Storm Water Management Criteria Manual

CITY OF AMARILLO

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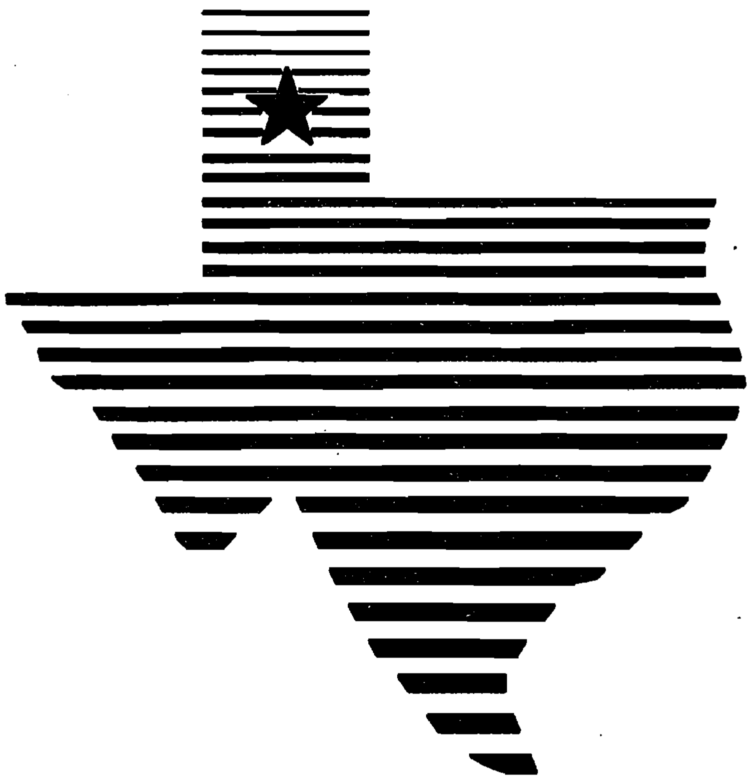

CITY OF AMARILLO

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1.1 INTRODUCTION

These policies shall govern the planning, design, construction, operation and maintenance of all storm drainage facilities within the City of Amarillo and all areas subject to its extraterritorial jurisdiction. Definitions, formulas, criteria, procedures and data presented herein have been developed to support these policies. If a conflict arises between the technical data and these policies, the policies shall govern.

1.2 ADMINISTRATIVE POLICY

1.2.1 Approvals

All storm drainage related plans, public and private, must have written approval from the City Engineer prior to bidding or beginning construction.

1.2.2 Manual Revisions

Revisions to the Storm Water Management Criteria Manual will be issued in writing and can only be made by the City Engineer with approval of the City Commission. Minor changes or modifications will be summarized in errata sheets and will be distributed as necessary. Major changes affecting policy, criteria, methodologies, or engineering data will be distributed as addenda or as replacement pages at the time the change is adopted. It shall be the responsibility of the Manual user to provide the City with a current address in order to receive written changes. Changes requested by users shall be accompanied by detailed comparative engineering data supporting the reasons and justifications for the change.

1.2.3 Design Requirements

The design criteria presented in the Storm Water Management Criteria Manual represent good engineering practice and should be utilized in the preparation of drainage plans. The criteria are not intended to be an iron-clad set of rules within which the developer and engineer must work; they are intended to establish guidelines, standards and methods for sound planning and design. Alternative methods of design should be submitted in a timely fashion to the City Engineer for consideration.

The design criteria shall be revised and updated as necessary to reflect advances in the field of urban drainage engineering, urban water resources management, and as water quality and other regulations change.

The City of Amarillo and engineers will utilize the Manual in the planning of new facilities and in their reviews of proposed works by developers, private parties, and other governmental agencies.

1.2.4 Financial Responsibility

The City Drainage Policy requires that developers pay for all storm drainage facilities that directly serve their development. These facilities shall be adequate to meet the standards of the major and minor storm events.

Construction cost funding of off-site (downstream) drainage facilities, whether new or upgrades to existing, required as a result of a new subdivision development shall be the responsibility of the City. The City shall also be responsible for funding the difference in construction cost of the drainage facility required by an area master plan, drainage study or any other planning information and the facility required to serve the subdivision development (oversize cost). The drainage facility required to serve the subdivision development shall be designed and sized to accommodate storm water from all contributing upstream areas in their present condition. Therefore, oversize cost shall be the construction cost difference in drainage facilities to handle the post-development and pre-development runoff from the upstream areas. The City's funding responsibility for oversize cost shall be limited to runoff from upstream areas not owned or controlled by the subdivision developer. The City's funding responsibility shall not include the cost of acquisition or dedication of any required off-site drainage easements or rights-of-way.

The City shall develop and maintain a Capital Improvement Program for drainage improvements. The Program will include facilities required by anticipated future development. However, when City funding is not immediately available for construction of off-site or oversize drainage improvements required by a new subdivision development, the developer may, at his/her option, wait for such funding to become available before proceeding with development or install all required off-site or oversize drainage improvements at his expense.

1.3 DESIGN POLICY

1.3.1 Qualifications

All drainage related plans shall be reviewed and sealed by a Registered Professional Engineer with a valid license from the State of Texas. The Engineer shall attest that the design was conducted in accordance with the Storm Water Management Criteria Manual.

1.3.2 Computations

Computations shall be submitted for review to the City Engineer and shall be in accordance with the procedures, standards, and criteria of the Storm Water Management Criteria Manual.

1.3.3 Drainage Systems and Design Storm Frequencies

Every area shall be evaluated for two separate and distinct drainage systems. The first drainage system to be evaluated is the minor system and the other is the major system as defined below. To provide for orderly urban growth, reduce costs to future generations, and to prevent loss of life and major property damage, both systems must be planned and properly designed and implemented.

Minor Storm Provisions

The minor storm drainage system is necessary to reduce maintenance costs, to provide protection against frequently occurring storms, to implement an orderly urban system, and to provide convenience to the residents. The minor storm drainage system shall be designed to convey the runoff from a 2-year return period storm. A 2-year storm is one that has a 50% chance of being equalled or exceeded in any given year.

Major Storm Provisions

In addition to providing storm drainage facilities for the minor storm runoff, provisions shall be made to prevent significant property damage and loss of life from the major storm runoff. These provisions shall be known as the major drainage system. The major storm drainage system shall be designed to accommodate a 100-year return period storm. A 100-year storm is one that has a 1% chance of being equalled or exceeded in any given year. The impact of the major storm shall be investigated and adequate major storm facilities shall be provided. A well-planned major drainage system can reduce the need for minor storm drainage systems.

The City Drainage Policy requires all development to include planning, design, and construction of minor and major storm drainage systems in accordance with the Storm Water Management Criteria Manual.

There are many developed areas within the City of Amarillo which do not conform to the drainage standards contained in the Manual. It is recognized that the upgrading of these developed areas to conform to all of the policy, criteria, and standards contained in the Manual will be difficult to obtain, short of complete redevelopment or renewal. Therefore, in the planning of drainage improvements and the designation of floodplains for existing developed areas, the use of the criteria and standards contained in the Manual may be varied or waived as determined by the City Engineer in accordance with established variance procedures.

1.3.4 Hydrologic Analysis

The determination of peak runoff magnitude shall be accomplished using either the Rational Method, HEC-1, SCS TR-20, or other computer modeling techniques and accepted methods approved by the City Engineer. In addition, peak flow curves from which peak rates for various return periods may be determined for the City of Amarillo have been included in Section 2 of the Storm Water Management Criteria Manual. The Amarillo Peak Flow Curve Method is based on drainage area size and land use characteristics and may be used in lieu of the methods noted above for areas up to 2,000 acres. When using HEC-1, the SCS Unit Hydrograph option is preferred.

Areas Under 200 Acres

Design peak discharges for the minor and major storm drainage systems for areas less than 200 acres may be computed using the Rational Method or the Amarillo Peak Flow Curve Method described in Section 2.

Areas Over 200 Acres

Design peak discharges for areas over 200 acres may be computed using the Amarillo Peak Flow Curve Method, HEC-1, SCS TR-20, or other approved computer modeling programs. These methods shall be used for designing both the minor and major storm drainage systems.

Where hydrographs are needed for flood routing through detention basins and other major drainage systems, they shall be computed using HEC-1, SCS TR-20, or another hydrograph modeling technique approved by the City Engineer.

For areas over 200 acres, where the computation of peak flows for individual subbasins less than 200 acres is involved, the Amarillo Peak Flow Curve Method should be used. Use of the peak flow curves in this instance provides a composite approach which ensures the consistency of results obtained using the Rational Method and hydrograph methods over the range of drainage areas up to 2,000 acres.

Areas Over 2,000 Acres

For areas over 2,000 acres, peak flows and design hydrographs should be based on the HEC-1 or SCS TR-20 hydrograph models, and **not** on the Amarillo Peak Flow Curve Method.

1.3.5 Accuracy

The peak discharges determined by analytical methods are approximations. The drainage system will rarely operate at the design discharge. Flow will always be more or less in actual practice, merely passing through the design flow as it rises and falls. Thus, the engineer should not overemphasize the accuracy of his computed discharges. The Engineer should emphasize the design of a practical and hydraulically balanced system based on sound logic and engineering.

1.3.6 Computer Models

A variety of computer models are available for hydrologic and hydraulic analyses. Approval of the City Engineer must be obtained before using models other than those specifically identified in the Storm Water Management Criteria Manual. The most recent versions of the chosen model must be used.

1.3.7 Open Drainage Channels

Open channels for conveyance of storm water runoff are desirable in urban areas and use of such channels is recommended. Wherever possible, natural drainageways should be used for storm runoff waterways. Consideration should be given to the floodplains and open space requirements of the area.

Natural watercourses, perhaps wet only during and after large rainstorms, generally should not be filled, straightened, or altered significantly. Channelizing a natural waterway tends to reduce natural storage, increase flow velocity and cause higher downstream peaks, often to the detriment of those downstream as well as those adjacent to the channel. Effort must be made to reduce flood peaks and control erosion so that the natural channel is preserved. Therefore, drainage designs which include new or reconstructed drainage channels should be carefully weighed against the environmental and financial considerations of maintaining a natural drainageway.

Open channels should generally follow the natural flowlines and should receive early attention in planning stages of a new development, along with other storm runoff features. Optimal benefits from open channels can often be obtained by incorporating parks and greenbelts with the channel layout.

A dedicated maintenance easement shall be provided with all drainage channels. This easement shall provide a minimum access width of 20 feet from the channel bank on each side unless otherwise approved by the City Engineer. For some small channels, this easement may be provided on one side only. These dedicated maintenance easements shall be sufficiently cleared and graded to allow easy access by maintenance equipment.

1.3.8 Playa Effect and Tailwater

The depth of flow in the receiving drainageway or playa must be taken into consideration for backwater computations for both the minor and major storm runoff. Backwater computations shall assume a starting elevation greater than or equal to that for the same return period as the minor and major design storms assuming a fully developed watershed.

1.3.9 Coordination of Planning Efforts

The planning for drainage facilities should be coordinated with planning for open space, transportation, utilities, recreation and solid waste collection. By coordinating these efforts, new opportunities can be identified which will assist in the solution of drainage problems. The planning of drainage works in coordination with other urban needs results in more orderly development and lower cost.

Open space may provide significant urban social benefits. Use of natural drainageways is often less costly than constructing and maintaining artificial channels. Combining the open space needs of a community with major drainageways can be a desirable conjunctive use.

The design and construction of new streets, alleys and highways should be fully integrated with drainage needs of the urban area to promote efficient drainage and avoid the creation of flooding hazards.

1.3.10 Use of Streets and Alleys

Streets are significant and important in urban drainage and shall be used for storm runoff up to reasonable limits, recognizing that the primary purpose of streets is for traffic. Limits of the use of streets for conveying storm runoff shall be governed by the design criteria in Table 1-1.

TABLE 1-1 Minor Storm Runoff Allowable Street Use

Street Classification	Maximum Pavement Encroachment
Local and Collector	No curb overtopping. Flow may spread to crown of street.
Arterial	No curb overtopping. Flow spread must leave at least one lane in each direction free of water.
Freeway	No significant encroachment is allowed on any traffic lanes.

When the above maximum encroachment is reached, a separate storm drainage system or additional storm drainage capacity shall be provided, designed on the basis of the minor storm. However, construction of the major drainage system is encouraged to quickly drain the minor storm runoff from the street.

While it is the intent of this policy to have major storm runoff removed from public streets at frequent and regular intervals into major drainageways, it is recognized that water will often tend to follow streets and roadways. Therefore, streets and roadways often may be aligned so that they will provide a specific runoff conveyance function. Planning and design objectives for the major drainage system with respect to public streets shall be based upon the limiting criteria in Table 1-2.

TABLE 1-2 Major Storm Runoff Allowable Street Inundation

Street Classification	Allowable Depth and Inundated Areas
Local and Collector	Residential dwellings and public, commercial, and industrial buildings shall not be inundated at the lowest finished floor elevation unless buildings are flood-proofed. The depth of water over the gutter flowline shall not exceed 24 inches.
Arterial	Residential dwellings and public, commercial, and industrial buildings shall not be inundated at the lowest finished floor elevation unless buildings are flood-proofed. The depth of water at the street crown shall not exceed 12 inches in order to allow operation of emergency vehicles. The depth of water over the gutter flowline shall not exceed 24 inches.
Freeway	No inundation is allowed.

The allowable flow across streets shall be limited within the criteria shown in Table 1-3.

TABLE 1-3 Allowable Cross Street Flow

Street Classification	Minor Storm Runoff	Major Storm Runoff
Local and Collector	6 inches of depth in valley gutter	24 inches of depth in valley gutter
Arterial	None	12 inches or less over crown
Freeway	None	None

In general, an arterial street crossing will require installation of a storm drain system or other suitable means to transport the minor storm runoff under the arterial street. Collector streets shall have cross valley gutters only at infrequent locations, as specified in accordance with good engineering practices.

Lowering of the standard height of street crown shall not be allowed for the purposes of hydraulic design, unless approved by the City Engineer. In no case will reduced crowns be allowed on arterial streets.

Where additional hydraulic capacity is required on a street, the gradient must be increased and/or inlets and storm drains or other storm water conveyance facilities shall be installed to remove the required portion of the flow.

Alleys are not an integral part of the drainage system. In general, alleys shall be designed to convey only the runoff from the rear of adjacent lots and direct it to the street at the end of the block. In no case shall runoff in any street be directed to flow into an alley or an alley be used as a drainage way.

1.3.11 FEMA National Flood Insurance Program

The Federal Emergency Management Agency (FEMA) administers the National Flood Insurance Program (NFIP) which enables property owners to purchase flood insurance at a reduced cost. In return for making subsidized flood insurance available for existing structures, the participating community agrees to regulate new development in Special Flood Hazard Areas (SFHAs). These SFHAs are areas that have special flood, mudslide and/or flood-related erosion hazards. These regulations are adopted in the form of a floodplain ordinance.

FEMA publishes a Flood Hazard Boundary Map (FHBM) and Flood Insurance Rate Maps (FIRM) showing communities flood hazard areas and the degree of risk in those areas. An FHBM is based on approximate data and identifies, in general, the Special Flood Hazard Areas within a community. FHBM's are used for floodplain management and insurance purposes. When a detailed Flood Insurance Study has been conducted the FIRM will show base flood elevations, insurance risk zones and flood plain boundaries and may show the floodway delineated.

If a flood map is believed to be incorrect, three procedures have been established to change or correct a flood map. They include: 1) Letter of Map Amendment (LOMA); 2) Letter of Map Revision (LOMR), and 3) Physical map revision. A LOMA results from a technical data or scientific data review submitted by the owner who believes his/her property has been incorrectly included in a designated Special Flood Hazard Area. A LOMR is used to change flood zones, flood delineations, flood elevations, and planimetric features. A LOMR is an amendment to the effective FEMA map and is usually followed by a physical map revision. A physical map revision is an official republication of a map to effect changes to flood insurance zones, floodplain delineations, flood elevations, floodways, and planimetric features.

1.3.12 Playa Management

City policy is to provide protection to properties adjacent to playas against flood damage up through the major storm event considering ultimate watershed development conditions. For those areas which do not meet this criterion, protection will be achieved through diversion (pumping or gravity outfall), deepening excavation, or a combination of both. Prior to protecting vulnerable properties through diversion or deepening of the playa, the relative cost of non-structural measures shall be considered.

Projects which propose reclaiming low-lying or flood prone areas by raising the ground surface with fill must conform to the City's Flood Hazard Areas Ordinance which regulates development around playas by requiring developers to provide compensatory storage volume to make up for the capacity lost through the filling project. Proposed filling projects must also address reduced evaporation losses from the smaller free water surface of the playa, the influence of the local water table on playa capacity to store storm runoff, and backwater effects on existing adjacent developed areas.

Predicted levels for the playas for return intervals of 2-year through 100-year shall be as determined by the Playa Simulation Model (ASAPP). Base flood elevations in the Flood Hazard Areas Ordinance may be modified by the City Engineer with approval of the City Commission to match the 100-year flood level as predicted by the Playa Simulation Model.

1.3.13 Erosion and Sediment Control

The need for sediment and erosion control facilities, either permanent or temporary, shall be determined according to the standards for sediment and erosion control in developing areas as stated in the Storm Water Management Criteria Manual.

A temporary erosion and sediment control plan is required for all developments. Any temporary erosion and sedimentation control facilities shall be constructed prior to any grading or extensive land clearing. These facilities must be maintained until construction and landscaping are completed and the potential for significant erosion has passed.

1.3.14 Storm Water Transfer

Planning and design of storm drainage systems should not result in the transfer of drainage problems from one location to another. Channel modifications which create or increase flooding downstream shall be avoided, both for the benefit of the public and to prevent damage to private parties. Erosion and downstream sediment deposition, increase of runoff peaks, and debris transportation should be avoided.

The subdivision development process can significantly alter historical or natural drainage paths. When these alterations result in a subdivision outfall system that discharges back into the natural drainageway at or near the historical location, the alterations are generally acceptable.

Master planning must be based upon potential future upstream development as determined by the City Engineer.

The policy of the City is to avoid transfer of storm drainage runoff from one basin to another and to maintain historical drainage paths. However, the transfer of drainage from basin to basin is a viable alternative in certain instances and will be reviewed on a case by case basis.

1.3.15 Detention and Retention

Storm water runoff may be stored in detention and retention basins and playas. Such storage reduces the drainage capacity required, thereby reducing the land area and expenditures required downstream. Multipurpose utilization of such storage areas is encouraged. Detention/retention basins shall be analyzed both individually and as a system to assure compatibility with one another and with the overall Storm Water Management Master Plan. Playas shall be analyzed using the Playa Simulation Model (ASAPP). All methods of analysis shall be approved by the City Engineer.

Storage of storm water runoff close to the points of rainfall occurrence such as the use of parking lots, ball fields, property line swales, parks, road embankments, borrow pits and on-site ponds is desirable and encouraged.

Parking lots, such as at shopping centers, create rapid runoff with high discharge rates. Parking lots should provide for temporary storage of runoff except where such storage is impractical. Wherever reasonably acceptable, parks should be used for short-term detention of storm runoff to create drainage benefits.

Maintenance of detention and retention basins requires the periodic removal of debris and sediment. Without maintenance, a basin will become unsightly, a social liability and eventually ineffective as a detention or retention basin. Maintenance of these basins will be assumed by the City when such basins are designed and built in accordance with the Storm Water Management Criteria Manual and adequate maintenance provisions, including access, are provided.

1.3.16 Storm Water Runoff Quality

The policy of the City of Amarillo is to include water quality considerations in planning for storm drainage facilities. Sediment and debris must be collected and removed from storm waters by using detention storage or other means. Storm water facilities shall be compatible with the City's National Pollutant Discharge Elimination System (NPDES) permit. The City drainage policy shall require planning for water quality management of storm water runoff. The management of overall water quality of storm drainage will require submittal of a water quality management plan for all new developments compatible with the City's NPDES permit. Any construction activity may require a general storm water discharge permit from the Environmental Protection Agency.

1.3.17 Drainage Plan Requirements

(Revised 10/17/94)

A Drainage Plan shall be prepared for any development within a drainage basin. The purpose of the drainage plan is to identify existing and proposed hydrology and hydraulics of the site and the proposed storm drainage system. The plan shall also propose specific solutions to drainage problems that would occur as a result of development. Detailed analysis of drainage basin hydrology and hydraulics is required. Alternative solutions to drainage problems shall be noted and the capacity of facilities on and off-site shall be evaluated. Specific improvements including open channels, storm drains, grading, erosion and sediment control, inlets, culverts, detention/retention basins and other improvements shall be located and sized to meet the requirements of the minor and major drainage systems. The plan must describe the general treatment of drainageways, including safety and maintenance, and outline the protection of public facilities and the protection of private property adjacent to the drainageways.

There are many properties within the City of Amarillo that offer opportunities for redevelopment or in-fill development. It is recognized that most of the existing developed area around these properties will not conform to the drainage standards established in the Manual. It is also recognized that upgrading the existing developed areas to conform to all the policy, criteria and standards in the Manual will be difficult to achieve. Therefore, in planning the drainage improvements and the design of redevelopment or in-fill projects, the use of the critieria and standards contained in the Manual may be varied or waived as appropriate by the City Engineer.

At a minimum, a drainage plan shall be submitted for all developments that certifies that buildings set 18" above the curb will be protected from the major storm. If there is no curb and gutter, a minimum building floor elevation shall be established for each building site that is at or above the level of the major flood.

1.3.18 Drainage Plan Submittal Standards

Plans and profiles shall be drawn on sheets 24" x 36" to a horizontal scale of 1"=50' and to a vertical scale of 1"=2' or 1"=5' (scales may vary on special projects, such as culverts and channel cross sections).

Good quality reproducibles of the original drawings shall be presented to the City of Amarillo Engineering Department prior to the receipt of final approval and shall remain the permanent property of the City of Amarillo.

Plans for the proposed storm drainage system shall include property lines, lot and block numbers, dimensions, right-of-way and easement lines, flood plains, street names, paved surfaces (existing or proposed), location, size and type of inlets, manholes, culverts, pipes, channels and related structures, contract limits, outfall details, miscellaneous riprap construction, contour lines, and title block.

Profiles shall indicate the proposed system (size and material) with elevations, flowlines, gradients, left and right channel bank profiles, station numbers, inlets, manholes, groundline and curbline elevations, typical sections, riprap construction, filling details, minimum permissible building floor elevations within 100-year floodplains and adjacent to open drainage features, pipe crossings, design flow capacities and velocities, and title block.

1.3.19 Operations and Maintenance

Operation and maintenance of storm water facilities and playas is required to ensure that they will perform as designed. Channel bed and bank erosion, drop structures, pipe inlets, and outlets, pumping facilities and overall condition of the facilities shall be routinely inspected and repaired as necessary to avoid reduced conveyance capacity, displeasing aesthetics and ultimate failure. Sediment and debris shall be periodically removed from channels, storm drains, detention basins and retention basins. Trashracks and inlets shall also be routinely cleared of debris to maintain system capacity.

The developer shall provide for perpetual maintenance of private drainage facilities. Private drainage facilities are those drainage improvements which remain on private property, are designed to serve only private property and are not owned by the City. The City will provide for perpetual maintenance of public drainage facilities after the warranty period.

The City Drainage Policy requires that access be provided to all storm water facilities for maintenance and inspection. Developers shall be responsible for providing system features to facilitate maintenance of minor drainage systems, including inlets, pipes, culverts, channels, ditches, detention basins and retention basins.

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2.1 INTRODUCTION

The primary objective of this section is to describe how a hydrologic model can be used to develop peak discharge-frequency estimates for a watershed. The hydrologist can use the material presented herein to help choose the appropriate level and detail of a study, based on the availability of the data, time and funds available, and the accuracy requirements of the study results. Any available data for the basin should be used in the calibration of the hydrologic model and in the development of discharge-frequency curves at gaged locations. By using all the resources available, including data from regional sources such as the United States Geological Survey in conjunction with the analytical techniques described herein, the hydrologist can usually make reasonable determinations of discharge-frequency relations for ungaged basins.

This section presents guidelines and methods for determining storm runoff for watersheds within the City of Amarillo and its extraterritorial jurisdiction area. It describes the method used for determining storm runoff from watersheds of less than 200 acres. It then briefly describes hydrologic models which can be used on watersheds greater than 200 acres. The recommended models for the major watersheds in Amarillo are provided.

There are several methods for determining the appropriate storm runoff from a watershed. The Rational Method may be used as the primary tool for the determination of peak storm water runoff rates from areas 200 acres or less and is especially useful for the design of storm sewer systems. However, the Rational Method shall not be used where determining detention storage requirements. The Amarillo Peak Flow Curve Method may also be used for various basin sizes for estimating peak flows for a range of flood events and drainage areas. In instances where detention is modeled, a hydrograph producing method, such as HEC-1, is required. The U.S. Army Corps of Engineers HEC-1[17] computer program computes the storm runoff and volume utilizing a variety of methods. The most extensively used methodology is based on the Soil Conservation Service (SCS) Unit Hydrograph procedures. These procedures are used to quantify the effects of urbanization, to determine peak flows for large drainage areas, and to design storm water storage facilities. The SCS Unit Hydrograph Method is used and accepted nationwide.

The presentation of these methods is not intended to preclude the use of other methods. However, the designer is advised to secure approval from the City Engineer before utilizing different methods.

2.2 RATIONAL METHOD

The Rational Method is an empirical runoff formula which has gained wide acceptance because of its simple intuitive treatment of peak storm runoff rates in areas less than 200 acres. This method relates runoff to rainfall intensity, surface area and surface characteristics by the formula:

$$Q = C_f \, CiA \tag{2-1}$$

where:

Q	=	peak runoff rate, in cubic feet per second (cfs)
C_f	=	correction factor to adjust the runoff coefficient for less frequent high intensity storms
C	=	runoff coefficient
i	=	average rainfall intensity, for a duration equal to the time of concentration, in inches per hour
A	=	drainage area of the tributary to the point under consideration, in acres

The Rational Method is based on the following assumptions:

A. The peak rate of runoff at any point is a direct function of the average uniform rainfall intensity during the time of concentration to that point.

B. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

C. The time of concentration is the time required for the runoff to become established and flow from the most hydraulically remote part of the drainage area to the point under design. This assumption applies to the most remote in time, not necessarily in distance.

Although the basic principles of the Rational Method apply to drainage areas greater than 200 acres, practice generally limits its use to some maximum area. For larger areas, storage and subsurface drainage flow cause an attenuation of the runoff hydrograph so that the rates of flow tend to be overestimated by the Rational Method. In addition, the assumption of uniform rainfall distribution and intensity becomes less appropriate as drainage area increases. Because of the trend for overestimation of flows and the additional cost in drainage facilities associated with this overestimation, the application of a more sophisticated runoff computation technique is usually warranted for larger drainage areas.

2.2.1 Runoff Coefficient, C

The runoff coefficient, C, is a variable of the Rational Method which is least susceptible to a precise determination and provides the designer with a degree of latitude to exercise his independent judgment. The following discussion is intended to provide a guide to promote the uniform application of runoff coefficients.

The runoff coefficient, C, accounts for abstractions for losses between rainfall and runoff which may vary with time for a given drainage area. These losses are caused by interception by vegetation, infiltration into permeable soils, retention in surface depressions, and evaporation and transpiration. In determining this coefficient, differing climatological and seasonal conditions, antecedent moisture conditions and the intensity and frequency of the design storm should be considered.

Table 2-1 represents recommended C values. Where ranges are shown, adjustments should be made for level of development, surface type, soil type and slope. It is often desirable to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure can be applied to typical "sample" areas as a guide to the selection of usual values of the coefficient for the entire area. Suggested coefficients with respect to surface types are given in Table 2-2.

TABLE 2-1 Recommended Runoff Coefficients **(Revised 9/7/94)**

Description of Area	Runoff Coefficient
• Single Family Residential Land Use:	
Estate greater than 20,000 sq. ft. lot	0.35 - 0.50
20,000 to 10,000 sq. ft. lot	0.40 - 0.60
10,000 to 7,500 sq. ft. lot	0.45 - 0.65
7,500 to 5,000 sq. ft. lot	0.50 - 0.70
• Multiple Family Residential Land Use:	
Low Density (4 stories or less)	0.50 - 0.70
Medium Density (7 stories or less)	0.55 - 0.80
High Density (more than 7 stories)	0.60 - 0.85
• Commercial Land Use:	
Limited and General Office Building Sites	0.60 - 0.85
Shopping Center Sites	0.65 - 0.95
Neighborhood Business District Sites	0.65 - 0.95
Central Business District Sites	0.85 - 0.95
Office Parks	0.65 - 0.95
• Industrial Land Use:	
Limited (service station, restaurant, light industrial, car wash)	0.65 - 0.95
General (auto sales, auto repair, body repair, convenience storage, food sales)	0.65 - 0.95
Heavy (surface parking, parking structure, transportation terminals, warehousing)	0.85 - 0.95

TABLE 2-2 Suggested Runoff Coefficient for Surface Types

Character of Surface	Runoff Coefficient
· Pavement	
Asphaltic and Concrete	0.95
Brick	0.85
· Roofs	0.95
· Turf Slopes	
Flat, 0 to 1%	0.25
Average, 1 to 3%	0.35
Hilly, 3 to 10%	0.40
Steep, 10%	0.45
· Cultivated Ground	
Flat, 0 to 1%	0.10
Average, 1 to 3%	0.20
Hilly, 3 to 10%	0.25
Steep, 10%	0.30

The designer shall use the above values as a rule of thumb. Areas not conforming to the above descriptions will be evaluated by calculating a composite runoff coefficient. Areas will be evaluated based upon the ultimate development.

