograph, CSP Arch

EXAMPLE
Size = 58 x 36 in.
Q = 40 cfs
K_e = 0.25
L = 380 ft.
L' = 380 x 0.7 = 266 ft.
Find H = 1.25 ft.

HEAD FOR STANDARD CORRUGATED
STEEL PIPE-ARCH CULVERTS
FLOWING FULL-OUTLET CONTROL
n=0.024

Pipe-Arch

| PIPE DIAMETER IN INCHES	ROUGHNESS FACTOR n' FOR HELICAL CORR.	LENGTH ADJUSTMENT FACTOR $\left(\frac{n'}{n}\right)^2$
12	.011	.21
24	.016	.44
36	.019	.61
48	.020	.70

LENGTH ADJUSTMENT FOR IMPROVED HYDRAULICS

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

Figure 7-15 Outlet Control Nomograph, RCP Arch

SUBMERGED OUTLET CULVERT FLOWING FULL

$$HW = H_L + h_o - LS_o$$

Source: 'Concrete Pipe Design Manual', ACPA 1970
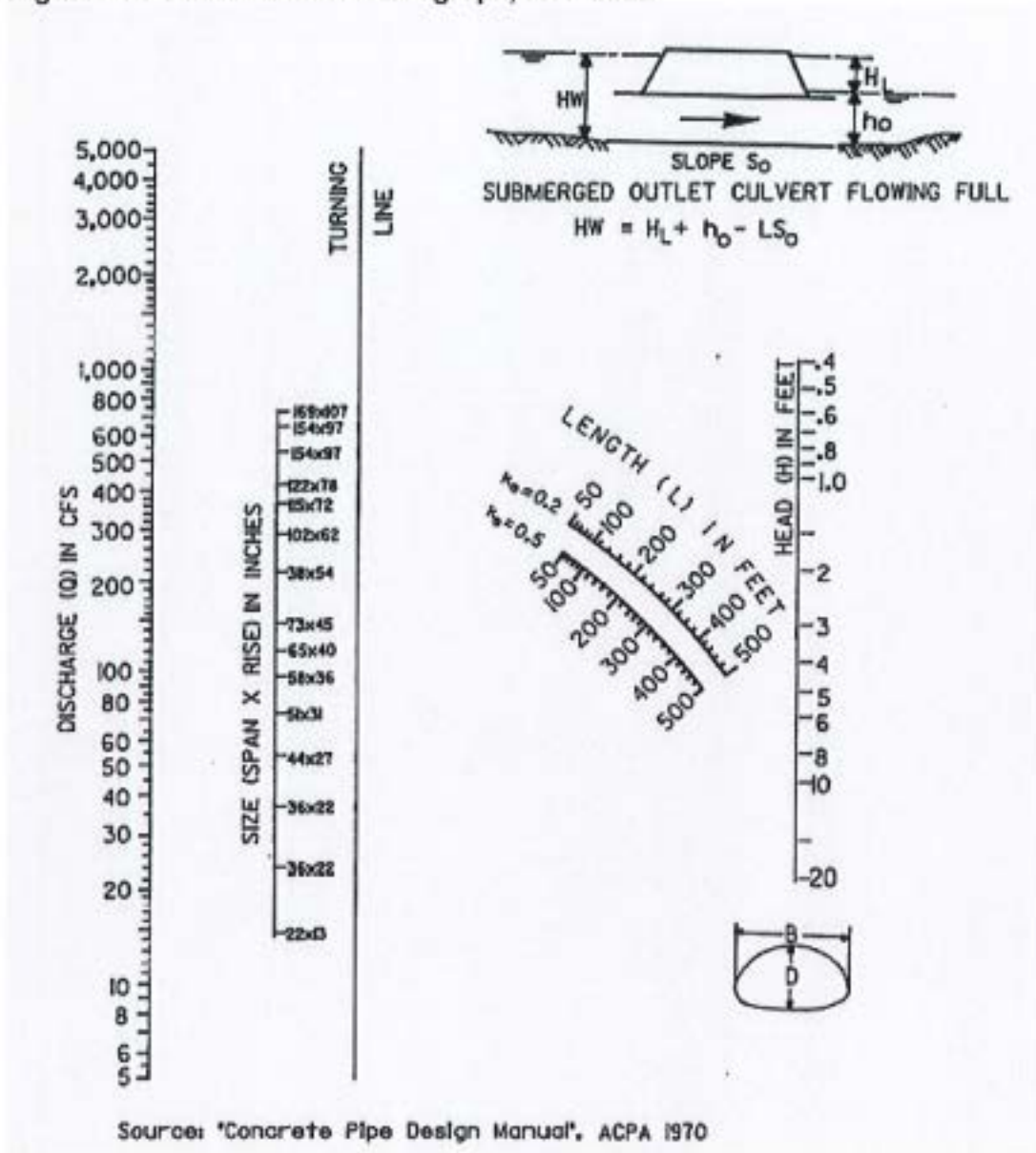
Figure 7-16 Outlet Control Nomograph, SPP Arch

SUBMERGED OUTLET CULVERT FLOWING FULL
$HW = H_L + h - LS_o$

EXAMPLE
SIZE-11.4 × 7.2
Q=400
K_a =0.5
L=172 ft.
n'= .030
L'=156
H=1.9 ft.

