ORDINANCE NO. 2010-31

AN ORDINANCE OF THE CITY COUNCIL OF THE CITY OF COPPERAS COVE, TEXAS AMENDING SECTION 17.5-93(B); REPEALING ALL ORDINANCES IN CONFLICT WITH THIS AMENDMENT; PROVIDING A SAVINGS CLAUSE; DECLARING AN EFFECTIVE DATE.

WHEREAS, it has been determined that the City of Copperas Cove desires to adopt comprehensive drainage design criteria in the form of a Drainage Criteria Manual.

NOW, THEREFORE, BE IT ORDAINED BY THE CITY COUNCIL OF THE CITY OF COPPERAS COVE, TEXAS:

SECTION 1.

<u>.CNOA</u>

That the City's Code of Ordinances is hereby amended by amending Sec. 17.5-93(b) in its entirety to read as follows:

Sec. 17.5-93 Drainage Criteria

b. The most current Drainage Criteria Manual as approved by the City Council of the City of Copperas Cove is hereby adopted and included in its entirety as part of the Subdivision Ordinance. Any references to "Drainage Master Plan" or "DMP" shall be considered as referring to the Drainage Criteria Manual.

SECTION 2.

That all ordinances for which provision has heretofore been made are hereby expressly repealed if in conflict with the provisions of this ordinance.

SECTION 3.

That should any section, clause, or provision of this ordinance be declared by a court of competent jurisdiction to be invalid, the same shall not affect the validity of this ordinance or any other ordinance of the City as a whole or any part thereof, other ordinance of the City as a whole or any part thereof, other than the part so declared to be invalid.

SECTION 4.

That this ordinance shall go into effect on August 18, 2010.

PASSED, APPROVED AND ADOPTED this 17th day of August 2010, at a regular meeting of the City Council of the City of Copperas Cove, Texas, which meeting was held in compliance with the Open Meetings Act, *Tex. Gov't. Code* §551.001, et.seq., at which meeting a quorum was present and voting.



ATTEST: Jane Lees, City Secretary

APPROVED AS TO FORM:

Denton, Navárro, Rocha & Bernal, P.C., City Attorney



DRAINAGE CRITERIA MANUAL

CITY OF COPPERAS COVE 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 COPPERAS COVE 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 design engineer shall make provisions within the design. All plans must be signed and sealed by a Professional Engineer registered in the State of Texas.

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 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) inches 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) inches above the roadway crown elevation or three (3) inches 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.
 - 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 through a storm sewer inlet or culvert. No utilities shall be allowed to cross under a storm sewer inlet.
 - 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) feet horizontal to one (1) feet vertical shall be provided to allow access to drainage channels and detention ponds. The minimum bottom width for any channel with vegetative side slopes shall be four (4) feet.
 - 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) feet wide for ponds with depths of five (5) feet or less (back slopes included) and a minimum of fifteen (15) feet wide for ponds over five (5) feet deep or with back slopes in excess of five (5) feet 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 be allowed with a minimum of 12 inch diameter rock or stone.
- 5. Storm drains between lots (crossing blocks) shall be avoided as much as possible. When unavoidable, such drains shall be underground storm drains, located entirely on one (1) lot, laid along an alignment that retains the conduit within the dedicated drainage easement. Storm drains along rear of residential lots (through back yards) shall not be permitted. Easements shall be a minimum of fifteen (15) feet in width or 1.5 times the depth of the storm drain, whichever is greater. Fences may cross easements with underground facilities, but may not run parallel. Fences may not cross or run parallel within drainage easements designed for surface flow.
- 6. All bends, wyes and pipe size changes in storm sewers shall be prefabricated or shall occur at manholes/junction boxes. All alignment changes of 45 degrees or more shall occur at a manhole or junction box.
- 7. Bedding of storm sewer shall be to six (6) inches above the top of pipe or to current Public Works Standards (whichever is greater).
- 8. Storm drains shall be reinforced concrete pipe (RCP), ASTM C76, minimum Class III, and minimum eighteen (18) inch diameter. The Engineer shall provide load analysis to the Engineering 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) feet of cover in unpaved areas and a minimum of one and five tenths (1.5) feet of cover from bottom of the 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 in diameter 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 for all changes in alignment less than 45 degrees. 45 degree alignment changes require a manhole or junction box.
- 12. Channels
 - a. Concrete Channels

Concrete channels shall be of sufficient cross section and slope (minimum 0.5%) 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 (minimum 1.0%) but with velocities that will not be so great as to create erosion. Side slopes shall

not be steeper than three (3) feet horizontal to one (1) feet vertical for channels four (4) feet or less in depth and no steeper than four (4) feet horizontal to one (1) feet 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 eighty five percent (85%) established, with no bare spots greater than ten (10) square feet, prior to acceptance by the City of Copperas Cove. If vegetation cannot be adequately established prior to the desired acceptance date, up to three (3) months additional grow-in time may be granted by the City Engineer. Such an extension must be requested in writing with details of the efforts to be taken to ensure adequate vegetation will be established within 3 months. Extension requests must also be accompanied by an irrevocable line of credit, surety, or maintenance bond equal to one hundred percent (100%) of the cost to fully sod the entire area to be vegetated. This guarantee shall be separate from any other required maintenance bonds.

- c. Major streams 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), 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. Final determination of any cost sharing will be determined by the City Council through a development agreement.

1.2.7 Computations

A. Computations to support all drainage designs shall be submitted to the appropriate City Departments 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 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 design 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 is 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. Available capacity downstream shall not be considered as sufficient justification to waive detention.

1.2.9 Flood Plain Management

A. City of Copperas Cove

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 design 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) two (2) 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 Copperas Cove 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 with revised floodplain delineation included, the applicant must provide to the City evidence of a FEMA approved CLOMR/CLOMA or LOMR/LOMA.
 - b. Prior to issuance of building permits on lots within the current FEMA FIRM floodplain, the applicant must provide to the City evidence of final acceptance by FEMA of the LOMR/LOMA 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 issuance of building permits on affected lots, 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 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 construction plans. Finished floor elevations shall be a minimum of one (1) feet above the average top of curb elevation fronting the lot (one and a half (1.5) feet above the average edge of pavement where no curb is present). 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 can not 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) feet above the base flood elevations.
- D. Lot to lot drainage is prohibited except in residential developments where one (1) lot may drain onto one (1) adjacent lot to the rear. Residential lots may not drain from side to side unless directly adjacent to a city maintained facility (right-of-way or easement). The cumulative storm water runoff on any single residential lot may not exceed the cumulative storm water runoff generated from a total of two (2) residential lots.
- E. The applicant for a building permit for a developed lot that is graded from front to rear shall prepare a detailed site grading plan that includes elevations for all corners of the subject lot, all corners of the downstream 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.
- **F.** All earthen swales must have a minimum of one percent (1%) slope.
- **G.** Easements must be a minimum of fifteen (15) feet wide or 1.5 times the depth of any buried pipe, whichever is greater. All easements must be located entirely on one (1) lot.

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. Erosion control protection must be provided along all disturbed areas adjacent to city maintained facilities. Such measures must be installed prior to city acceptance and must be maintained until a certificate of occupancy is issued on the property. 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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Drainage Criteria Manual

SECTION 2 - DETERMINATION OF STORM RUNOFF

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 <u>Figure 2-1</u> 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 paragraph 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 paragraph 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-year 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-year 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 <u>will be</u> 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 of 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 100 acres or less. For larger drainage systems, the Soil Conservation Service hydrologic methods (available in TR-20, 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/Graphical⁵ TR-20, HEC-1 or HEC-HMS						
Greater than 400 Acres	SCS TR-20, HEC-1 or HEC-HMS						
 VRIM, Variable Rainfall Intensity Method in Section 2.4.5 SCS, Tabular/Graphical and TR-20 Methods in Section 2.6.4 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 following equation:

Where:

- Q_p is defined as the peak runoff in cubic feet per second. Actually, Q_p is in units of inches per hour per acre. Since this rate of in/hr/ac differs from cubic feet per second by less than one (1) percent (1 in/hr/ac = 1.008 cfs), the more common units of cfs are used.
- C is the coefficient of runoff representing the ratio of peak runoff rate "Q_p" to average rainfall intensity rate "i" for a specified area "A".
- i is the average intensity of rainfall in inches per hour for a period of time equal to the time of concentration (t_c) for the drainage area to the point under consideration.
- A is the area in acres contributing runoff to the point of design.

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). On-site 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 character 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 five (5) minutes.

A. **Sheet Flow.** Sheet flow is shallow flow over land surfaces which usually occurs 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) feet or less. The following equation 2-2 has been developed for sheet flow of less than three hundred (300) feet.

$$t_c = Ln/(42s^{0.5})$$
 (Eq. 2-2)

where,

- t_c = Time of concentration in minutes
- L = Length of the reach in ft.
- n = Manning's n (see Table 2-3)
- s = Slope of the ground in ft/ft
- B. **Shallow Concentrated Flow.** After a maximum of three hundred (300) feet 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:

where,

- t_c = Time of concentration in minutes
- L = Length of the reach in ft.
- n = Manning's n (see Table 2-3)
- s = Slope of the ground in 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 "n" values for channels can be obtained from Section 6 of this Manual.

(Eq. 2-3)

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

$$V = Q/A$$
 .

Where:

- V = Average velocity, ft/s Q = Design discharge, cfs
- A = Cross-sectional area, ft^2

TABLE 2-2								
RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS Runoff Coefficient (C)								
				Return F	Period			
Character								
of Surface	2	5	10	25	50	100	500	
	Years	Years	Years	Years	Years	Years	Years	
DEVELOPED		•	•	1		•	I	
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 (La	wns, Par	ks, 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	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	

(Eq. 2-4)

TABLE 2-2 (Continued)							
RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS Runoff Coefficient (C)							
			Retur	n Period			
Character of Surface	2 Years	5 Years	10 Years	25 Years	50 Years	100 Year	500 Years
Pasture/Range Flat, 0-2% Average, 2-7% Steep, over 7%	0.25 0.33 0.37	0.28 0.36 0.40	0.30 0.38 0.42	0.34 0.42 0.46	0.37 0.45 0.49	0.41 0.49 0.53	0.53 0.58 0.60
<u>Forest/Woodlands</u> Flat, 0-7% Average, 2-7% Steep, over 7%	0.22 0.31 0.35	0.25 0.34 0.39	0.28 0.36 0.41	0.31 0.40 0.45	0.35 0.43 0.48	0.39 0.47 0.52	0.48 0.56 0.58
* Grass cover less than 50 percent of the area. ** Grass cover on 50 to 75 percent of the area. *** Grass cover larger than 75 percent of the area. Source: 1. Rossmiller, R.L. "The Rational Formula Revisited." 2. City of Austin, Watershed Engineering Division							

	AND SHALLOW CONCENTRATED FLOW						
Manning's "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						

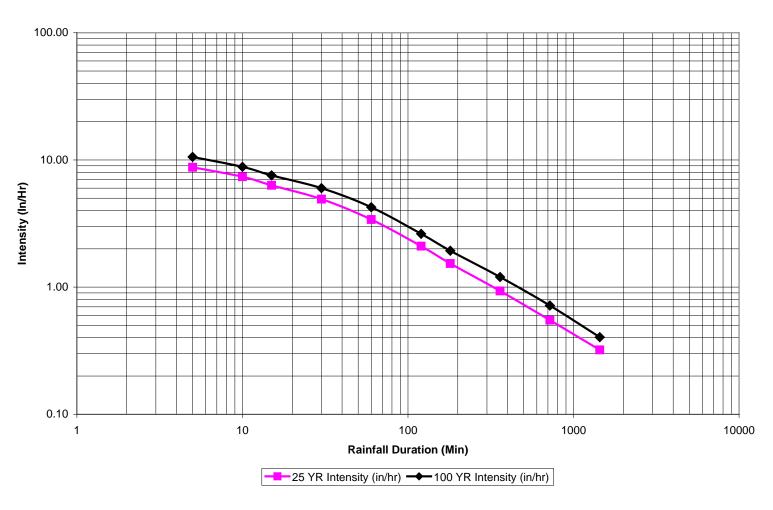
2.4.3 Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour, 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 Copperas Cove must be designed to intercept and carry the runoff from a twenty-five (25) year frequency storm, with an auxiliary or overflow system capable of carrying a one hundred (100) year 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 Copperas Cove intensity-duration-frequency curves are shown in <u>Figure 2-2</u> in Appendix B of this Manual.



Intensity Duration Frequency Curve for Copperas Cove, TX

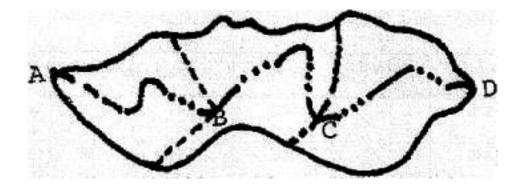
The intensity-duration-frequency curves and the intensity-duration equations are applicable for all design frequencies shown and for storm durations from five (5) minutes to 3 hours. They are required for use in determining peak flows by the Rational Method or other appropriate methods.

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

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



Reach	Description of Flow	Slope (%)	Length (Ft.)	Drainage Area (Acre)	"n" Value
A to B	Sheet flow (grass lawn)	4.5	300	3	0.3
B to C	Shallow concentrated flow (gutter)	2.0	840	20	0.016
C to D	Storm drain with inlets n=0.015D=3	1.5	1,200	30	

For reaches A-B and B-C, the time of concentration can be calculated from Equations 2-2 and 2-3.

$$\begin{split} t_c \ (A-B) &= 300(0.3)/42(s)^{0.5} \\ &= 2.14/(0.045)^{0.5} \\ &= 10.1 \ \text{min.} \\ t_c(B-C) &= 840(0.016)/60(s)^{0.5} \\ &= 0.22/(0.02)^{0.5} \\ &= 1.6 \ \text{min.} \end{split}$$

The flow velocity in reach C-D needs to be calculated from Manning's Equation, 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}
= 10.0ft/s

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

Reach	Length (ft.)	Velocity (fps)	t _c (min)	С	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 year storm (from <u>Figure 2-2</u> in Appendix B of this Manual) for 13.7 minutes = 9.2 inches per hour.

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

Thus the peak flow $Qp = CiA = 0.80 \times 9.2$ in/hr X 53 acre = 390 cfs

2.4.5 Variable Rainfall Intensity Method

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

Example 2-2

Variable Rainfall Intensity Method

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

Drainage area = one hundred (100) acres

Runoff coefficient C = 0.45

Design rainfall frequency: ten (10) year

Copperas Cove rainfall intensity-duration-frequency curves (Figure 2-2 in Appendix B of this Manual)

Time of concentration = forty (40) minutes.

Find: The ten (10) year design storm and resulting flood hydrograph.

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

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

Column 1:	Duration (minutes) = 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 (inches) = total precipitation for storm
	of specified duration (from Table 2-11).
Column 4:	Incremental Depth (inches) = difference in total

precipitation between specified duration and duration of five (5) minutes less than specified duration (e.g., P35 minutes - P30 minutes). Incremental Intensity (inches/hour) = Incremental Depth

Column 5: Incremental Intensity (inches/hour) = Incremental Depth (inches) x (60 minutes/hour)/(five (5) minutes).

	Table 2-6Development Of A Ten (10) Year Frequency Storm								
Duration (Min) (1)	Intensity (In/hr) (2)	Accumulated Depth (In) (3)	Incremental Depth (In) (4)	Incremental Intensity (In/hr) (5)					
5	8.64	.034	0.34	.41					
10			0.36	.43					
15	6.16	.108	.038	.46					
20			.04	.48					
25	5.00	.19	.04	.48					
30			.05	.60					
35	4.30	.29	.05	.60					
40			.06	.72					
45	3.73	.41	.06	.72					
50			.07	.84					
55	3.33	.56	.08	.96					
60			.09	1.08					
65	3.00	.76	.11	1.32					
70			.13	1.56					
75	2.74	1.07	.18	2.16					

	Table 2-6 (Continued)Development Of A Ten (10) Year Frequency Storm								
Duration (Min) (1)	Intensity (In/hr) (2)	Accumulated Depth (In) (3)	Incremental Depth (In) (4)	Incremental Intensity (In/hr) (5)					
80			.24	2.88					
85	2.50	1.67	.36	4.32					
90			.72	8.64					
95	2.32	2.89	.5	6.0					
100			.29	3.48					
105	2.17	3.38	.20	2.4					
110			.15	1.8					
115	2.05	3.65	.12	1.44					
120			.1	1.2					
125	1.94	3.83	.08	.96					
130			.08	.96					
135	1.85	3.98	.07	.84					
140			.06	.72					
145	1.77	4.09	.05	.60					
150			.05	.60					
155	1.69	4.19	.05	.60					
160			.04	.48					
165	1.62	4.27	.04	.48					
170			.04	.48					
175	1.56	4.34	.03	.36					
180			.03	.36					
185	1.50	4.38		.36					

Table 2-7 illustrates the computed 10 year 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 (minutes) = time from the beginning of the storm.