The coefficients in these two tables are applicable for storms of 2 to 10 year frequencies. These coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen. Table 2-3 represents correction factors to adjust the runoff coefficient for less frequent high intensity storms.

TABLE 2-3 Frequency Factors for the Rational Formula

Recurrence Interval (years)	Adjustment Factor, C_f
2 to 10	1.00
25	1.10
50	1.20
100	1.25

2.2.2 Rainfall Intensity, i

Rainfall intensity, i, is the average rate of rainfall in inches per hour. Intensity is selected on the basis of design frequency of occurrence, a statistical parameter established by design criteria, and rainfall duration. For the Rational Method, the critical rainfall intensity is the rainfall having a duration equal to the time of concentration of the drainage basin.

Rainfall intensity can be determined for the 2-, 5-, 10-, 25-, 50- and 100-year return periods from Figures 2-1 and 2-2 for durations from five minutes to 24 hours. The curves in Figure 2-1 are applicable for design frequencies up to a 100-year storm and for durations expressed in minutes, while Figure 2-2 presents the duration in hours.

2.2.3 Time of Concentration, t_c

One of the basic assumptions underlying the Rational Method is that runoff is a function of the average rainfall rate during the time of concentration. The time of concentration, t_c, is defined as the time it takes for runoff to become established and flow from the most hydraulically remote distant part of the drainage area to the point of reference. It is usually computed by determining the water travel time through the watershed. Overland flow or shallow concentrated flow, storm drain or road gutter flow, and channel flow are the three phases of direct flow commonly used in computing travel time.

For urban areas, the time of concentration consists of an inlet time or overland flow time (t_i) plus the travel time (t_t) in the storm drain, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time (t_i) plus the time of travel (t_t) in a combined form, such as a small swale, channel, or drainageway. The travel portion (t_t) of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway. Inlet time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. The time of concentration can be represented by Equation 2-2 for both urban and non-urban areas.

$$t_c = t_i + t_t \qquad (2\text{-}2)$$

where:

t_c = time of concentration, in minutes

t_i = initial, inlet, or overland flow time, in minutes

t_t = travel time in the ditch, channel, gutter, or storm sewer, in minutes

Overland Flow

The travel time for overland flow consists of the time it takes water to travel from the uppermost part of the watershed to a defined channel or inlet of the storm sewer system. Overland flow is significant in small drainage areas because a high proportion of travel time is due to overland flow. The velocity of overland flow can vary greatly with the surface cover and tillage. If the slope and land use of the overland flow segment are known, the travel time can be calculated using Equation 2-3.

The Kerby Equation is used to compute t_c for overland flow only. If channelized flow occurs in the sub-basin area, other methods must be used to determine the flow time in the channel.

The addition of these two flow times will provide the inlet time of concentration. The Kerby Equation is as follows:

$$t_c = 0.83\left[\frac{NL}{S^{0.5}}\right]^{0.467} \tag{2-3}$$

where:

t_c	=	time of concentration, in minutes
N	=	coefficient of roughness, Table 2-4
L	=	length, in feet, measured from the extremity of the catchment area in a direction parallel to the slope until a defined channel is reached
S	=	slope, in feet per foot, the difference in elevation between the extreme point of the catchment area and the point in question, divided by the distance between the two points

TABLE 2-4 Values of N for Use in the Kerby Formula

N	Type of Surface
0.02	smooth impervious surfaces
0.10	smooth bare packed soil, free of stones
0.20	poor grass, cultivated row crops or moderately bare surfaces
0.40	pasture or average grass cover
0.60	deciduous timberland
0.80	conifer timberland, deciduous timberland with deep forest litter or dense grass cover

The time of concentration calculated for fully developed land use should not be less than 5 minutes to avoid the oversizing of inlets, storm sewers and open channels.

Overland flow length should not exceed 500 feet for developed areas or 1,000 feet for undeveloped areas before being intercepted by a defined channel or storm sewer inlet. Beyond these distances, use gutter flow or channel flow velocities based on Manning's Equation. For preliminary work, the travel time (t_t) can be estimated with the help of Figure 2-3.

Storm Sewer or Street Gutter Flow

Travel time through the storm sewer or street gutter system to the main open channel is the sum of travel times in each individual component of the system between the uppermost inlet and the outlet. In most cases, average velocities can be used without a significant loss of accuracy. During major storm events, the sewer system may be fully taxed and additional channel flow may occur, generally at a significantly lower velocity than the flow in the storm sewers. By using the average conduit size and the average slope (excluding any vertical drops in the system), the average velocity can be estimated using Manning's Formula.

Since the hydraulic radius of a pipe flowing half full is the same as when flowing full, the respective velocities are equal. Travel time may be based on the pipe flowing full or half full. The travel time through the storm sewers is computed by dividing the length of flow by the average velocity.

Channel Flow

The travel time for flow in an open channel can be determined by using Manning's Formula to compute average velocities. Average velocities for channel flow should be computed assuming bankfull conditions.

Example 1 Time of Concentration of an Urbanized Watershed

Given: A model urbanized watershed is shown below. Three types of flow exist from the furthermost point of the watershed to the outlet. The following data describes the watershed.

Reach	Description of Flow	Slope, S (Percent)	Length, L (Feet)
A to B	Overland (park, N = 0.40)	2.0	300
B to C	Shallow concentrated (paved)	0.5	900
C to D	Storm sewer (n = 0.015; diameter = 3 ft)	0.4	2,000
D to E	Open channel, gunite, trapezoidal (b = 5 ft; d = 3 ft; R = 1.78 ft; Z = 1; n = 0.019)	0.3	3,000

Find: Compute time of concentration (t_c) for a 10-year recurrence interval

Solution:

1) Compute the overland flow travel time.
Reach A to B (park). From Equation 2-3 for S = 0.02 ft/ft; L = 300 feet; and N = 0.4; t_i = 19.4 minutes.

2) Reach B to C (paved). From Figure 2-3 for a S = 0.5 percent, L = 900 feet; velocity, V, = 1.4 fps.

$$t_t = \frac{L}{60V} = \frac{900}{60(1.4)}$$

t_t = 10.7 minutes

3) Compute the storm sewer flow travel time.
Reach C to D. Use Manning's Equation to compute full-pipe velocity. R=D/4 for a circular pipe.

$$V = \frac{1.49}{n}\left[\frac{D}{4}\right]^{2/3} S^{1/2}$$

$$V = \frac{1.49}{0.015}\left[\frac{3}{4}\right]^{2/3}(0.004)^{1/2} = 5.2\ fps$$

$$t_t = \frac{L}{60V} = \frac{2{,}000}{60(5.2)} = 6.4\ minutes$$

4) Compute the open-channel flow travel time.

Reach D to E. Use Manning's Equation to compute bankfull velocity.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$V = \frac{1.49}{0.019}(1.78)^{2/3}\ (0.003)^{1/2} = 6.3\ fps$$

$$t_t = \frac{L}{60V} = \frac{3{,}000}{60(6.3)} = 7.9\ minutes$$

5) Summary

Reach	Description of Flow	Length (ft)	Velocity (fps)	Travel Time (min)
A to B	Overland	300	0.3	19.4
B to C	Shallow concentrated (paved)	900	1.4	10.7
C to D	Storm sewer	2,000	5.2	6.4
D to E	Open channel	3,000	6.3	7.9
Totals		6,200		44.4

t_c = 45 minutes, for the basin

2.3 CITY OF AMARILLO PEAK FLOW CURVE METHOD

2.3.1 City of Amarillo Peak Flow Curves

Standard Peak Flow Curves for various basin sizes were developed for estimating peak flows for a range of flood events and drainage areas. The curves were developed based on hydrologic parameters which were found to be typical in Amarillo. Peak flow curves were developed for the 2-, 5-, 10-, 25-, 50- and 100-year flood events and are based on drainage area, average basin slope, and the SCS Runoff Curve Number.

Average Basin Slope

The average basin slope is an indicator of the general slope of the basin. It is determined by calculating the difference in elevation between the highest point in the basin and the elevation of the basin outlet and dividing it by the straight-line distance between these two points. **If the average basin slope is found to be greater than 3%, then the curves should not be applied to the basin.**

SCS Runoff Curve (CN)

The SCS uses an index called the runoff curve number (CN) to represent the combined hydrologic effect of the soil type, land use, hydrologic condition of the soil cover, and the antecedent soil moisture. The CN indicates the runoff potential of soil which is not frozen. Higher CN's reflect a higher runoff potential.

The SCS soil classification system consists of four soil groups which are characterized as follows:

Group A: deep sand, deep loess, aggregated silts

Group B: shallow loess, sandy loam

Group C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay

Group D: soils that swell significantly when wet, heavy plastic clays and certain saline soils

County soil surveys made by the SCS districts give a detailed description of the soils in a county. This is generally the best means of identifying the soil group.

The SCS cover classification includes three factors: 1) land use, 2) treatment, and 3) hydrologic condition. The land uses are subdivided by treatment practices (Table 2-5). The hydrologic condition reflects the level of land management which are given as poor, fair and good.

Antecedent soil moisture has a significant effect on runoff potential (CN). The SCS has developed three antecedent moisture conditions which are described below:

Condition I: soils are dry but not to wilting point
Condition II: average conditions
Condition III: heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated soils

The antecedent moisture condition (AMC) selected should produce reasonable runoff hydrographs results for a specific design problem. The authors obtained acceptable results for watersheds in Amarillo using AMC II for all storm frequencies.

Development of the City of Amarillo Peak Flow Curves

The peak flow curves were developed assuming a standard rectangular basin shape having dimensions 3L by L. The peak flows for a range of flood events were determined using the Rational Method for drainage areas of 200 acres and less and using the HEC-1, Flood Hydrograph Package[17], for drainage areas greater than 200 acres. The unit peak discharge rates were transitioned between the two methods from a drainage area of 50 acres to 400 acres.

Computation of the peak flow for the Rational Method was based on procedures outlined in Section 2.2. The Rational Method runoff coefficient, C, was determined by equating the SCS Runoff Curve Number to a C value using Rossmiller's equation[10]. Computation of the peak flow using HEC-1 was conducted using the SCS Type II Rainfall Distribution, SCS Runoff Curve Number Loss Rate procedure, and the SCS Unit Hydrograph Method. Time of concentration for each of the drainage areas and slope categories was computed following procedures discussed in Section 2.2. The average basin slope used in the computation of time of concentration was not the mid-range value, but the 75% value. For example, if the slope category is 0.5% to 1%, the average basin slope used in the time of concentration computations was 0.875%.

2.3.2 Application of the City of Amarillo Peak Flow Curves

In order to apply the City of Amarillo Peak Flow Curves, the user must first determine three parameters: drainage area (acres), average basin slope, and SCS Runoff Curve Number. There are limitations in the application of the curves. The curves only apply to basins with a drainage area of less than 2,000 acres and an average basin slope of 3% or less. The curves are also based on a standard basin rectangular shape having dimensions 3L by L. **If the basin shape varies significantly from the 3L by L rectangular shape, the curves should not be used.**

TABLE 2-5 SCS Runoff Curve Numbers for Urban Areas and Other Agricultural Lands

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	**Average Percent Impervious Area[2]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm 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
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier), desert shrub with 1-to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	91	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82

TABLE 2-5 SCS Runoff Curve Numbers for Urban Areas and Other Agricultural Lands

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	**Average Percent Impervious Area**[2]	**A**	**B**	**C**	**D**
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Pasture, grassland, or range--continuous forage for grazing.[6]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow--continuous grass, protected from grazing and generally mowed for hay.	--	30	58	71	78
Brush--brush-weed-grass mixture with brush the major elements.[7]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[8]30	48	65	73
Woods--grass combination (orchard or tree farm)[9]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[10]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads--buildings, lanes, driveways, and surrounding lots.	--	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using Figures 2-3 or 2-4 located in TR-55.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using Figures 2-3 or 2-4 located in TR-55 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figures 2-3 or 2-4, located in TR-55 based on the degrees of development (impervious area percentage) and the CN's for the newly graded pervious area.
6 *Poor:* <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.
7 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
8 Actual curve number is less than 30; use CN = 30 for runoff computations.
9 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pastures.
10 *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, Urban Hydrology for Small Watersheds, Technical Release No. 55.

The peak flow curves are presented for the 2-year through 100-year flood events for slope ranges of 0 to 3%. There are eight curves for each flood event and slope category which show SCS Runoff Curve Numbers ranging from 70 to 100. Refer to Table 2-5 for selection of the SCS Runoff Curve Number. Once the drainage area, average basin slope, and runoff curve number are known, the peak flow can be determined for a particular site. For example, the 100-year peak discharge for a 100 acre drainage area with an average basin slope of 1.5% and an SCS Runoff Curve Number of 85 would be determined to be 240 cfs. This is determined by using the 100-year Peak Flow Curves for the average basin slope range of 1% to 2%. For a 100 acre drainage area and a curve number of 85, the unit peak discharge is found to be 2.4 cfs/acre. Multiplying 2.4 cfs/acre times the 100 acre drainage area yields the 100-year peak discharge rate of 240 cfs for the site.

2.4 HYDROLOGIC MODELS

For basins with areas larger than 200 acres or if storage is evaluated, a more complex approach to computing runoff is used. This involves hydrologic models which allow for the temporal variation of rainfall intensity and describe the shape of a hydrograph in a realistic manner (i.e., the initial onset of the storm produces little runoff and as a larger portion of the basin contributes the runoff increases, and finally a peak occurs. The storm reduces in intensity and the runoff diminishes until it has reached its baseflow or a no flow condition.)

Basically, in hydrologic models, the transformation from rainfall excess to streamflow is accomplished through unit hydrograph routing procedures. These procedures allow hydrograph analysis concepts to be applied to watersheds through the development and application of generalized functions for estimating the amount of precipitation lost due to interception and infiltration (loss rates), unit hydrographs, and base flow. The unit hydrograph is usually assumed to give a unique relationship between rainfall excess and surface runoff for a basin regardless of storm size, losses, or other factors. Because of its ease of use, the unit hydrograph has received the most attention by hydrologists.

2.5 COMPUTERIZED HYDROLOGIC ANALYSIS PACKAGES

There are various hydrologic analysis packages available to calculate the runoff from a watershed. HEC-1 is recommended, but if another hydrograph procedure is used approval of the method should be obtained from the City Engineer prior to the design or analysis. Several hydrologic programs are summarized below.

A. The Flood Hydrograph Package, HEC-1[17], was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC). HEC-1 is a compilation of several methods including various unit hydrograph procedures, loss rate functions, and channel and reservoir routing options. If a hydrograph procedure is required, other models may be used, but a HEC-1 analysis is generally recommended.

B. The Technical Release No. 20, "Computer Program for Project Formulation - Hydrology", TR-20[13] was originally developed by the USDA, Soil Conservation

Service (SCS) and has been modified by the SCS and other groups. TR-20 uses the procedures described in the SCS National Engineering Handbook, Section 4, Hydrology (NEH-4), except for the revised reach routing procedure (Att-Kin Method) which has superseded the Convex method.

C. Under the sponsorship of the U.S. Environmental Protection Agency (EPA), a comprehensive mathematical model capable of representing urban storm water runoff and combined sewer overflow phenomena was developed, named the Storm Water Management Model (SWMM)[6]. SWMM simulates the runoff of a drainage basin for any prescribed rainfall pattern. A total watershed is segmented into a number of smaller basins or subcatchments that can be readily described by its hydraulic or geometric properties. Manning's equation is used to route the excess uniform rainfall across overland surfaces, and through gutters, pipes and streams. The SWMM model simulates both water quantity and quality aspects which are associated with urban runoff and combined sewer systems.

D. The A&M Watershed Model[16] was developed to simulate a flood event caused by a rain storm. The model can be used to compute the discharge, water velocity, and water surface elevation in a stream during a flood, and was developed for the microcomputer. The model was designed to limit the number of technical options available to the user and at the same time provide a maximum number of data input options. The model has one loss rate function (curve number procedure), one unit hydrograph (two parameter gamma function), two stream routing methods (storage and hydraulic), one pipe routing procedure (hydraulic), one reservoir routing procedure (continuity equation storage-discharge method), and one method for water surface profile computation (standard step). The technical options were selected to minimize the requirement for historical streamflow data and field measurements and maximize the use of available data such as topographic maps and aerial photography.

2.6 HEC-1 MODEL

The Flood Hydrograph Package[17], HEC-1, model was developed and is supported by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC), Davis, California. The model is designed to simulate the surface runoff response of a drainage basin to a precipitation input. For purposes of this Manual, all references in the HEC-1 manual to a "river basin" have been changed to "drainage basin." The model represents the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin commonly referred to as a subbasin. A component may be a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component, and mathematical relationships which describe the physical processes. The result of the modeling is the computation of streamflow hydrographs at desired locations in the watershed basin.

The major components of the model are discussed briefly. It is recommended that the user solicit additional information in the Users Manual for HEC-1, Flood Hydrograph Package[17].

2.6.1 Stream Network Model Development

Using topographic maps and other geographic information, a drainage basin is subdivided into an interconnected system of stream network components, (see Figure 2-28). A basin schematic diagram, Figure 2-29, is developed by the following steps:

A. The study area watershed boundary is delineated with the aid of topographic maps. For urban basins, municipal drainage maps may also be necessary to obtain an accurate depiction of the basin's extent.

B. Segmentation of the basin into a number of subbasins determines the number and type of stream network components to be used in the model. Two factors have impact on the basin segmentation: the study purpose and the hydrometeorological variability throughout the basin. First, the study purpose defines the areas of interest in the basin, and hence, the points where subbasin boundaries should occur. Second, the variability of the hydrometeorological processes and basin characteristics determines the number and location of subbasins. Each subbasin is intended to represent an area of the watershed which, on the average, has the same hydraulic/hydrologic properties. Usually, the assumption of uniform precipitation and infiltration over a subbasin becomes less accurate as the subbasin size increases. If the subbasins are chosen appropriately, the average parameters used in the components will more accurately model the subbasin.

C. Each subbasin is represented by a combination of model components. Subbasin runoff, basin routing, reservoir, diversion and pump components are available.

D. The subbasins and their components are linked together to represent the topology of the drainage basin. HEC-1 has several methods for combining or linking together outflow from the various components.

2.6.2 Rainfall-Runoff Simulation

The HEC-1 model components are used to simulate the rainfall-runoff process as it occurs in an actual drainage basin. The model components are based upon mathematical relationships which are intended to represent individual meteorologic, hydrologic and hydraulic processes which comprise the precipitation-runoff process. These processes consist of precipitation, interception/infiltration, transformation of precipitation excess to subbasin outflow, addition to baseflow and flood hydrograph routing.

Hypothetical Frequency-Storm Derivation

Development of a hypothetical storm is derived from National Weather Service (NWS) rainfall data. The following steps are performed to develop the synthetic storm.

A. Establish the appropriate storm duration and the time interval for subdividing the storm rainfall;

B. Extract the rainfall values from Table 2-6 for the study area;

C. Make adjustments to the rainfall depth for size of drainage area, if needed;

D. Adjust for partial to annual series (if required);

E. Compute incremental rainfall amounts, and;

F. Arrange the storm rainfall increments in time.

Each of these steps is described in the following paragraphs. Steps E and F are optional in HEC-1. The individual may create a rainfall distribution, using a standard SCS curve distribution or another method of rainfall distribution. The method detailed here allows a "balanced" storm which produces a consistent depth-frequency relation for each duration of the total storm.

Storm Duration

Before constructing any hypothetical event, total duration and time interval for each rainfall increment must be estimated. Both parameters must reflect the type and size of the drainage area and the type and location of basin features one intends to analyze. The total duration of the hypothetical storm is directly related to the time of concentration of the watershed. The duration should be increased considerably if the total volume of runoff as well as peak discharge is of importance in the study. Drainage basins having an unusually large amount of flood plain storage may require a storm of longer-duration to capture the attenuation effect of large natural storage areas. Reservoir studies require long-duration events for full assessment of the reservoir flood storage needed.

Time Interval

The computation time interval must be small enough to accurately define the flood hydrograph (especially the peak); however, too small an interval will result in excess computations by the individual or the computer. The time interval will generally be established by the fastest peaking subarea of the overall basin model for which the peak discharge is required (i.e., for later use in developing water surface profiles, to evaluate the effects of a flood control component, etc.) The time interval must be small enough to define the rising limb and peak for the hydrograph for this subarea.

Precipitation

Precipitation input to HEC-1 is in terms of hyetographs (rainfall-intensity-duration pattern), historical storms or synthetic storms. The hyetograph pattern is assumed to be uniformly distributed over the subbasin. Precipitation for an observed or historical storm event can be supplied to the program by a basin-average precipitation method or weighted precipitation gage method. The synthetic storm is frequently used in planning and design studies and is typically generated from depth-duration data. A synthetic storm of any duration from 5 minutes to 10 days can be generated based on given depth-duration data. A triangular precipitation distribution is constructed such that the depth specified for any duration occurs during the central part of the storm. This is referred to as a "balanced storm."

Precipitation data can be entered on the PH card in HEC-1. Table 2-6 provides the depth-duration frequency data for the City of Amarillo.

TABLE 2-6 Depth-Duration-Frequency Data for Amarillo, Texas

Duration (Min)	Rainfall Depth (inches)					
	Frequency					
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
5	0.45	0.54	0.60	0.69	0.77	0.84
10	0.69	0.86	0.97	1.14	1.28	1.41
15	0.85	1.08	1.23	1.46	1.64	1.81
30	1.18	1.53	1.77	2.12	2.38	2.65
60	1.52	2.00	2.33	2.80	3.16	3.52
120	1.65	2.15	2.48	3.00	3.37	3.80
180	1.80	2.30	2.65	3.20	3.63	4.01
360	2.15	2.75	3.15	3.65	4.20	4.70
720	2.40	3.10	3.55	4.05	4.70	5.20
1,440	2.72	3.45	3.96	4.49	5.23	5.79

Areal Adjustment

The rainfall data listed in Table 2-6 are "point rainfall depths"; that is, as measured at a rain gage, i.e., a single point. The hypothetical storm will apply to a specific watershed having a defined drainage area and an areal adjustment may be necessary. An adjustment is often not made unless the study area is more than 5 to 10 square miles. When the drainage area is larger than about 10 square miles, the adjustment becomes significant, particularly for the 30- and 60-minute durations. Figure 2-30 is used to adjust the rainfall for area. The areal adjustment factor is automatically accounted for on the PH card.

2.6.3 Interception and Infiltration

Land surface interception, depression storage and infiltration are referred to in HEC-1 as precipitation losses. Interception and depression storage include the surface storage of water by trees or grass, local depressions in the ground surface, or in water areas where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface.

Two important factors should be noted about the precipitation loss computation in HEC-1. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. This makes the HEC-1 model a single event-oriented model.

The precipitation loss computations can be used with the unit hydrograph model components. In the case of the unit hydrograph component, the precipitation loss is considered to be a subbasin average (uniformly distributed over an entire subbasin).

In HEC-1, the precipitation loss can be calculated by one of four methods. Using any of these methods, an average precipitation loss is determined for a computational interval and subtracted from the rainfall/snowmelt hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a subbasin. The four methods used to calculate precipitation losses are: 1) initial and constant loss rate; 2) exponential loss rate; 3) SCS Runoff Curve Number; and 4) Holtan loss rate. It is recommended to use the SCS Runoff Curve Number method for calculation of interception and infiltration losses.

2.6.4 Lag Time and Concentration Time

Two basic equations are used in defining the shape of the unit hydrograph. The first equation defines the lag time of the basin, L, or the time from the midpoint of unit excess rainfall to the unit hydrograph peak, t_p. A number of relationships are given for the dimensions of the SCS synthetic unit hydrograph (Figure 2-31). The lag time is related to the time of concentration by the equation:

$$L = 0.6\, t_c \tag{2-4}$$

where:

L	=	lag time, in hours
t_c	=	time of concentration, in hours

2.6.5 Basin Routing

A basin routing, referred to as "river routing" in the HEC-1 manual, component, such as element 1020, Figure 2-28, is used to represent flood wave movement in a channel. The input to the component is an upstream hydrograph, which may have resulted from an individual or a combined contribution of subbasin runoff, basin routing or diversions. The hydrograph is routed to a downstream point based on the characteristics of the channel. The hydrograph routed through channel reaches and reservoirs is based on the continuity equation and some relationship between discharge and storage. These methods are: 1) Muskingum; 2) Kinematic Wave; 3) Modified Puls; 4) Working R and D; and 5) Level Pool Reservoir Routing.

With reference to Figure 2-28, the runoff from subbasin 10 is calculated and routed to control point 20 via routing reach 1020. Runoff from subbasin 20 is calculated and combined with the outflow hydrograph at control point 20. However, if runoff from subbasin 20 is concentrated near the upstream end of reach 1020, runoff from subbasin 10 and 20 can be combined prior to routing through reach 1020.

A suitable combination of subbasin runoff and basin routing components can be used to represent the topology of a drainage basin. The connectivity of the stream network components is determined by the order in which the data components are arranged. Simulation must always begin at the uppermost subbasin in a branch of the stream network. The simulation proceeds downstream until a confluence is reached. Before simulating below the confluence, all flows above that confluence must be computed and routed to that confluence. The flows are combined at the confluence and the combined flows are routed downstream.

2.6.6 Assumptions and Limitations of HEC-1

A drainage basin is represented as an interconnected group of subbasins. The assumption is made that the hydrologic processes can be represented by model parameters which reflect average conditions within a subbasin. If such averages are inappropriate for a subbasin, it is necessary to consider smaller subbasins within which average parameters apply. Model parameters represent spatial and temporal averages. Thus the time interval to be used should be small enough so that temporal averages are adequate.

There are several limitations of the HEC-1 model. HEC-1 simulations are limited to a single storm event, due to the fact that provision is not made for soil moisture recovery. Streamflow routings are performed by hydrologic routing methods and do not reflect dynamic effects applicable to very flat basin slopes, as encountered in the City of Amarillo.