HEAD FOR STRUCTURAL PLATE PIPE-ARCH CULVERTS 18 IN. CORNER RADIUS FLOWING FULL OUTLET CONTROL
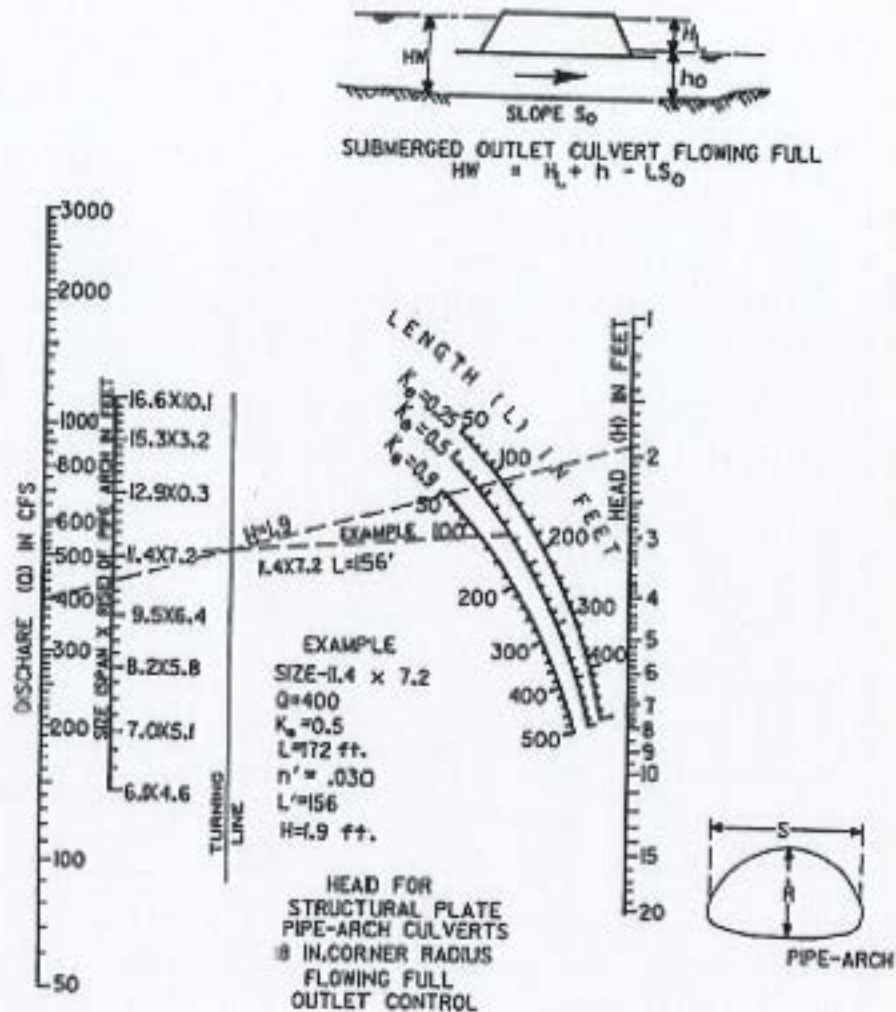
PIPE-ARCH

OUTLET CONTROL. Head for structural plate pipe-arch culvert with 18 in corner radius with submerged outlet and flowing full. For 31 in. corner radius, use structure sizes with equivalent areas on the 18-in. corner radius scale.

Length Adjustment for Improved Hydraulics

Pipe-Arch Size in Feet	Roughness Factor		Length Adjustment Factor $\left(\dfrac{n'}{n}\right)^2$
	Curves based on n	Actual n'	
6.1 × 4.6	.0327	.0327	1.0
8.1 × 5.8	.0321	.032	1.0
11.4 × 7.2	.0315	.030	0.907
16.6 × 10.1	.0306	.028	0.837

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

Figure 7-17 Outlet Control Nomograph, RCP Elipse
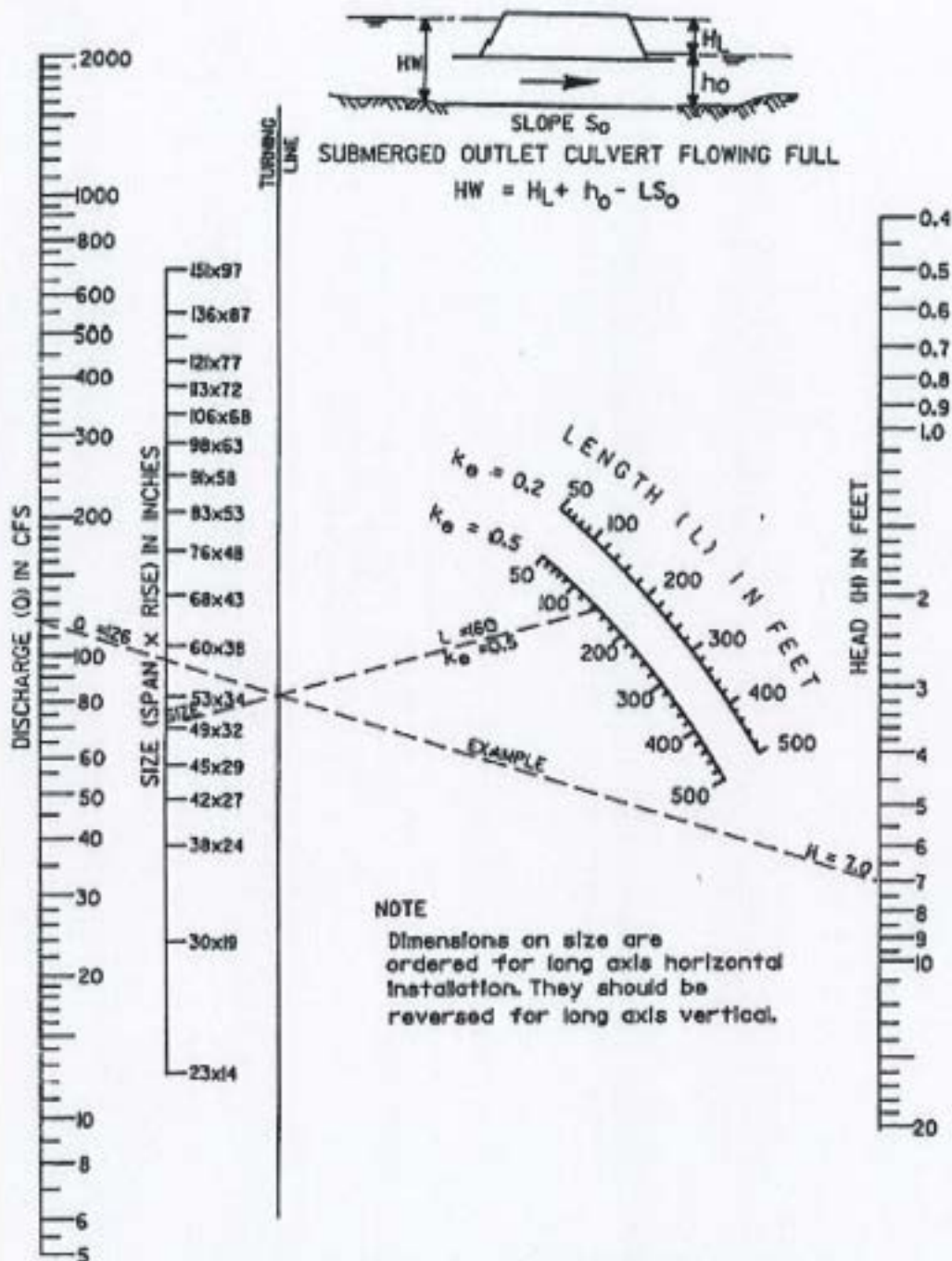
SUBMERGED OUTLET CULVERT FLOWING FULL

$$HW = H_L + h_o - LS_o$$

NOTE

Dimensions on size are ordered for long axis horizontal installation. They should be reversed for long axis vertical.