Column 2: i (inches/hour) = 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 (cubic feet per second) = column 6 x "C" x A (for the 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 five (5) minute increments doubled to ten (10) minute 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 one half (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 Minutes And C = 0.45								
Time (Min) (1)	l ₁₀ (In/Hr) (2)	Sum I ₁₀ (3)	Sum I ₁₀ (Lagged 40 min) (4)	Time (3) - (4) (5)	Q40 (In/Hr) (6)	Q (cfs) (7)	Folded (8)		
0							330		
5	0.41	0.41		.41	.05	2.3	320		
10	0.43	0.84		.84	.10	4.5	310		
15	0.46	1.3		1.3	.16	7.2	300		
20	0.48	1.78		1.78	.22	9.9	290		
25	0.48	2.26		2.26	.28	12.6	280		
30	0.6	2.86		2.86	.36	16.2	270		
35	0.6	3.46		3.46	.43	19.3	260		
40	0.72	4.18		4.18	.52	23.4	250		
45	0.72	4.9	.41	4.5	.56	25.2	240		
50	0.84	5.7	.84	4.9	.61	27.4	230		
55	0.96	6.7	1.3	5.4	.67	30.1	220		
60	1.08	7.8	1.78	6.0	.75	33.7	210		
65	1.32	9.1	2.26	6.8	.85	38.2	200		
70	1.56	10.7	2.86	7.8	.97	43.6	190		

	Table 2-7 (Continued)Runoff Computations From A 100 AcreArea With A Time Of Concentration Of40 Minutes And C = 0.45								
Time (Min) (1)	l ₁₀ (In/Hr) (2)	Sum I₁₀ (3)	Sum I ₁₀ (Lagged 40 min) (4)	Time (3) - (4) (5)	Q40 (In/Hr) (6)	Q (cfs) (7)	Folded (8)		
75	2.16	12.8	3.46	9.3	1.16	52.2	180		
80	2.88	15.7	4.18	11.5	1.44	64.8	170		
85	4.32	20.0	4.9	15.1	1.89	85.1	160		
90	8.64	28.7	5.7	23.0	2.87	129.1	150		
95	6.0	34.7	6.7	28.0	3.5	157.5	140		
100	3.48	38.1	7.8	30.3	3.8	171.0	130		
105	2.4	40.5	9.1	31.4	3.92	176.4	120		
110	1.8	42.3	10.7	31.6	3.95	177.7	(peak)		
115	1.44	43.8	12.8	31.0	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) hour storm duration, which is considered to be acceptable for the Copperas Cove area; however, the design storm most representative of the Copperas Cove area has a three (3) hour duration. 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 method 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: <u>A Method for Estimating Volume and Rate of Runoff in Small Watersheds</u>

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," <u>National Engineering Handbook</u>. 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 hydroiogic 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-55 or "Hydrology, Section 4," <u>National Engineering Handbook</u> should be used.

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

<u>Group A:</u> (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.

<u>Group B:</u> 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.

<u>Group C:</u> 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.

<u>Group D:</u> (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 claypan 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-55 publication. The minimum infiltration rates for the four (4) soil groups are:

Group	Minimum Infiltration Rate (in/hr)
А	0.30 - 0.45
В	0.15 - 0.30
С	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 Copperas Cove area, the SCS <u>Soil Survey of Coryell County, Texas</u> should be used.

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover type and Hydrologic Condition	Average % Impervious Area ¹	A	В	С	D
Fully developed urban ar	eas (vegetatio	n establi	shed)		
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		Curve Numbers for Hydrologic Soil Group					
Description							
Cover type and Hydrologic Condition	Average % Impervious Area ¹	Α	В	С	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 business Industrial	85 72	89 81	92 88	94 91	95 93		
Residential districts by average lot size: 1/8 acre or less (town houses) 1/4 acre 1/3 acre 1/2 acre 1 acre 2 acres	65 38 30 25 20 12	77 61 57 54 51 46	85 75 72 70 68 65	90 83 81 80 79 77	92 87 86 85 84 82		
Developing urban areas				I	l		
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94		
Agricultural lands				1			
Grassland, or range- continuous forage for grazing ²	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 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	Α	В	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 Fair Good	48 35 30	67 56 48	77 70 65	83 77 73
Woods—grass combination (orchard or tree farm). ⁴	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods⁵	Poor Fair Good	45 36 30	66 60 55	77 73 70	83 79 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 50 percent ground cover or heavily grazed with no mulch.
 Fair: 0 to 75 percent ground cover and not heavily grazed.
 Good: greater than 75 percent ground cover and lightly or only occasionally grazed.

³ Poor: less than 50 percent ground cover.
 Fair: 50 to 75 percent ground cover.
 Good: greater than 75 percent ground cover.

⁴ Curve numbers shown were computed for areas with 50 percent woods and 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. 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) feet, sheet flow for developed conditions should be limited to one hundred fifty (150) feet.

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 methods are considered acceptable for the Copperas Cove 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 sub-areas. 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) hour rainfall distributions. The engineer should apply those tables corresponding to a Type III rainfall distribution which is acceptable for the Copperas Cove 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) hours and reach travel times of zero (0) to three (3) hours. The discharge values were developed from the TR-20 program by computing hydrographs for a one square mile 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 for calculating the flow

at any time is:

$$=q_t A_m Q_v \tag{Eq. 2-6}$$

where,

q

- q = Hydrograph ordinate at hydrograph time t, cfs
- q_t = Individual value read from the tabular discharge tables, CSM/inch
- A_m = Drainage area of individual subwatershed, mi²
- Q_v = Total runoff volume, inches.

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(hrs)
 - 2. Drainage Area (mi²)
 - 3. Type III rainfall distribution
 - 4. 24-hr, rainfall (in.)
 - 5. CN

The peak discharge equation for the graphical method is:

 $q_p = q_u A_m Q$

(Eq. 2-7)

*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 Fp 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 <u>National Engineering Handbook</u> 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), an arithmetic equation for a storm without any initial abstraction can be expressed as:

$$F/S = Q/P$$

(Eq. S-1)

where,

Q = Actual runoff volume

P = Rainfall (P is equal or greater than Q)

F = Actual retention after runoff begins

S = Potential maximum retention after runoff begins (S is equal to or greater than

F)

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:

(Eq. S-2)

The actual runoff (Q) can be solved as:

$$Q = p^2/(p+s)$$
 (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 arithmetic expression becomes:

$$F/S = Q/(P-I_a)$$
(Eq.S-4)

where 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:

$$Q = ((P-I_a)+S)$$
 (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:

l_a=0.2S (Eq.S-6)

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

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

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

$$S = (1000/CN)-10$$
 (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/qp 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 <u>Figure 2-3</u> 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-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.

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

$$Q = q_p(T_p + T_r)/2$$
 (Eq.S-9)

where,

Q = Direct runoff, inches

 T_p = Time to peak, hours

T_r = Recession time, hours

q_p = Peak discharge, inches per hour

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:

$q_p = 2Q/(T_p + T_r)$	(Eq.S-10)
$q_{\mu} = \langle \langle \cdot \rho \cdot \cdot \cdot \rangle \rangle$	

Let $K = 2/(1 + T_r/T_p))$

therefore, $q_p = KQ/T_p$

where, Q = Direct runoff, inches

 T_p = Time to peak, hours

 T_r = Recession time, hours

q_p= Peak discharge, inches per hour

In making the conversion from inches per hour to cubic feet per second and putting the equation in terms ordinarily used, including drainage area (A) in square miles, and time (T) in hours, equation S-12 becomes the general equation:

q_p= (645.33 KAQ)/T_p

(Eq. S-13)

(Eq. S-11)

(Eq. S-12)

Where q_p is peak discharge in cubic feet per second and the conversion factor 645.33 is the rate required to discharge one (1) inch of excess rainfall from one (1) square mile in one (1) hour.

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:

$$q_p=484A \text{ Q/T}_p$$

(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 <u>Figure 2-3</u> 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	.000	.001		
0.1	.030	.001		
0.2	.100	.006		
0.3	.190	.012		
0.4	.310	.035		
0.5	.470	.065		
0.6	.660	.107		
0.7	.820	.163		
0.8	.930	.228		
0.9	.990	.300		
1.0	1.000	.375		
1.1	.990	.450		
1.2	.930	.522		
1.3	.860	.589		
1.4	.780	.650		
1.5	.680	.700		
1.6	.560	.751		
1.7	.460	.790		
1.8	.390	.822		
1.9	.330	.849		
2.0	.280	.871		
2.2	.207	.908		

	Hydrograph and mass Curve	9
Time Ratios (t/T _p)	Discharge Ratios (q/q _p)	Mass Curve Ratios (Q _a /C
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

SECTION 3 - STREET FLOW

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Drainage Criteria Manual <u>SECTION 3 - STREET FLOW</u>

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 and 100 year 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 year storm which is designated as the design storm. Conveyance provisions for the 100 year 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 three (3) cubic feet per second for the 25 year 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 three (3) cubic feet per second (cfs) for the 25 year storm shall enter the intersection. For a cul-de-sac with a slope greater than seven (7) percent, no more than three (3) cfs for the 25 year 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 where slope across the intersection is less than one and two tenths (1.2%) percent. 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 when slope across the intersection is less than one and two tenths (1.2%) percent. 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.

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

where,

W = Street Width, feet

 y_0 = Water depth in the gutter, feet

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 feet per foot (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:

$$Q_0 = 0.56(z/n)S_0^{-1/2}Y_0^{-8/3}$$
 (Eq. 3-2)

where,

Q_o = Gutter discharge, cfs

z = Reciprocal of the crown slope, ft/ft

 $S_o =$ Street or gutter slope, ft/ft

n = Roughness coefficient

 $Y_o =$ Depth of flow in gutter, feet

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

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, an n value equal to 0.016 is recommended. For gutters with small slope less than one (1) percent where sediment may accumulate, an n value of 0.02 is recommended.

2)

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:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0$$

(Eq. 3-3)

where,

Q = Gutter flow, cfs

 S_0 = Street grade, ft/ft

 y_0 = Water depth in the gutter, feet

 K_0 , K_1 , K_2 = Constant coefficients shown in Table 3-2 for different street widths:

Coeffici		ole 3-2 3-3, Streets Without	Curb Split
Street		Coefficients	
Width* (ft)	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

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

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

where,

Q = Gutter flow, cfs

 s_0 = Street grade, ft/ft

 Y_0 = Water depth in the gutter, feet

CS = Curb split, feet

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

-		C	oefficients		
Street Width (ft)	K ₀	K ₁	K ₂	K ₃	Curb Split Range (ft)
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

C. Streets with Curb Split - Lower Gutter.

The gutter flow equation for the lower gutter is:

 $\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3(CS)$

where,

- Q = Gutter flow, cfs
- S_0 = Street grade in ft/ft
- y_0 = Water depth in the gutter in feet

CS = Curb split in feet

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

		Co	pefficients		
Street	K	K	K		Curb Split
Width (ft)	K ₀	K ₁	K ₂	K ₃	Range (ft)
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

All the crown heights for different street widths are calculated by the following equation: Crown Height (feet) = $0.5 + [(W \ 30)/120]$ (Eq. 3-6)

(Eq. 3-5)

where,

W =street width, feet 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 (%) 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 (%) point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 feet per foot of street width.

Example: Determination of Crown Location

Given:	0.4 feet Design	split on 30-foot wide street.
Maxim	num curb split	= 0.02 x street width
		= 0.02 x 30 feet = 0.6 feet Maximum
Mover	ment	= 1/4 street width for 30 foot street

	= ¼ x 30 feet = 7.5 feet
Split Movement	=(Design split x W/Maximum Split x 4)
	= (0.4 x 30/.6 x 4) = 5 feet

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) inch above the top of curb.

SECTION 4-INLETS

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Drainage Criteria Manual SECTION 4 - INLETS

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) feet.
- C. All curb inlets (whether in a sump or on grade) incorporate a standard five (5) inch depression. Unless otherwise approved in writing by City Engineer, all curb inlets shall be a minimum of ten (10) feet 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: (1) inlets in sumps where flow contributes from two (2) or more sides (Type S); and (2) inlets on grade (Type G). The following list references the various inlet types. (See <u>Figures 4-1 through 4-7</u> 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 <u>Figure 4-8</u> in Appendix B of this Manual for an unsubmerged inlet or <u>Figure 4-9</u> in Appendix B of this Manual for submerged conditions, regardless of what depth of depression exists at the inlet.

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:

$$Q = 3.0h^{1.5}L$$

(Eq. 4-1)

where,

- Q = Capacity of curb opening inlet or of area inlet, cfs
- h = Head at the inlet, feet, = $a + Y_0$
- L = Length of opening through which water enters the inlet, feet

<u>Figure 4-8</u> in Appendix B of this Manual provides for direct solution of the above equation.

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:

$$Q = 4.82Ah^{0.5}$$

(Eq. 4-2)

where,

Q = Capacity, cfs

h = Depth of flow at inlet, feet

A = Area of grate opening, square feet

The curves shown in <u>Figure 4-9</u> 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 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:

Street Grade = five (5) 0 percent

Cross Slope = zero (0) ft/ft

Parabolic crown street width = thirty (30) feet

 Q_a in one gutter = twelve (12) cfs

- Find: Capacity of a ten (10) foot curb inlet on grade (Type G-1) with a five (5) inch gutter depression.
- Step 1. From Equation 3-3 (Section 3 of this Manual) depth of flow in gutter is $y_0=0.43$ feet, of 5.1 inches.
- Step 2. Enter Fig. 4-10 with $y_0 = 0.43$ feet and a = 5 inches 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 $L/L_a0.75$ and a/y = 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 Qpass

 Q_{pass} = 12-10.1 = 1.9 cfs

Step 8. The by-pass flow is 1.90 cubic feet per second.

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 run-off, 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 feet." Y_0 " can be determined from Equation 3-1 or <u>Figure 3-1</u> (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 <u>Figure 4-10</u> in Appendix B of this Manual by 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 percent (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 (in feet) to water depth in the gutter (in feet).
- **Column 16.** The ratio of Q/Q_a. The value is read from <u>Figure 4-11</u> 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.

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Table 4-1 Inlet Flow Calculation Table

Source: City of Austin Drainage Criteria Manual

SECTION 5 - STORM DRAINS

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Drainage Criteria Manual

SECTION 5 - STORM DRAINS

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 year storm with provisions made for the 100 year 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 forty-five (45) 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 Copperas Cove Standards for Public Works Construction.
- H. The 25 year hydraulic grade line shall remain six (6) inches 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 one half (2.5) feet per second (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							
Туре	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) inches.

5.3.4 Roughness Coefficients

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

Table5-2 Roughness Coefficients "n" For Storm Drains						
Materials of Construction	Minimum Design Coefficient					
Concrete	0.013					
Corrugated-metal Pipe	0.024					
Plain or Coated Paved Invert (Asphalt)	0.020					
Plastic Pipe Smooth Perforated	0.010 0.020					

5.4.0 FLOW IN STORM DRAINS

All storm drains shall be designed by the application of the Continuity Equation and Manning's Equation either through the appropriate charts and nomographs or by direct solution of the equations as follows:

5.4.1 Flow Equation Method

Q A

V

n

R

S

Wp

= Pipe Flow, cfs

= Velocity of flow, ft/sec

= Wetted perimeter, ft

= Cross-sectional area of flow, ft²

= Coefficient of roughness of pipe

= Hydraulic radius = A/Wp, ft

= Friction slope in pipe, ft/ft

Q	= AV and	(Eq.5-1)
Q	=(1.49/n)AR ^{2/3} S ^{1/2}	(Eq. 5-
where,	,	2)

5-4	
-----	--

5.4.2 Nomograph Method

Nomographs for determining flow properties in circular pipe, elliptical pipe and pipearches are given here as <u>Figures 5-1 through 5-9</u> in Appendix B of this Manual. The nomographs are based upon a value of "n" of 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 "n" other than 0.012, the value of Q should be modified by using the following formula:

 $Q_c = 0.012 Q_n / n_c$

where, Q_c = Flow based upon n_c

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 feet D = diameter = 36 inches n = 0.018

Find: Discharge (Q).

First determine d/D = 1.8'/3.0' = 0.6. then enter <u>Figure 5-1</u> (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/ftD = diameter = 36 inches Q = 22.7 cfs n = 0.018

Find: Velocity of flow (ft/sec).

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 <u>Figure 5-1</u> (in Appendix B of this Manual) to determine d/D = 0.6. Now enter <u>Figure 5-3</u> (in Appendix B of this Manual) to determine V = 7.5 ft/sec.