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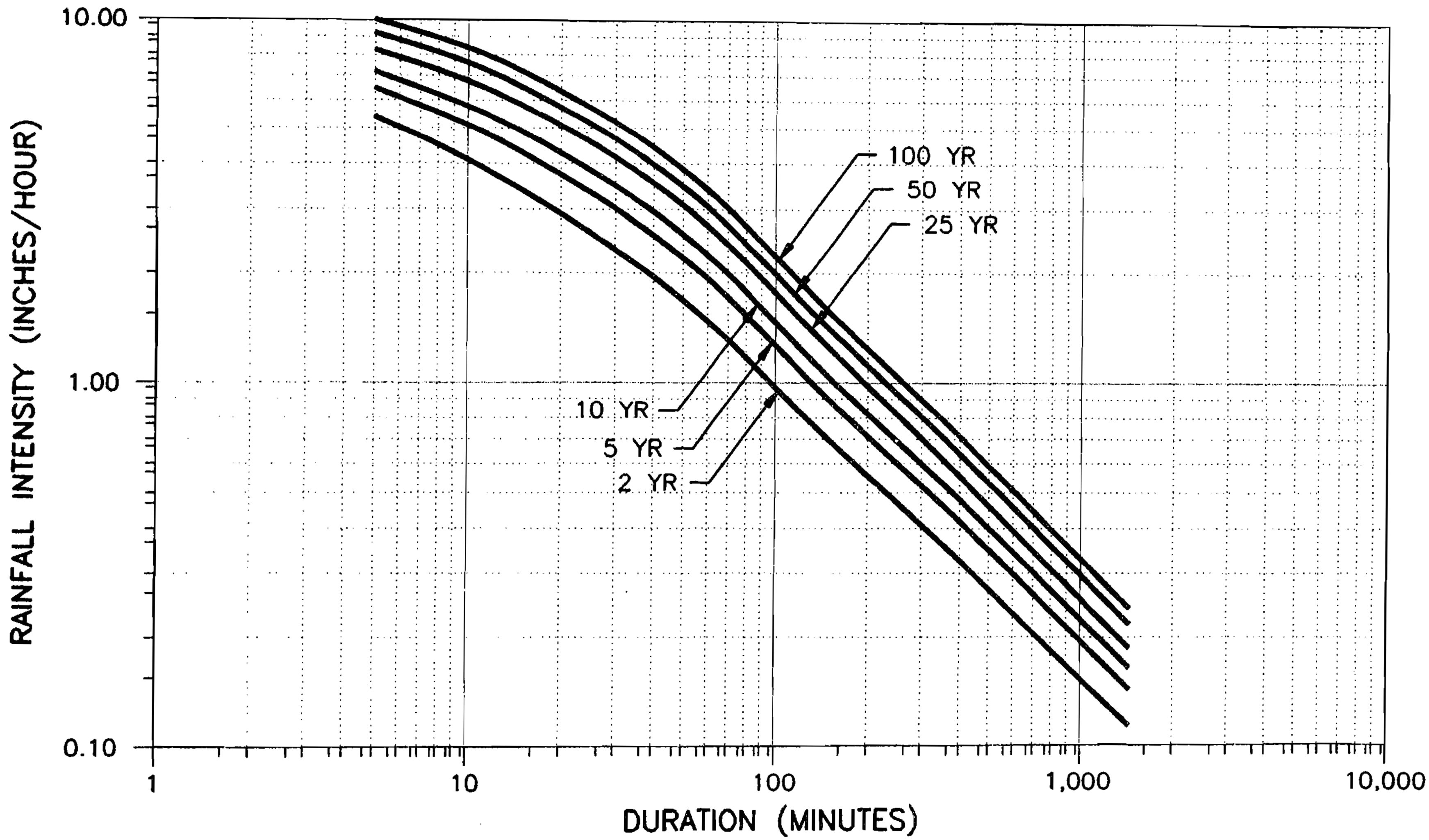
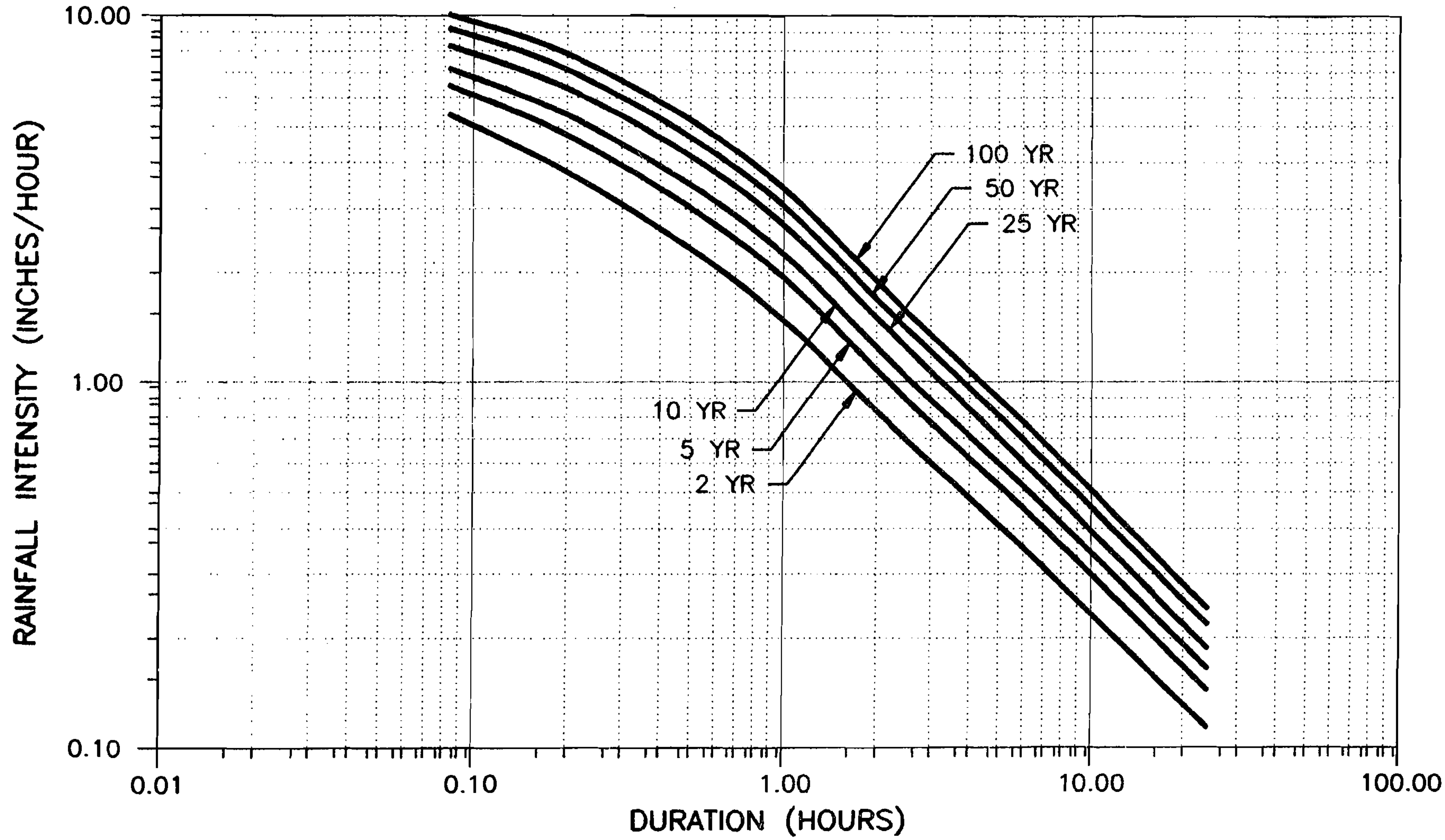


Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

Intensity Duration Frequency Curves **Duration in Minutes** **Amarillo, Texas**	Date NOV 1992
Storm Water Management Criteria Manual	Figure 2-1



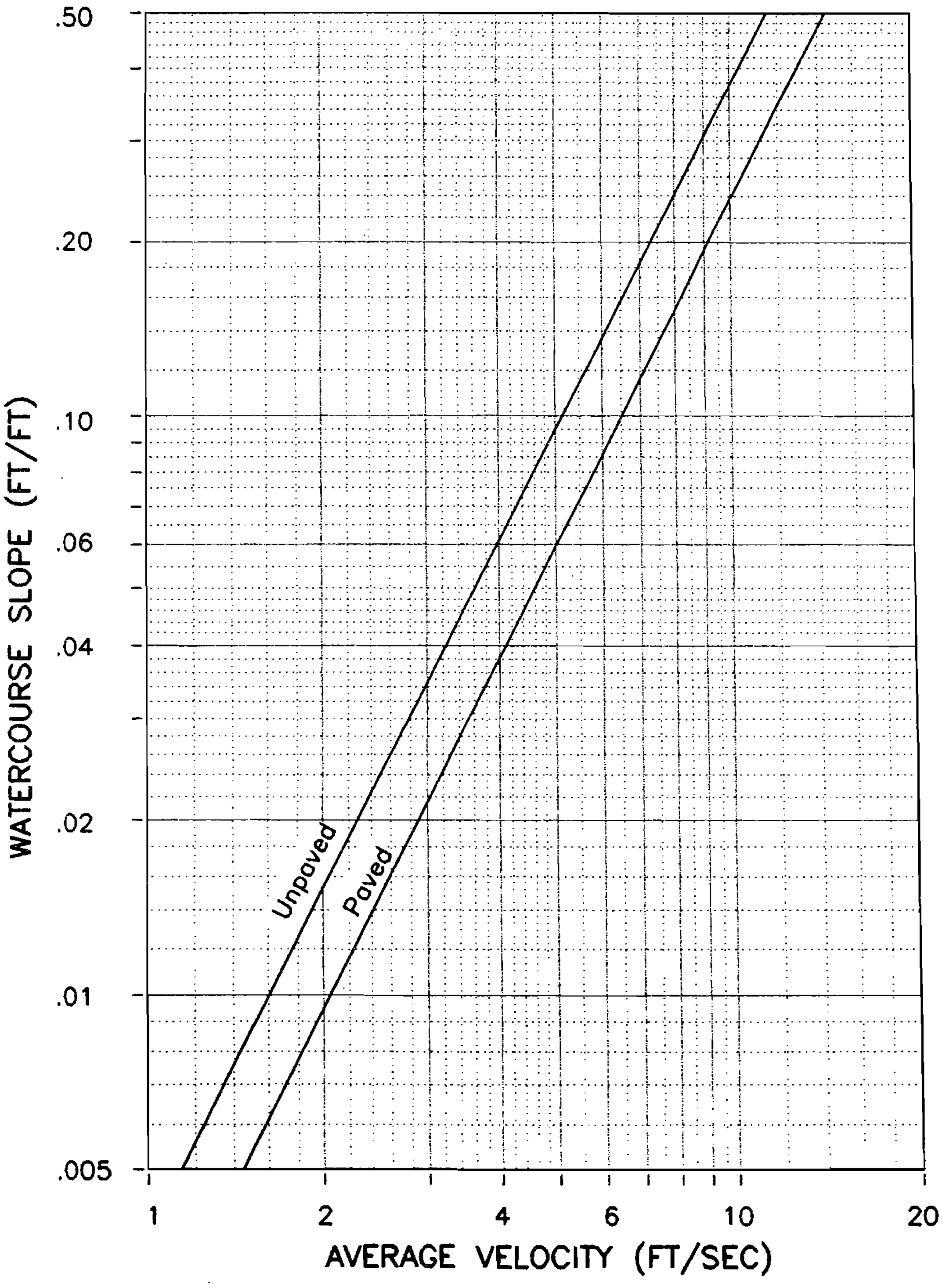
Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

Intensity Duration Frequency Curves
Duration in Hours
Amarillo, Texas

S m Water Management Criteria Manual

Date	NOV 1992
Figure	2-2



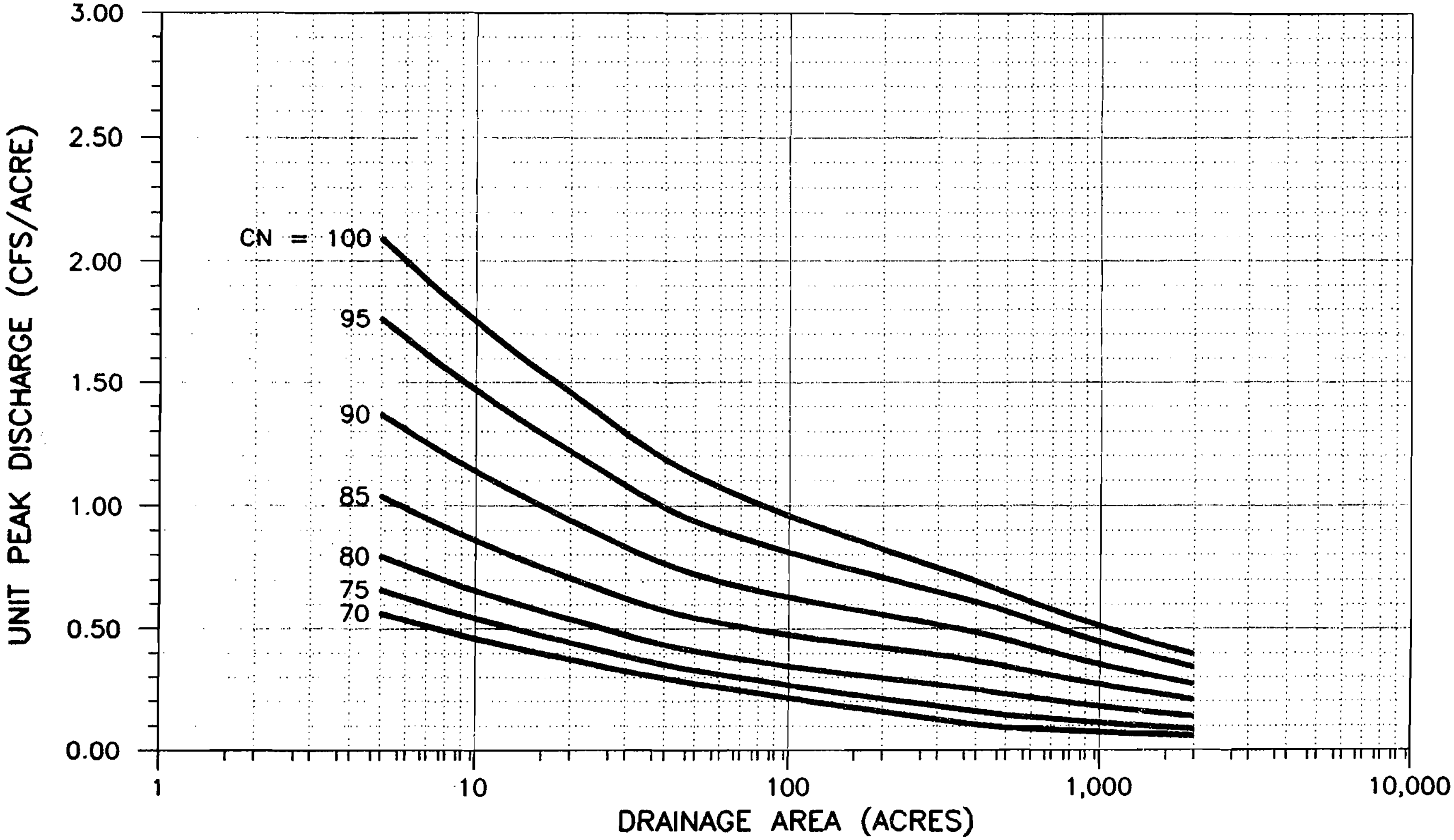
Source: Soil Conservation Service, TR-55, Urban Hydrology for Small Watersheds.

CITY OF AMARILLO

Average Velocities for Travel Time for Shallow Concentrated Flow

Date NOV 1992

Figure 2-3



Source: HDR Engineering, Inc., _Amarillo Storm Water Management Master Plan_.

CITY OF AMARILLO

2-Year Peak Flow Curves (Average Basin Slope = 0% - 0.5%) Amarillo, Texas

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Date	NOV 1992
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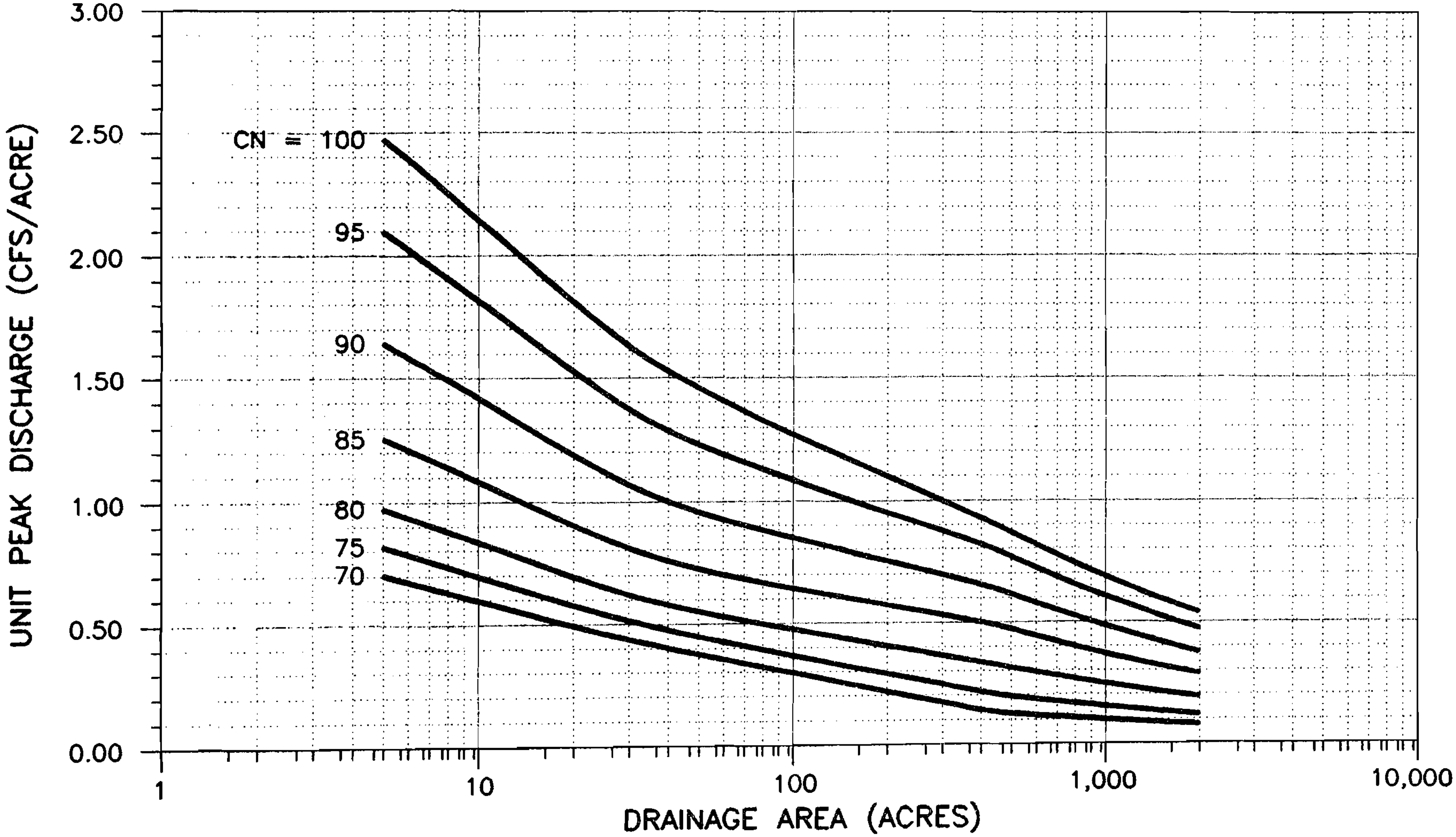


Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

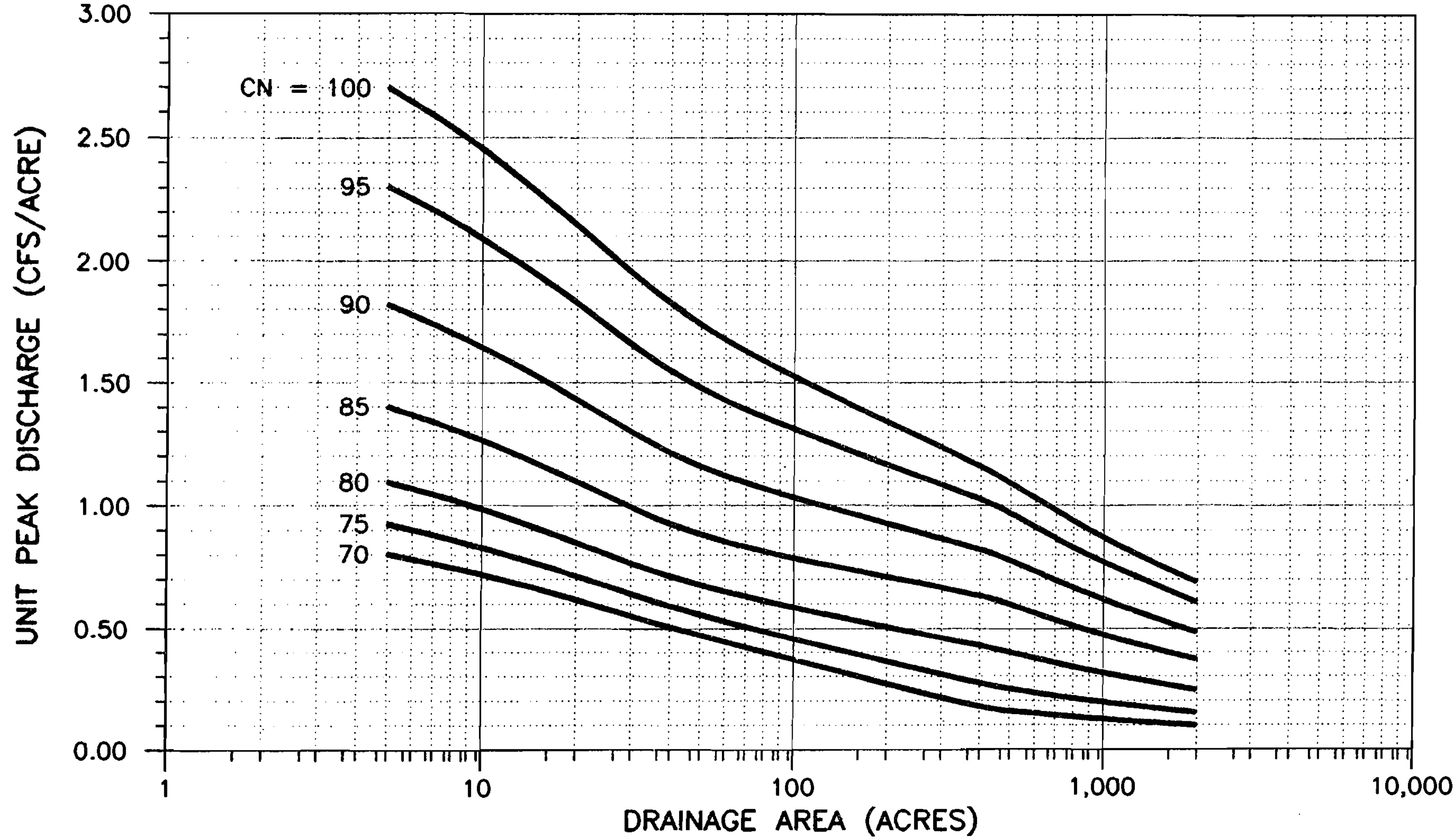
CITY OF AMARILLO

2-Year Peak Flow Curves
(Average Basin Slope = 0.5% - 1%)
Amarillo, Texas

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 2-5



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

2-Year Peak FLow Curves (Average Basin Slope = 1% - 2%) Amarillo, Texas

Date: NOV 1992

Figure: 2-6

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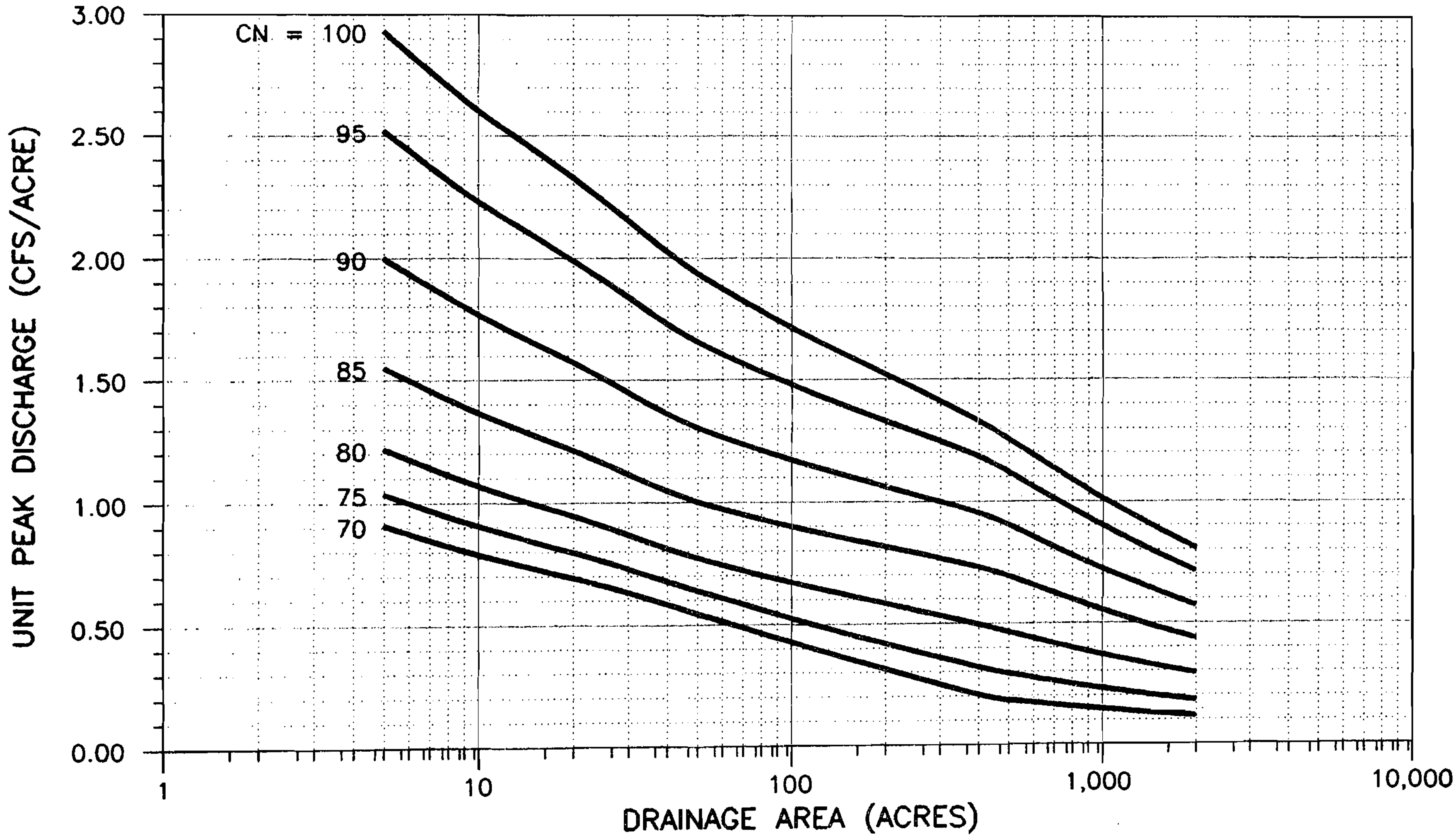


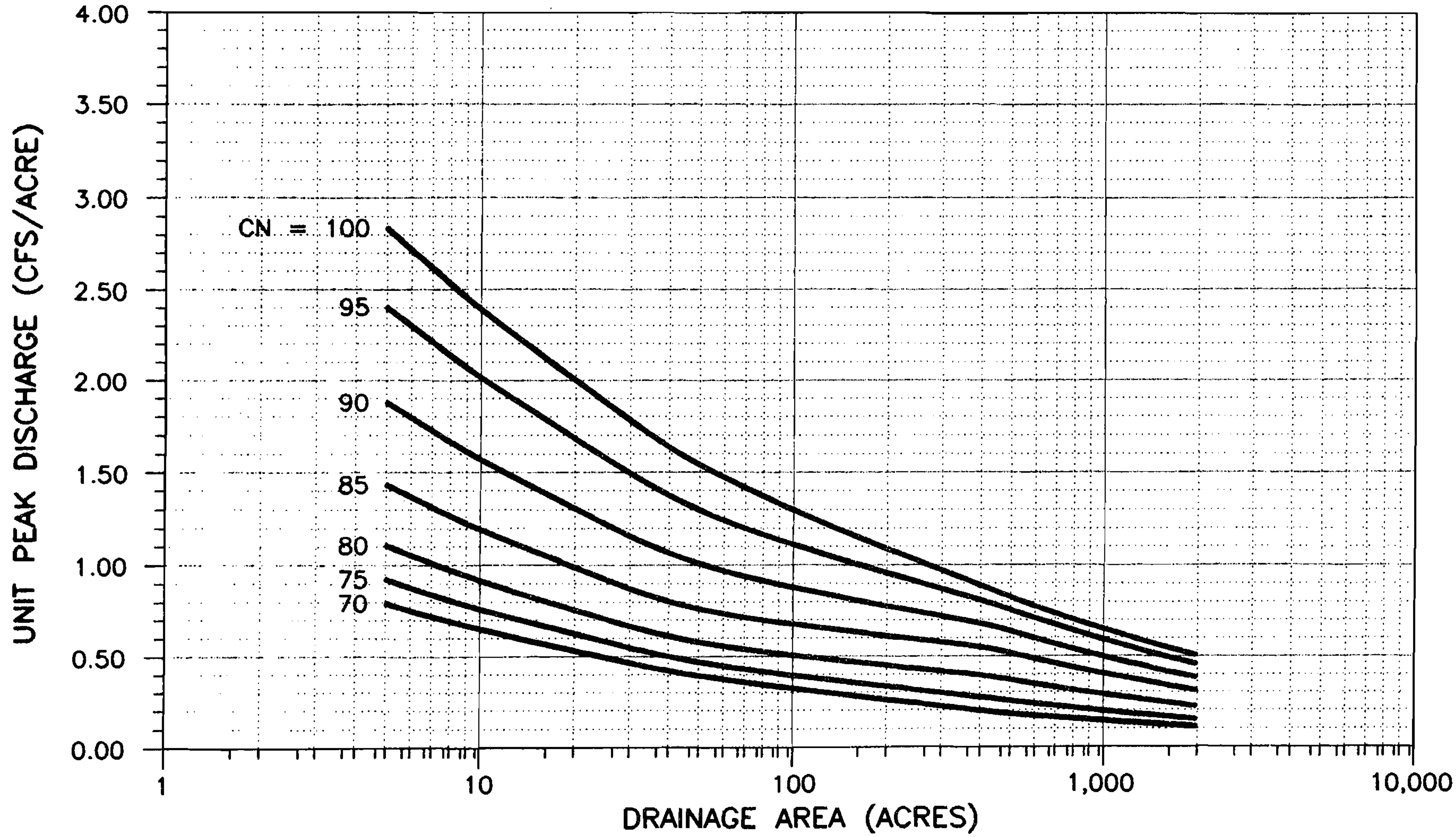
Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