Source: "Hydraulic Charts for the Selection of Highway Culverts", HEC-5, USDOT, FHA Dec. 1965

Figure 7-18 Critical Depth Curves, Circular Pipe
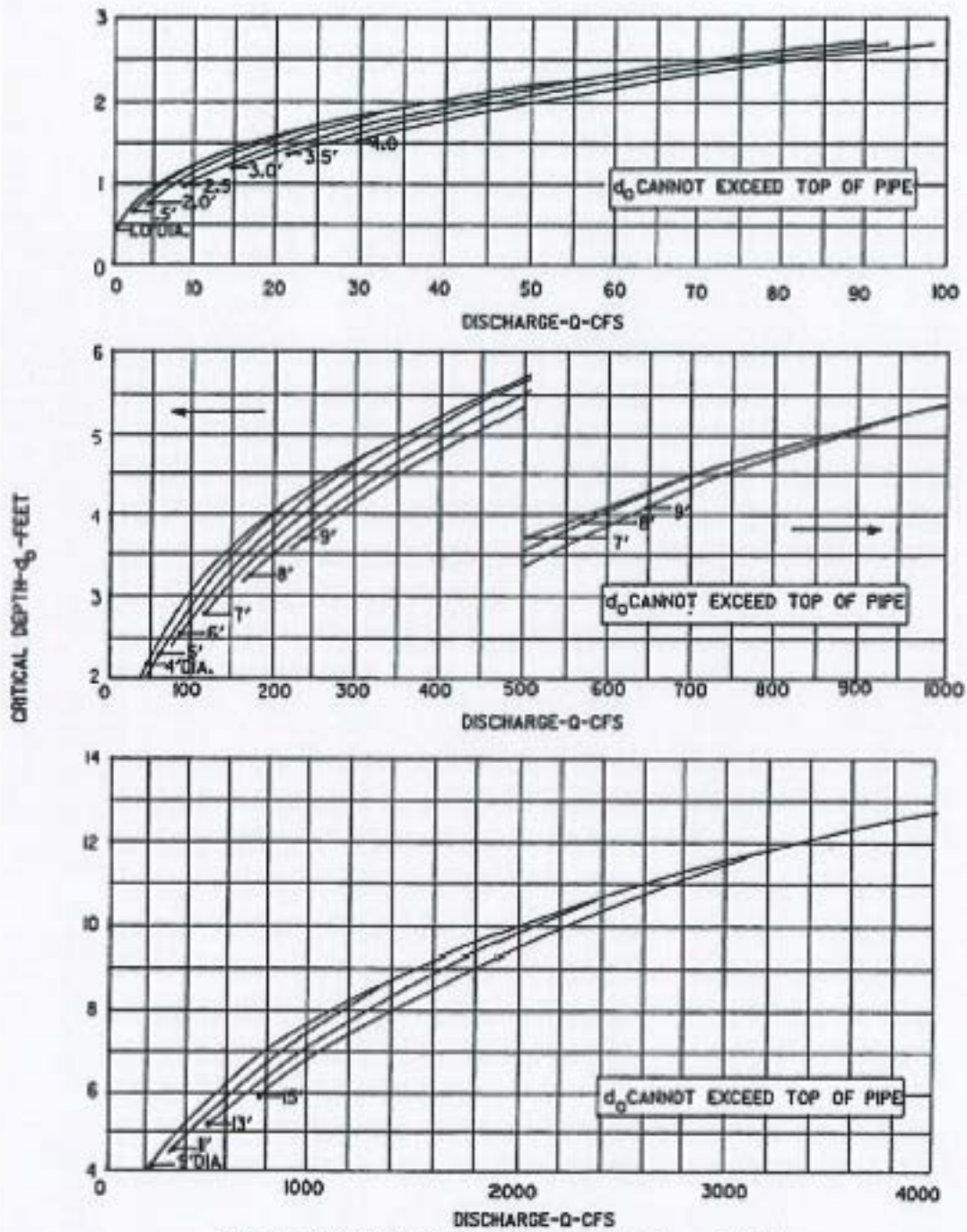
Source: 'Hydraulic Charts for the Selection of Highway
Culverts', HEC-5, USDOT, FHA   Dec. 1965