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:

$$h_{f} = (29n^{2}/R^{1.33})(V^{2}/2g)L$$

where,

- h_f = Friction loss, ft
- n = Manning's Coefficient
- L = Length of pipe, ft
- R = Hydraulic radius, ft
- V = Velocity of flow, ft/sec
- g = Acceleration due to gravity, 32 ft/sec^2

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_fL

where,

- h_f = Friction loss, feet
- S_f = Friction slope, feet = $(Q/C)^2$
- L = Length of pipe, feet
- C = Full flow coefficient from Table 5-3
- Q = Discharge, cfs

Example 5-3:

Given: Discharge Q = 24 cfs, diameter D = 24 inches, the length of pipe L=300 feet and n = 0.013

Find: The friction loss H_f

(Eq.5-4)

(Eq. 5-3)

First, from Table 5-3 for D = 24 inches and n = 0.013, the full flow coefficient C = 226. Second, the friction slope $S_f = (Q/C)^2 = 0.011$

The friction loss $H_f = S_f L = 3.3$ feet

F	Table 5-3Full Flow Coefficient Values for Circular Concrete Pipe									
D Pipe	A Area	R Hydraulic	; Value of C* for							
Diameter (inches)	(square feet)	Radius (feet)	n = 0.010		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	124	114	105				
21	2.405	0.437	206	187	172	158				
24	3.142	0.500	294	267	245	226				
27	3.976	0.562	402	366	335	310				
30	4.909	0.625	533	485	444	410				
33	5.940	0.688	686	624	574	530				
36	7.069	0.750	867	788	722	666				
42	9.621	0.875	1308	1189	1090	1006				
54	15.904	1.125	2557	2325	2131	1967				
60	19.635	1.250	3385	3077	2821	2604				

D Pipe Diameter	A Area	R Hydraulic	Value of C* for							
(inches)	(square feet)	Radius (feet)	n = 0.010	n = 0.011	n = 0.012	n = 0.013				
66	23.758	1.375	4364	3967	3636	3357				
72	28.274	1.500	5504	5004	4587	4234				
78	33.183	1.625	6815	6195	5679	5242				
84	38.485	1.750	8304	7549	6920	6388				
90	44.170	1.875	9985	9078	8321	7681				
96	50.266	2.000	11850	10780	9878	9119				
102	56.745	2.125	13940	12670	11620	10720				
108	63.617	2.250	16230	14760	13530	12490				
114	70.882	2.375	18750	17040	15620	14420				
120	78.540	2.500	21500	19540	17920	16540				
126	86.590	2.625	24480	22260	20400	18830				
132	95.033	2.750	27720	25200	23100	21330				
138	103.870	2.875	31210	28370	26010	24010				
144	113.100	3.000	34960	31780	29130	26890				
* C = (1.486	6/n)AR ^{0.667}									

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:

$$h = KV^2/2g$$
 (Eq. 5-5)

where,

h = velocity head loss, feet K = coefficient for head loss

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 Ki given in Table 5-4.

$$h^{j} = (V_{2}^{2} - K_{j}V_{1}^{2})/2g$$
 (Eq. 5-6)

where, V_1 = Velocity for inflowing pipe, ft/sec.

- V_2 = Velocity for outflowing pipe, ft/sec.
- K_i = Junction or structure coefficient of loss.

The detailed design information for junction losses can be found in the Bibliography 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:

 $h_b = K_b v^2/2g$

(Eq. 5-7)

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

Cases	Reference Figure	Description of Condition	Coefficient K _j		
A	5-10	Manhole on Main Line with 45° Branch Lateral	0.50		
В	5-10	Manhole on Main Line with 90° Branch Lateral	0.25		
С	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 22 ¹ / ₂ °bend Manhole on line with 60° Lateral Manhole on line with 22 ¹ / ₂ ° Lateral	0.50 0.43 0.35 0.20 0.35 0.75		
	ther than 90 degrees lowing percentage f	s are used, the 90 degree bend coe actor applied:	efficient can be		
60° Bend - 85%;	45° Bend - 70%; 22	2 ¹ / ₂ ° Bend - 40%			

- 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.

 $h_{gc} = 0.04 V^2/2g$

(Eq. 5-8)

where, V = velocity for smaller pipe.

Table 5-5 Values of K for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula H = K (V²/2g)

d_2/d_1					Velo	city, V,	fps				
	2	3	4	5	6	7	8	10	12	15	20
1.2	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84
	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86

 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 5-6Values of K for Determining Loss of Head Due to GradualEnlargement in Pipes from the Formula H = K (v²/2g)

d₂/d ₁ 1.1	2° .01	4° .01	6° .01	8° .02	10° .03	15° .05	20° .10	25° .13	30° .16	35° .18	40° .19	45° .20	50° .21	60° .23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
	.03	.04	.04	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

Angle of cone*

* 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. <u>Handbook of Hydraulics</u>, 1976.

					rmining			Due to S H = K (V			
	Velocity, V in feet per second										
d_2/d_1	2	3	4	5	6	7	8	10	12	15	20
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42
10.0	.49 .49	.48 .49	.48 .48	.48 .48	.48 .48	.47 .47	.47 .47	.46 .47	.46 .46	.45 .45	.43 .44
$d_2/d_1 =$	ratio of		of larg				smaller p	•		1	

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 feet.
- 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 $(Q/C)^2=S_f$).
- Column 7. Elevation of hydraulic gradient (hg) 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 hg 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 <u>Figures 5-1 and 5-3</u> 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 <u>Figures 5-10</u> and <u>5-11</u> 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.

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Table 5-8 Hydraulic Computations Storm Sewers

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) feet from an access point for drains thirty (30) inches in diameter or smaller. For storm drains greater than thirty (30) inches in diameter, manholes shall be placed so that there is a maximum distance of three hundred (300) feet 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 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) feet of each other (See <u>Figure 5-12</u> 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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Drainage Criteria Manual

SECTION 6 - OPEN CHANNELS

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: (1) Natural channels, which include all watercourses that have been carved by nature through erosion; and (2) 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 year storm with provisions for the 100 year 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 concentration times 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 design 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 design 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.

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

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

(Eq. 6-1)

where,

- Q = Total discharge, cfs
- n = Roughness coefficient
- A = Cross-sectional area of channel, ft^2
- R = Hydraulic radius of channel, feet (R=A/P)
- S = Slope of the frictional gradient, ft/ft
- P = Wetted perimeter, feet

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 <u>Figure 6-1</u> 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-2, 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:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m$$

where n_0 is a basic n value for a straight, uniform, smooth channel in the natural materials involved, n_1 is a value added to n_0 to correct for the effect of surface irregularities; n_2 is a value for variations in shape and size of the channel cross section; n_3 is a value for obstructions; n_4 is a value for vegetation and flow conditions; and m is a correction factor for meandering of the channel. Proper values of n_0 to n_4 and m may be selected from Table 6-1 according to the given conditions.

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 n₂, 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 n₃ 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 reevaluated 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.

(Eq. 6-2)

Table 6-1Computation of Composite Roughness CoefficientFor Excavated and Natural Channelsn = $(n_0 + n_1 + n_2 + n_3 + n_4)m$						
Channel Co	onditions	Values				
n ₀ Material Involved	Earth React Fine Gravel Coarse Gravel	0.020 0.025 0.024 0.028				
n ₁ Degree of Irregularity	Smooth Minor Moderate Severe	0.000 0.005 0.010 0.020				
n ₂ Relative Effect of Channel Cross Section	Gradual Alternating Occasionally Alternating Frequently	0.000 0.005 0.013				
n ₃ Relative Effect of Obstructions	Negligible Minor Appreciable Severe	0.000 0.013 0.025 0.050				
n ₄ Vegetation	Low Medium High Very High	0.008 0.018 0.038 0.075				
m Degree of Meandering	Minor Appreciable Severe	1.000 1.150 1.300				
Source: Chow, V.T. Open Cha	nnel Hydraulics, 1959.					

In selecting the value of n₄, the degree of effect of vegetation is considered in the following way:

- A. Low for conditions comparable to the following: (a) 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 (b) 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. <u>Medium</u> for conditions comparable to the following: (a) turf grasses where the average depth of flow is one (1) to two (2) times the height of vegetation; and (b) 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) feet.
- C. <u>High</u> for conditions comparable to the following: (a) turf grasses where the average depth of flow is about equal to the height of vegetation, (b) 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) feet; and (c) 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) feet.
- D. <u>Very high</u> for conditions comparable to the following: (a) turf grasses where the average depth of flow is less than half (1/2) the height of vegetation, (b) growing season -- bushy willows about 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) feet and (c) growing season-trees intergrown with weeds and brush, all in full foliage, with any value of hydraulic radius up to ten (15) feet.

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 (1.0) 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 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) feet. The method applies only to unlined natural streams, floodways, and drainage channels and shows a minimum value of 0.02 for the n value of such channels. The minimum value of n in general, however, may be as low as 0.012 in lined channels and as low as 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 year storm is six (6) feet per second and includes all transitions to or from channels and waterways with similar or different materials. In all cases, the velocity for the 100 year storm must be non-erosive. The minimum permissible velocity for the 2 year storm is two (2) feet per second.
- B. **Roughness Coefficient.** The roughness coefficients selected shall be based on the degree of retardance of vegetation. Table 6-2 provides minimum Manning's Coefficients 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 year storm flow is greater than two (2) feet per second, 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) feet wide, two (2) inches 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 one-half (0.5) percent.
- D. **Side Slopes.** Side slopes shall be four (4) to one (1) or flatter for channels equal to or over four (4) feet deep and three (3) to one (1) or flatter for channels less than four (4) feet deep.
- E. **Curvature.** The center line curvature shall have a minimum radius of twice the top width of the 100 year storm flow.
- F. Bottom Width. The minimum flat bottom width of the channel is three (3) feet.

G. Freeboard. All grass-lined channels shall be designed to convey the one hundred (100) year storm event. The freeboard for the channel shall be the velocity head for the one hundred (100) year 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 and 100 year 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 Coefficients for concrete-lined channels.
- C. **Freeboard.** Adequate channel freeboard shall be provided for the 100 year storm in reaches flowing at critical depth by Equation 6-3 or using the energy grade line, whichever is less.

$$H_{FB} = 2.0 + 0.025 V (d)^{1/3}$$

where,

 $\begin{array}{ll} H_{FB} & = Freeboard \ height, \ ft \\ V & = Velocity, \ ft/sec \\ d & = Depth \ of \ flow, \ ft \end{array}$

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:

$$h = V_2 T_w/gr_c$$

where

h = Superelevation, ft

V = Flow velocity, ft/sec

 T_w = Top width of channel, ft

r_c = Centerline radius of curvature, ft

 $g = Acceleration due to gravity, ft/sec^2$

(Eq. 6-4)

(Eq. 6-3)

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 two (2) year storm flow of at least two (2) feet per second. 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 year storm (except for channel drop structures and energy dissipators as approved by the City) and velocities for the 100 year 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 commonly used drop structures are shown in <u>Figure 6-2</u> 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) foot 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 shall be no steeper than 2:1. The length, L, of the chute depends upon the hydraulic characteristics of the channel and drop. For a unit discharge, q, of thirty (30) cubic feet per second per foot, L would be about fifteen (15) feet, that is, about one-half (1/2) of the q value. The L should not be less than ten (10) feet, 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 Coefficients				
 Grass lined a. Bermuda (with regular mowing) b. St. Augustine (with regular mowing) c. Native grasses and vegetation not 	.040 .045 .060				
 2. Concrete a. Concrete lined (rough finish) b. Concrete lined (smooth finish-culverts) c. Concrete rip-rap (exposed rubble) 	.020 .015 .025				
3. Gabion	.035				
4. Rock-cut	.025				
Source: 1. Chow, V.T. <u>Open Channel Hydraulics</u> . 1959. 2. WRC Engineering, Inc. <u>Boulder County Storm Drainage Criteria Manual</u> . 1984.					

Table 6-3 Length of Downstream Apron						
Maximum Unit Discharge, q (cfs/ft) Length of Downstream Apron, I						
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 4:1. The riprap should extend up the side slopes to a depth equal to one (1) foot above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is four (4) feet 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 <u>Figure 6-2</u> in Appendix B of this Manual.

- A. <u>Approach Channel:</u> The upstream and downstream channels will normally be grasslined trapezoidal channels.
- B. <u>Approach Apron:</u> A minimum ten (10) foot 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. <u>Chute Apron:</u> The riprap stilling basin is designed to force the hydraulic jump to occur within the basin and is designed for essentially zero 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 <u>Hydraulic Design of Stilling Basins and Energy Dissipators, 1984</u>. 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 2:1 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 per foot for the 100 year 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 <u>Figure 6-3</u> in Appendix B of this Manual is the critical velocity in a rectangular channel, $V_c=(gq)^{1/3}$.
- C. **Chute Design.** The chute is usually constructed on a 2:1 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) foot (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 (1/3 to 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:

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

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 10 year storm shall be calculated with Manning's "n" values in accordance with Tables 6-2 and 6-3. This channel is designed to separate the more frequent 10 year storm via an unobstructed pilot channel. Side slopes of the pilot channel shall not exceed 3:1 slope gradient and shall have a bottom width of no less than six (6) feet. The remaining cross-sectional area is designed to convey the additional storm flows up to the 100 year storm. This upper platform will accommodate vegetation with minimal maintenance requirements.
- B. The ultimate 100 year floodplain shall be contained within overbanks on each side of the pilot channel. These overbanks shall be a minimum width often (10) feet and have a slope gradient not to exceed 6:1. 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 "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.

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

SECTION 7 - CULVERTS

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Drainage Criteria Manual SECTION 7 - CULVERTS

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 year frequency 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) and twenty-four (24) inches diameter.

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:

$$h_e = K_e (V_2^2 - V_1^2)/2g$$

(Eq.7-1)

where,

h_e = Entrance head loss, feet

- V_2 = Velocity of flow in culvert, ft/sec
- V_1 = Velocity of flow approaching culvert, ft/sec
- K_e = Entrance loss coefficient as shown in Table 7-1
- g = Acceleration due to gravity

7.2.3 Type Of Headwall

The common types of headwall entrances are shown in <u>Figure 7-1</u> 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 7-1Values of Culvert Entrance Loss Coefficients					
Type of Entrance	Entrance Coefficient, K _e				
Pipe					
Headwall (no wingwalls) Grooved edge Rounded edge (0.15D radius) Rounded edge (0.25D radius) Square edge (cut concrete and CMP)	0.20 0.15 0.10 0.40				
Headwall with 45° Wingwalls Grooved edge Square edge	0.20 0.35				
Headwall with Parallel Wingwalls Spaced 1.25D apart Grooved edge Square edge	0.30 0.40 0.25				
Projecting Entrance (no headwall or wingwalls) Grooved edge (RCP) Square edge (RCP) Sharp edge, thin walls (CMP)	0.25 0.50 0.90				
Sloping Entrance (no headwall or wingwalls) Mitered to conform to slope Flared-end section	0.70 0.50				

Table 7-1 (Continued)Values of Culvert Entrance Loss Coefficients							
Type of EntranceEntrance Coefficient, Ke							
Box, Reinforced Concrete							
Headwall Parallel to Embankment (no wingwalls) Square edge on sides of opening Rounded on 3 edges to radius of 1/12 barrel dimension	0.50 0.20						
Wingwalls at 30° to 75° to barrel axis Square edged at crown Crown edge rounded to radius of 1/12 barrel dimension	0.40 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 u capacity operating under outlet control.	sed to evaluate the culvert or sewer						
Source: WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual, 1984							

A. Parallel Headwall.

- 1. Approach velocities are low (below six (6) feet per second).
- 2. Backwater pools are permitted.

B. Flared Headwall.

- 1. Approach velocities are between six (6) and ten (10) feet per second.
- 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 <u>Figure 7-1</u> in Appendix B of this Manual.

C. Warped Headwall.

Approach velocities are between eight (8) and twenty (20) feet per second. 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-<u>2</u> 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 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	Outlet Protection
Below six (6) ft/sec	Riprap protection. (Four (4) inch minimum thickness) or alternate approved material.
Above six (6) ft/sec	Structurally reinforced apron, six (6) inch 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

(Eq. 7-2)

where,

- L = Apron length, feet
- V = culvert discharge velocity, ft/sec
- D = height of box culvert or diameter of pipe culvert, feet

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. <u>Unsubmerged:</u> 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).
- 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, <u>Figure 7-3</u> in Appendix B of this Manual).

The discharge capacity for several culvert materials, shapes, and inlet configurations under inlet control conditions are presented in <u>Figures 7-5 to 7-1D</u> 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:

- The headwater submerges the culvert opening, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, <u>Figure 7-3</u> in Appendix B of this Manual).
- The headwater submerges the culvert opening, the culvert outlet is not submerged by the tailwater (Condition B or C, <u>Figure 7-3</u> 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, <u>Figure 7-3</u> 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 <u>Figure 7-4</u> in Appendix B of this Manual).

Bernoulli's Equation is:

 $d_1 + V_1^2/2g + LS_0 = TW + h_e + h_f + h_v$

(Eq. 7-3)

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:

$$HW = d_1 + V_1^2 / 2g$$
 (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:

 $HW + LS_{0}-TW = h_{e} + h_{f} + h_{v}$ (Eq. 7-5)

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

HW + LS₀= H_L + TW

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

$$HL = HW + LS_0 \quad TW \tag{Eq.7-6}$$

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

 $H_{L} = h_{e} + h_{f} + h_{v}$ (Eq. 7-7)

in which $h_v = V^2/2g.$ (Eq. 7-8)

For inlet losses, the governing equation is Equation 7-1:

 $h_e = K_e (V_2^2 - V_1^2)/2g$

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,

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

where K_e is the entrance loss coefficient, as shown in Table 7-1 and V is the velocity of flow in the culvert.