2-Year Peak Flow Curves
(Average Basin Slope = 2% - 3%)
Amarillo, Texas

Storm Water Management Criteria Manual

Date	NOV 1992
Figure	2-7



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

5-Year Peak Flow Curves (Average Basin Slope = 0% - 0.5%) Amarillo, Texas

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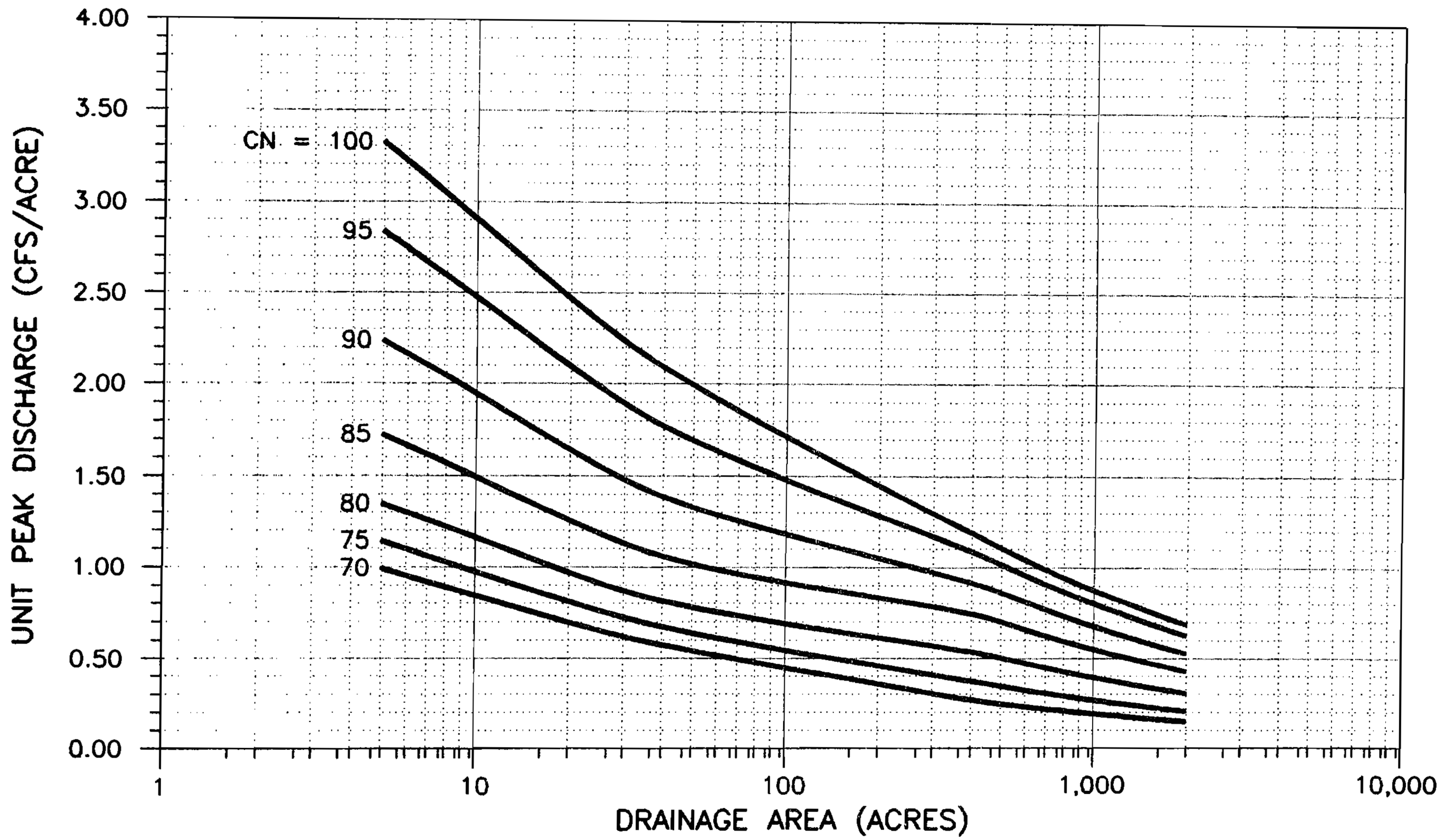
Date NOV 1992

Figure 2-8



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

5-Year Peak Flow Curves (Average Basin Slope = 0.5% - 1%) Amarillo, Texas

Storm Water Management Criteria Manual

Date	NOV 1992
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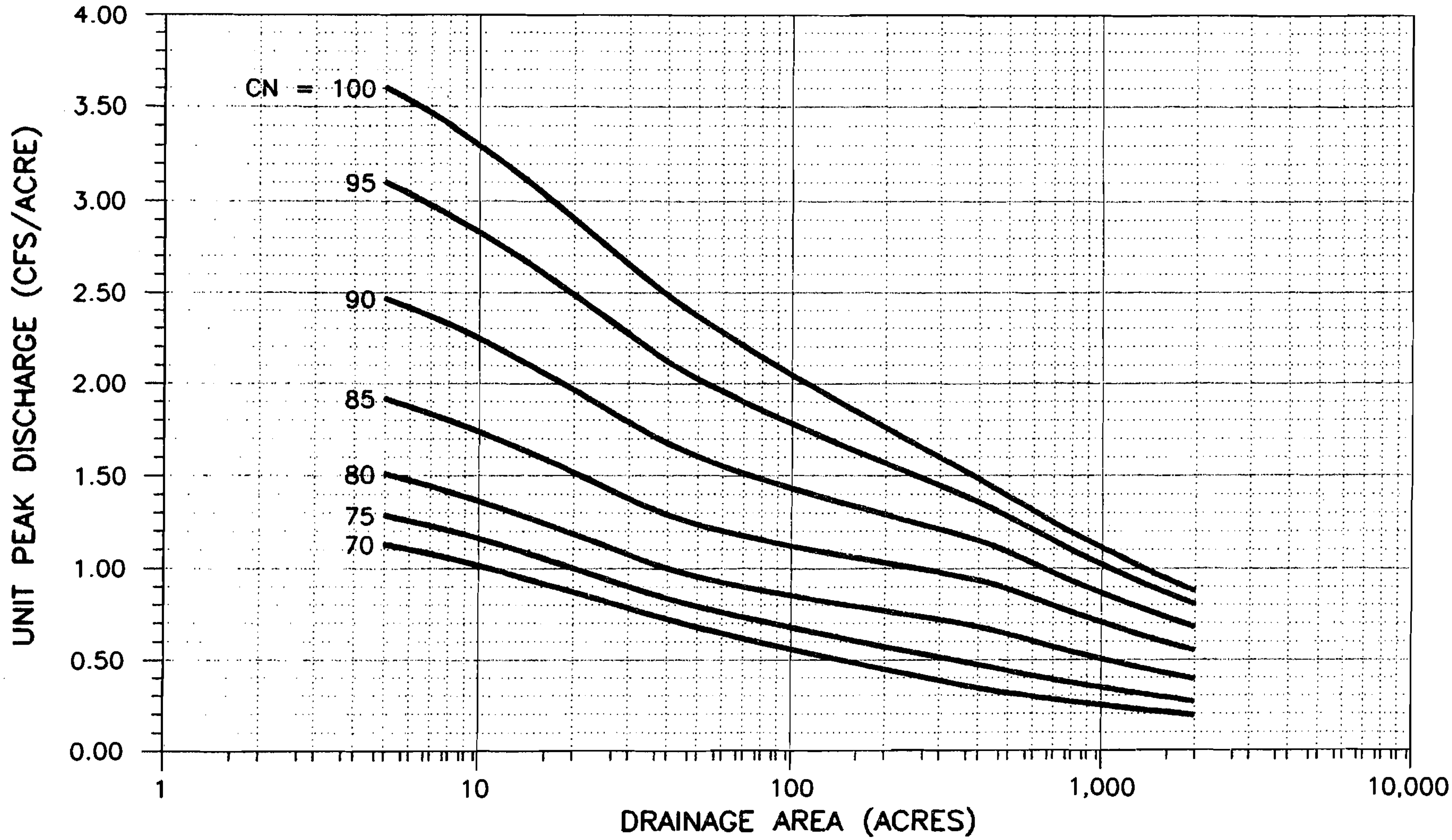


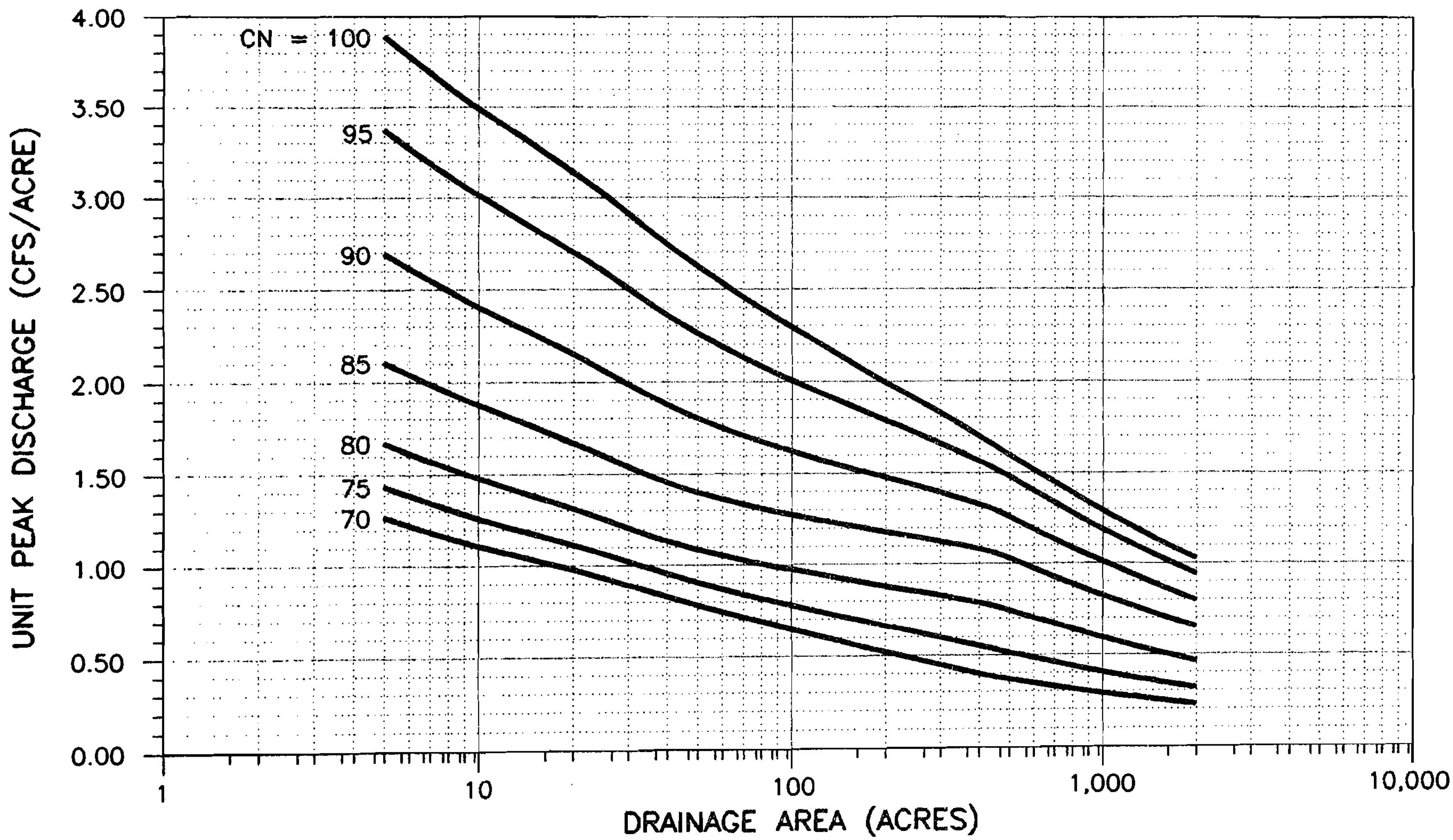
Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

5-Year Peak Flow Curves
(Average Basin Flow = 1% - 2%)
Amarillo, Texas

S m Water Management Criteria Manual

Date	NOV 1992
Figure	2–10



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

5-Year Peak Flow Curves
(Average Basin Slope = 2% - 3%)
Amarillo, Texas

Storm Water Management Criteria Manual

Date
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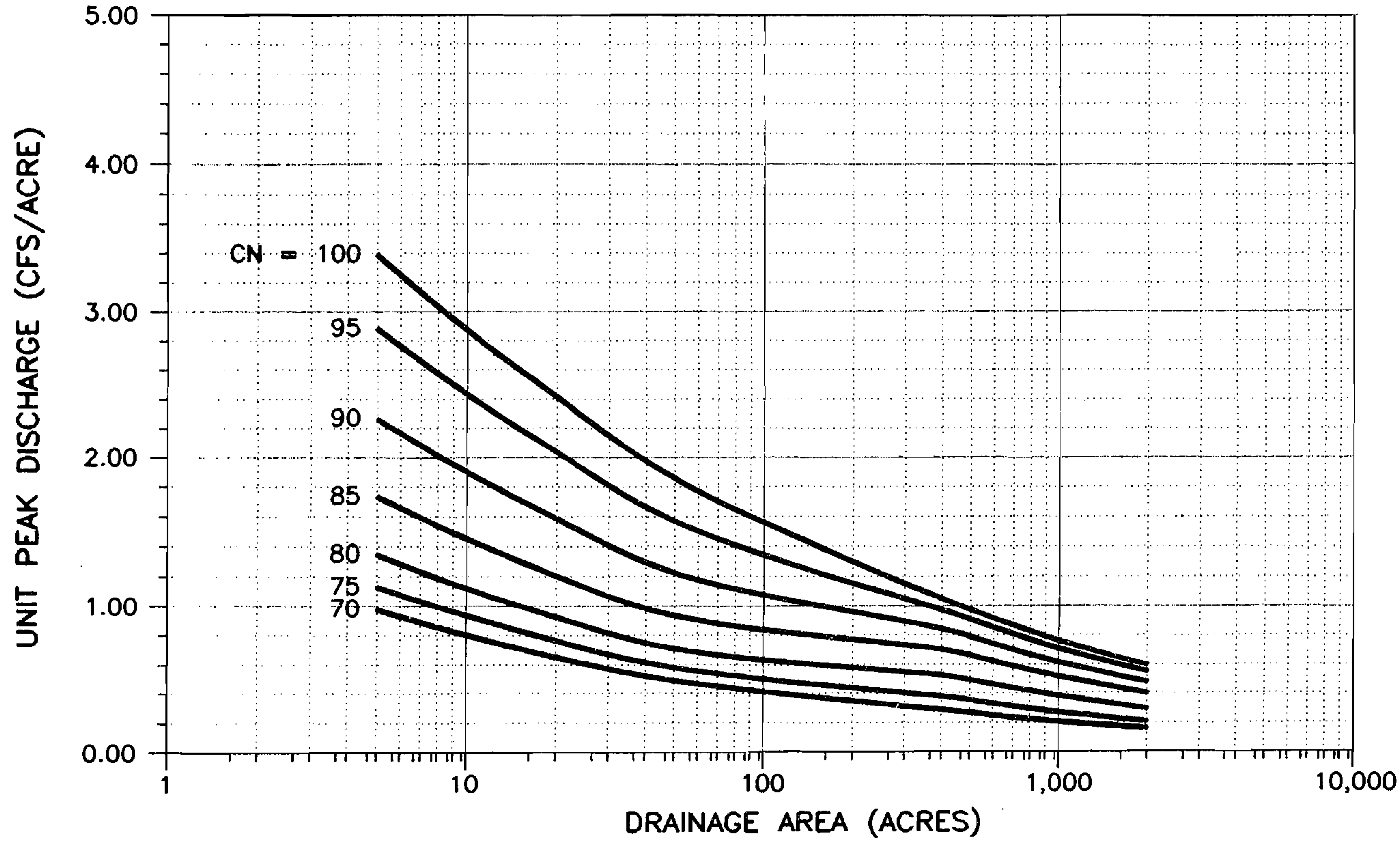
Figure
2-11



Source: HDR Engineering, Inc., *Amarillo Storm Water Management Master Plan*.

CITY OF AMARILLO

10-Year Peak Flow Curves (Average Basin Slope = 0% - 0.5%) Amarillo, Texas

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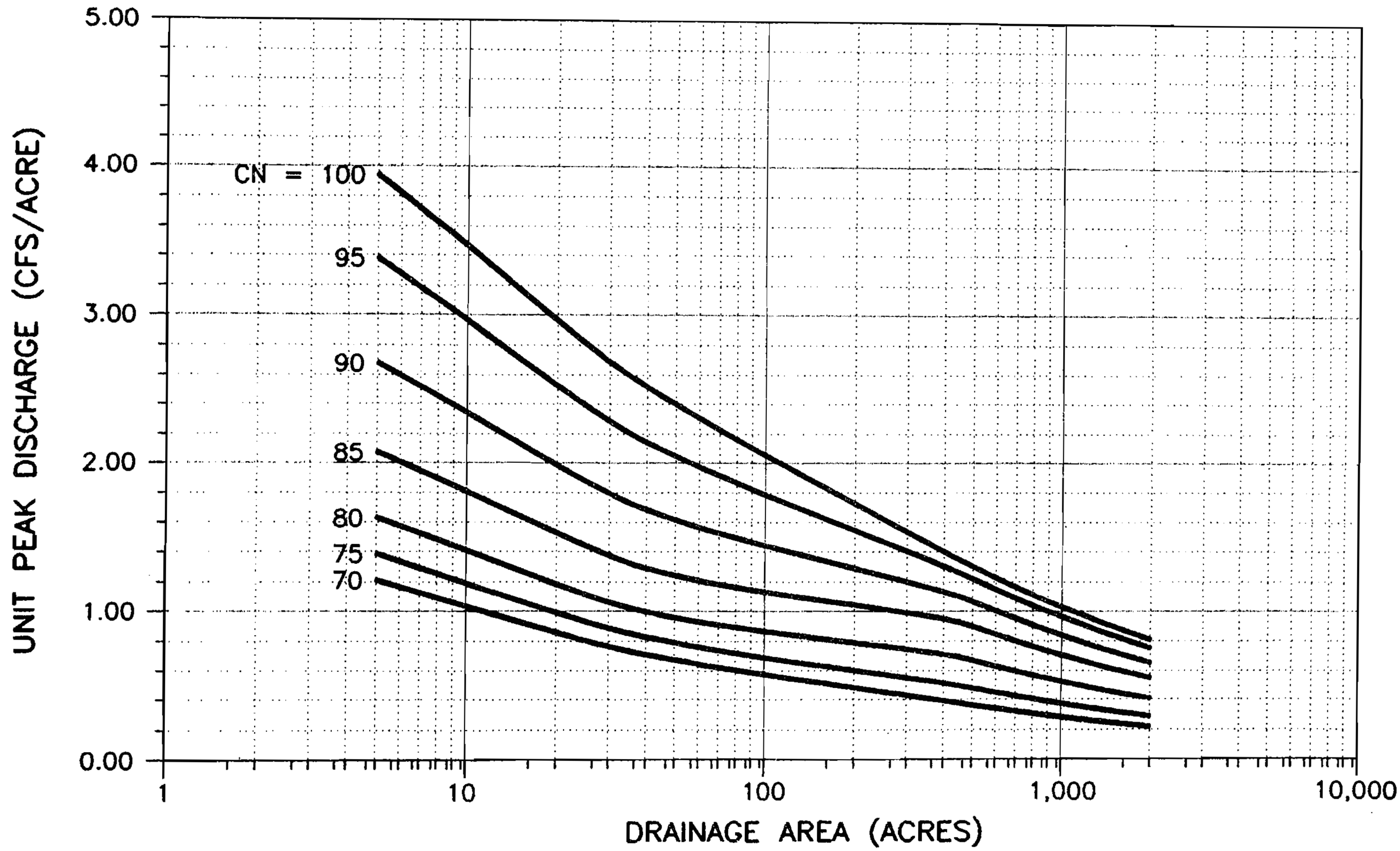
Figure 2–12



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

10-Year Peak Flow Curves (Average Basin Slope = 0.5% - 1%) Amarillo, Texas

Storm Water Management Criteria Manual

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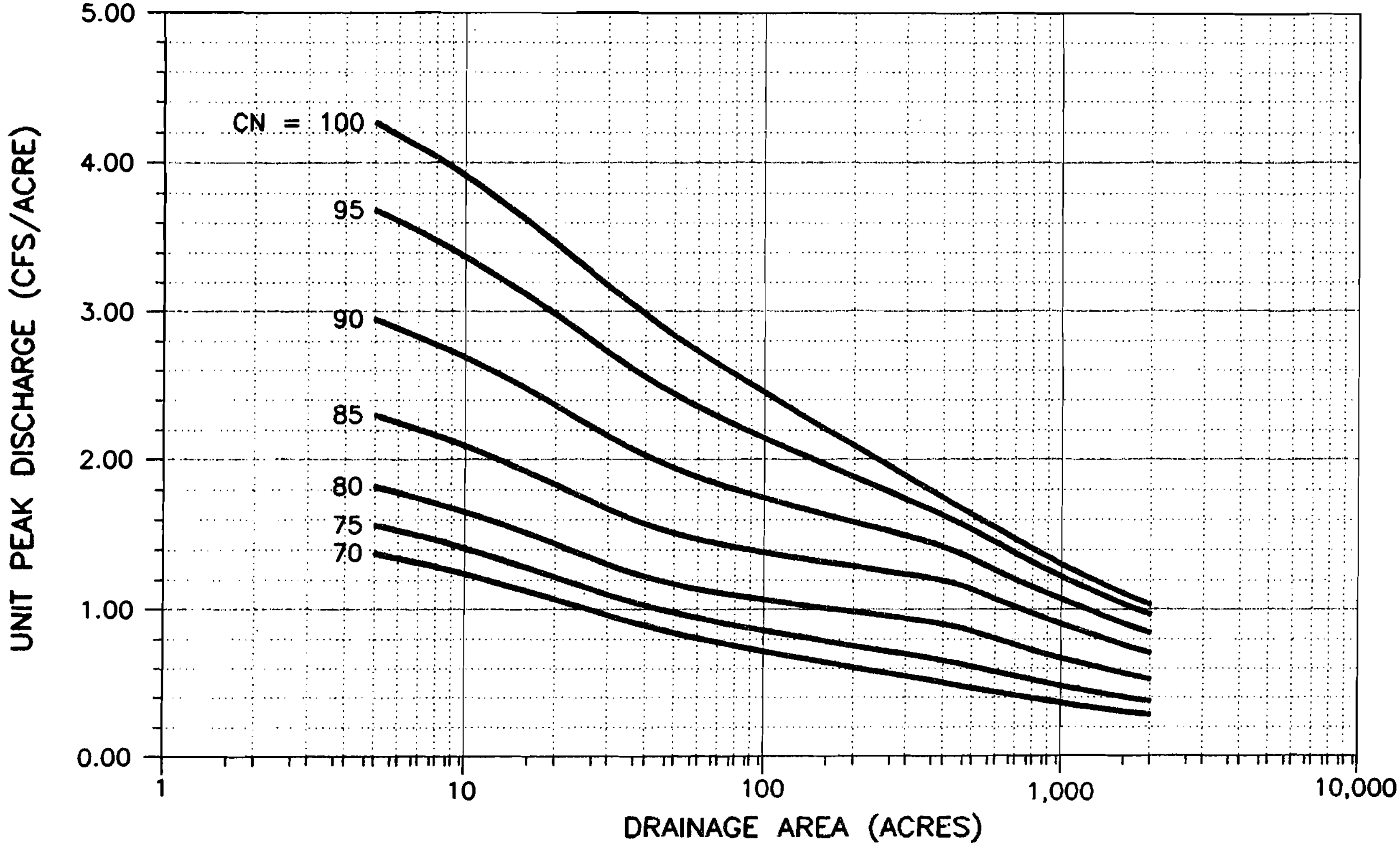
Figure 2-13



Source: HDr Engineering, Inc., _Amarillo Storm Water Management Master Plan_.

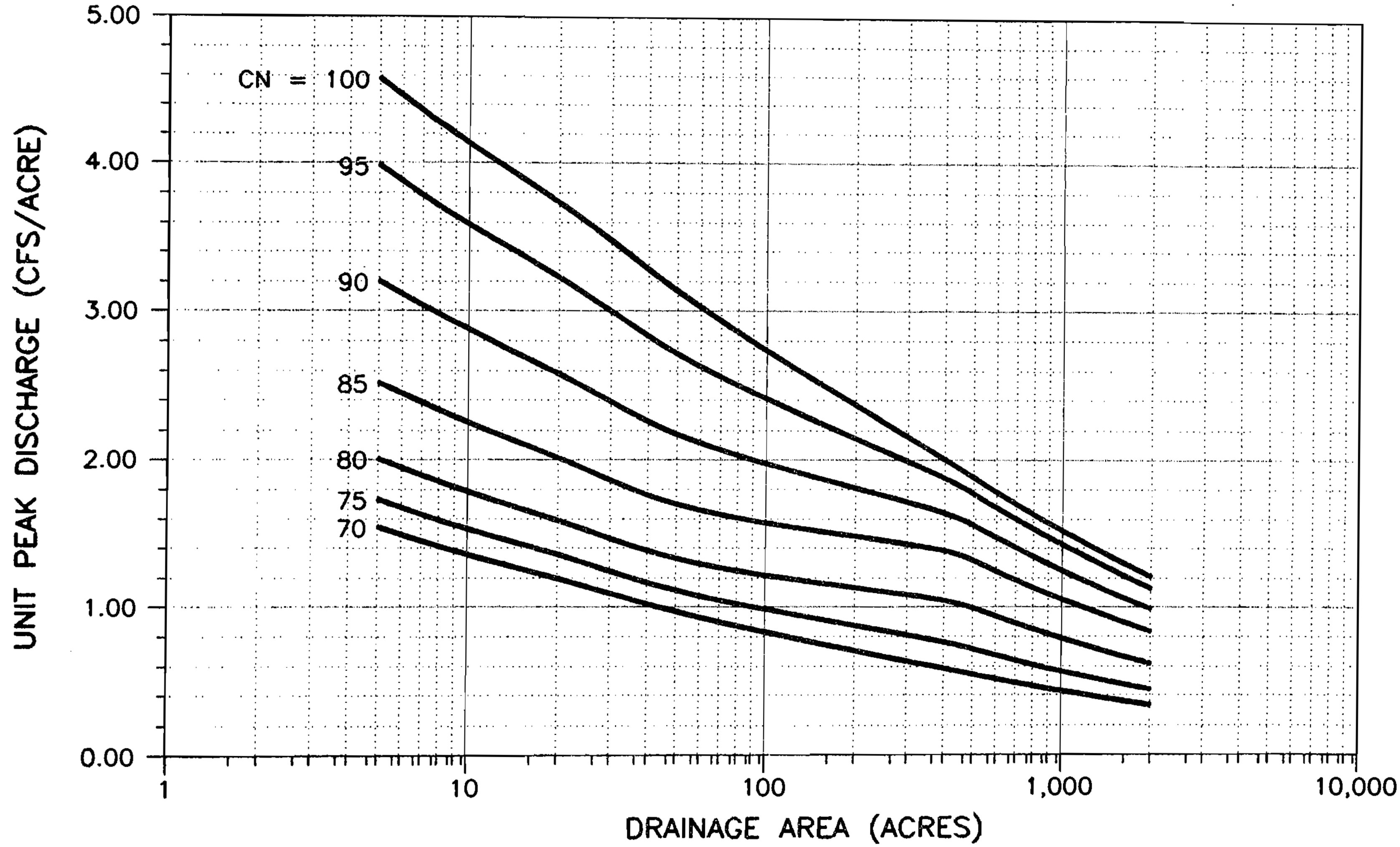
CITY OF AMARILLO

10-Year Peak Flow Curves (Average Basin Slope = 1% - 2%) Amarillo, Texas

St n Water Management Criteria Manual

Date NOV 1992

Figure 2-14



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

10-Year Peak Flow Curves (Average Basin Slope = 2% - 3%) Amarillo, Texas

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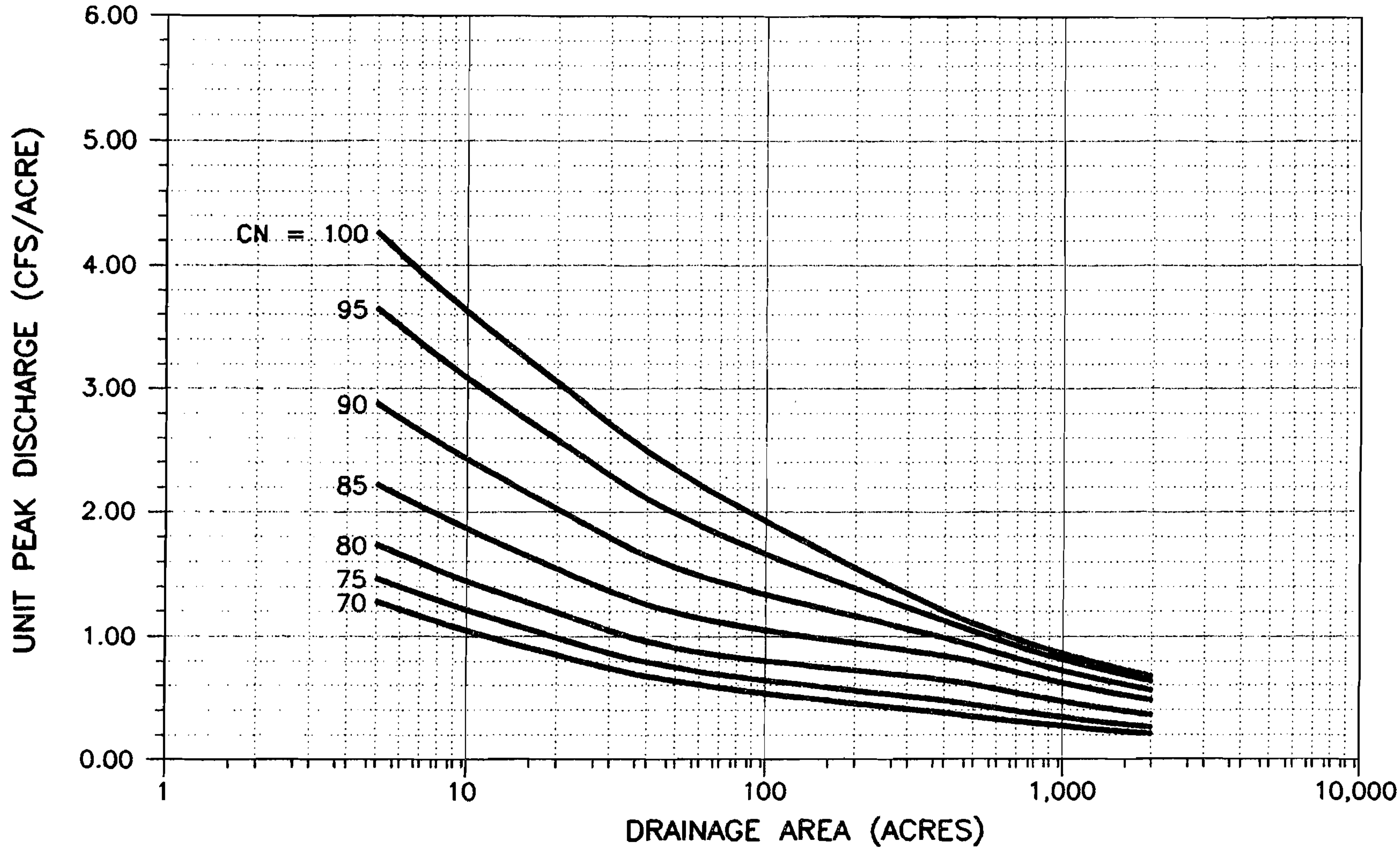

Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

25-Year Peak Flow Curves (Average Basin Slope = 0% - 0.5%) Amarillo, Texas

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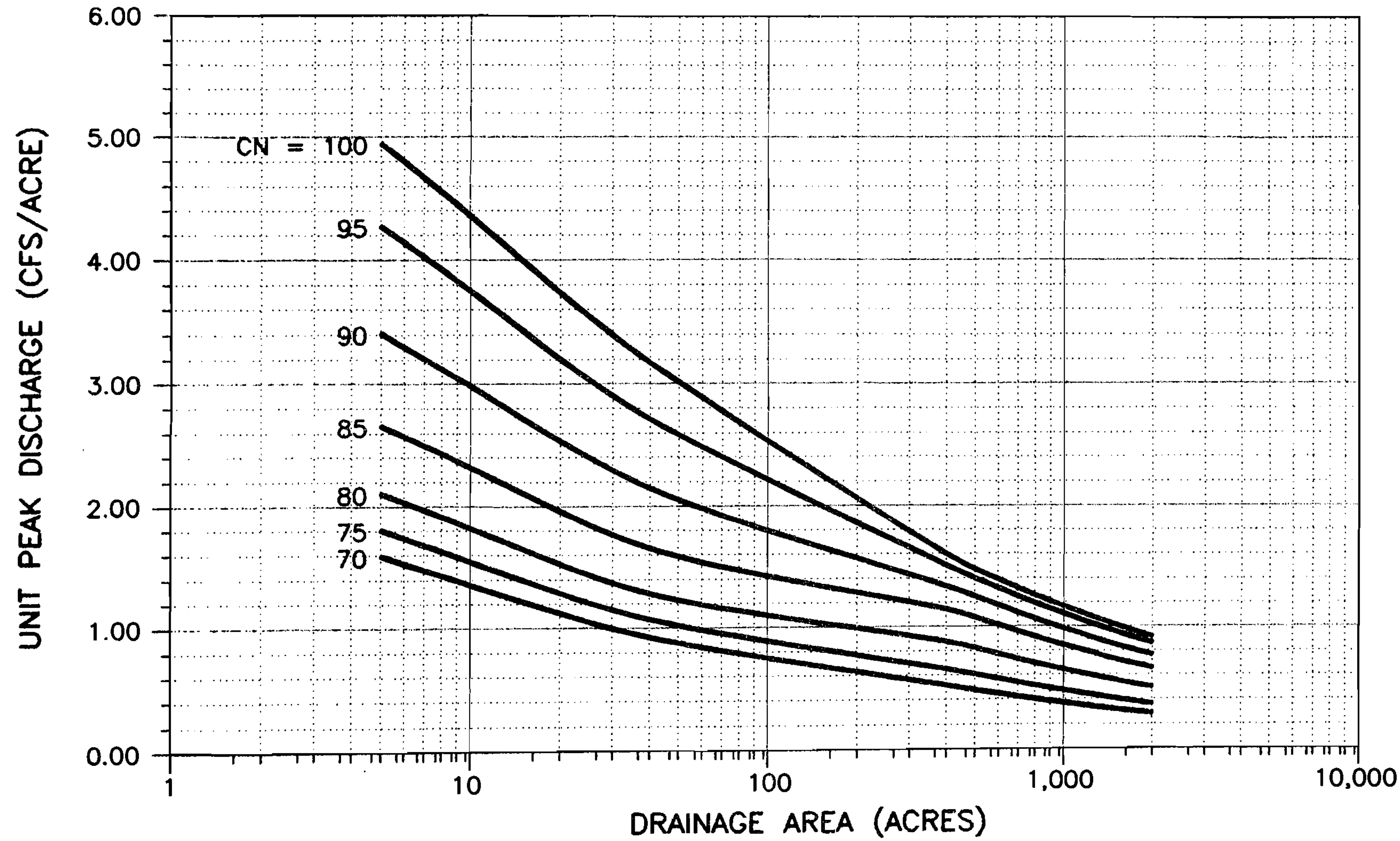
Date NOV 1992

Figure 2-16



Source: HDR Engineering, Inc., *Amarillo Storm Water Management Master Plan*.

CITY OF AMARILLO

25-Year Peak Flow Curves (Average Basin Slope = 0.5% - 1%) Amarillo, Texas

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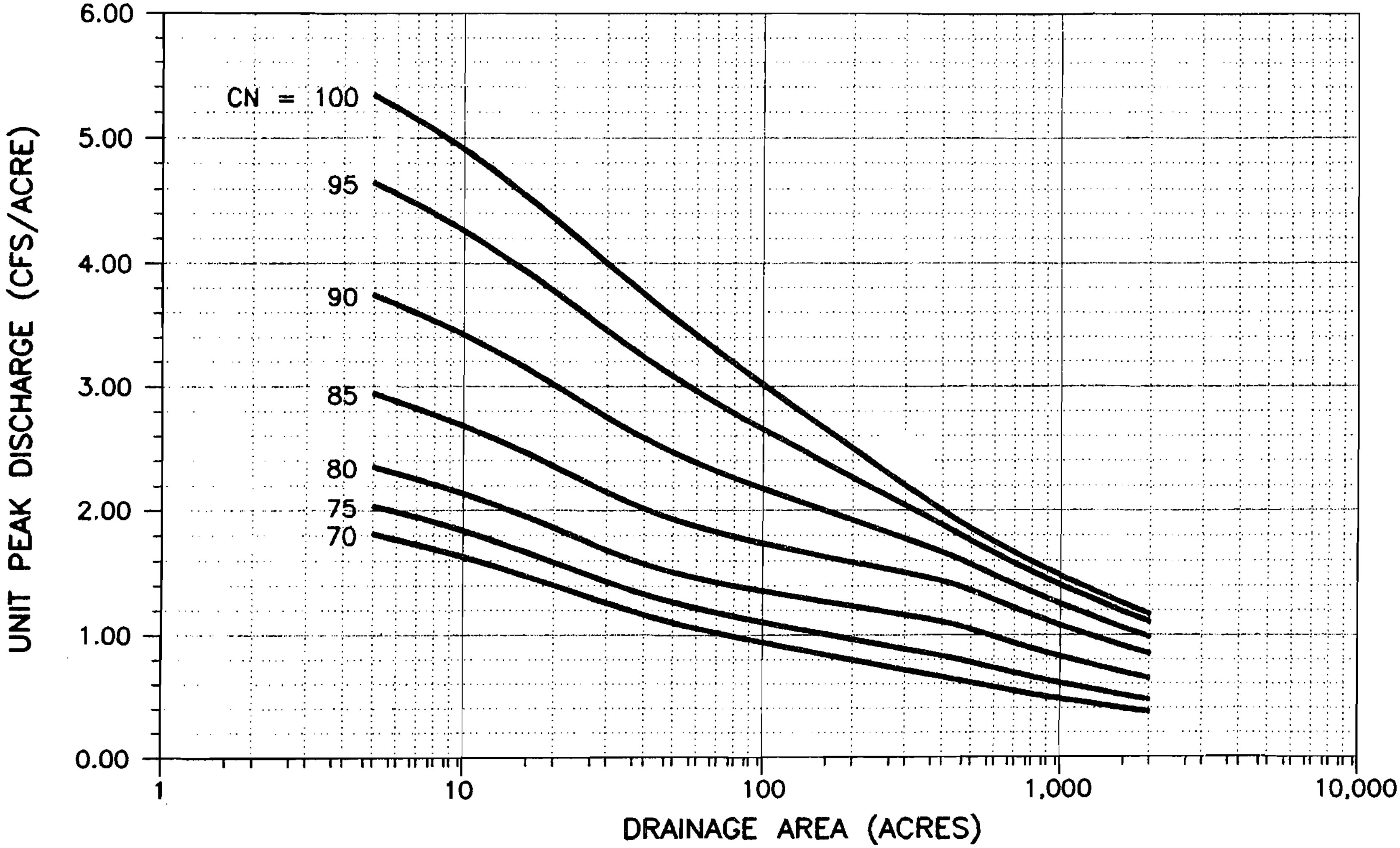

Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

25-Year Peak Flow Curves (Average Basin Slope = 1% - 2%) Amarillo, Texas

St n Water Management Criteria Manual

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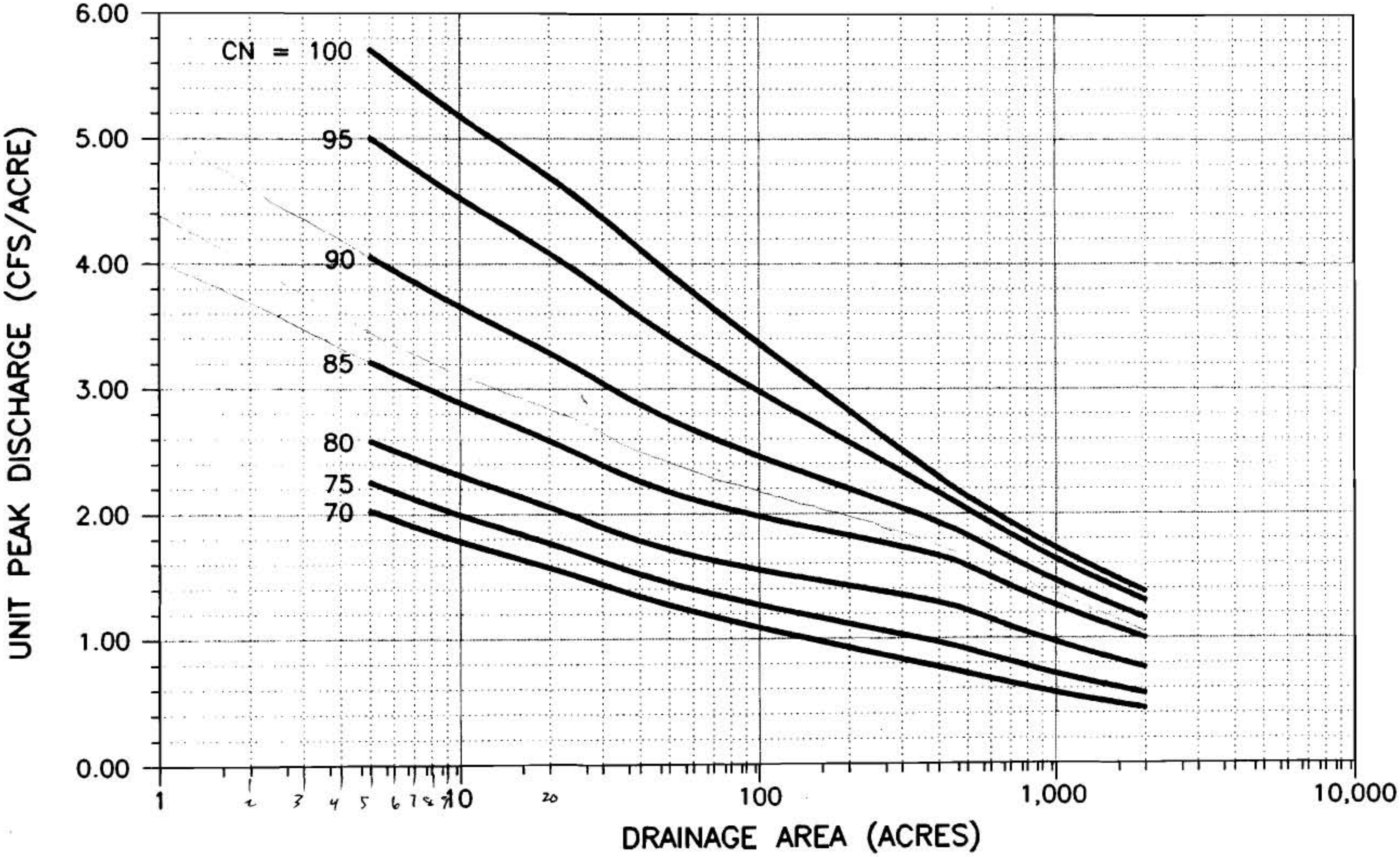
Figure: 2-18



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

25-Year Peak Flow Curves (Average Basin Slope = 2% - 3%) Amarillo, Texas

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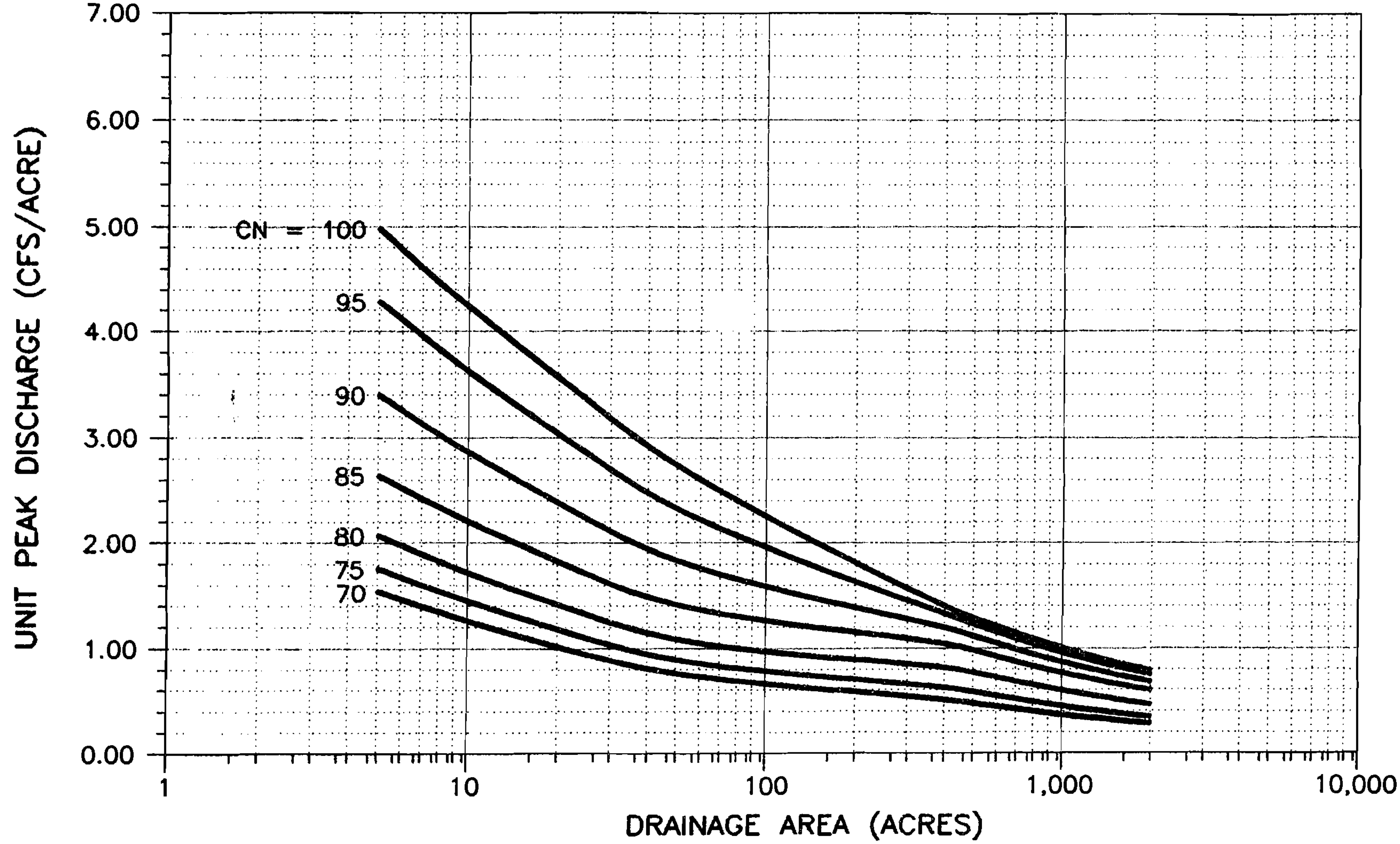
Figure 2–19



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

50-Year Peak Flow Curves (Average Basin Slope = 0% - 0.5%) Amarillo, Texas	Date NOV 1992
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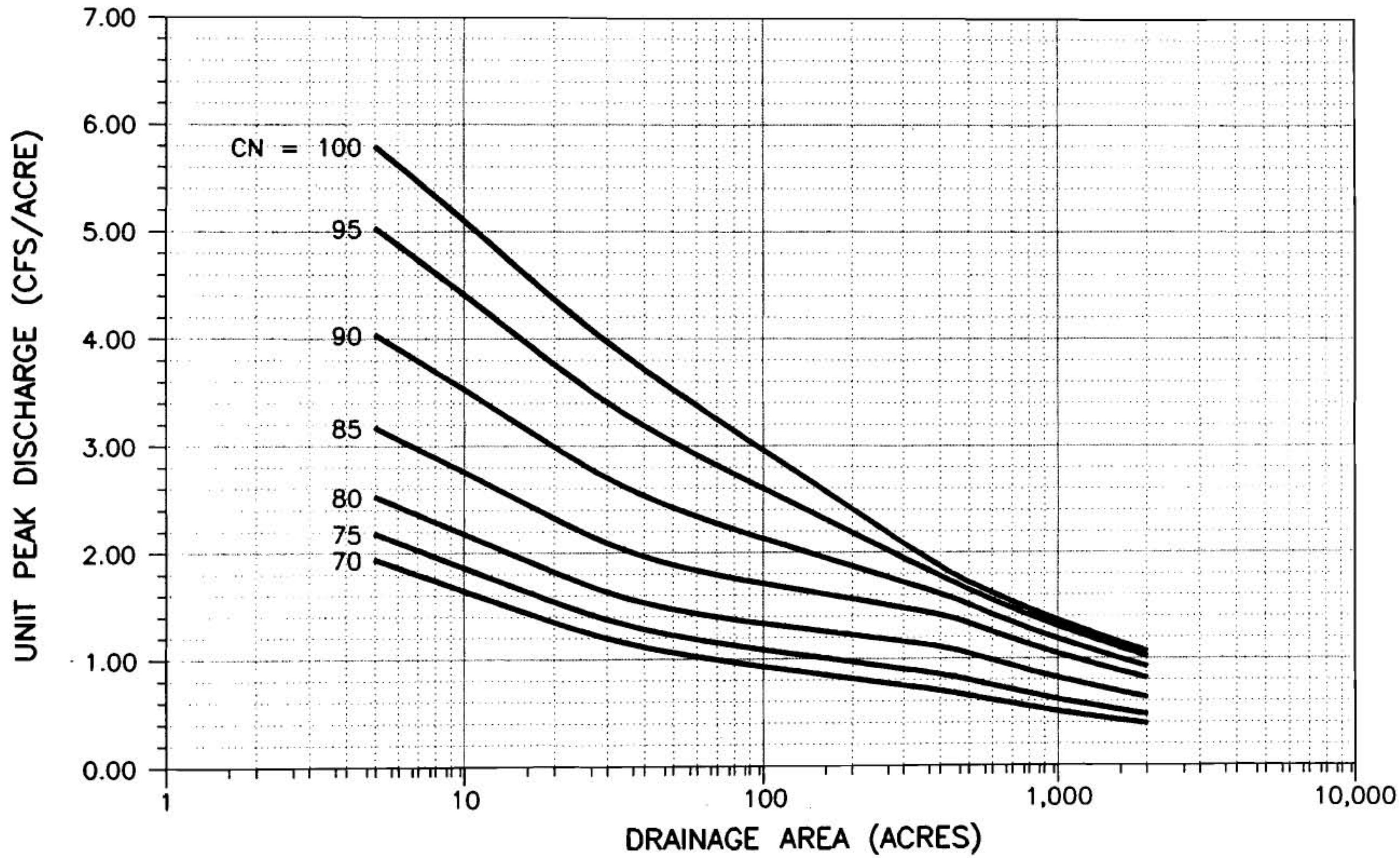
CITY OF AMARILLO



Source: HDR Engineering, Inc., _Amarillo Storm Water Management Master Plan_.

CITY OF AMARILLO

50-Year Peak Flow Curves (Average Basin Slope = 0.5% - 1%) Amarillo, Texas

Storm Water Management Criteria Manual

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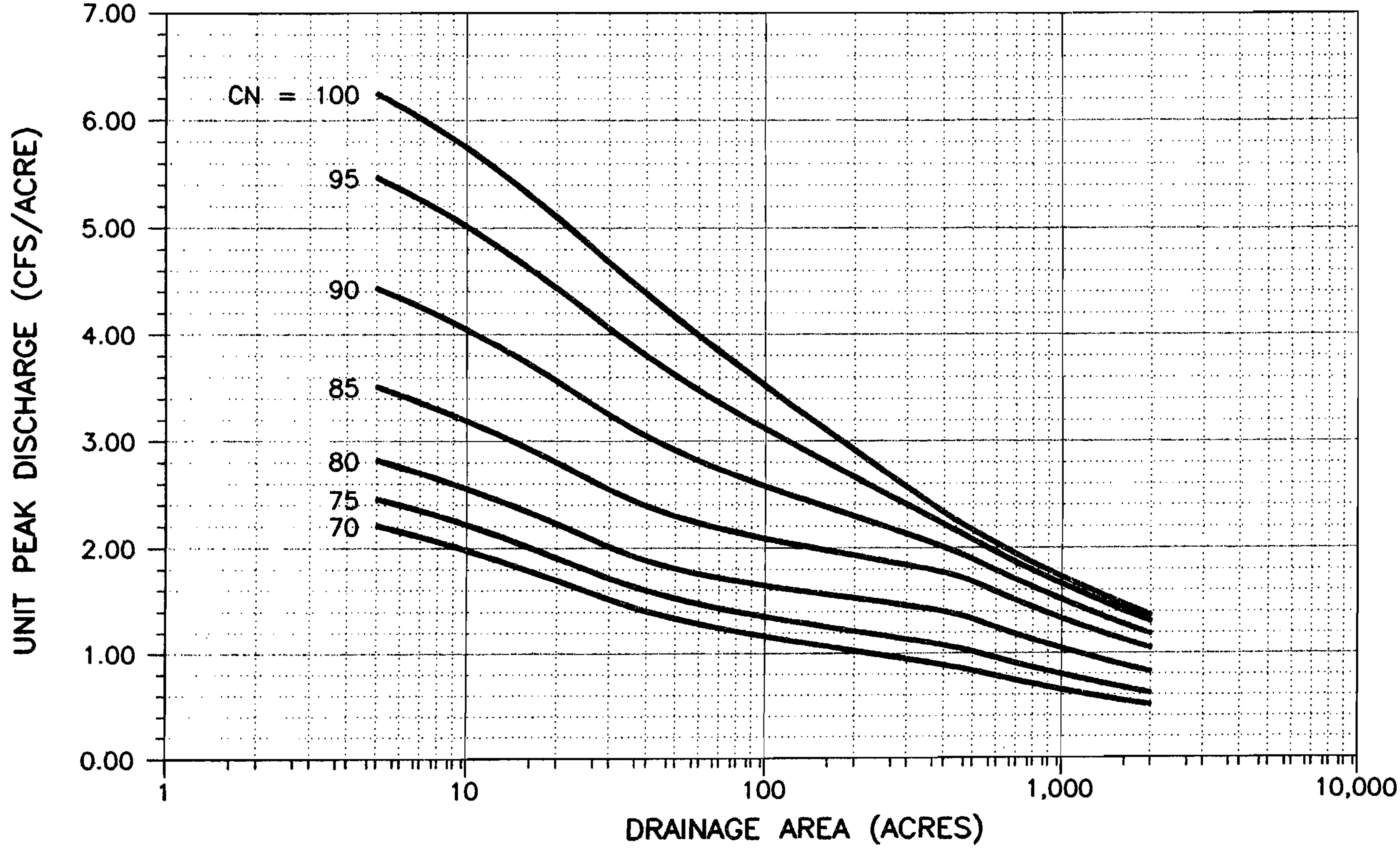

Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

50-Year Peak Flow Curves (Average Basin Slope = 1% - 2%) Amarillo, Texas

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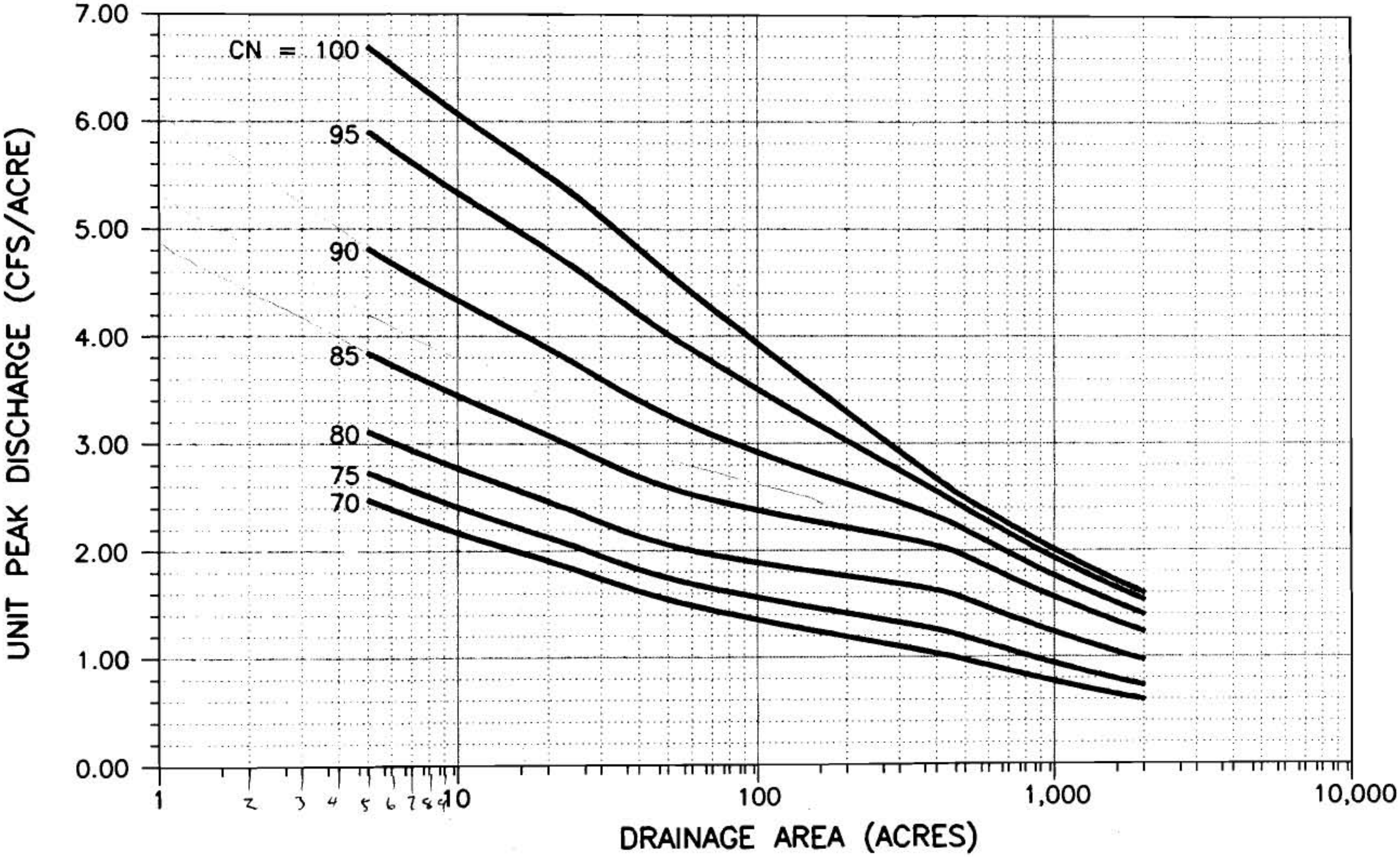
Figure 2-22