Figure 7-19 Critical Depth Curves, CSP Arch
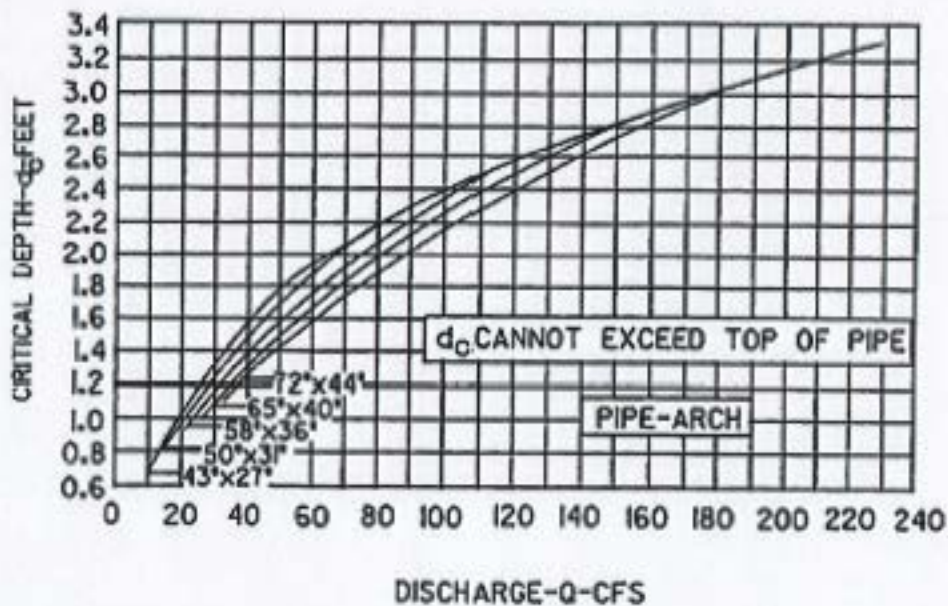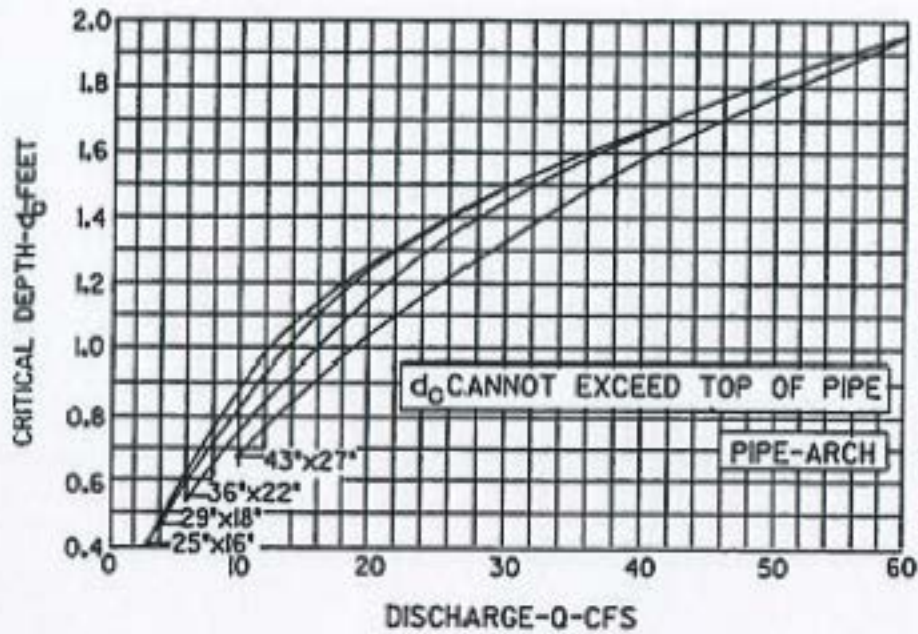
Source: "Handbook of Steel Drainage & Highway Construction
Products", AISI 1971
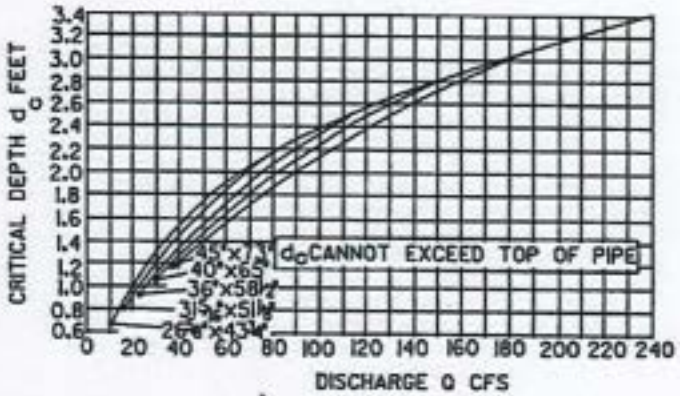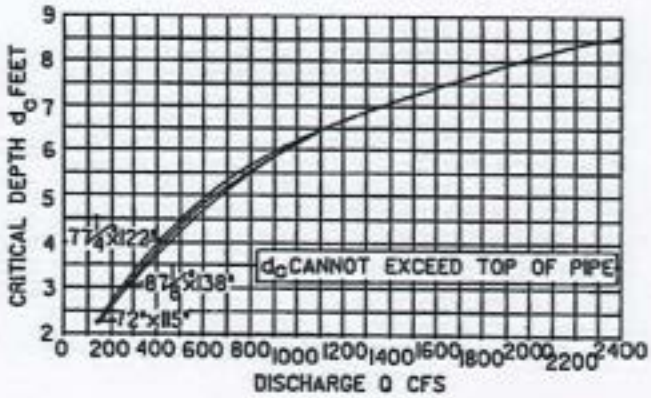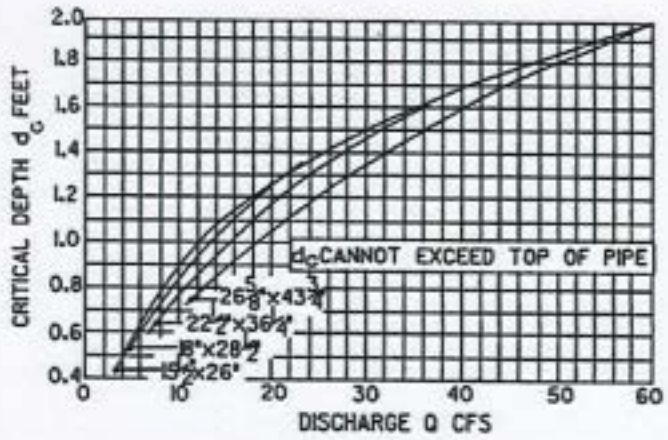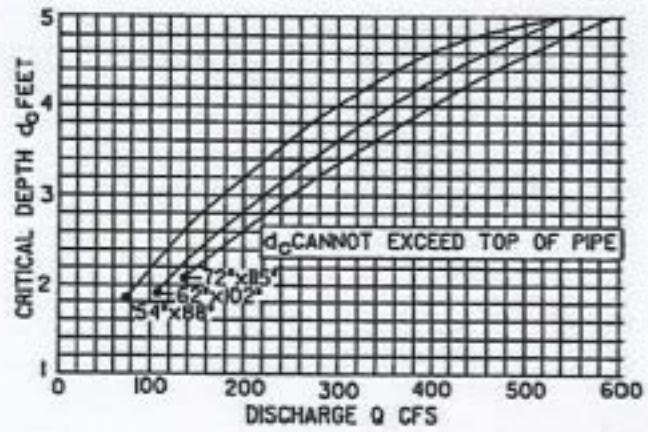
Figure 7-20 Critical Depth Curves, RCP Arch