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

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

where

n = Manning's coefficient
L = Length of culvert, feet
R = Hydraulic radius, feet
V = Velocity of flow in the culvert, ft/sec

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

$$H = (K_e + 1 + 29n^2 L/R^{1.33})V^2/2g$$

(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 <u>Figure 7-3</u> 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:

$$HW = H + h_0 LS_0$$
 (Eq.7-12)

where,

HW = vertical distance from flow line at the entrance to the pool surface, feet

H = head loss, feet (use appropriate nomograph)

 h_0 = vertical distance from flow line at the outlet to the hydraulic grade line, feet (In this case h_0 equals TW, measured in feet above the flow line.)

 S_0 = slope of barrel, ft/ft

L = culvert length, feet

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

$$h_0 = (d_c + D)/2 \text{ or TW},$$

(Eq. 7-13)

where,

 d_c = critical depth in feet (d_c cannot exceed D)

D = height of culvert opening in feet whichever value is greater.

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 (<u>Figures 7-5 to 7-10</u> in Appendix B of this Manual) and outlet control conditions (<u>Figures 7-11 to 7-17</u> in Appendix B of this Manual). The critical depth for pipes of different shapes are shown in <u>Figures 7-16 to 7-22</u> 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, feet
 - 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, feet
 - 5. Allowable flow velocities in natural stream
 - 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.
- Use inlet control nomographs for the culvert type selected (<u>Figures 7-5 to 7-10</u> 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 f 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), in feet, 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 ho 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.
- If outlet control governs in Step 3-3 above, the outlet velocity equals Q/A₀, where A₀ 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 A₀ corresponding to d_c or TW depth, whichever gives the greater area of flow. A₀ 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. <u>Step 6.</u> 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 (<u>Figures 7-11 to 7-17</u> 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 n1 than that of the chart n, use the length scales shown with an adjusted length L_1 , calculated by the following equation:

 $L_1 = L(n_1/n)^2$

(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 feet 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 feet on the head (H) scale.

7.4.4 Example 7-1

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

Given:	Culvert size length L n value Inlet elevation outlet elevation slope S ₀	= 48 inches RCP =110 feet =0.012 = 720.0 feet = 718.8 feet =0.010
Entrance condition (squared)	= 0.50	
Maximum elevation for er	=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 <u>Figure 7-5</u> 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, <u>Figure 7-12</u> 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 ho-

Step 6. The critical depth (cd) is found from <u>Figures 7-18 to 7-22</u> 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 twelve (12) inches for the 100 year frequency storm, and for any street other than residential, the allowable maximum is six (6) inches for the 100 year 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 <u>Figure 7-23</u> in Appendix B of this Manual. The three (3) flow types are described below:

A. Type I Flow.

Referring to Item A of <u>Figure 7-23</u> 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 <u>Figure 7-23</u> 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 <u>Figure 7-23</u> 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 <u>Figure 7-23</u> 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 <u>Figure 7-23</u> 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 <u>Hydraulics of Bridge Waterways</u> 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.

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Table 7-2 Calculation Table for Culvert Design

CALCULATION TABLE FOR CULVERT DESIGN PROJECT: TOTAL LOSSES EL Hehe + hf + hv 2 H dia in HW Ŧ EL ho(Tw) ()) Hw = H + ho - LSo EL. (2) For Tw < D the =do+D or Tw (Whichever is greater) TYPE= 48' RCP So=0.010 L= 110' Sol.=1.10 For Tw>D ; ho=Tw n= 0.012 0 FULL=160 CFS V FULL=12.4 fps (3) For Box Culvert 1 do =0.315 $(g)^3 \leq_D$ INLET=SQUARE EDGE Ke= 0.5 OUTLET CONTROL EQUATIONS. INLET CONTROL CON-CAL-CONTROL CULATED TROLLED Tw< D Tw>D ELEV. HW HW H. 0 Tw ho=do+D HW HW do ELEV. no D 1 2 3 4 5 6 7 8 9 10 1 12 13 720 0 ٥ 0 720.0 724 4 T. 70 0.94 2.55 3.28 3.12 4 INLET 724.0 5 1.25 95 725 1.72 2.90 3.45 4.07 5 INLET 725.0 726 6 1.5 115 5.10 2.60 3.20 726.0 3.60 6 INLET 728 B 4.50 2.0 150 7.20 3.60 3.80 8 INLET 728,0 730 10 2.5 170 5.80 3.85 3.93 8.63 INLET 730.0 10

Example Table 7-2 Calculation Table for Culvert Design

SECTION 8- STORMWATER MANAGEMENT

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Drainage Criteria Manual section 8 - stormwater management

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 <u>Figure 8-1</u> 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 on-site and regional SWM. Essentially, the distinction between the two approaches is that on-site 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 on-site 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 on-site 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: On-site and regional.

In general, on-site 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 on-Site SWM Ponds

A. On-site SWM ponds are further classified as either small or large, as follows:

ON-SITE SWM POND CLASS	DRAINAGE AREA
Small	<25 acres
Large	25-64 acres

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

B. On-site SWM ponds shall be designed to reduce post-development peak rates of discharge to existing pre-development peak rates of discharge for the 2, 10, and 25 year 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	IMPOUNDED	
POND CLASS	<u>VOLUME, AC-FT</u>	
Small	0-150	
Large	>150	

Any regional pond with a height of dam over fifteen (15) feet 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 on-site and off-site areas, shall be assumed to be fully developed. Any orifice with a dimension smaller than or equal to twelve (12) inches 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.
On-site:	Small	25 year	0
	Large	25 year	1.0
Regional:	Small	100 year	2.0
	Large	100 year	*

*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 an engineer licensed in the State of Texas.

- C. All SWM ponds shall be designed using a hydrograph routing methodology.
- D. The minimum embankment top width of earthen embankments shall be as follows:

TOTAL HEIGHT	MINIMUM TOP	
OF EMBANKMENT, FT,	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.

- F. Earthen embankment side slopes shall be no steeper than four (4) horizontal to one (1) 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. Stagedischarge 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) feet, 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 year storm) in parking lots are limited to an average of eight (8) inches with a maximum of twelve (12 inches). Maximum depths shall be permitted only in overflow parking areas not typically in daily business.

- K. All pipes discharging into a public storm sewer system shall have a minimum diameter of eighteen (18) inches 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 <u>Figure 8-2</u> in Appendix B of this Manual)

$$Q = CLH^{3/2}$$

where

Q = Weir discharge, cubic feet per second

C = Weir Coefficient

L = horizontal length, feet

H = Head on weir, feet

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

$$Q = C_v tan (0/2) H^{2.5}$$

where

Q = Weir Flow, cubic feet per second

C_v = Weir Coefficient

- O = Angle of the weir notch at the apex (degrees)
- H = Head on Weir, feet
- C. Orifice flow equation (See Figure 8-2 in Appendix B of this Manual)

$$Q = C_o A (2gH)^{0.5}$$

where

Q = Orifice Flow, cubic feet per second

 C_o = Orifice Coefficient (use 0.6)

- A = Orifice Area, square feet
- g = Gravitation constant, 32.2 feet/sec²
- H = Head on orifice measured from centerline, feet

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.

(Eq. 8-3)

(Eq. 8-1)

(Eq. 8-2)

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 hydrologic methods (available in TR-20, 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. FIGURE AND DIAGRAMS

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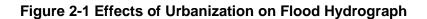
APPENDIX B. FIGURE AND DIAGRAMS

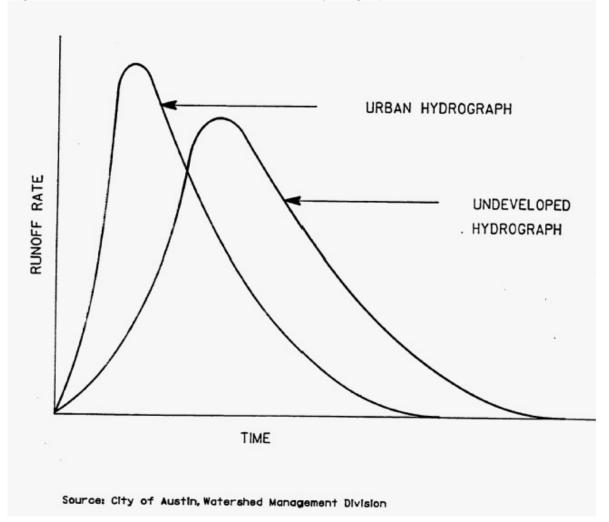
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FIGURES FROM SECTION 2





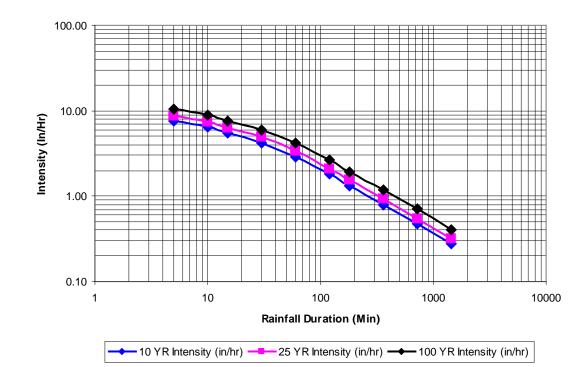


Figure 2-2 Intensity Duration Frequency Curve for Copperas Cove, TX

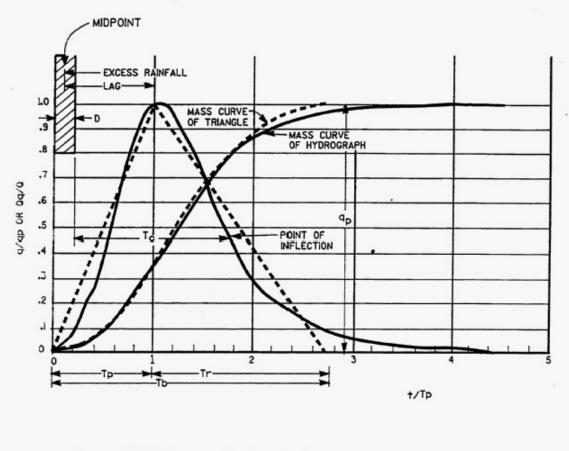


Figure 2-3 Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph

Source: U.S. Soll Conservation Service, "Hydrology Section 4," National Engineering Handbook, (NEH-4), 1972.

FIGURES FROM SECTION 3

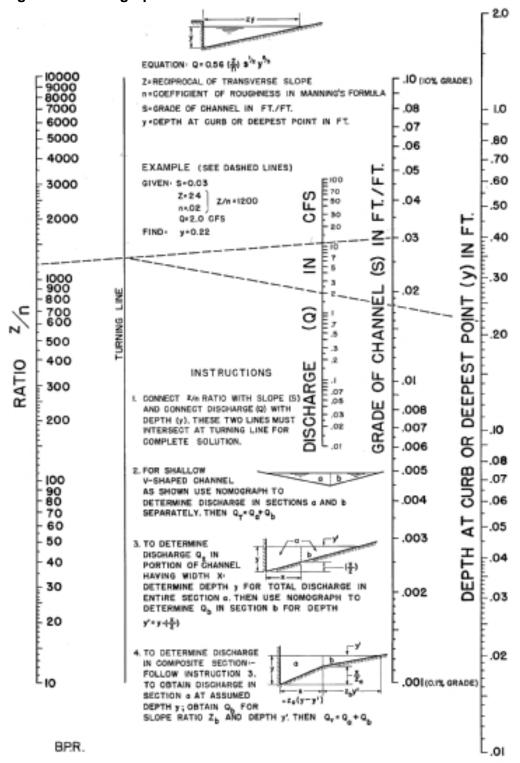


Figure 3-1 Nomograph for Flow in Gutters

Source: Texas Highway Department, Bridge Division

FIGURES FROM SECTION 4

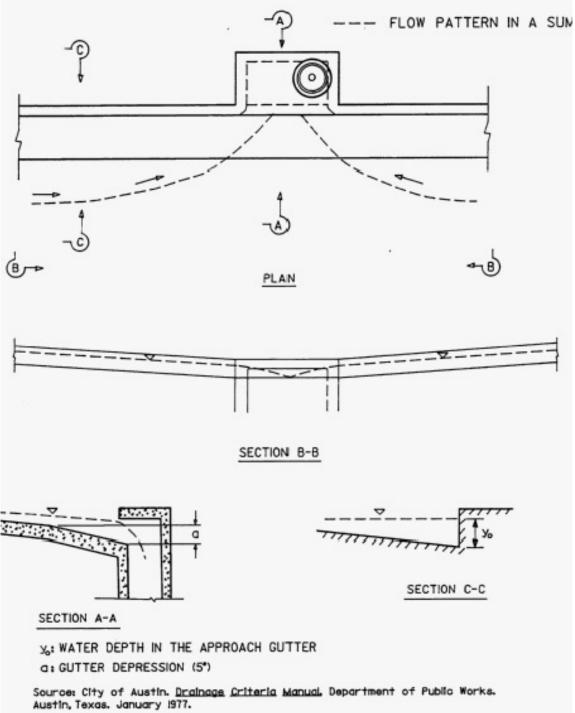
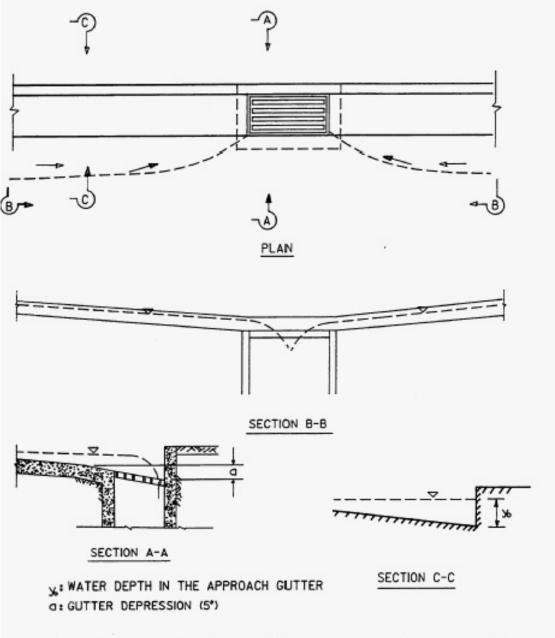




Figure 4.2 Grate Inlet in a Sump (Type S-2)





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

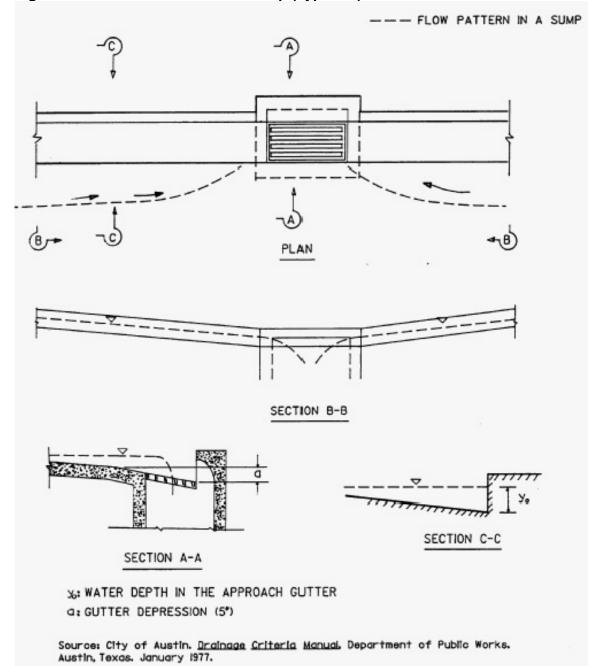
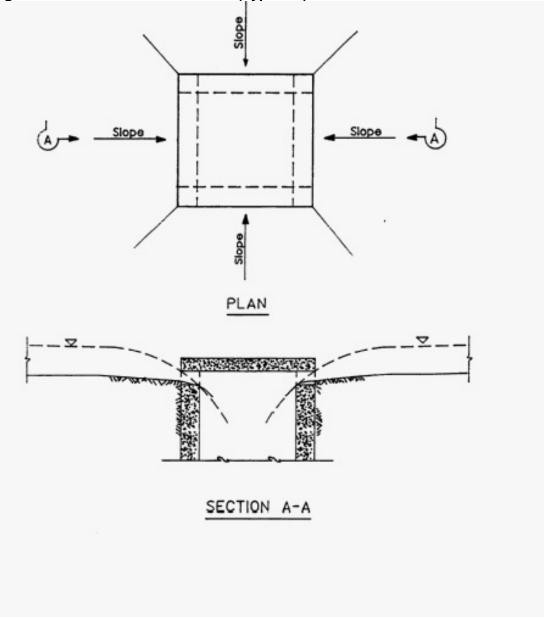


Figure 4-3 Combination Inlet in a Sump (Type S-3)