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

50-Year Peak Flow Curves (Average Basin Slope = 2% - 3%) Amarillo, Texas

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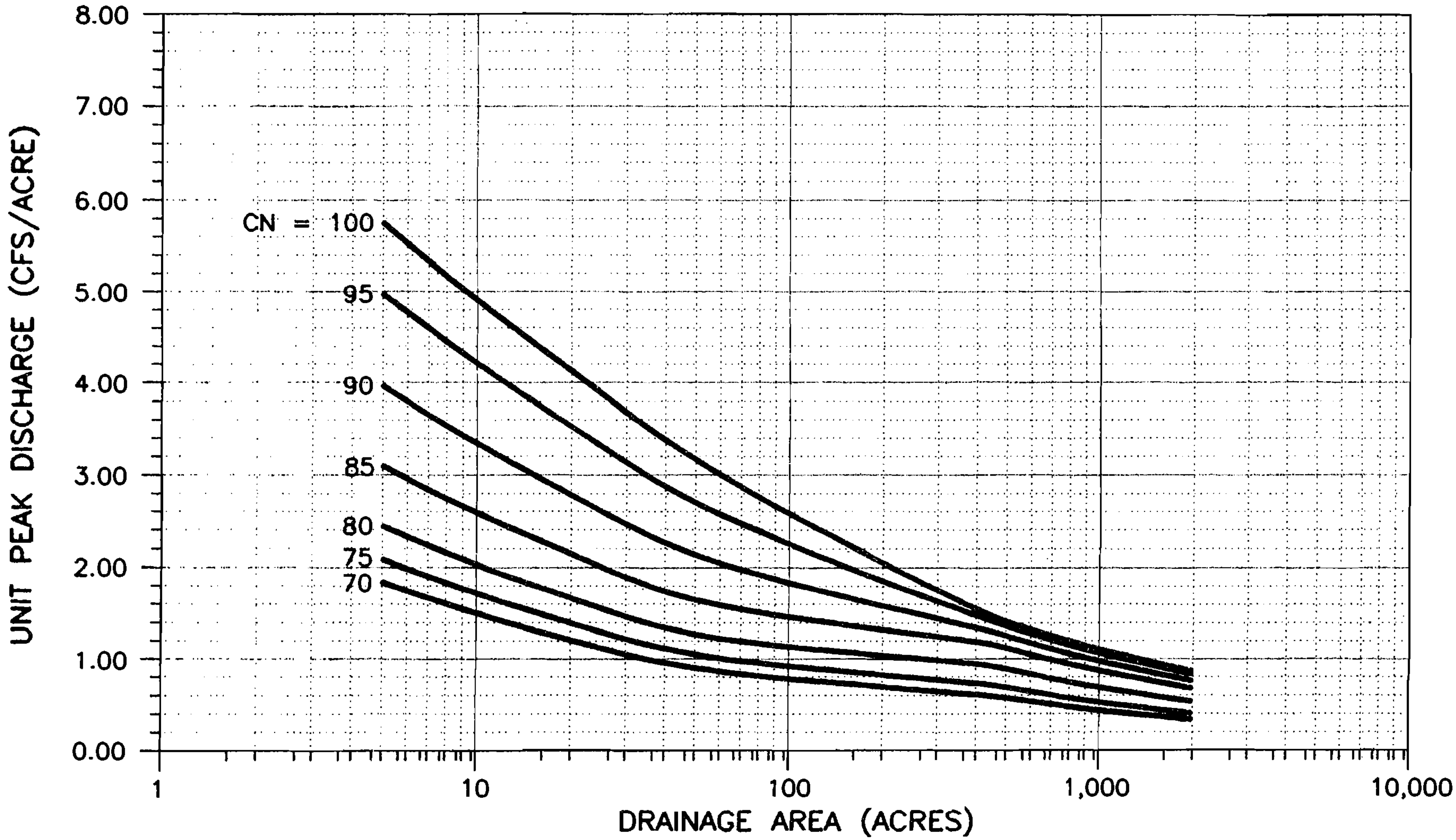
Figure 2-23



Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

100-Year Peak Flow Curves (Average Bain Slope = 0% - 0.5%) Amarillo, Texas

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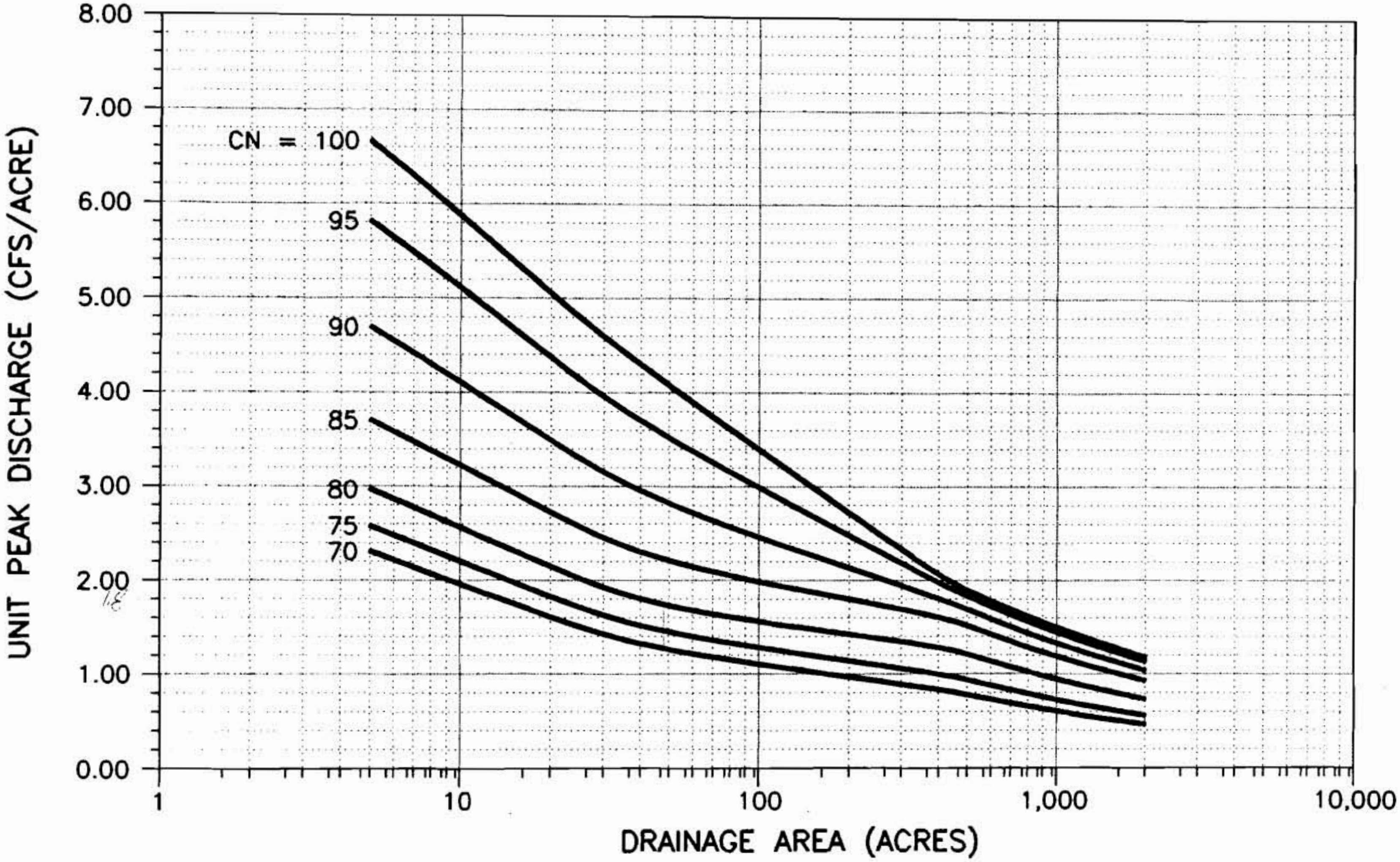

Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

100-Year Peak Flow Curves (Average Basin Slope = 0.5% - 1%) Amarillo, Texas

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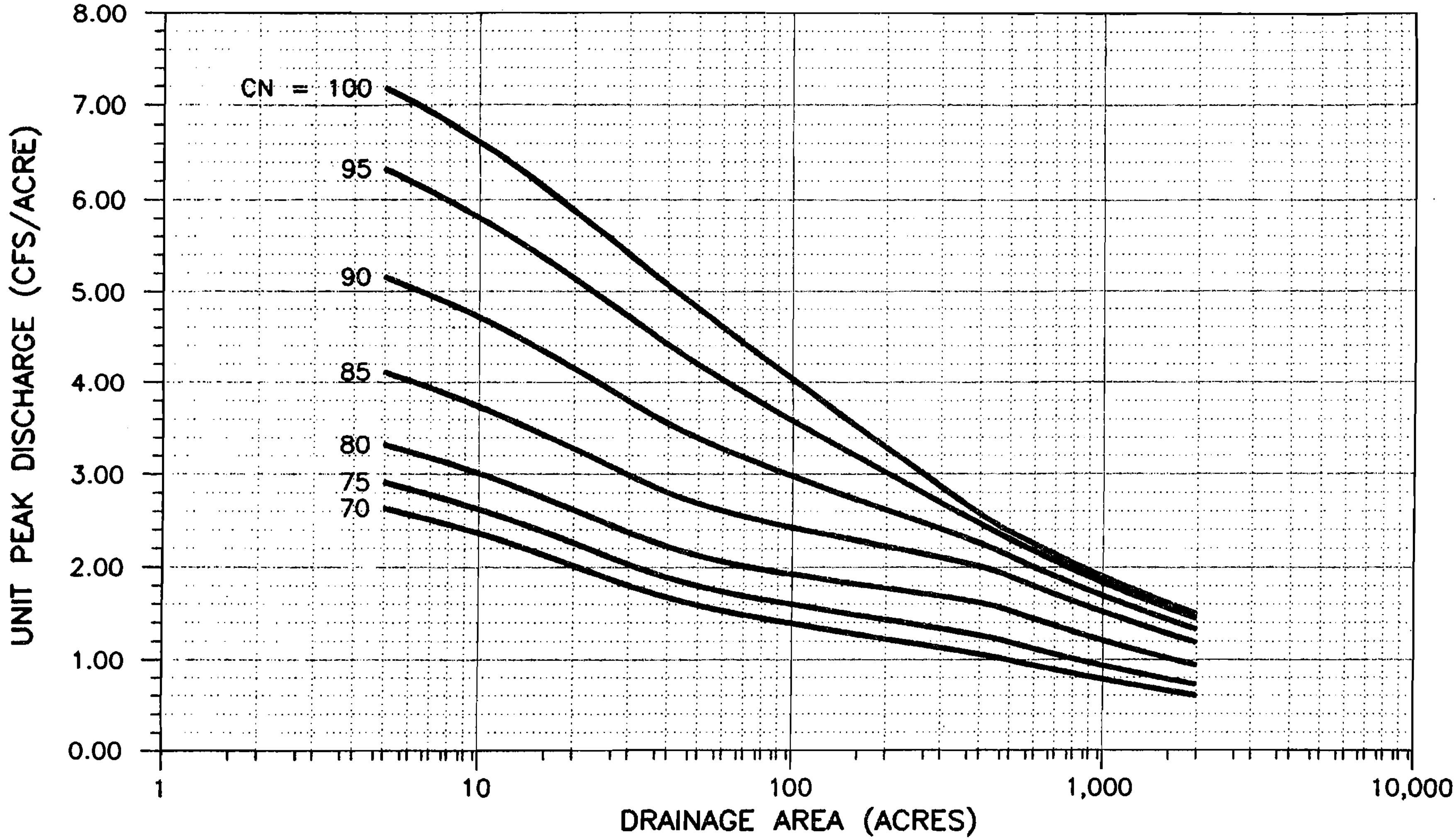

Source: HDR Engineering, Inc., Amarillo Storm Water Management Master Plan.

CITY OF AMARILLO

100-Year Peak Flow Curves (Average Basin Slope = 1% - 2%) Amarillo, Texas	Date NOV 1992
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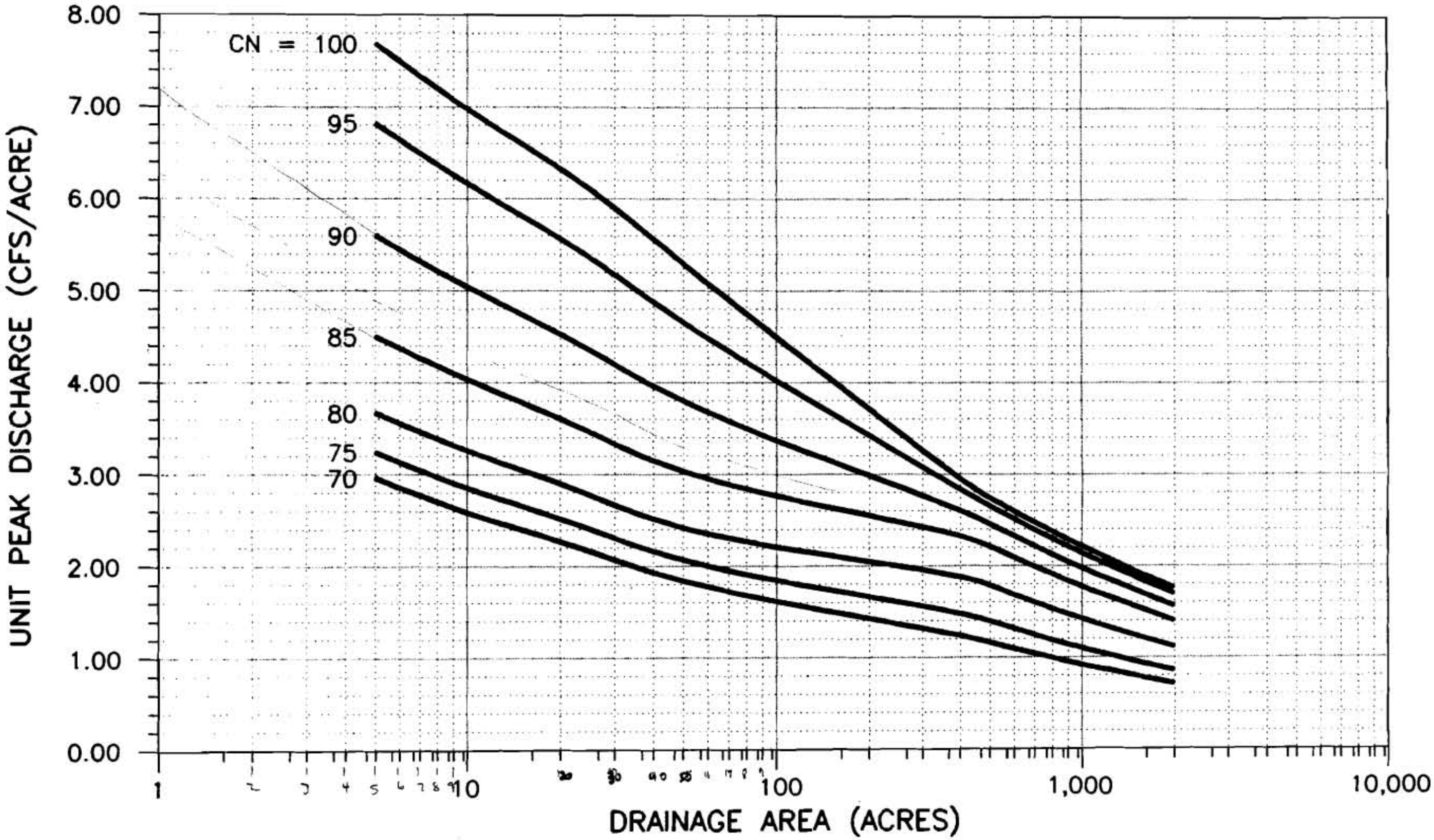

Source: HDR Engineering, Inc., _Amarillo Storm Water Management Master Plan_.

CITY OF AMARILLO

100-Year Peak Flow Curves (Average Basin Slope = 2% - 3%) Amarillo, Texas

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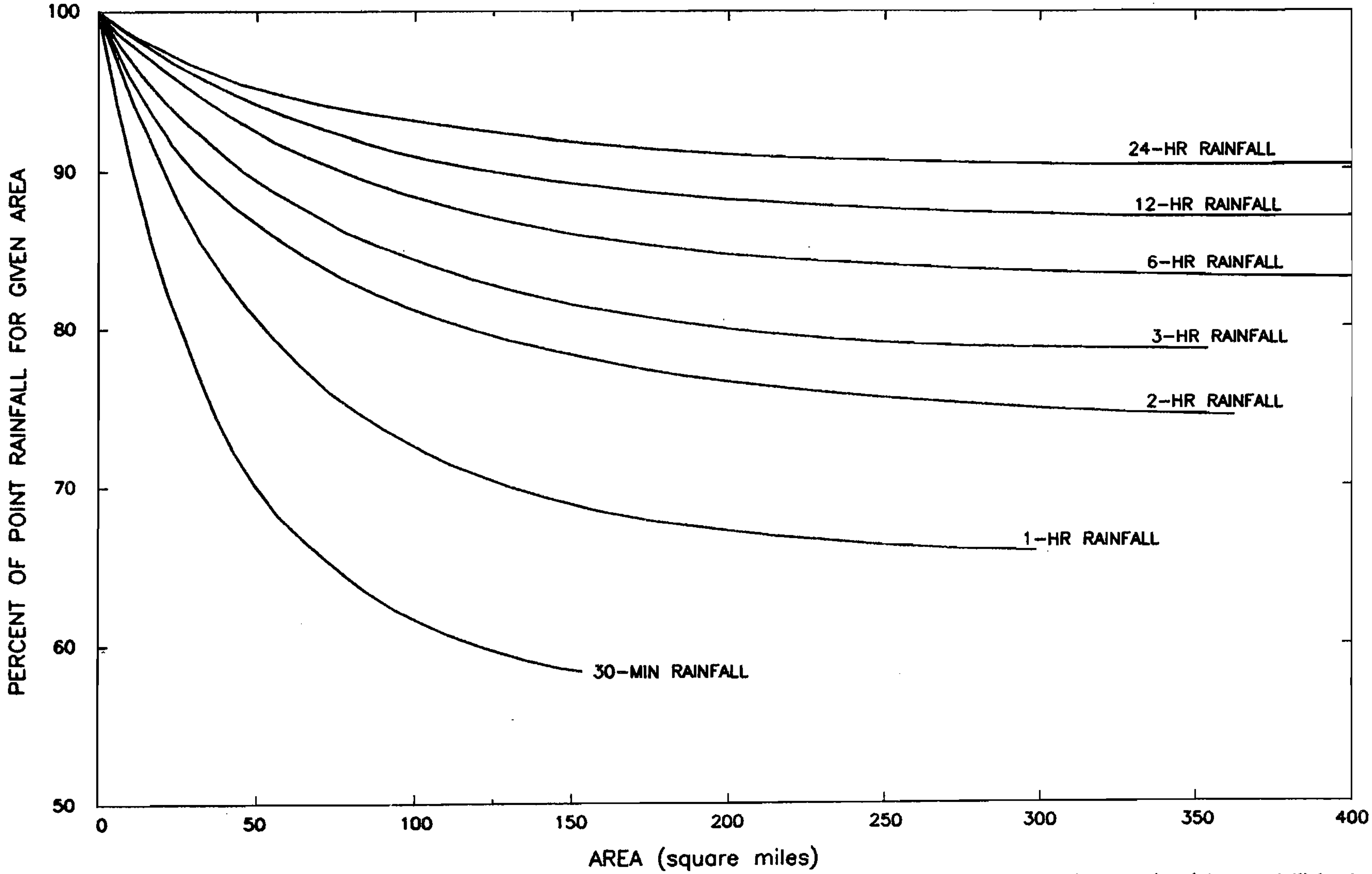
Figure: 2-27

NOTES: 1. From Weather Bureau Technical Paper No. 40, Figure 15
2. The 2-hr and 12-hr curves are interpolated from the TP40 data.



Source: Hydrologic Engineering Center, Hydrologic Analysis of Ungaged Watershed Using HEC-1.

CITY OF AMARILLO

Adjustment of Point Rainfall for Watershed Area

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Figure ?-30

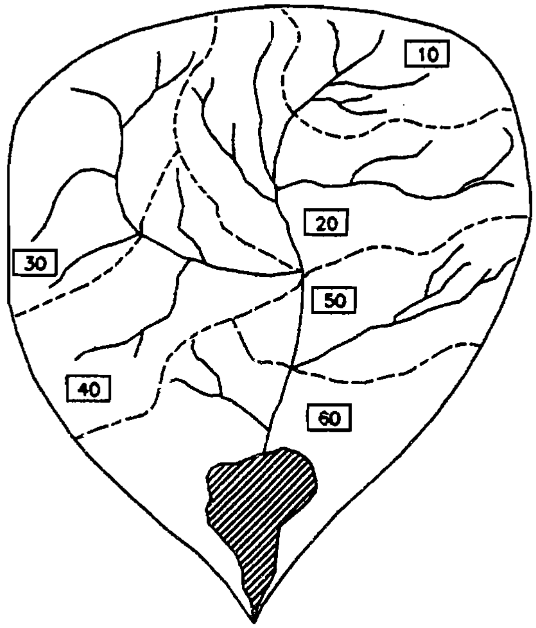


Figure 2-28 Example Drainage Basin

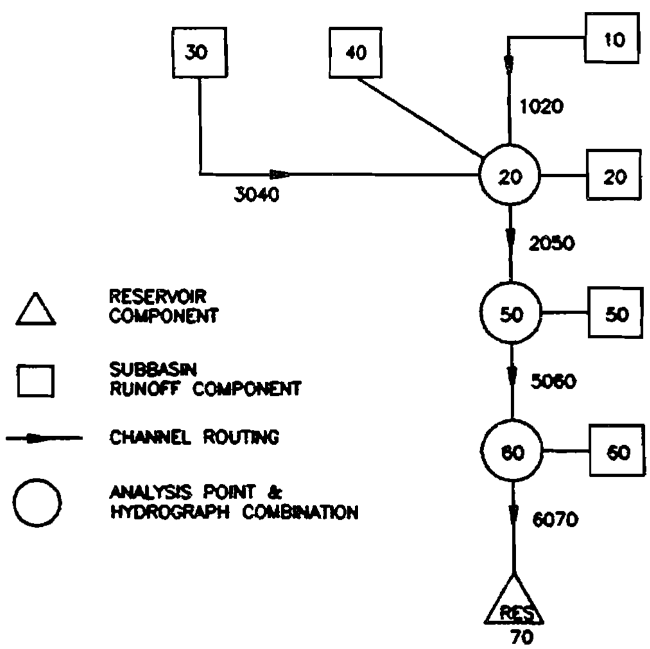
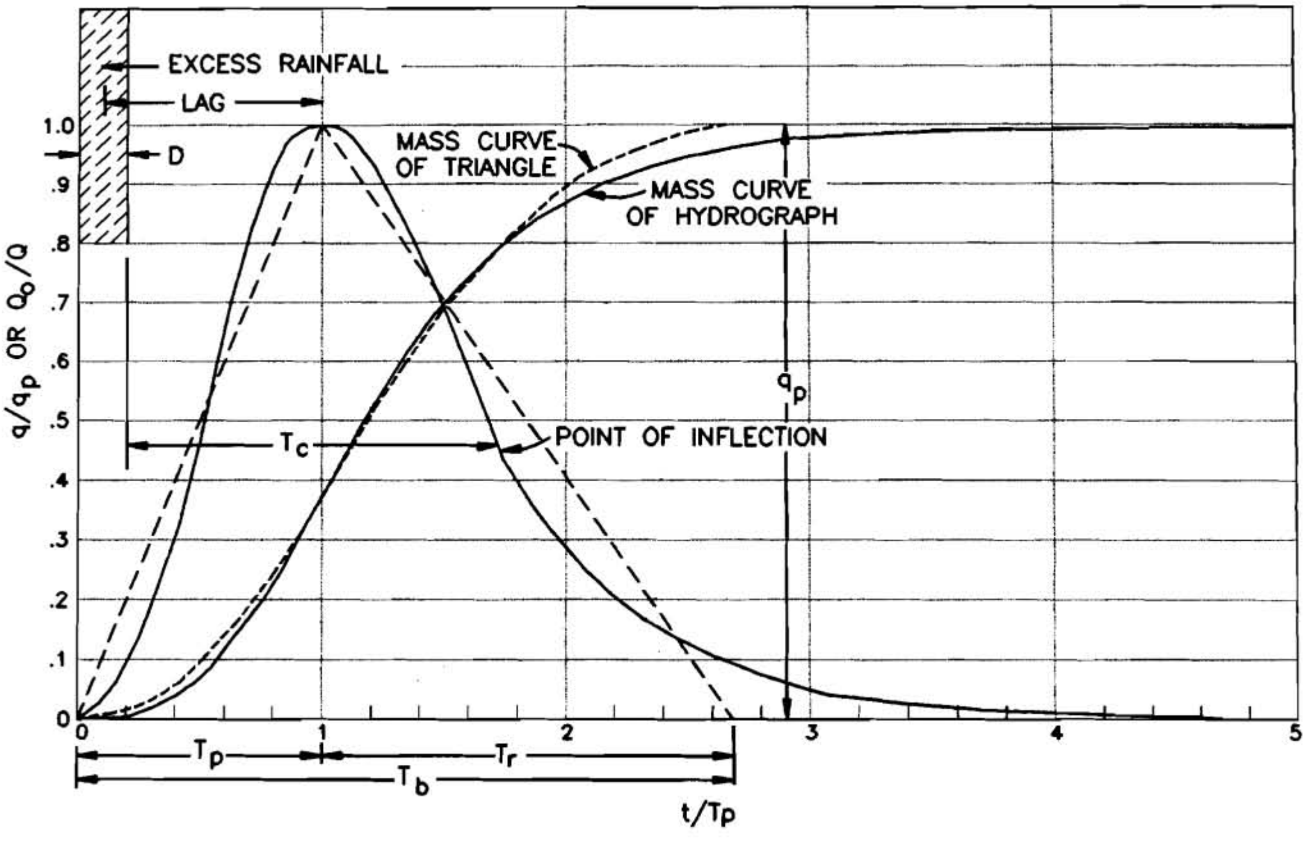


Figure 2-29 Example Drainage Basin Schematic

Source: Hydrologic Engineering Center, HEC-1, Flood Hydrograph Package.



Source: Soil Conservation Service, National Engineering Handbook Section 4.

CITY OF AMARILLO

Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph

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Figure 2-31

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3.1 INTRODUCTION

Roadways or streets in the urban areas of Amarillo serve an important and necessary drainage service even though their primary function is for the movement of traffic. However, good planning of streets can substantially help in reducing the size of, and sometimes eliminating the need for a storm drainage system in newly urbanized areas. Traffic and drainage uses are compatible up to a point, beyond which drainage must be secondary to traffic needs.

3.2 EFFECTS OF STORM WATER ON STREET CAPACITY

The storm runoff which influences the traffic capacity of a street can be classified as follows:

A. Sheet flow across the pavement as falling rain flows to the edge of the pavement.

B. Runoff flowing adjacent to the curb.

C. Storm water ponded at low points.

D. Flow across the traffic lane from external sources, or cross-street flow (as distinguished from water falling on the pavement surface).

E. Splashing of any of the above types of flow on pedestrians.

Each of these types of storm runoff must be controlled within acceptable limits so that the street's main function as a traffic carrier will not be unduly restricted. The effect of each of the above categories of runoff on traffic movement are discussed in the following sections.