Source: "Concrete Pipe Design Manual", ACPA, 1970

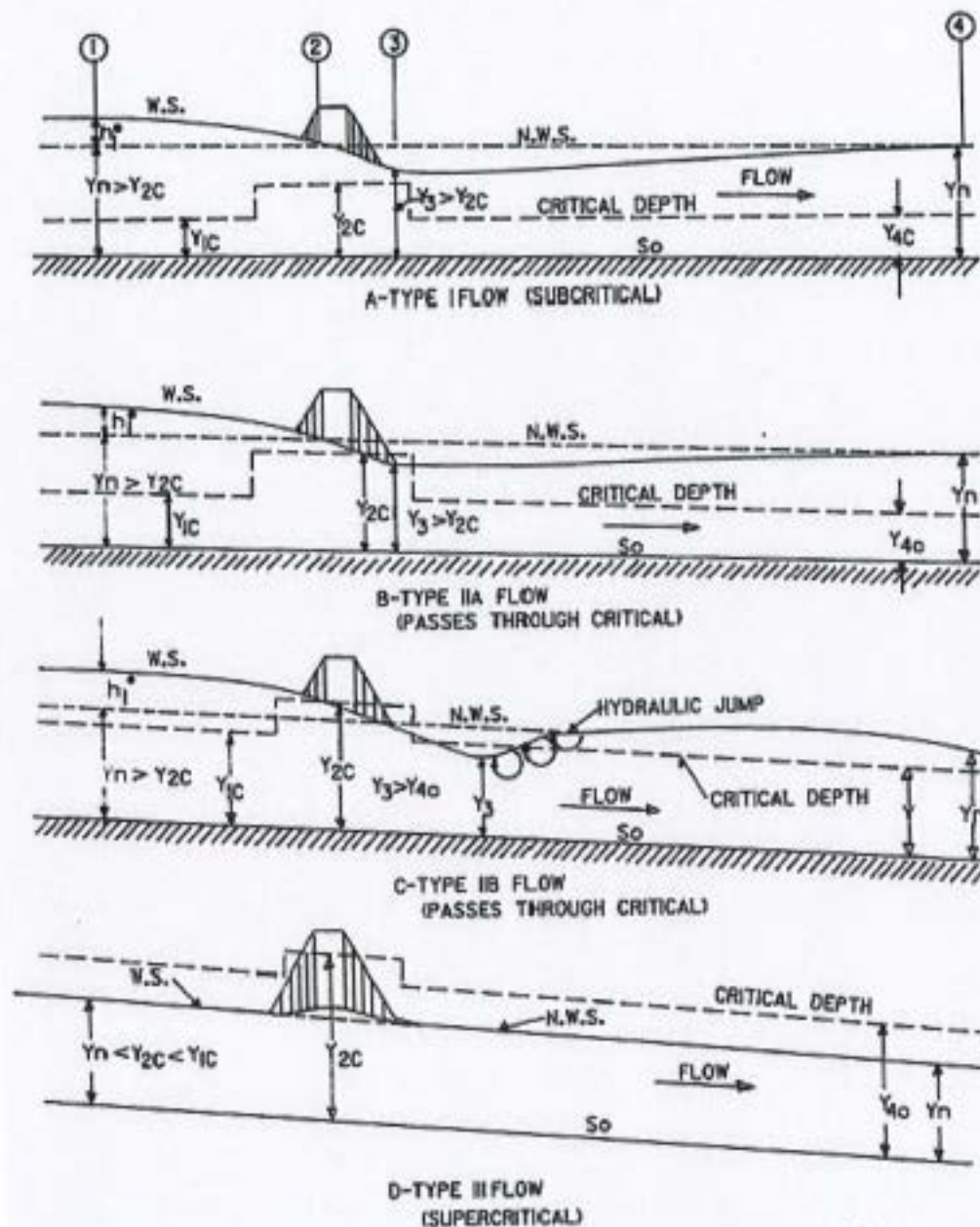## Figure 7-21 Critical Depth Curves, SSP Arch
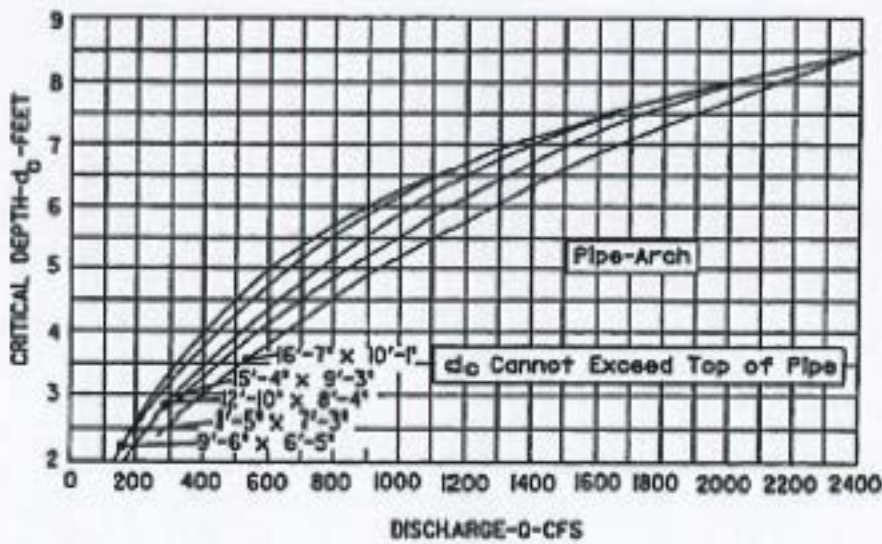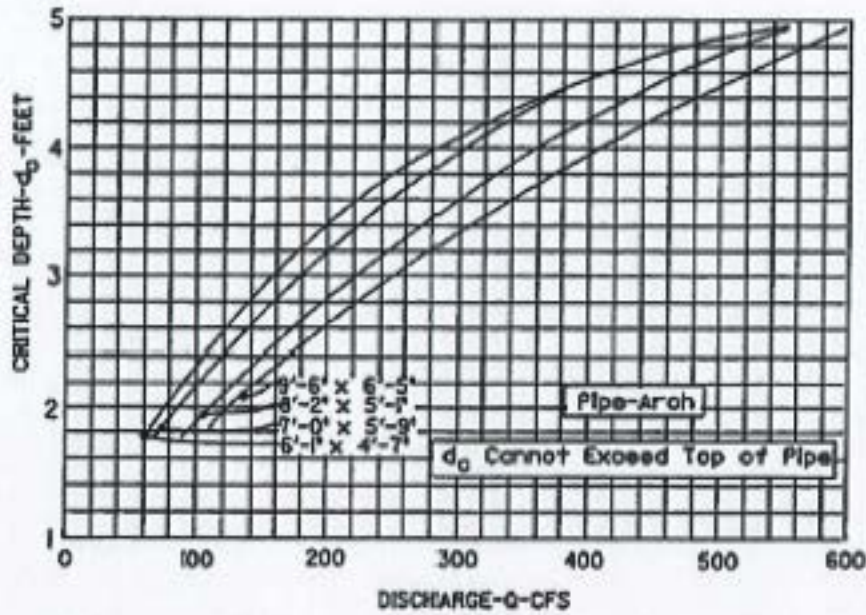
Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