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



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

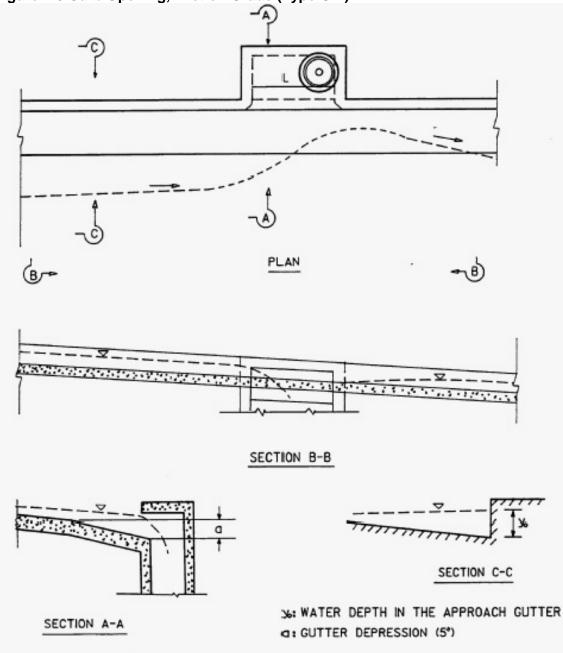
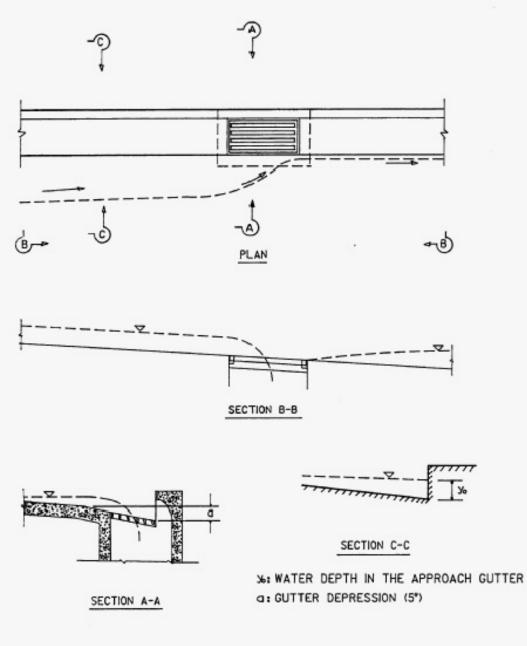


Figure 4-5 Curb Opening, Inlet on Grade (Type G-1)

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

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



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

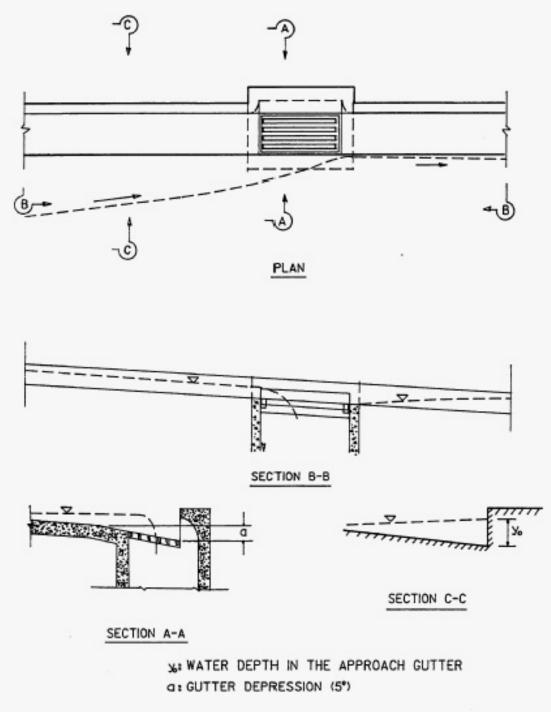


Figure 4-7 Combination Inlet on Grade (Type G-3)

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

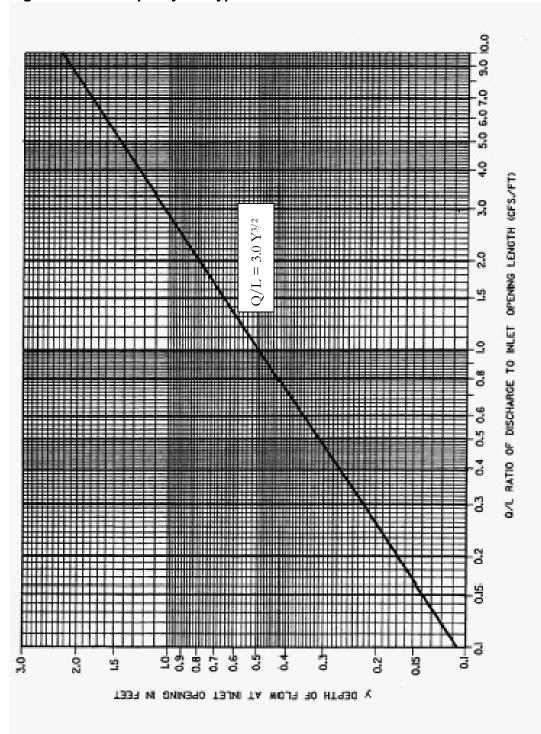
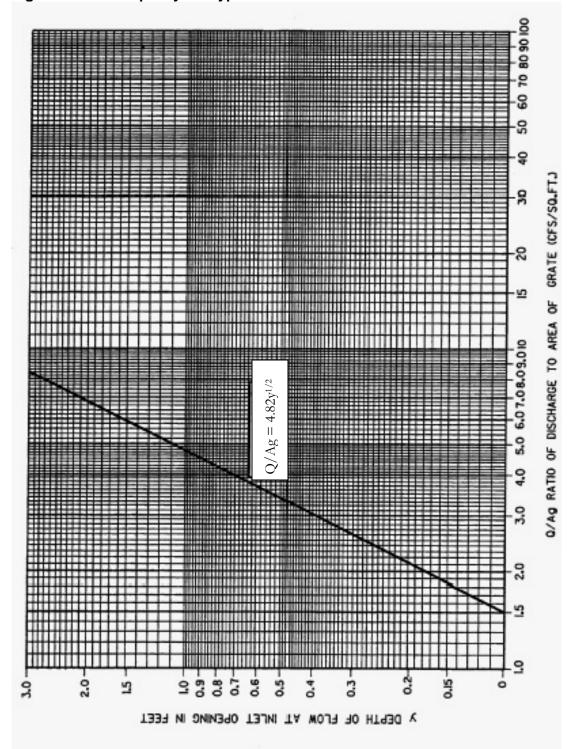


Figure 4-8 Inlet Capacity for Type S-1 and S-3

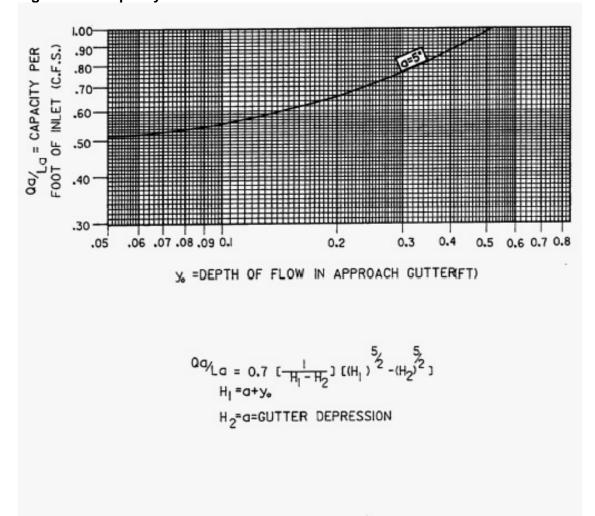
Source: City of Austin. <u>Brainage Criteria Manual</u> Department of Public Works. Austin, Texas. January 1977.

Figure 4-9 Inlet Capacity for Type S-2



Source: City of Austin. <u>Drainage Criteria Manual</u> Department of Public Works. Austin. Texas. January 1977.

Figure 4-10 Capacity for Inlets on Grade



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

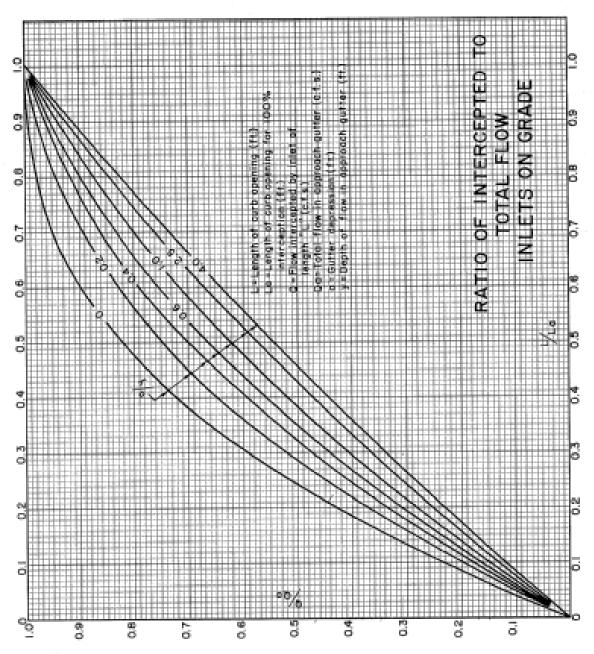


Figure 4-11 Ratio of Intercepted to Total Flow for Inlets on Grade

Source: Texas Highway Department, Bridge Division

FIGURES FROM SECTION 5

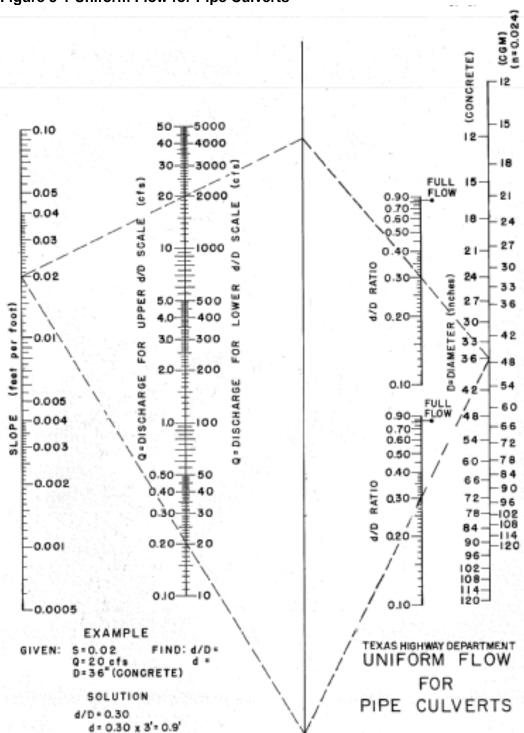


Figure 5-1 Uniform Flow for Pipe Culverts

Source: Texas Highway Department, Bridge Division

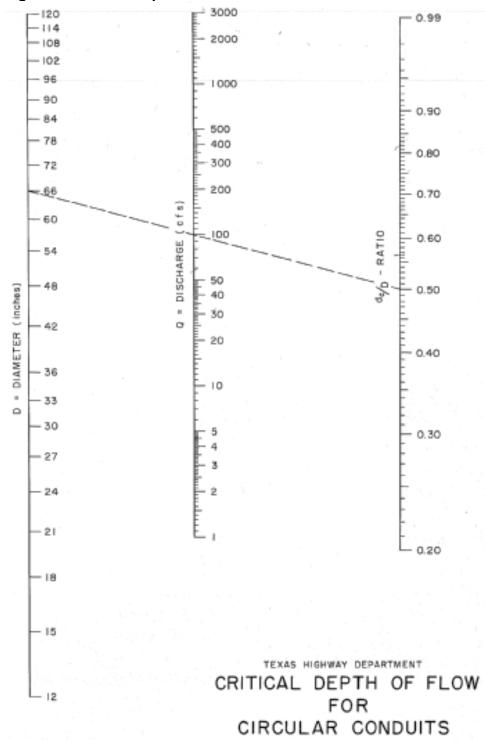
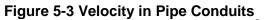
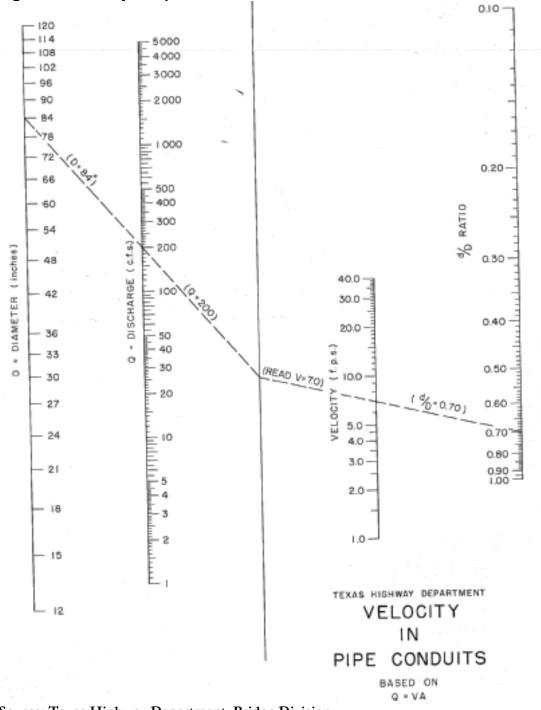