3.2.1 Interference Due to Sheet Flow Across Pavement

Rainfall which falls upon the paved surface of a street or road must flow overland as sheet flow until it reaches a channel. Channels can be created either by curbs and gutters or by roadside ditches. The direction of flow on the street may be determined by the vector addition of the street grade and the crown slope, which is equivalent to drawing the perpendicular to a contour line on the road as shown in Figure 3-1. The depth of sheet flow will be essentially zero at the crown of the street and will increase as it proceeds towards the channel. Traffic interference due to sheet flow is essentially of two types: hydroplaning and splash.

Hydroplaning

Hydroplaning is the phenomenon of vehicle tires actually being supported by a film of water which acts as a lubricant between the pavement and the vehicle. It generally occurs at speeds commensurate with arterial streets and its effect can be minimized by achieving a relatively rough pavement which will allow water to escape from beneath the tires by pavement grooving to provide drainage, or by reducing travel speed.

Splash

Traffic interference due to splash results from sheet flow of excessive depth caused by water traveling a long distance or at a very low velocity before reaching a gutter. Increasing the street crown slope will decrease both the time and distance required for water to reach the gutter. The crown slope, however, must be kept within acceptable limits to prevent side-slipping of traffic during frozen surface conditions and to allow the opening of doors when parked adjacent to curbs. An exceedingly wide pavement section contributing flow to one curb will also affect the depth of sheet flow. This may be due to superelevation of a curve, off-setting of the street crown due to warping of curbs at intersections, or many traffic lanes between street crown and the gutter. Consideration should be given to all of these factors to maintain a depth of sheet flow within acceptable limits.

3.2.2 Interference Due to Gutter Flow

Water which enters a street, either sheet flow from the pavement surface or overland flow from adjacent land areas, will flow in the gutter of the street until it reaches an outlet, such as a storm drain or a channel. Figure 3-1 shows the configuration of gutter flow moving down a street when there is a storm drain system. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively infringe upon the traffic lane. If vehicles are parked adjacent to the curb, the width of spread will have little influence on traffic until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, as with many arterial streets, whenever the flow width exceeds a few feet it will significantly affect traffic. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the flow width increases, it becomes impossible for vehicles to operate without driving through water, and they again begin to use the inundated lane. At this point the traffic velocities will be significantly reduced as the vehicles begin to drive through the deeper water. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed on the open lane.

Eventually, if width and depth of flow become great enough, the street will become ineffective 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.

The street classification is also important when considering the degree of interference to traffic. A local street, and to a lesser extent a collector or arterial street, could be inundated with little effect upon vehicular travel. The small number of cars involved could move at a low rate of speed through the water even if the depth were four to six inches. However, reducing the speed of arterial traffic affects a greater number of private, commercial, and emergency vehicles.

3.2.3 Interference Due to Ponding

Storm runoff ponded on the street surface because of a change in grade or the crown slope of intersecting streets has a substantial effect on traffic. A major problem with ponding is that it may reach depths greater than the curb and remain on the street for long periods of time. Another problem is that ponding is localized in nature and vehicles may enter a pond moving at a high rate of speed.

The manner in which ponded water affects traffic is essentially the same as for curb flow; the width of spread onto the traffic lane is the critical parameter. Ponded water will often bring traffic on a street to a complete halt. In this case, incorrect design of only one facet of an entire street and storm drainage system will render the remainder of the street system useless during the runoff period.

3.2.4 Interference Due To Water Flowing Across Traffic Lanes

Whenever storm runoff moves across a traffic lane, a serious impediment to traffic flow occurs. The cross flow may be caused by superelevation of a curve or a street intersection exceeding the capacity of the higher gutter on a street with cross fall. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when they reach the location. If the velocity of vehicles is naturally slow and use is light, such as on local streets, cross-street flow does not cause sufficient interference to be objectionable.

The depth and velocity of cross-street flow should always be maintained within such limits that it will not have sufficient force to affect moving traffic. If a vehicle which is hydroplaning enters an area of cross street flow, even a minor force could be sufficient to move it laterally towards the gutter.

At certain intersections the flow may be trapped between converging streets and must either flow over one street or be carried underground. If the vehicles crossing the intersection are required to stop, then very little hazard exists to the traveling public. This is the basis for the assumption that valley gutters are acceptable across a local street where it intersects another local or collector street. Another point in favor of the use of valley gutters is the continuation of the grade of the dominant street. If the crown of the local street is allowed to coincide with the crown of the major street, the outside traffic lanes of the major street will have a "hump" at the intersection.

3.2.5 Effect on Pedestrians

In areas where pedestrians frequently use sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem. It must also be kept in mind that under certain circumstances, pedestrians will be required to cross ponded water adjacent to curbs.

Since the majority of pedestrian traffic will cease during the actual rainstorm, less consideration need be given to the problem while the rain is actually falling. Ponded water, however, remaining after the storm has passed, must be negotiated by pedestrians.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets which are classified as local for vehicles and located adjacent to a school are arterials for pedestrian traffic. Allowable width of gutter flow and ponding should reflect this fact.

3.3 DESIGN CRITERIA

Design criteria for the collection and transport of runoff on public streets is based on a reasonable frequency of traffic interference. That is, depending on the street classification, certain traffic lanes can be fully inundated once during the minor design storm return period. For example, a local street flow is allowed to flood to a curb depth of 6-inches during a 2-year frequency storm. During the 2-year period, lesser storms will occur which will produce less runoff and will not inundate the entire street.

Planning and design for urban storm runoff must be considered from the viewpoint of both the regularly expected storm occurrence, that is, the minor storm, and the major storm occurrence. The minor storm will have a 2-year return period. The major storm will have a return period of 100 years. The objectives of the major storm runoff planning and design is to eliminate major damage and loss of life. The minor drainage system is necessary to eliminate inconvenience, frequently recurring minor damage, and high street maintenance costs.

3.3.1 Street Capacity for Minor Storm

Determination of street capacity for the minor storm shall be based upon pavement encroachment. The pavement encroachment for the minor storm shall be limited as set forth in Table 3-1.

TABLE 3-1 Minor Storm Runoff Allowable Street Use

Street Classification	Maximum Pavement Encroachment
Local and Collector	No curb overtopping. Flow may spread to crown of street.
Arterial	No curb overtopping. Flow spread must leave at least one lane in each direction free of water.
Freeway	No significant encroachment is allowed on any traffic lanes.

When the maximum encroachment is reached, a separate storm drainage system or additional storm drainage capacity shall be provided and designed on the basis of the minor storm. Development of the major drainage system is encouraged so that the minor storm runoff is removed from the streets, thus moving the point at which the storm drainage system must begin further downstream.

Calculating Capacity

In general, an arterial street crossing will require installation of a storm drain system or other suitable means to transport the minor storm runoff under the arterial street. Collector streets shall have cross valley gutters only at infrequent locations, as specified in accordance with good engineering practices.

Lowering of the standard height of street crown shall not be allowed for the purposes of hydraulic design, unless approved by the City Engineer. In no case will reduced crowns be allowed on arterial streets.

Where additional hydraulic capacity is required on a street, the gradient must be increased and/or inlets and storm drains or other storm water conveyance facilities shall be installed to remove the required portion of the flow.

Alleys are not an integral part of the drainage system. In general, alleys, shall be designed to convey only the runoff from the rear of adjacent lots and direct it to the street at the end of the block. In no case shall runoff in any street be directed to flow into an alley or an alley be used as a drainageway.

When the allowable encroachment has been determined, the gutter (that portion of the street used to convey runoff) capacity can be computed using the modified Manning's formula. Gutter cross sections usually resemble a triangular shape with the curb forming the vertical leg of the triangle. The gutter may have a straight transverse slope, a transverse slope composed of two straight lines (V-shape), composite transverse slopes or parabolic sections for older pavements.

The modified Manning's Equation is utilized in triangular channels to better describe the hydraulic radius of a gutter section. The equation in terms of cross slope and depth of flow at the curb is:

$$Q = 0.56 \left[\frac{Z}{n}\right] S^{0.5}\, d^{2.67} \tag{3-1}$$

where:

Q	=	discharge, in cubic feet per second, Figure 3-2
Z	=	reciprocal of cross slope, $1/S_x$, in feet per foot
n	=	Manning's roughness coefficient
S	=	longitudinal slope, in feet per foot
d	=	depth of flow at curb or deepest point, in feet

The resistance of the curb face is neglected in the equation since the resistance is negligible when the cross slope is 10 percent or less.

The width of flow or spread in a triangular channel can be calculated by the following equation.

$$T = Zd \tag{3-2}$$

where:

T	=	width of flow (spread), in feet
Z	=	reciprocal of cross slope, $1/S_x$, in feet per foot
d	=	depth of flow at curb or deepest point, in feet

Figure 3-2 can be used for a direct solution of Equation 3.1, using Manning's "n" value of 0.016. For other values of "n", divide the value of Qn by "n". Manning's "n" values for different street and gutter roughness conditions are presented in Table 3-2. Figure 3-3 provides the solution for a typical composite gutter section with varying cross slopes.

TABLE 3-2 Manning's Roughness Coefficients for Streets and Gutters

Surface Type	"n" Value
Concrete gutter troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
Brick	0.016

Calculating Velocity

The average velocity of flow in the gutter can be calculated by modifying Manning's Equation. Figure 3-4 is a nomograph of the velocity in a triangular gutter section. The equation in terms of cross slope and width of flow in the pavement, assumes that the discharge in the gutter varies uniformly between the section and is:

$$V = \frac{1.12}{n} S^{0.5} S_x^{0.67} T^{0.67} \tag{3-3}$$

where:

V	=	velocity, in feet per second, Figure 3-4

S = longitudinal slope, in feet per foot

S_x = cross slope, in feet per foot

T = width of flow (spread), in feet

n = Manning's roughness coefficient

If a channel has zero flow at the upstream end, the average velocity occurs at the point where spread is equal to 65 percent of the maximum spread. For channel sections with discharges greater than zero at the upstream section, the spread at average velocity, T_a, is given by Table 3-3. In Table 3-3, T_1 is the spread at the upstream section and T_2, is the spread at the downstream section. The average spread is then used in Figure 3-4 or Equation 3-3.

TABLE 3-3 Spread at Average Velocity in a Triangular Gutter Section

$\frac{T_1}{T_2}$	$\frac{T_a}{T_2}$
0.0	0.65
0.1	0.66
0.2	0.68
0.3	0.70
0.4	0.74
0.5	0.77
0.6	0.82
0.7	0.86
0.8	0.90

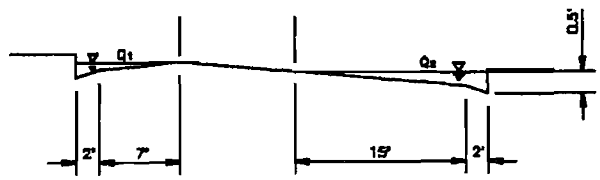
Example 1 Gutter Carrying Capacity, Minor Storm

Given: 6" vertical curb
2'-wide by 0.1'-deep gutter
2% pavement crown slope
36' street width, curb to curb
Crown offset to 1/4 point for cross fall
Collector street
Street grade = 0.3%
Assume n=0.016

Find: Capacity, each gutter

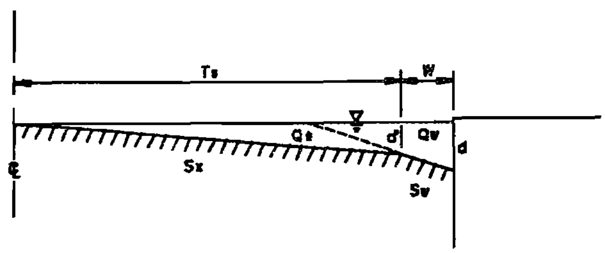
Solution:

1) Determine allowable pavement encroachment.
From Table 3-1, no curb overtopping, flow may spread to crown of street.



2) Calculate capacity for each gutter.

Right Side - Using Nomograph, Figure 3-2.



$W = 2$ ft $S = 0.003$ $d = 0.5$ ft
$S_w = 0.1/2 = 0.11$ $Z_w = 20$
$S_x = 0.02$ $Z_x = 50$

Calculate Q_w by subtracting the Q's at depth d based on an extension of slope ratio Z_w to intersect the water surface and at a depth d', where $d' = d - W/Z_w$, thus $Q_w = Q_d - Q_{d'}$. Calculate Q_s for a slope ratio of Z_s and depth d'. Then $Q = Q_w + Q_s$.

a) Find Q_w:

1) Find $Q_{d'}$

$d = 0.5$ ft $Z_w/n = 20/0.016 = 1{,}250$
Therefore from Figure 3-2, $Q_d = 6.0$ cfs

$d' = d - W/Z_w$
$d' = 0.5 - 2/20 = 0.4$ ft

2) Find $Q_{d'}$:

$d' = 0.4$ ft $\quad Z_w/n = 20/0.016 = 1{,}250$
Therefore from Figure 3-2, $Q_{d'} = 3.3$ cfs

$Q_w = 6.0 - 3.3 = 2.7$ cfs

b) Find Q_s:

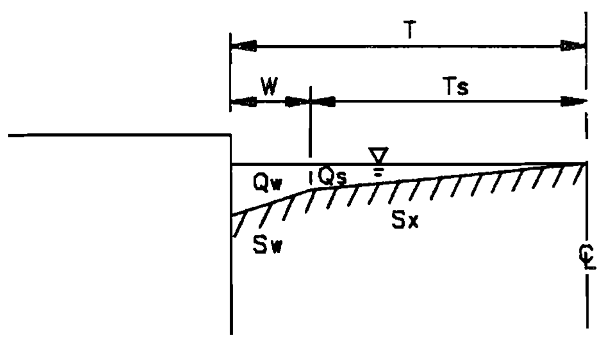
$d' = 0.4$ ft $\quad Z_x/n = 50/0.016 = 3{,}125$
Therefore from Figure 3-2, $Q_s = 8.3$ cfs

$T_s = Z_x d' = 50\ (0.4) = 20$ ft

c) Find Q:

$Q = Q_w + Q_s = 8.3 + 2.7 = 11$ cfs

Left Side



a) Calculate maximum depth at curb, when flow is spread to crown of street.

$T_s = 7$ ft

$d' = T_s S_x = 7\ (0.02) = 0.14$ ft

$d = d' + W/Z_w = 0.14 + 2/20 = 0.24$ ft

b) Find Q_d:

$d = 0.24$ ft $\quad Z_w/n = 1{,}250$
Therefore from Figure 3-2, $Q_d = 0.85$ cfs

c) Find $Q_{d'}$:

$d = 0.14$ ft $\quad Z_w/n = 1{,}250$
Therefore from Figure 3-2, $Q_{d'} = 0.2$ cfs

d) Find Q_w:

$$Q_w = Q_d - Q_{d'}$$
$$= 0.85 - 0.2 = 0.65 \text{ cfs}$$

e) Find Q_s:

$d' = 0.14$ ft $\quad Z_x/n = 3{,}125$
Therefore from Figure 3-2, $Q_s = 0.5$ cfs

f) Find Q:

$$Q = Q_w + Q_s$$
$$= 0.65 + 0.5 = 1.1 \text{ cfs}$$

3.3.2 Street Capacity for Major Storm

Determination of the allowable capacity for the major storm shall be based upon allowable depth and inundated area. The allowable depth and inundated area for the major storm shall be limited as set forth in Table 3-4.

Calculating Capacity

When the allowable depth and inundated area as determined from Table 3-4, the street capacity shall be calculated using Manning's Formula with an "n" value applicable to the actual boundary conditions encountered.

3.3.3 Ponding

The term ponding shall refer to areas where runoff is restricted to the street surface by sump inlets, street intersections, low points, intersections with drainage channels, or other reasons.

Minor Storm

Limitations for pavement encroachment by ponding for the minor storm shall be those presented in Table 3-1. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, and other hydraulic structures.

TABLE 3-4 Major Storm Runoff Allowable Street Inundation

Street Classification	Allowable Depth and Inundated Areas
Local and Collector	Residential dwellings, public, commercial, and industrial buildings, shall not be inundated at the lowest finished floor elevation, unless buildings are flood-proofed. The depth of water over the gutter flowline shall not exceed 24 inches.
Arterial	Residential dwellings, public, commercial, and industrial buildings, shall not be inundated at the lowest finished floor elevation, unless buildings are flood-proofed. The depth of water at the street crown shall not exceed 12 inches in order to allow operation of emergency vehicles. The depth of the water over the gutter flowline shall not exceed 24 inches.
Freeway	No inundation is allowed.

Major Storm

Limitations for depth and inundated area for major storms shall be those presented in Table 3-4. These limitations shall determine the allowable depth at inlets, gutter turnouts, culvert headwaters, and other hydraulic structures.

3.3.4 Cross-Street Flow

Cross-street flow is classified into two general categories. The first type is runoff which has been flowing down the street in a gutter and when the gutter capacity is exceeded the flow crosses the street to the opposite gutter. The second type is from some external source, such a drainage way which is intersecting the street, which will flow across the crown of a street when the conduit capacity beneath the street is exceeded.

Depth

Cross-street flow depth shall be limited as set forth in Table 3-5.

TABLE 3-5 Allowable Cross Street Flow

Street Classification	Minor Storm Runoff	Major Storm Runoff
Local and Collector	6 inches of depth in valley gutter	24 inches of depth in valley gutter
Arterial	None	12 inches or less over crown
Freeway	None	None

Capacity

Based upon the limitations in Table 3-5 and other applicable limitations (such as ponding depth), the quantity of cross-street flow shall be calculated. Where allowable ponding depth would cause cross-street flow, the limitation shall be the minimum allowable of the two criteria.

3.4 INTERSECTION LAYOUT CRITERIA

The following design criteria are applicable at intersections of urban streets. Gutter capacity limitations covered in Section 3.3 shall apply along the street, while this section shall govern at the intersection.

3.4.1 Gutter Capacity, Minor Storm

Pavement Encroachment

Limitations at intersections for pavement encroachment shall be as given in Table 3-1.

The capacity of each gutter approaching an intersection shall be calculated based upon the most critical cross section.

A. Continuous Grade Across Intersection

When the gutter flow will be continued across an intersection, the slope used for calculating capacity shall be that of the gutter flow line crossing the street. (Figure 3-5).

B. Flow Direction Change at Intersection

When the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 feet, 25 feet, and 50 feet from the geometric point of intersection of the two gutters. (Figure 3-5).

C. Flow Interception by Inlet

When gutter flow will be intercepted by an inlet on continuous grade at the intersection, gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet, and 50 feet upstream from the inlet. (Figure 3-5).

3.4.2 Gutter Capacity, Major Storm

Allowable Depth and Inundated Area

The allowable depth and inundated area for the major storm shall be limited as set forth in Table 3-4.

Capacity

The carrying capacity of each gutter approaching an intersection shall be calculated, based upon the most critical cross section.

The grade used for calculating capacity shall be as covered in Gutter Capacity, Minor Storm.

3.4.3 Ponding

Minor Storm

The allowable pavement encroachment for the minor storm shall be as presented in Table 3-1.

Major Storm

The allowable depth and inundated area for the major storm shall be as presented in Table 3-4.

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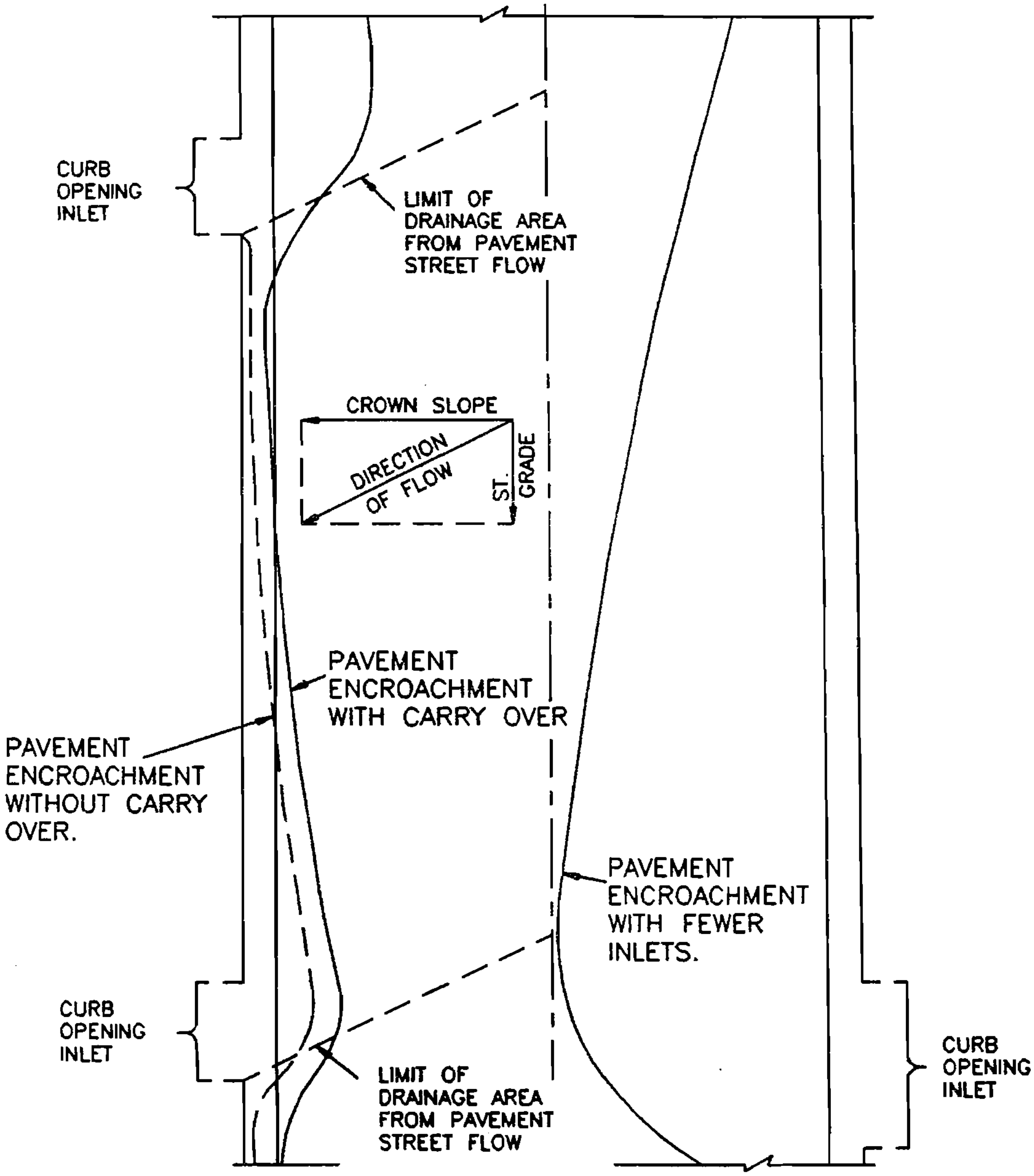
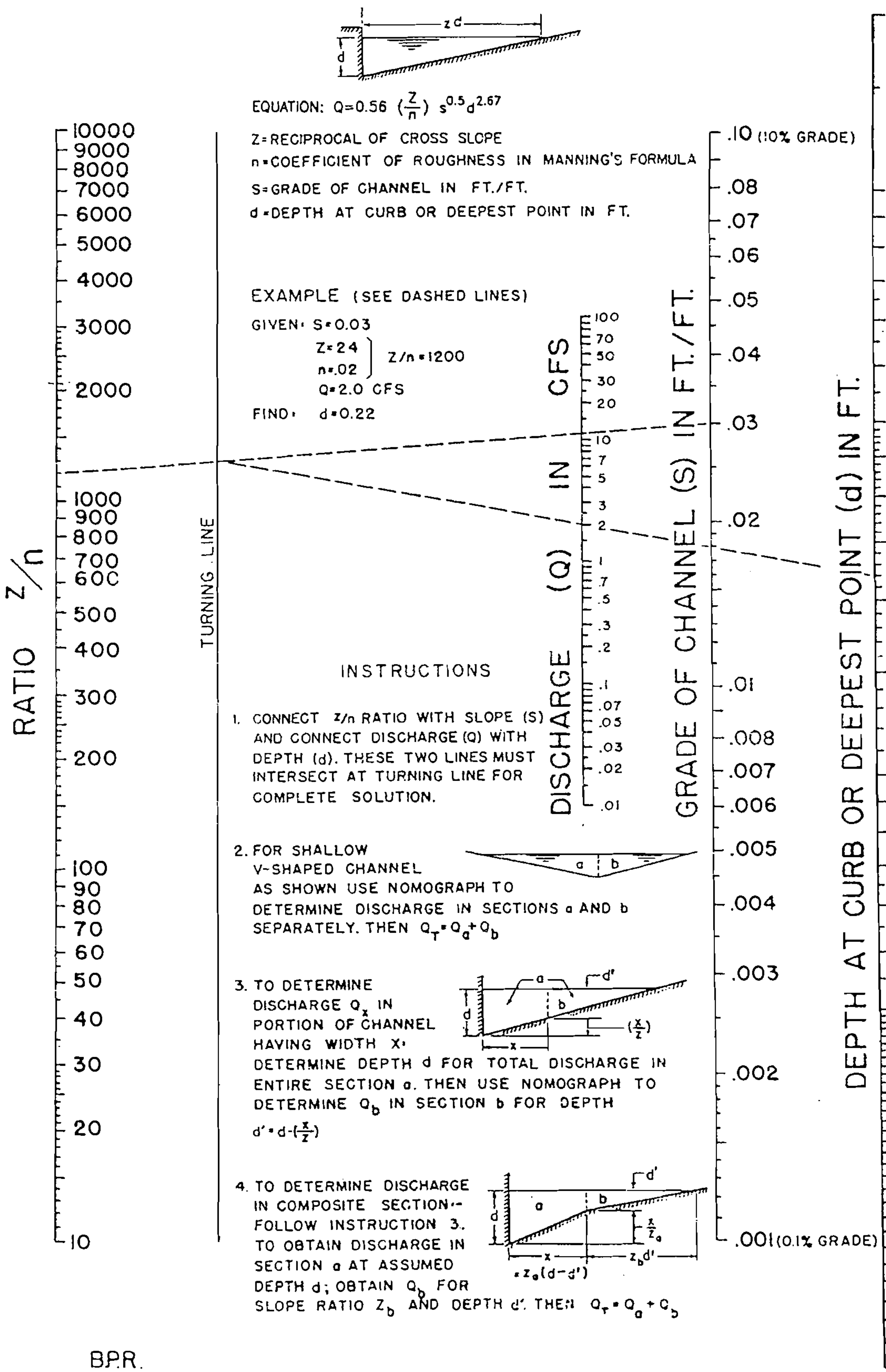


PLAN – TYPICAL
STREET WITH INLETS

Source: Denver Regional Council of Governments.

CITY OF AMARILLO	**Gutter and Pavement Flow Patterns**	Date: NOV 1992
	Storm Water Management Criteria Manual	Figure: 3–1



Source: Bureau of Public Roads.