Figure 7-22 Critical Depth Curves, RCP Elipse
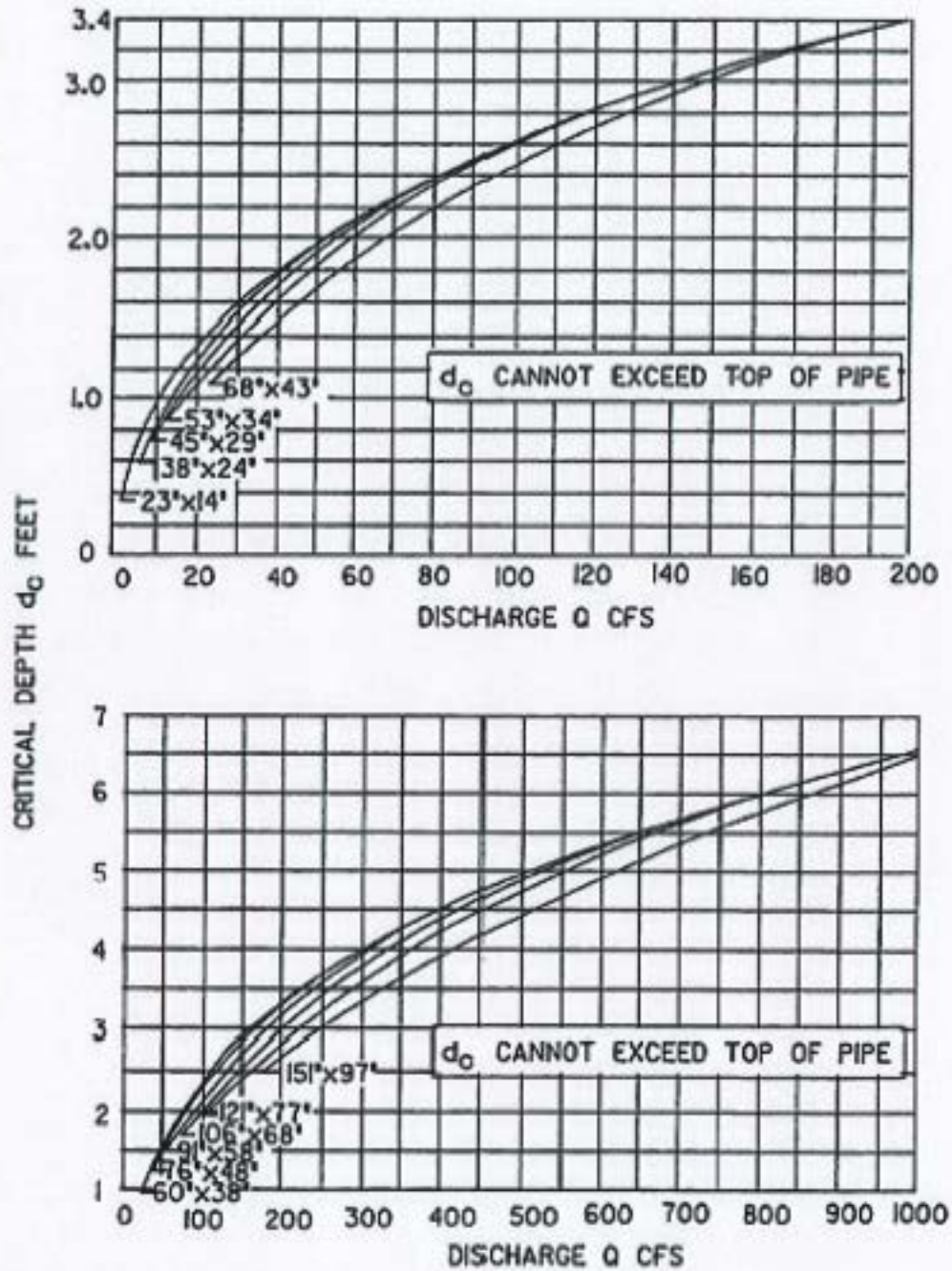
Source: "Concrete Pipe Design Manual", ACPA 1970

Figure 7-23 Types of Flow for Bridge Design

A-TYPE I FLOW (SUBCRITICAL)

B-TYPE IIA FLOW
(PASSES THROUGH CRITICAL)

C-TYPE IIB FLOW
(PASSES THROUGH CRITICAL)

D-TYPE III FLOW
(SUPERCRITICAL)

Source: Federal Highway Administration, "Hydraulics of Bridge Waterways,"
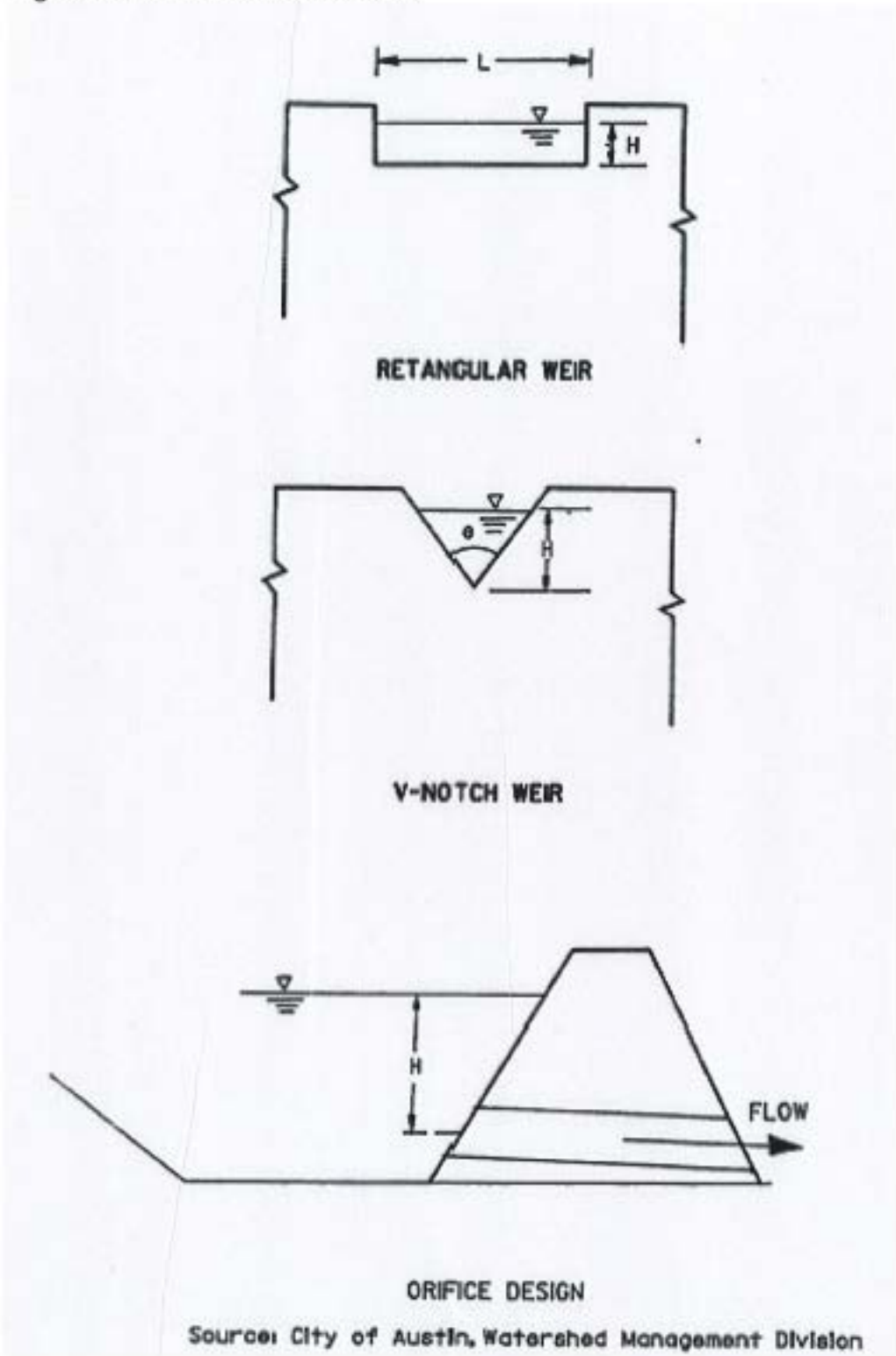Hydraulic Design Series No. 1, Second Edition, September, 1973.