Figure 5-2 Critical Depth of Flow for Circular Conduits

Source: Texas Highway Department, Bridge Division





Source: Texas Highway Department, Bridge Division

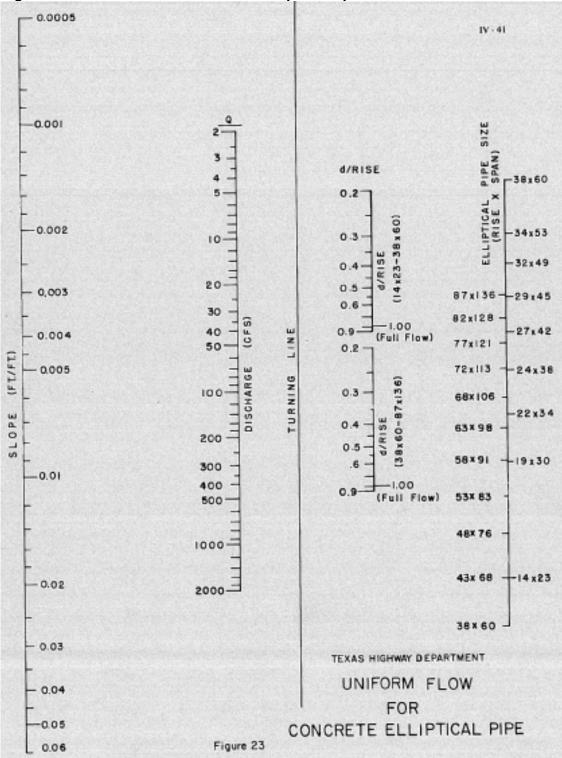


Figure 5-4 Uniform Flow for Concrete Elliptical Pipe

Source: Texas Highway Department, Bridge Division

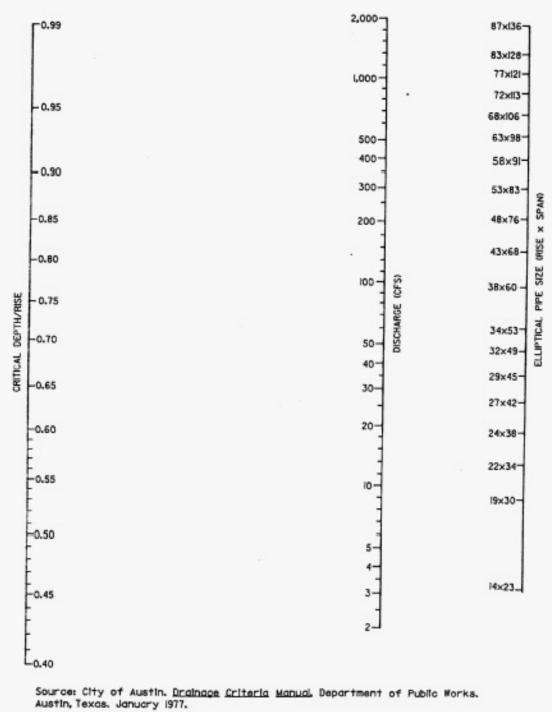


Figure 5-5 Critical Depth for Elliptical Pipe

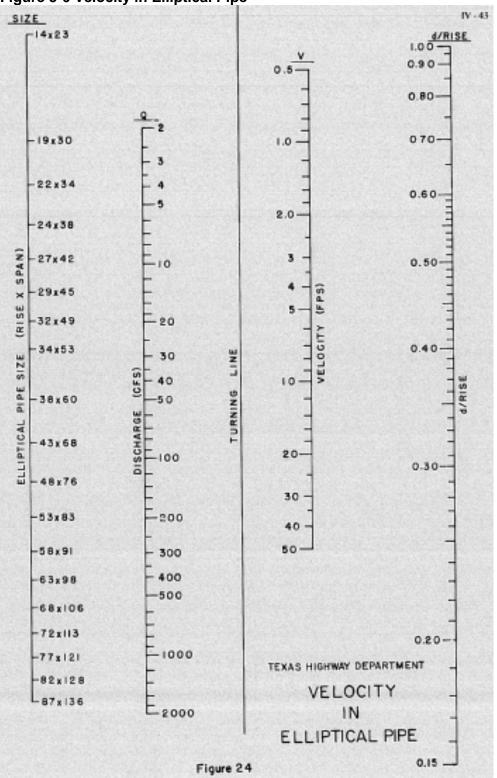


Figure 5-6 Velocity in Elliptical Pipe

Source: Texas Highway Department, Bridge Division

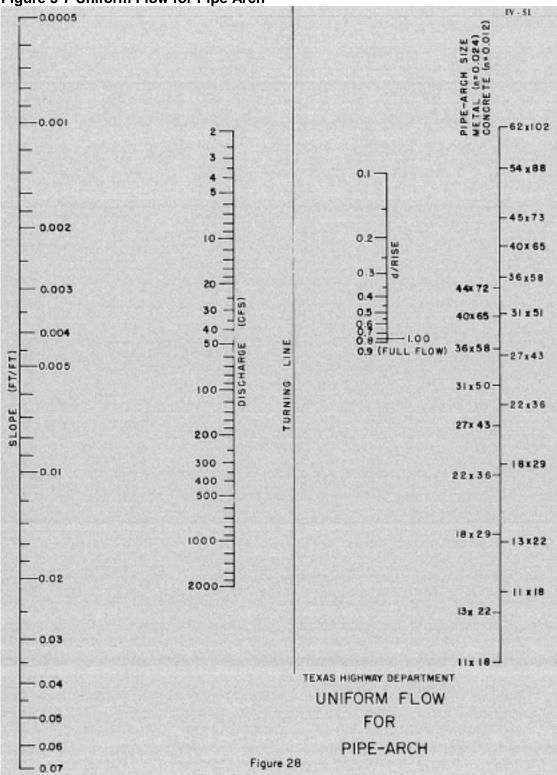
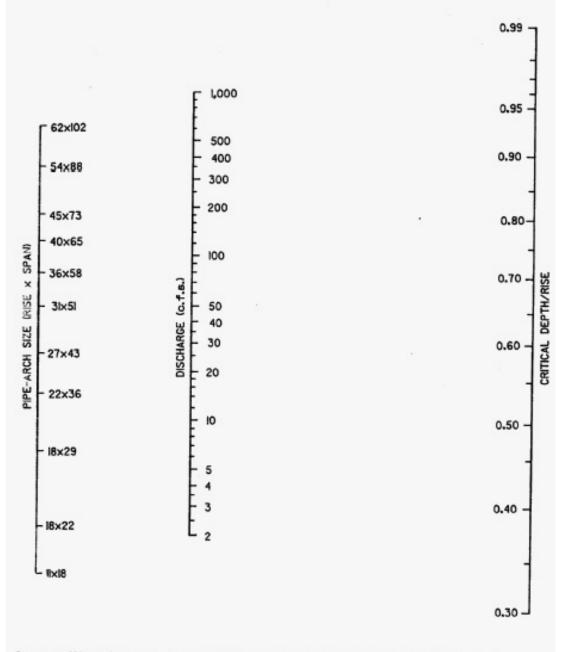


Figure 5-7 Uniform Flow for Pipe Arch

Source: Texas Highway Department, Bridge Division





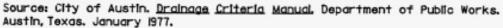
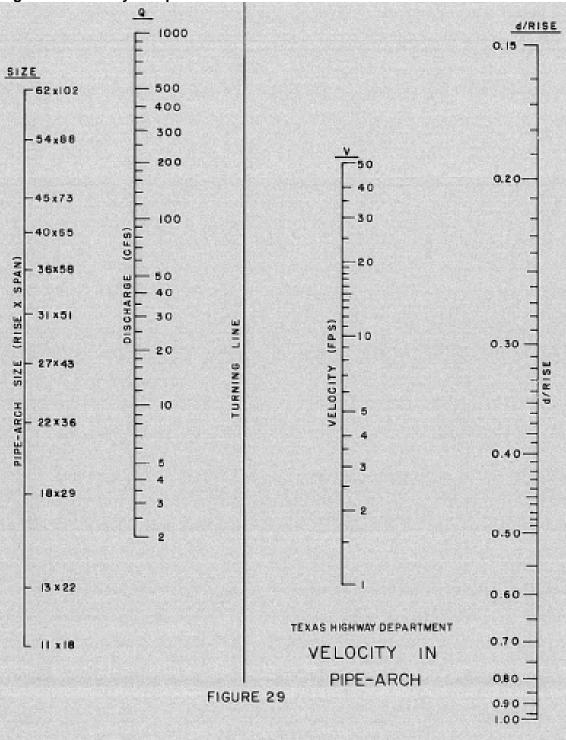


Figure 5-9 Velocity in Pipe-Arch



Source: Texas Highway Department, Bridge Division

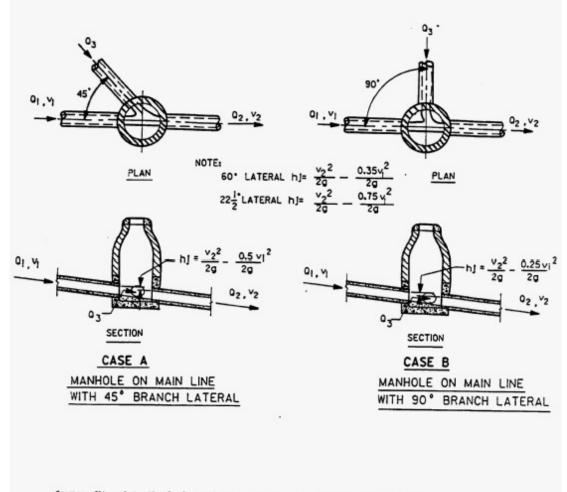


Figure 5-10 Minor Head Losses Due to Turbulence at Structures

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

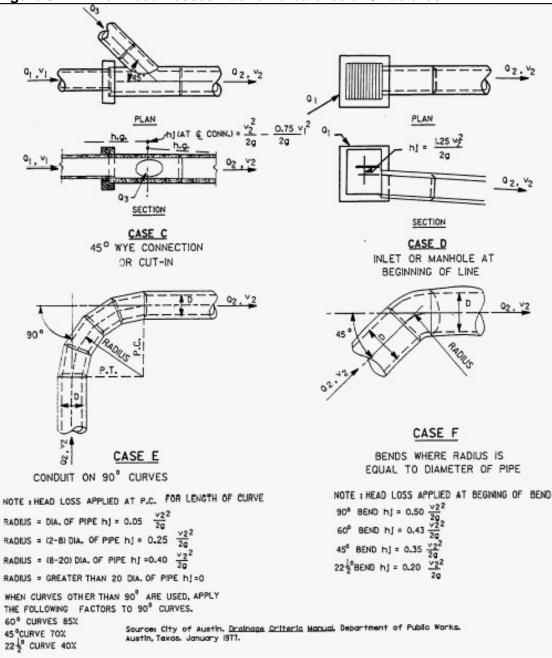
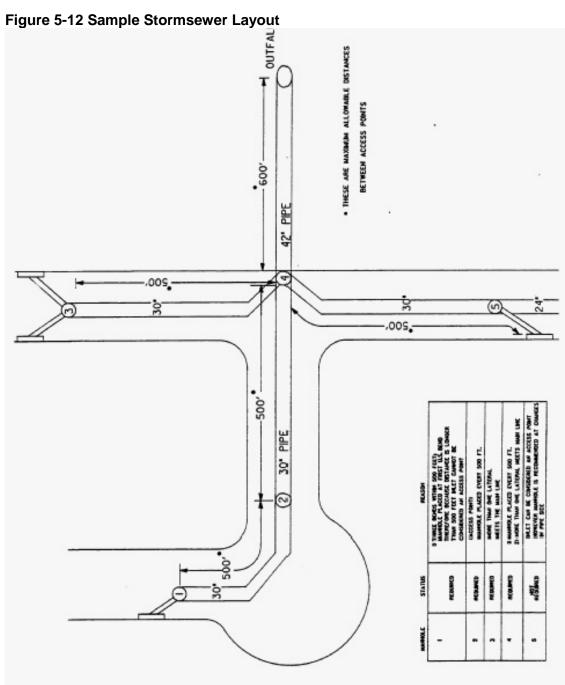


Figure 5-11 Minor Head Losses Due to Turbulence at Structures



Source: City of Austin, Watershed Management Division

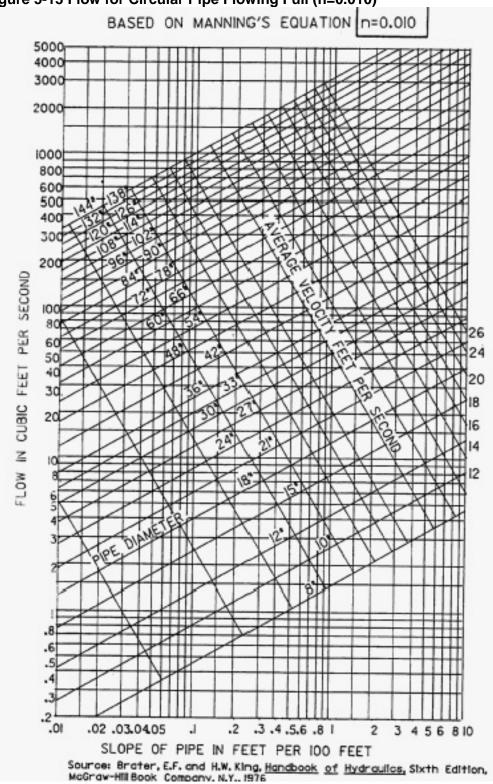


Figure 5-13 Flow for Circular Pipe Flowing Full (n=0.010)

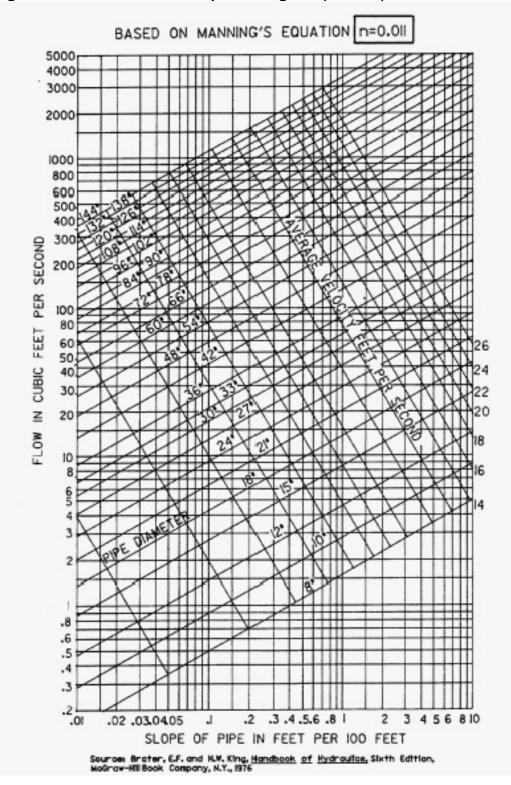


Figure 5-14 Flow for Circular Pipe Flowing Full (n=0.011)

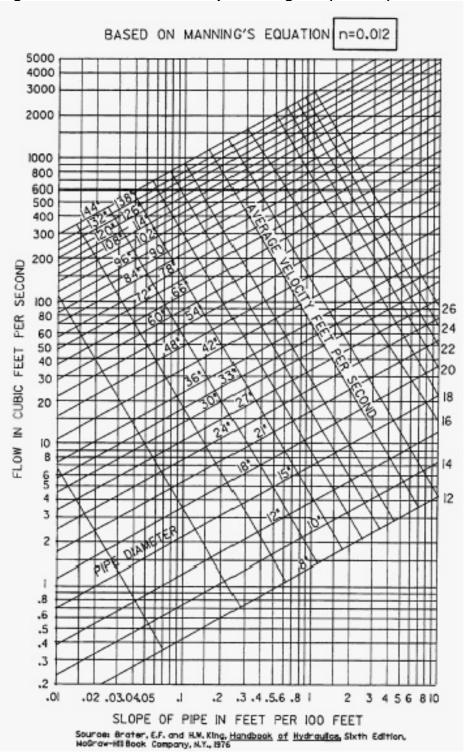


Figure 5-15 Flow for Circular Pipe Flowing Full (n=0.012)

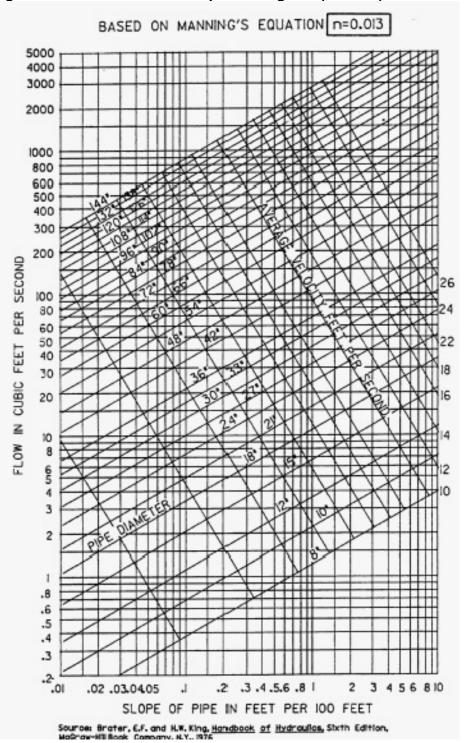


Figure 5-16 Flow for Circular Pipe Flowing Full (n=0.013)

FIGURES FROM SECTION 6

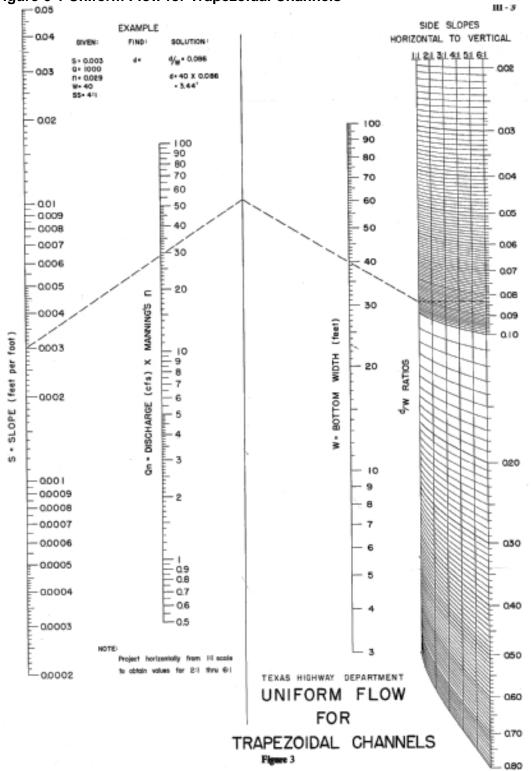


Figure 6-1 Uniform Flow for Trapezoidal Channels

Source: Texas Highway Department, Bridge Division

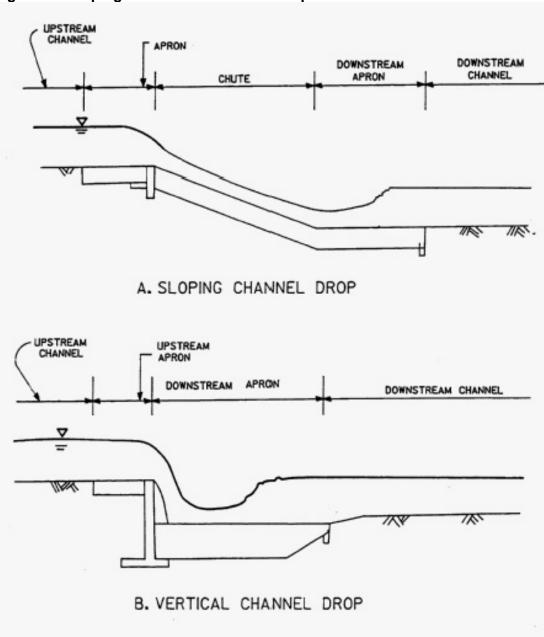


Figure 6-2 Sloping and Vertical Channel Drops

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

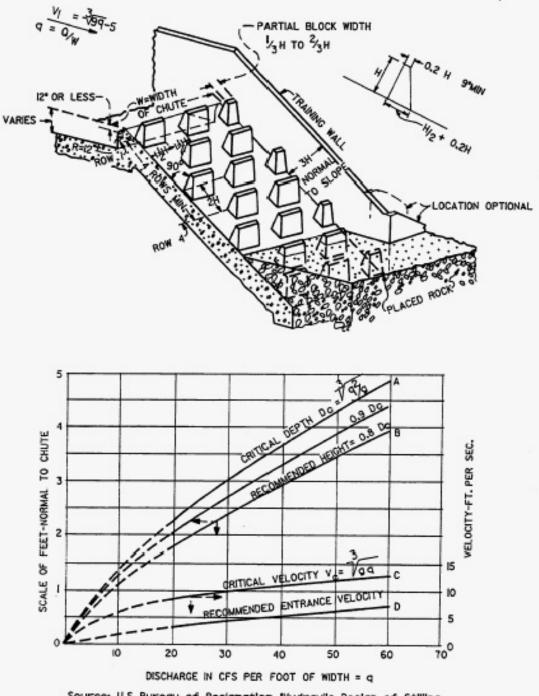
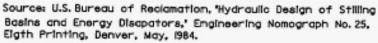