Figure 8-1 Concept of Detention Pond
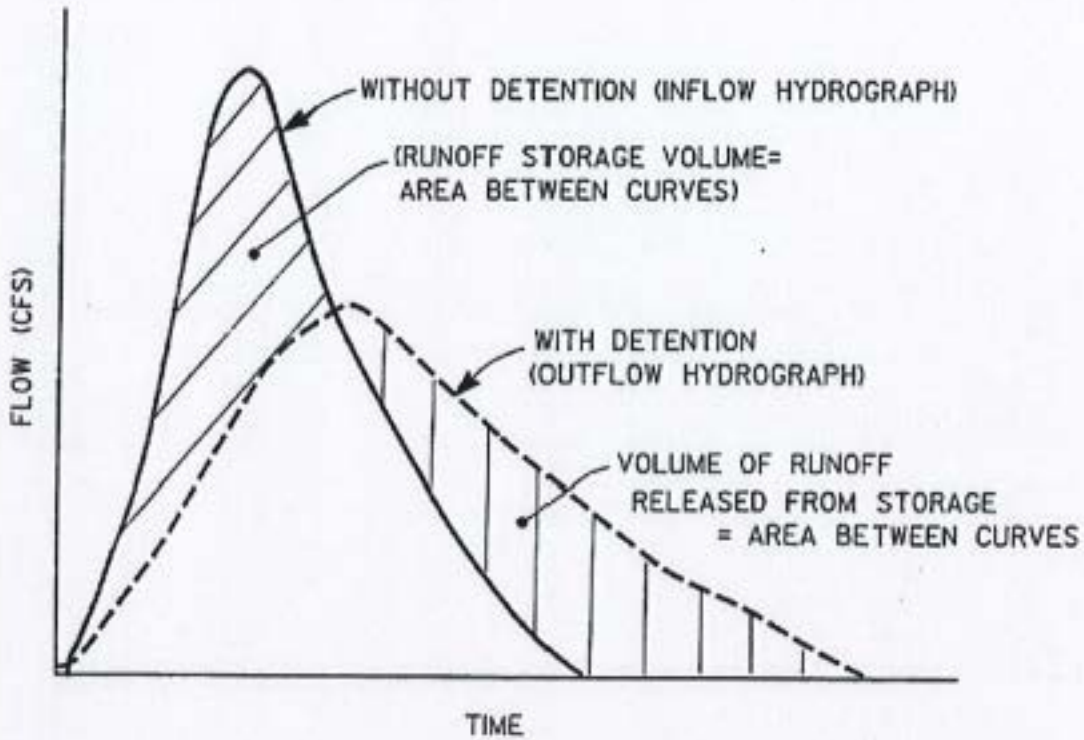
Source: City of Austin, Watershed Management Division

Figure 8-2 Weir and Orifice Flows

RETANGULAR WEIR

V-NOTCH WEIR

ORIFICE DESIGN

Source: City of Austin, Watershed Management Division

GLOSSARY

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

ACRE - A common variable unit of land measure, equal to 43,560 square feet or 1/640 square mile.

ACRE-FT - a unit of volume commonly used in the United States in reference to large-scale water resources, such as reservoirs, aqueducts, canals, sewer flow capacity, and river flows.

Apron - A floor or lining of concrete, timber, or other suitable material located at the inlet or discharge side of hydraulic structures (box culverts, spillways, etc.) designed 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 or outlet of a pond 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:

  (1) Cause a more uniform distribution of velocities

  (2) Dissipate energy

  (3) Divert, guide, or agitate the flow

  (4) Mitigate eddy currents.

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.

CFS - An abbreviation for cubic feet per second, a unit of measure of flow.

CFS/FT - An abbreviation for cubic feet per second per foot, a unit of measure of flow per foot of width.

Channel Roughness - The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the Manning Roughness Coefficient "n" used in the Manning Equation.

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

Concentrated Flow - Stormwater runoff that moves through an open waterway or channel that is bounded by banks or walls, such as a swale, ditch, creek, river, or open pipe or culvert.

Conduit - Any open or closed device for conveying flowing water.

Criteria - A standard or rule on which a judgment or decision is based.

Critical Flow - The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

Critical Slope - The minimum slope of a conduit which will produce critical flow

Crown - The highest point on a transverse section of a conduit or the highest point of a roadway cross section.

CSM/IN - An abbreviation for cubic feet per second per square mile per in of runoff, a unit of measure of flow for runoff.

Culvert - Pipe or other conduit through which flow passes under a road or street.

Curb - A vertical or sloping structure located 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.

Dam - A barrier constructed across a watercourse for the purpose of either temporarily or permanently impounding water.

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.

1. Off-site detention - A detention pond located outside the boundary of the area it serves.

2. Onsite detention - A detention pond which is located within the specific site or subdivision it serves.

3. On-stream detention - The temporary storage of storm runoff behind embankments or dams located in a channel.

4. Regional detention - Detention facilities provided to control excess runoff based on a watershed-wide hydrologic analysis.

Drainage Area - The area contributing storm runoff to a stream or drainage system at a particular point.

Drop Structures - The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes

Engineer - Shall mean a person who is duly licensed and registered to engage in the practice of professional engineering in the State of Texas.

Energy Grade Line - A line representing the energy in flowing water. The elevation of the energy line is equal to the summation of elevation of the flow line plus the depth, velocity head, and the pressure head.

Entrance Head - The head required to force 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.

Flood Control - The elimination or reduction of flood losses by the construction of flood storage reservoirs, channel improvements, dikes and levees, by-pass channels, or other engineering works.

Floodplain - Geographically the entire area subject to flooding.

FPS - An abbreviation of feet per sec, a unit of measure of velocity.

Freeboard - The distance between the calculated water surface elevation and the maximum physical elevation of the channel or pond, which is provided as an additional factor of safety.

Frequency (of storms, floods) - Average recurrence interval of events, over long periods of time. Mathematically, frequency is the reciprocal of the exceedance probability.

Friction Slope - The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradient, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.

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 (1) indicates supercritical flow and a value less than one (1) subcritical flow.

FT - An abbreviation for foot/feet, a unit of measure of length.

FT^2 - An abbreviation for square feet, a unit of measure of area.

FT/SEC^2 - An abbreviation for feet per square second, a unit measure of acceleration.

Gabion - A wire basket containing rocks which is placed uniformly with others to provide protection against erosion.

Grade - The inclination or slope of a channel, conduit, or natural ground surface, usually expressed in terms of the ratio of vertical rise to horizontal distance.

Gutter - A shallow concrete waterway adjacent to a curb for conveying street flow.

Headwall - The normal functions of properly designed headwalls and endwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence and to prevent adjacent soil from sloughing into the waterway opening.

Headwater - The upper reaches of a stream near its sources, the region where ground waters emerge to form a surface stream, or the headwater depth on the upstream side of a structure. (See Entrance Head)

HR - An abbreviation for hour, a unit of measure of time.

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 transitions to 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 or table showing discharge versus time at a given point on a stream or conduit.

1. Synthetic Hydrograph - Runoff or unit hydrographs which are devised by empirical means (as opposed to derivation based upon natural, measured data).

2. Unit Hydrograph - The direct runoff hydrograph resulting from one inch of precipitation excess distributed uniformly over a watershed for a specified duration.

Hydrology - The science that deals with the processes governing the depletion and replenishment of the water resources of the earth.

Hyetograph - A histogram or graph of rainfall intensity versus time for a storm.

Impervious - A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

IN - An abbreviation for inch, a unit of measure of length.

IN/HR - An abbreviation for inch per hour, a unit of measure of rainfall.

Infiltration - The absorption of water by the soil, either as it falls as precipitation, or from a stream flowing over the surface.

Inlet - The inflow point for a storm sewer system which is usually associated with streets (e.g., curb opening inlets, grate inlets, etc.).

Intensity - See Rainfall Intensity.

Invert - The floor, bottom, or lowest portion of the internal cross section of a conduit. Used particularly with reference to sewers, tunnels, and drains.

Lag Time - In hydrograph analysis lag time is the time from the centroid of the mass of excess rainfall to the peak of the runoff hydrograph.

Manning's Roughness Coefficient - The coefficient of roughness used in the Manning Equation.

Manning's Equation - A uniform flow equation used to relate velocity, hydraulic radius and the energy gradient slope.

May - A permissive condition. No requirement for design or application is intended.

MI - An abbreviation for mile, a unit of measure of length.

MIN - An abbreviation for minute, a unit of measure of time.

Must - This is a mandatory condition. Where certain requirements in the design or application of the guidelines are described with the "must" stipulation, it is mandatory that the requirements be met.

One Hundred (100) Year Storm - Size of storm equaled or exceeded on the average once in one hundred (100) years (with given duration), or that storm having a one (1) percent chance of occurring in any given year.

One Hundred (100) Year Flood - Size of flood which might be expected to be equaled or exceeded once in one hundred (100) years on the average, or has a one (1) percent chance of occurring in any given year. Usually associated with the one hundred (100) year storm.

Orifice - An opening with closed perimeter, and of regular form in a plate, wall, or partition, through which water may flow.

Overland Flow - Runoff which is not considered concentrated. Other term is sheet flow.

Peak Flow (Peak Rate of Runoff) - the maximum rate of flow past a particular point for a given storm.

Policy - A definite course or method of action selected to guide and determine present and future decisions.

Precipitation - Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.

Prismatic Channel - A channel built with unvarying cross section and constant bottom slope.

Probable Maximum Flood (PMF) - The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region.

Probable Maximum Precipitation - The critical depth-duration-area rainfall relationship which would result from a storm containing the most critical meteorological conditions considered probable of occurring.

Rainfall Duration - The length of time over which a discrete rainfall event lasts.

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 per hour.

Rational Formula - A traditional means of relating runoff from an area and the intensity of the storm rainfall (Q = CiA).

Reach - Any length of river or channel. Usually used to refer to sections which are uniform with respect to discharge, depth, area or slope, or sections between gaging stations.

Recommended - A condition which should be met if it is physically and economically reasonable to do so.

Required - This is a mandatory condition. Where certain requirements in the design or application of the guidelines are described with the "required" stipulation, it is mandatory that they be met.

Recurrence Interval - The average interval of time within which a given event will be equaled or exceeded once. For an annual series (as opposed to a partial duration series) the probability of occurrence in any one (1) year is the inverse of the recurrence interval. Thus a flood having a recurrence interval of one hundred (100) years has a one (1) percent probability of being equaled or exceeded in any one year.

Return Period - See Recurrence Interval.

Right-of-way - Land dedicated by a plat or separate instrument to and for use as a public roadway.

Riprap (Revetment) - Forms of bank protection, usually using rock. Riprap is a term applied to stone which is dumped rather than placed more carefully. In Austin concrete is often called riprap.

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 techniques may be classified into two categories - hydrologic routing and hydraulic routing.

Runoff - That part of the precipitation which reaches a stream, drain or sewer.

Runoff Coefficient (C) - A decimal number used in the Rational Formula which defines the runoff characteristics 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.

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

Shall - This is a mandatory condition. Where certain requirements in the design or application of the guidelines are described with the "shall" stipulation, it is mandatory that the requirements be met.

Sheet Flow - Stormwater runoff that flows downslope over relatively smooth surfaces in the form of a thin, continuous layer that does not vary in depth in a direction perpendicular to the direction of flow.

Should - An advisory condition. Where the word "should" is used, it is considered to be advisable usage, recommended but not mandatory.

Soffit - The bottom of the top of a pipe. In a sewer pipe, the uppermost point on the inside of the structure. In contrast, the crown is the uppermost point on the outside of the pipe wall.

Soil Conservation Service (SCS) Runoff Curve Number (CN) - Index number used by the SCS as a measure of the tendency of rainfall to run off into streams rather than evaporate or infiltrate.

Steady Flow - Open channel flow is said to be steady if the depth of flow does not change or if it can be assumed to be constant during the time interval of consideration.

Stilling Basin - Pool of water conventionally used, as part of a drop structure or other structure, to dissipate energy.

Tailwater - The depth of flow in the stream immediately downstream of a hydraulic structure.

Time of Concentration - The time associated with the travel of runoff from an outer point which best represents the shape of the contributing area.

Total Head - In the flow process, the total energy for a given point is represented by the summation of $V^2/2g$, p/γ and z. The units for these three (3) items are foot-pounds force per pound force. It is common practice to lump all these three (3) items together as total head in feet. The item of $V^2/2g$ is called velocity head (in feet) and p/γ is the pressure head (in feet).

Trunk Line - The primary collector line of a storm sewer system

Uniform Channel - A channel with a constant cross section 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, for a constant flow.

Unit Hydrograph - See Hydrograph.

Watershed - The total area contributing storm runoff to a stream or creek.

Weir - A weir is a notch of regular form through which water flows.

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