Figure 6-3 Baffled Apron and its Design Curve



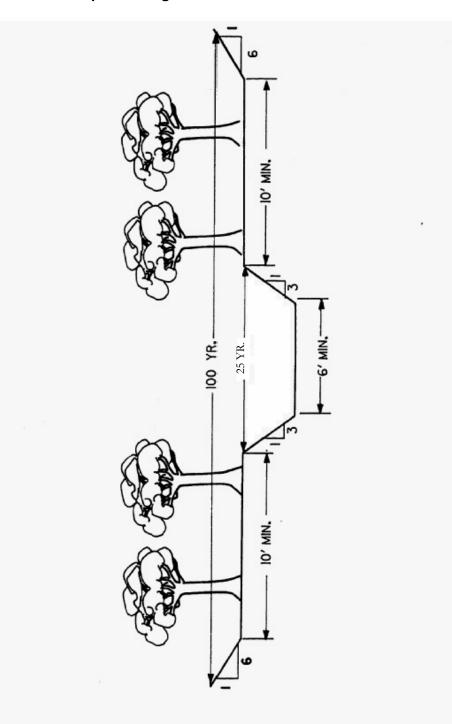


Figure 6-4 Conceptual Design of Alternative Channel

Source: City of Austin, Department of Environmental Protection

FIGURES FROM SECTION 7



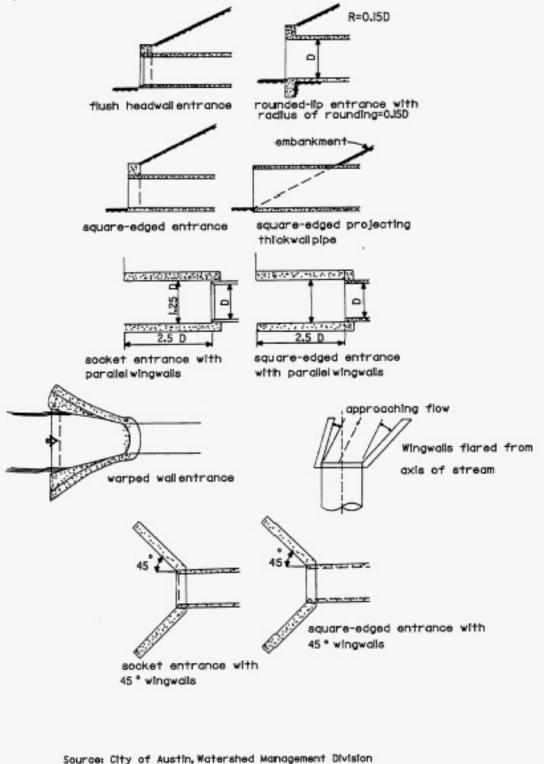
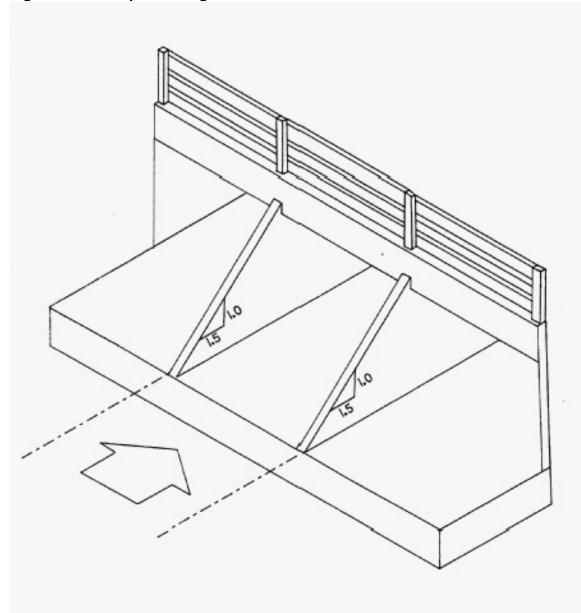
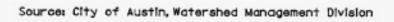


Figure 7-2 Conceptual Design of Debris Fins





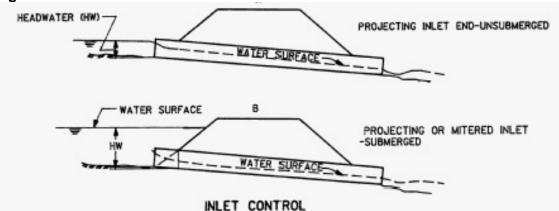
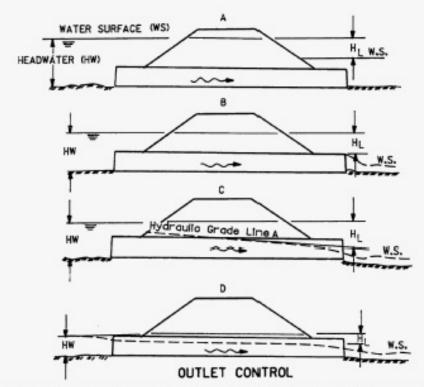


Figure 7-3 Inlet and Outlet Conditions for Culverts

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.



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

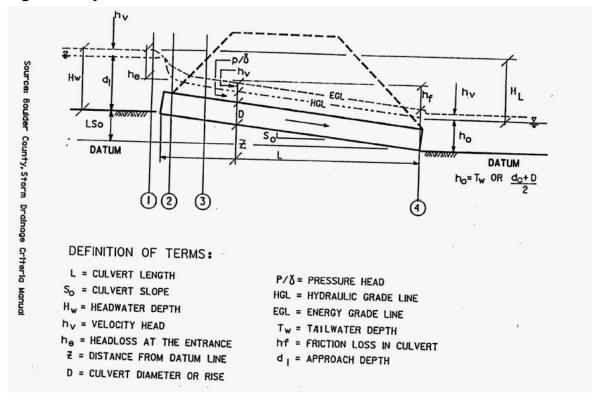


Figure 7-4 Hydraulics of a Culvert Under Outlet Control Condition

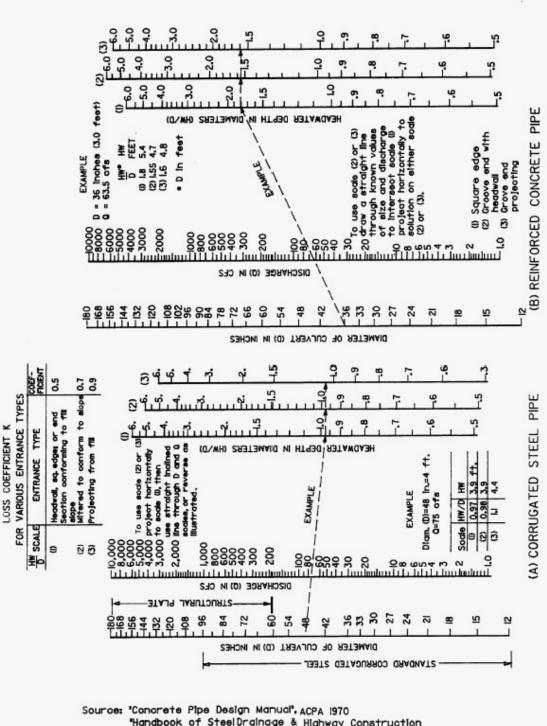


Figure 7-5 Inlet Control Nomograph, Circular Pipe

"Handbook of Steel Drainage & Highway Construction Products", AISI 1971

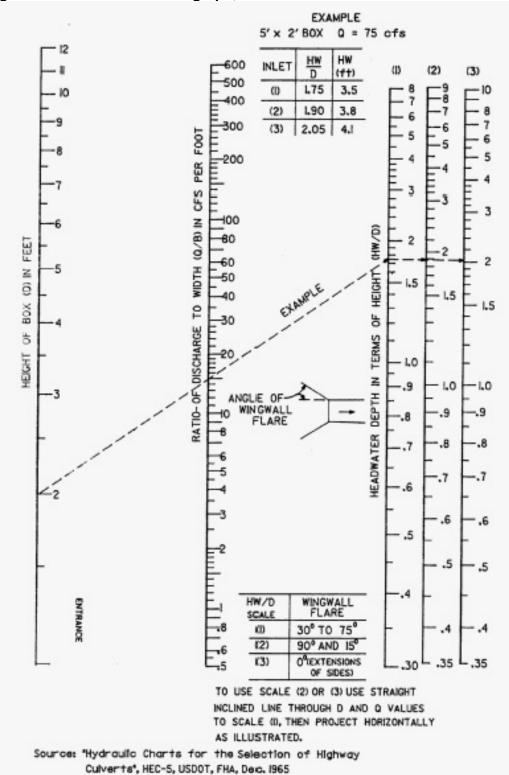
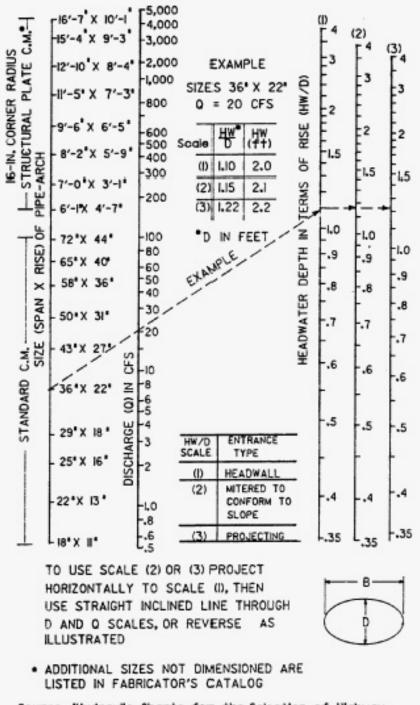


Figure 7-6 Inlet Control Nomograph, Box Culverts

Figure 7-7 Inlet Control Nomograph, CSP Arch



Source: 'Hydraulic Charts for the Selection of Highway Culverts', HEC-5, USDOT, FHA, Dec. 1965

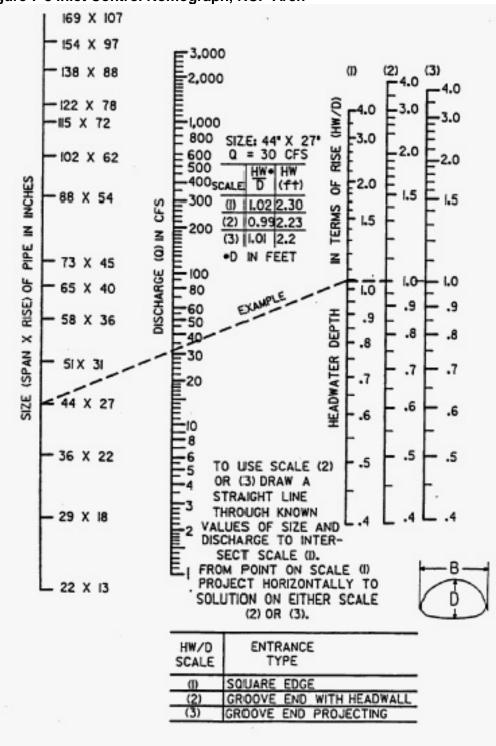


Figure 7-8 Inlet Control Nomograph, RCP Arch

Source: 'Concrete Pipe Design Manual', ACPA 1970

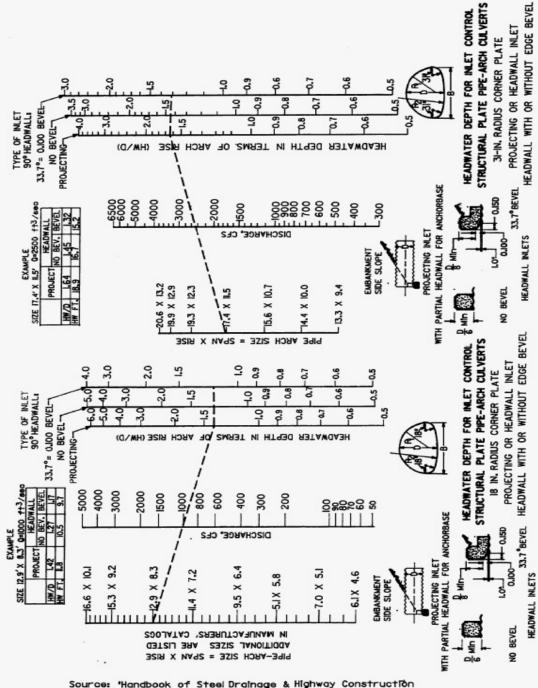


Figure 7-9 Inlet Control Nomograph, SSP Arch

ource: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

-151 × 9 EXAMPLE -3000 SIZE: 76" x 48" Q = 300 CFS (2) 136 x 87 2000 (3) 빵 HW 4.0 D RISE (HW/D) 4.0 (f†) æ -121 × 7 0 2.8 1.2 4.0 .0 (2)1000 3.0 -113 x 72 2.3 (3) 800 -106 x 68 .0 600 5 2.0 - 98 x 63 2.0 500 EXA HEADWATER DEPTH IN TERMS 400 91 × 58 **2.**0 Ē S INCHE .5 300 1.5 83 x 53 -1.5 -200 z 76 x 48 PIPE 100 (Q) IN CFS 68 x 43 RISED OF OVAL 1.0 80 1.0 -1.0 TO USE SCALE (2) OR (3) -60 x 38 60 .9 .9 DISCHARGE DRAW A STRAIGHT LINE .9 50 THROUGH KNOW VALUES 40 OF SIZE AND DISCHARGE -53 x 34 .8 -.8 TO INTERSECT SCALE (D) .8 30 (SPAN X FROM POINT ON SCALE (D) 49 x 32 PROJECT HORIZONTIALLY TO SOLUTION ON EITHER SCALE .7 .7 ringing to the 20 .7 45 x 29 (2) OR (3). SIZE 42 x 27 HW/D ENTRANCE .6 .6 SCALE TYPE .6 10 -38 x 24 8 0 SQUARE EDGE WITH HEADWALL 6 Litet a line hundred .5 .5 (2) GROOVE END WITH HEADWALL -5 5 4 GROOVE END PROJECTING (3) -30 x 19 3 2 1.0 23 x 14

Figure 7-10 Inlet Control Nomograph, RCP Elipse

Source: 'Hydraulic Charts for the Selection of Highway Culverts', HEC-5, USDOT, FHA Dec. 1965

INLET CONTROL NOMOGRAPH

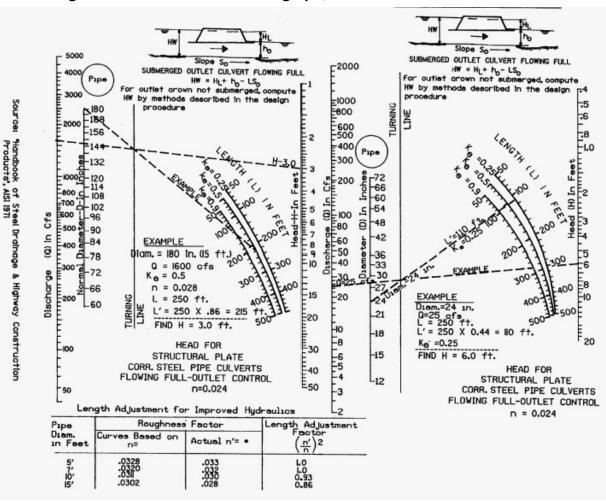
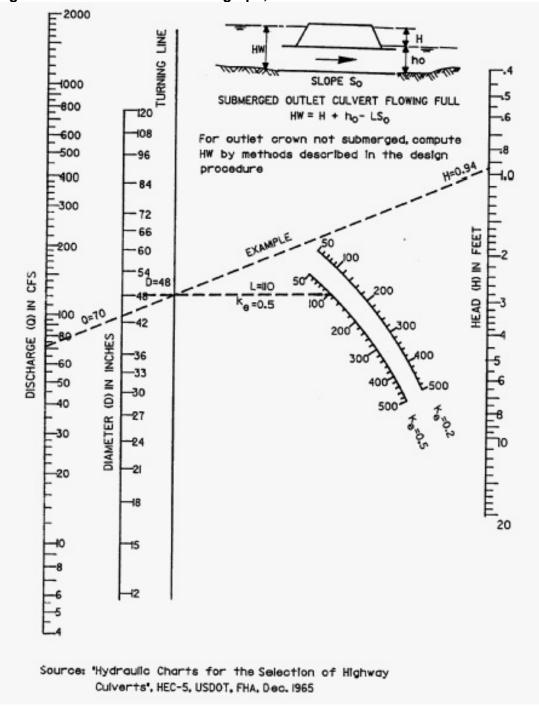


Figure 7-11 Outlet Control Nomograph, Circular CSP





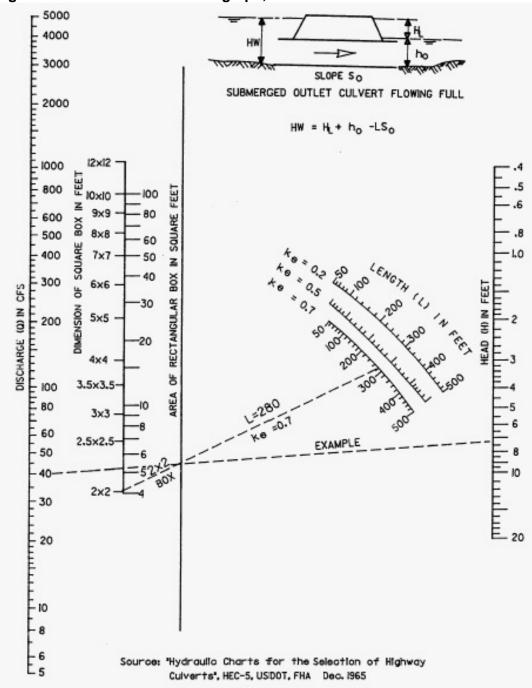


Figure 7-13 Outlet Control Nomograph, Box Culverts

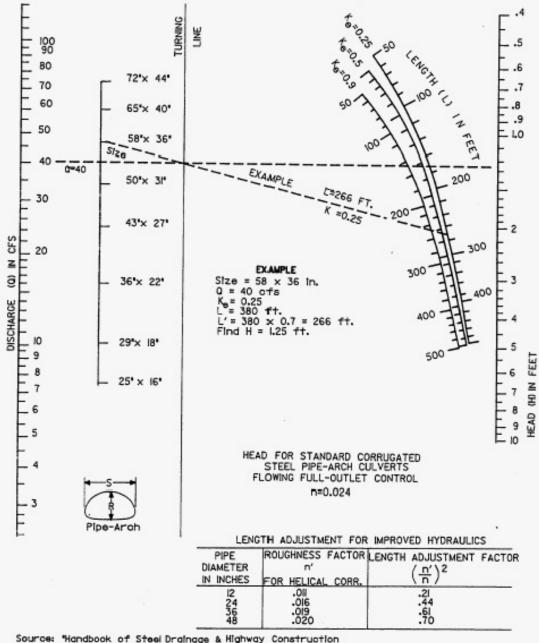


Figure 7-14 Outlet Control Nomograph, CSP Arch

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

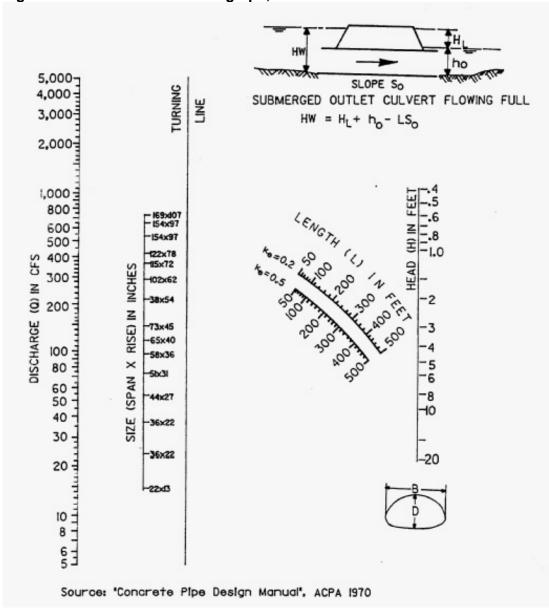


Figure7-15 Outlet Control Nomograph, RCP Arch

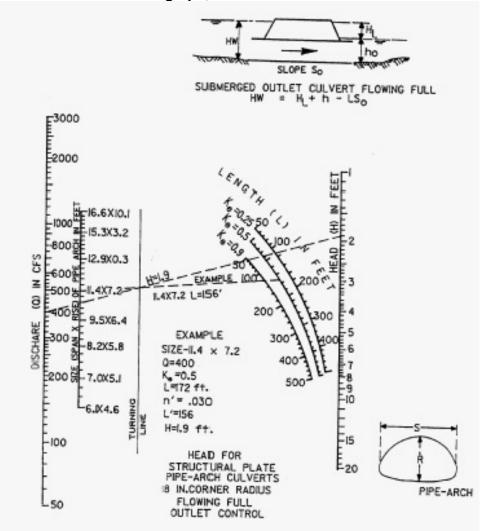


Figure 7-16 Outlet Control Nomograph, SPP Arch

OUTLET CONTROL. Head for structural plate pipe-arch cuivert 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 F	Length Adjustment Facto	
	Curves based on n	Actual n'	$\left(\frac{n'}{n}\right)^2$
6.1x 4.6	.0327	.0327	1.0
8.1× 5.8	.0321	.032	LO
1.4 x 7.2	.03/5	.030	0.907
16.6 × 10.1	.0306	.028	0.837

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

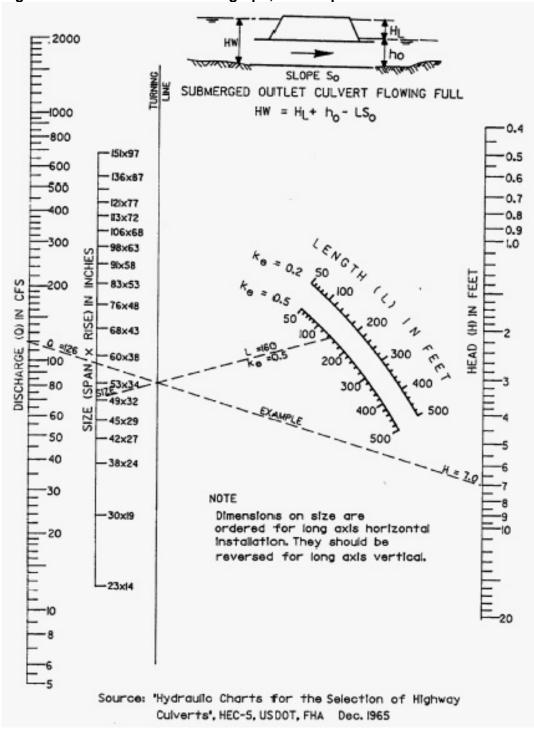


Figure 7-17 Outlet Control Nomograph, RCP Elipse

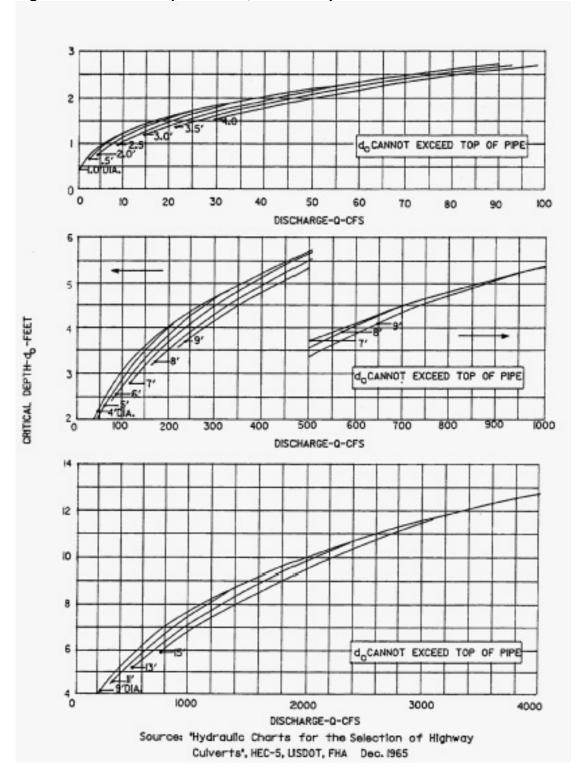


Figure 7-18 Critical Depth Curves, Circular Pipe

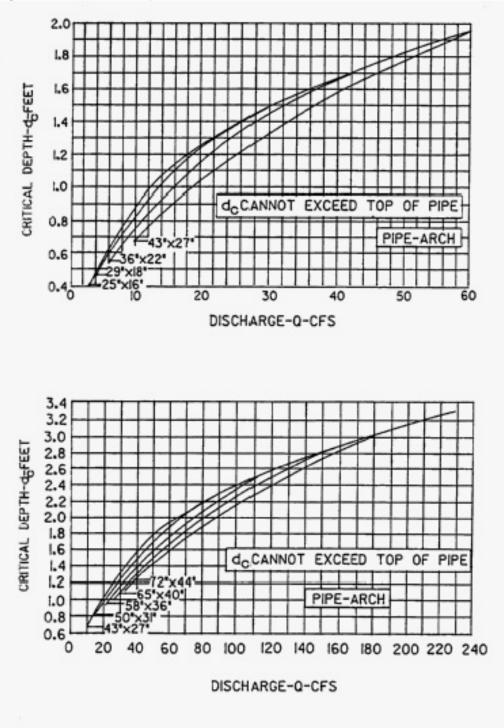


Figure 7-19 Critical Depth Curves, CSP Arch

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

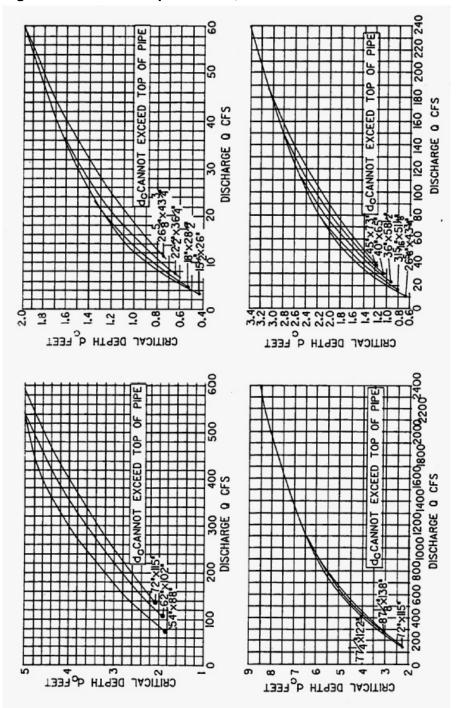


Figure 7-20 Critical Depth Curves, RCP Arch

Source: "Concrete Pipe Design Manual", ACPA 1970

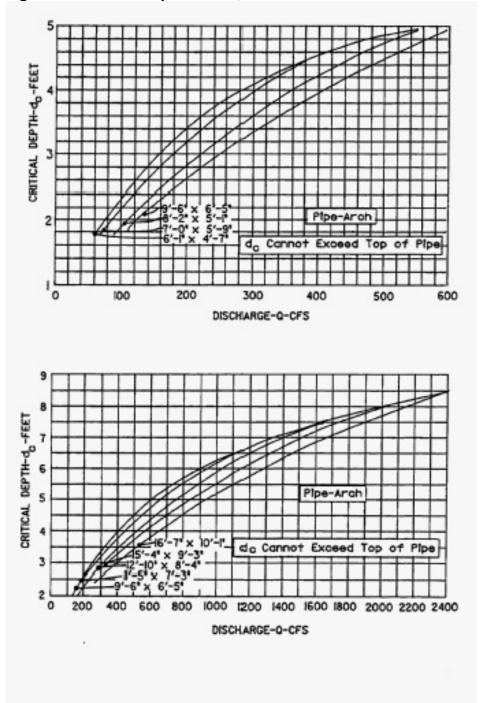


Figure 7-21 Critical Depth Curves, SSP Arch

Source: "Handbook of Steel Drainage & Highway Construction Products", AISI 1971

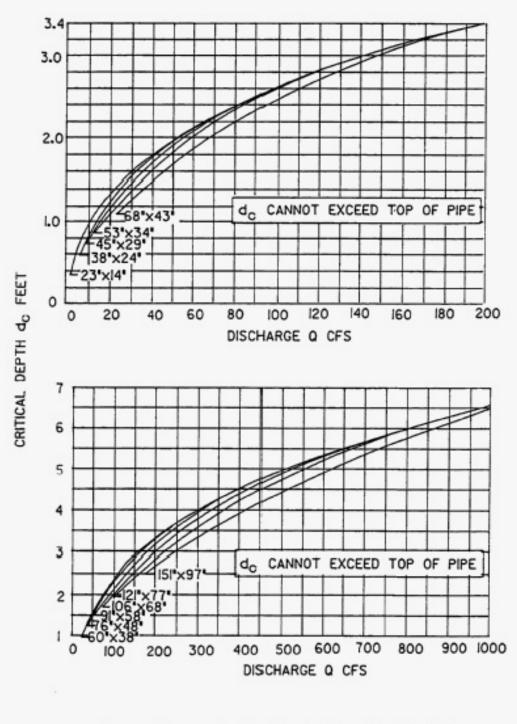


Figure 7-22 Critical Depth Curves, RCP Elipse

Source: 'Concrete Pipe Design Manual', ACPA 1970

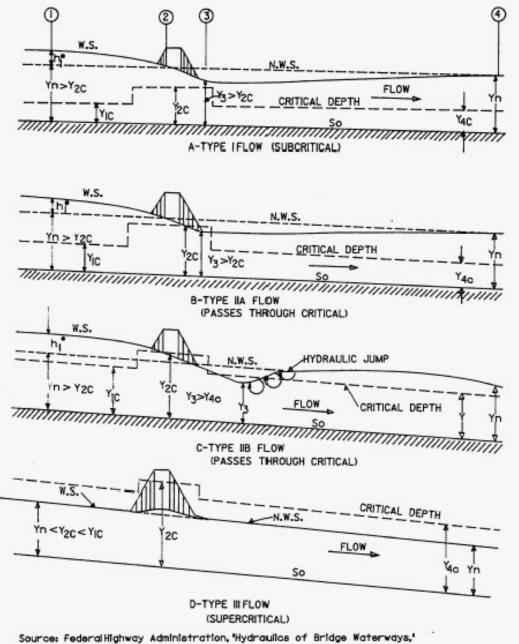


Figure 7-23 Types of Flow for Bridge Design

burce: Federal Highway Administration, Hydraulics of Bridge Waterways Hydraulic Design Series No. I, Second Edition, September, 1973.

FIGURES FROM SECTION 8



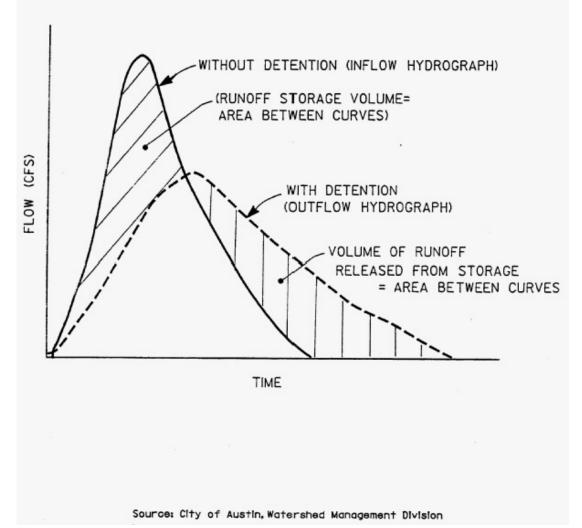
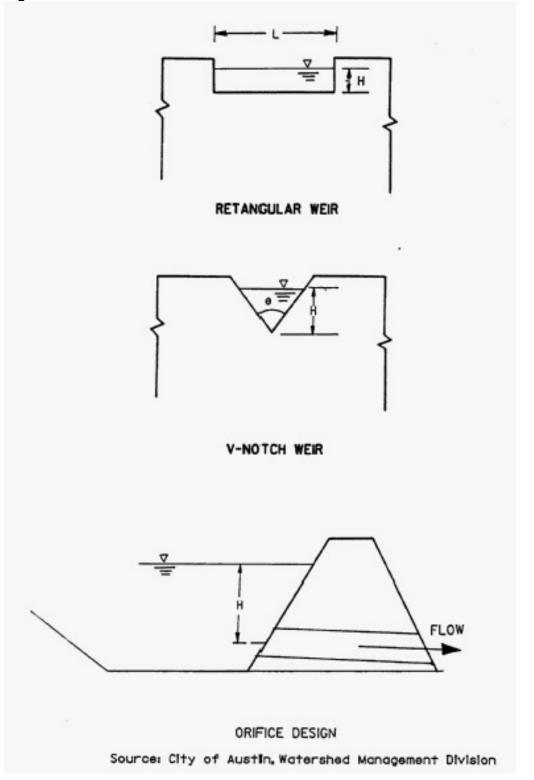


Figure 8-2 Weir and Orifice Flows



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GLOSSARY

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

Apron - A floor or lining of concrete, timber, or other suitable material 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; and (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.

Channel Roughness - The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the Manning 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 - (1) The highest point on a transverse section of a conduit. -

(2) The highest point of a roadway cross section.

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. On-site 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.

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 1 indicates supercritical flow and a value less than 1 subcritical flow.

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 - (1) The upper reaches of a stream near its sources; (2) the region where ground waters emerge to form a surface stream; and (3) the headwater depth on the upstream side of a structure. (See Entrance Head)

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).

 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.

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 Coefficient - The coefficient of roughness used in the Manning Equation.

Manning 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.

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 iayer 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/ 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/ 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.