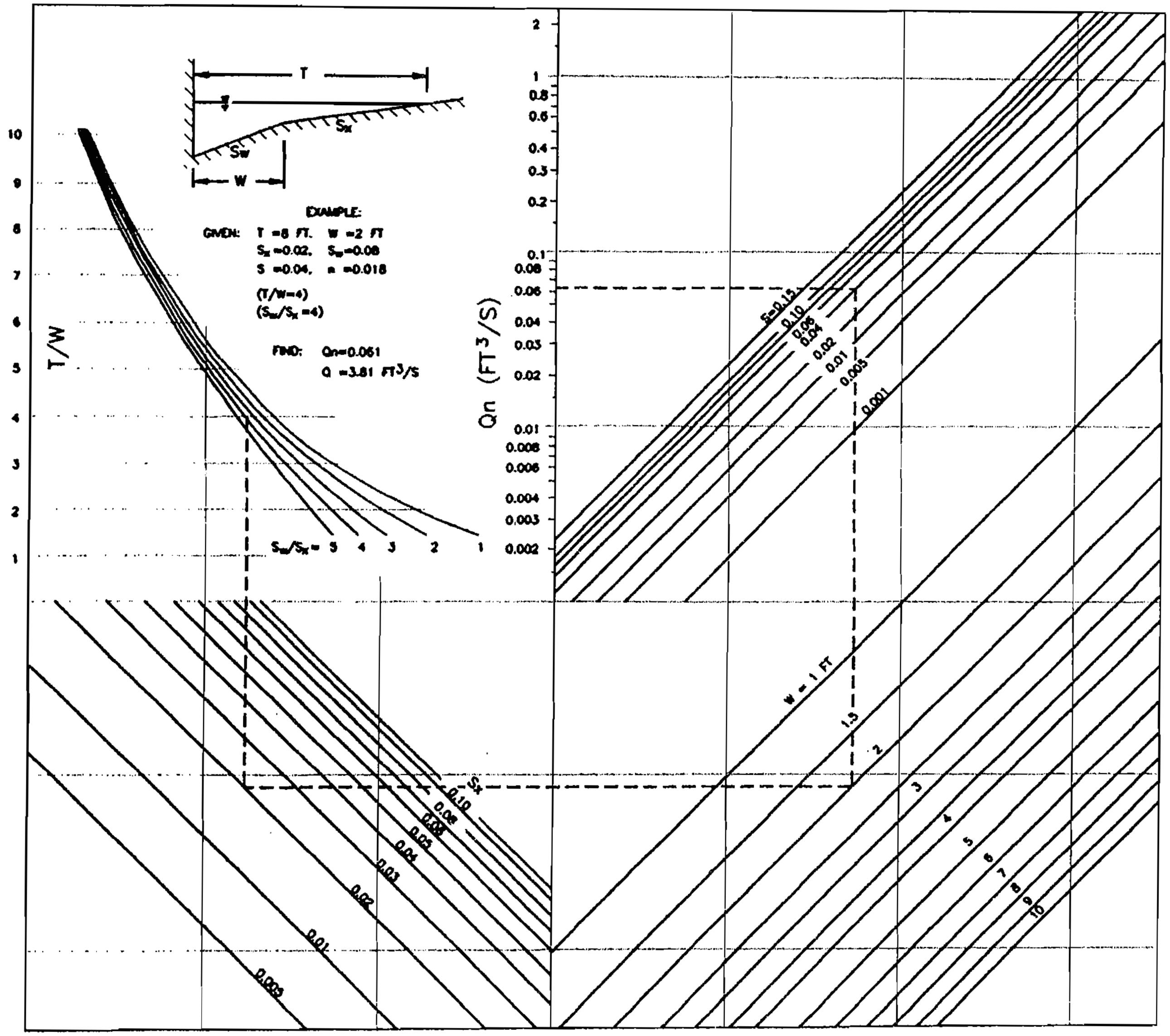
CITY OF AMARILLO

Flow in Triangular Channels

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
3-2



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

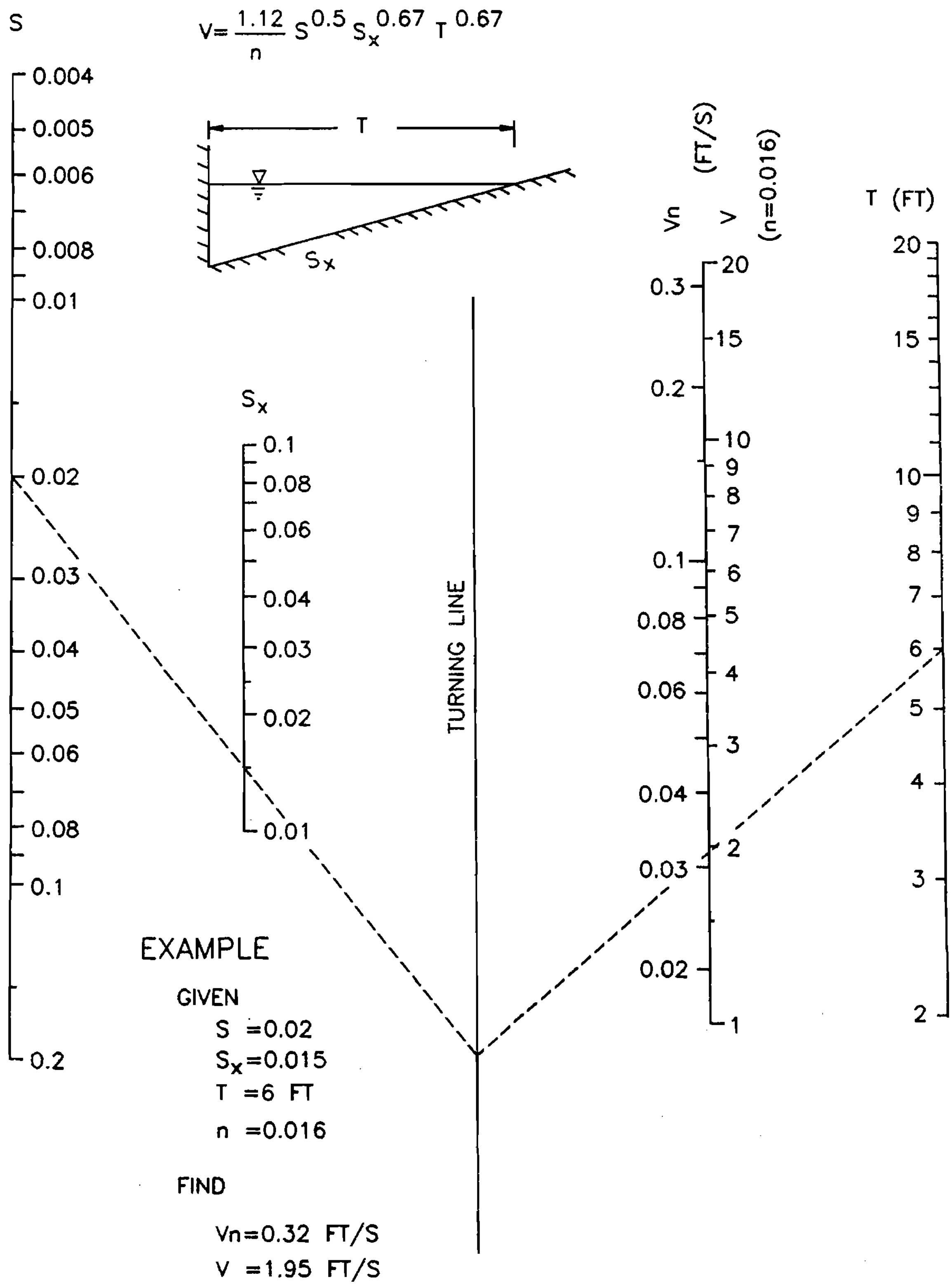
Flow in Composite Gutter Sections

Date NOV 1992

Figure 3-3

CITY OF AMARILLO

Storm Water Management Criteria Manual



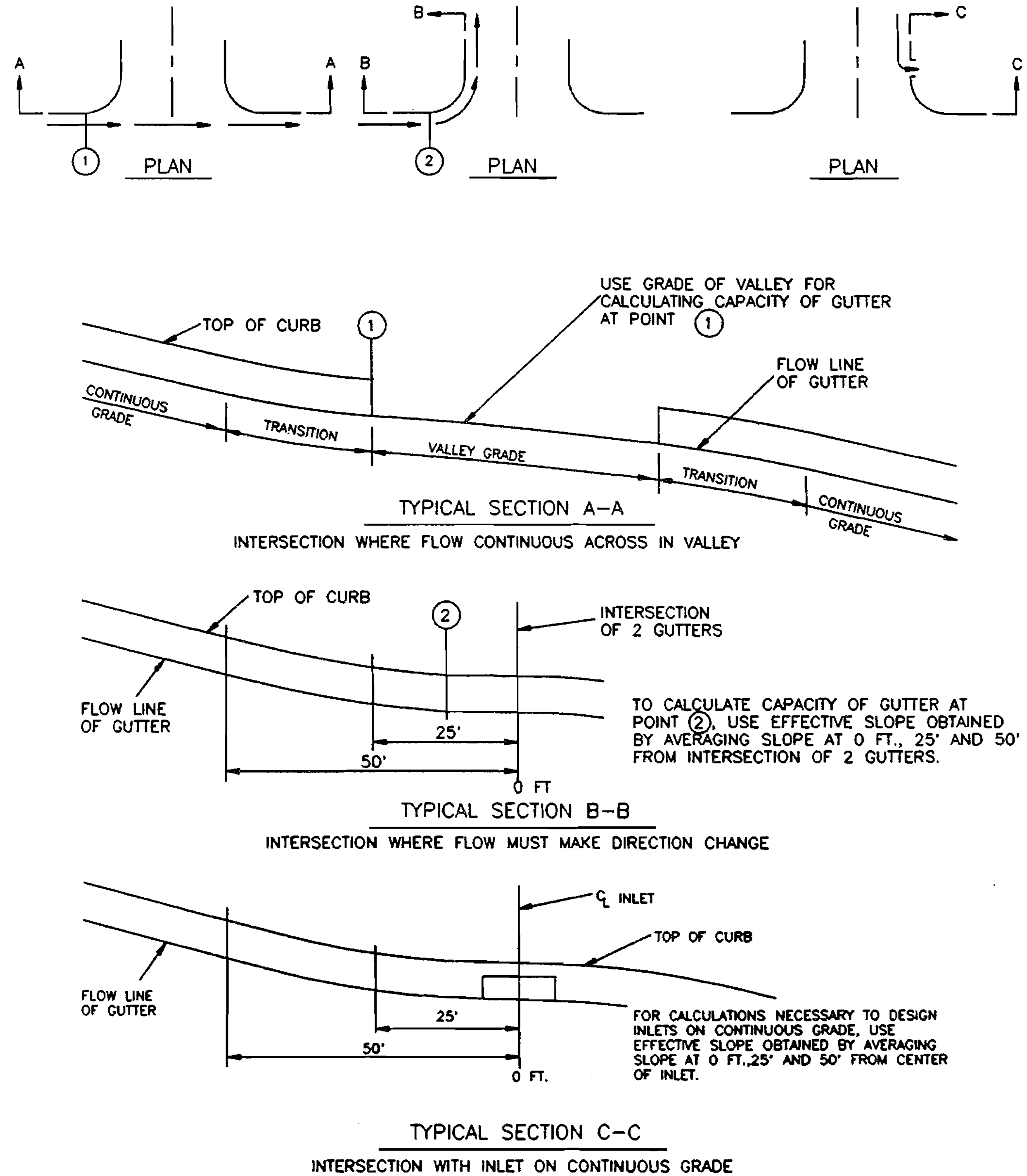
Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

CITY OF AMARILLO

Velocity in Triangular Gutter Sections

Date NOV 1992

Figure 3–4



CITY OF AMARILLO

Intersection Drainage

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
3–5

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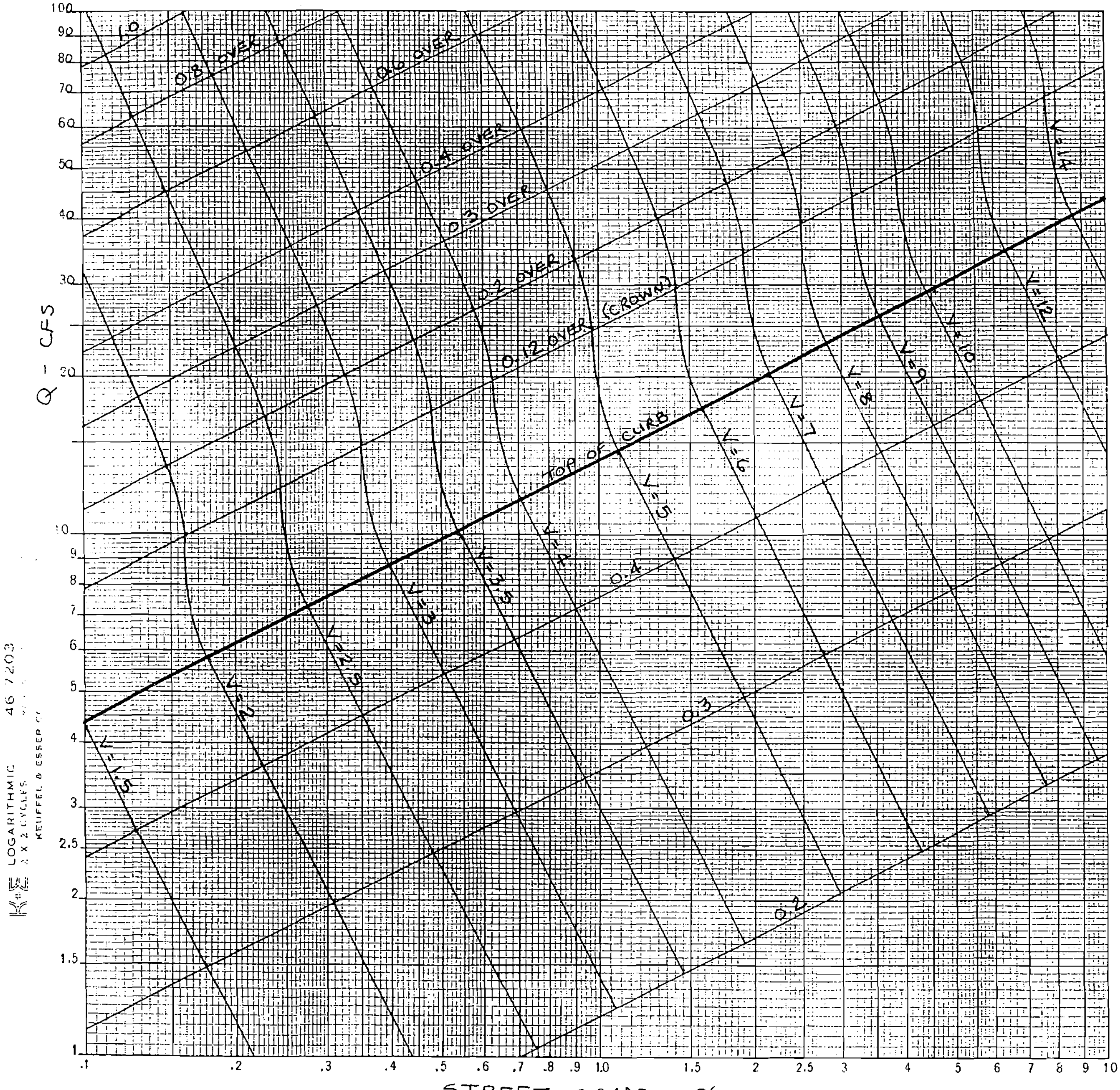
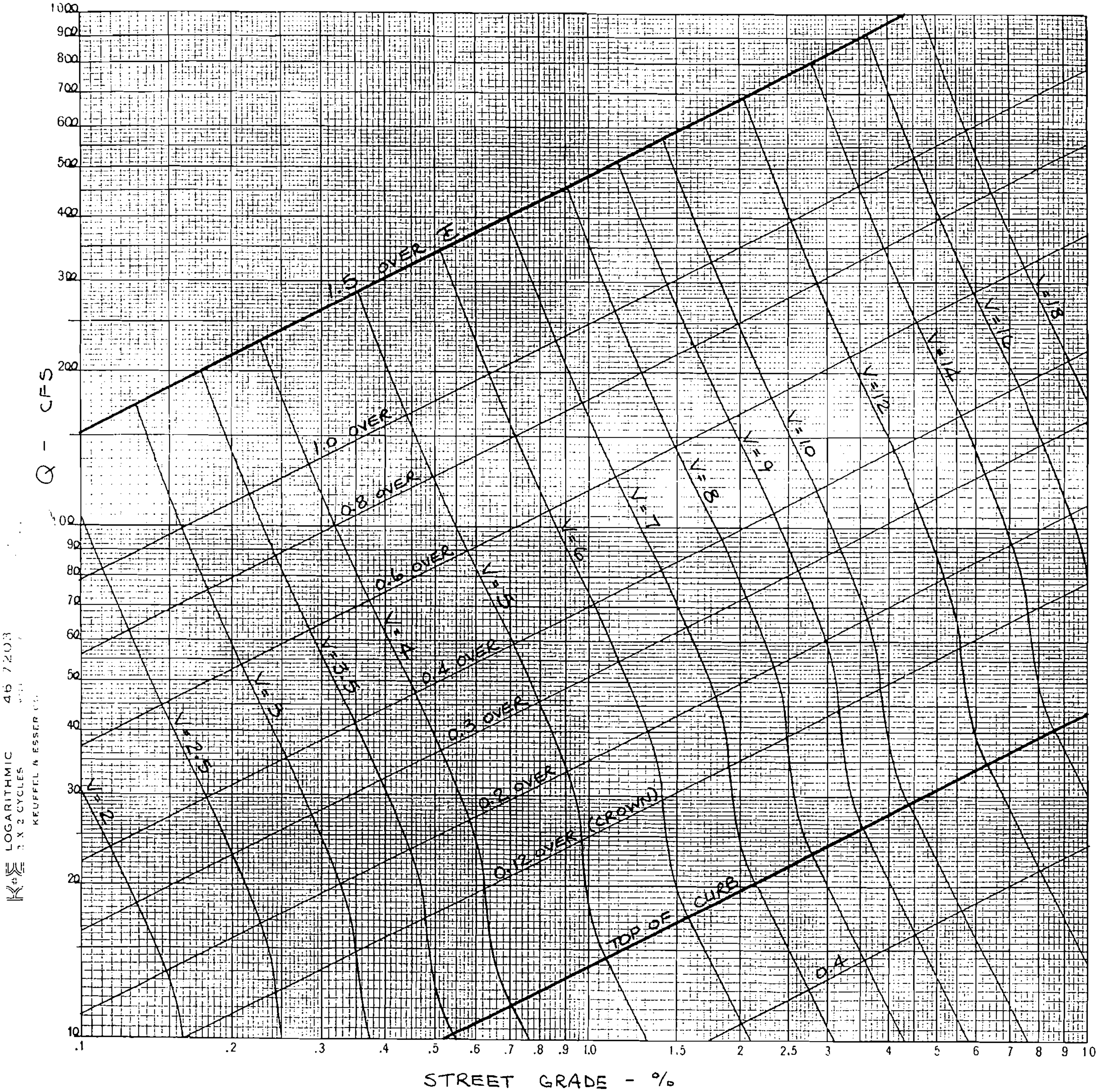


FLOW CAPACITY
1/2 STREET SECTION
37' B-B

1

4-1-94 MDK

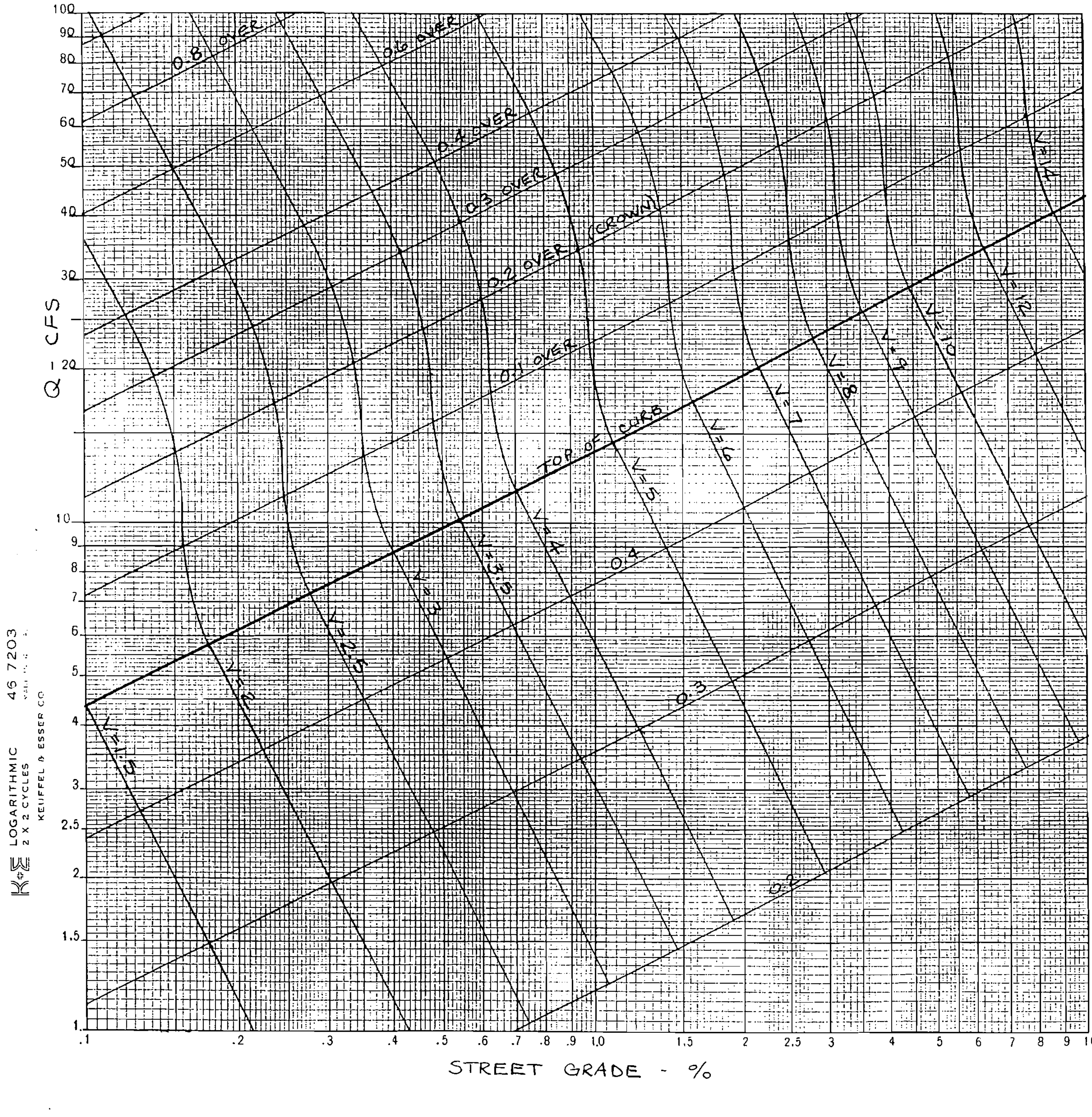


FLOW CAPACITY

1/2 STREET SECTION

37' B-B

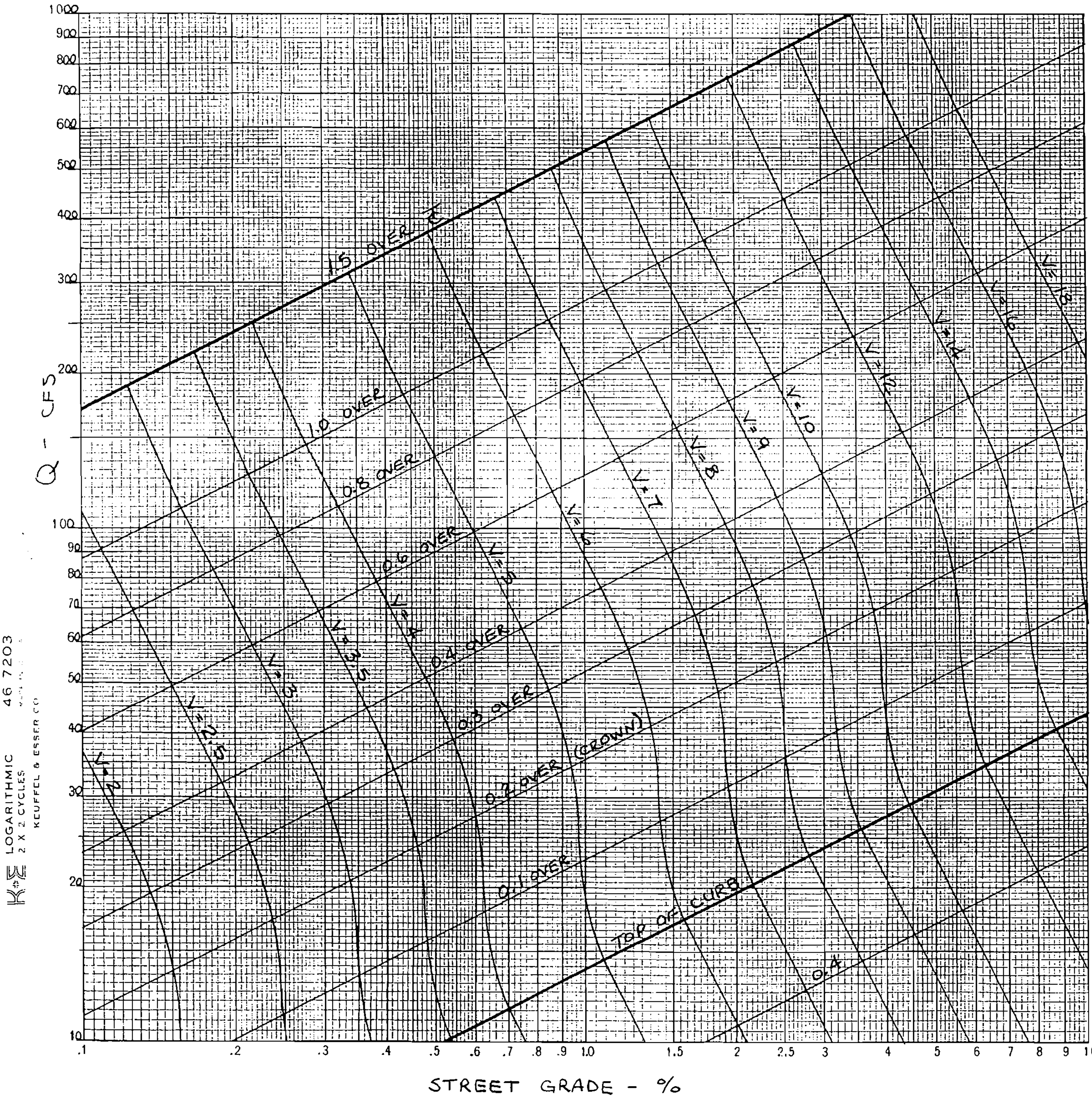
4-1-94 MDK



FLOW CAPACITY
1/2 STREET SECTION
45' B-B

3

4-1-94 MDI

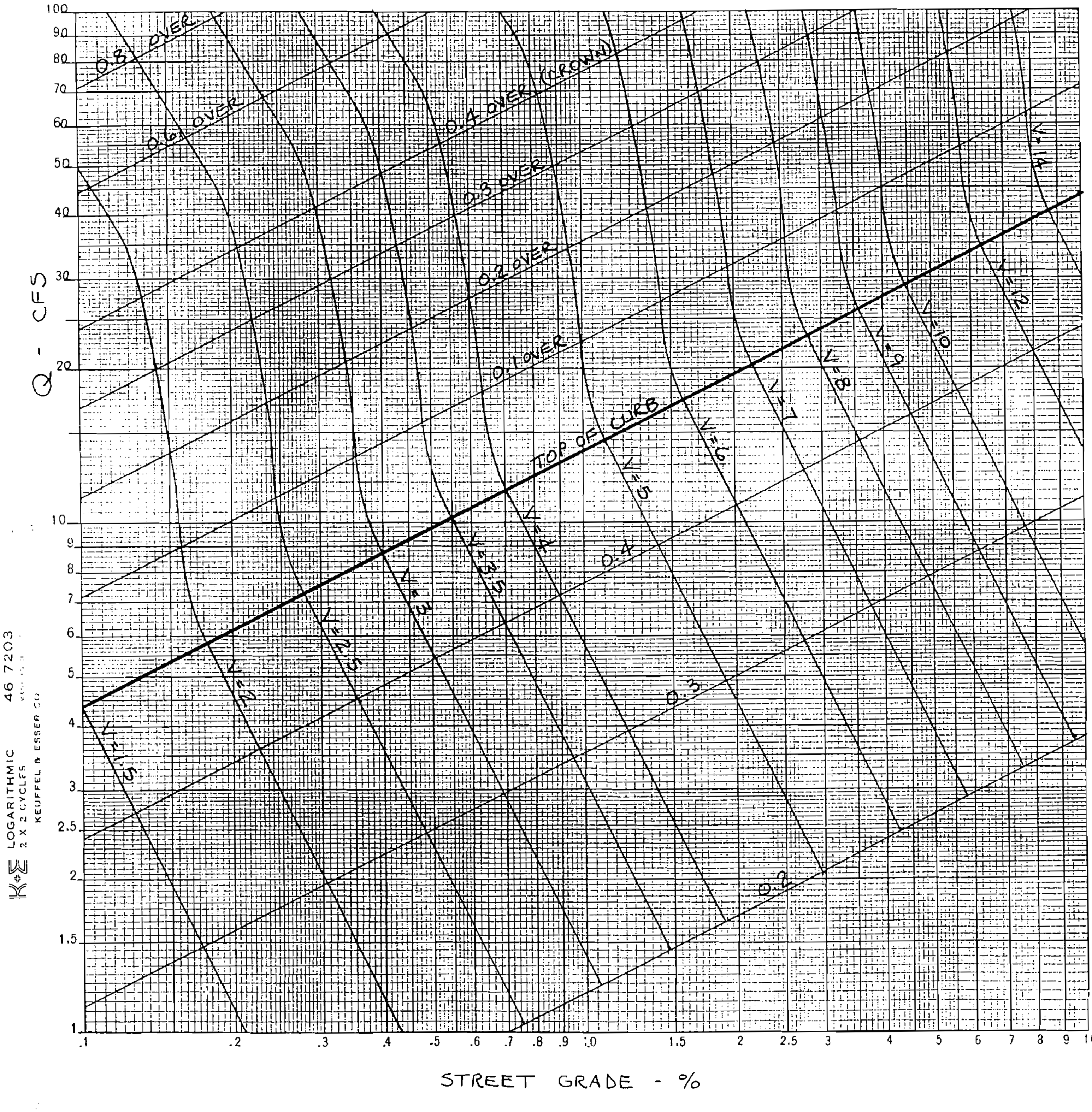


FLOW CAPACITY

1/2 STREET SECTION

45' B-B

4-1-94 MDK

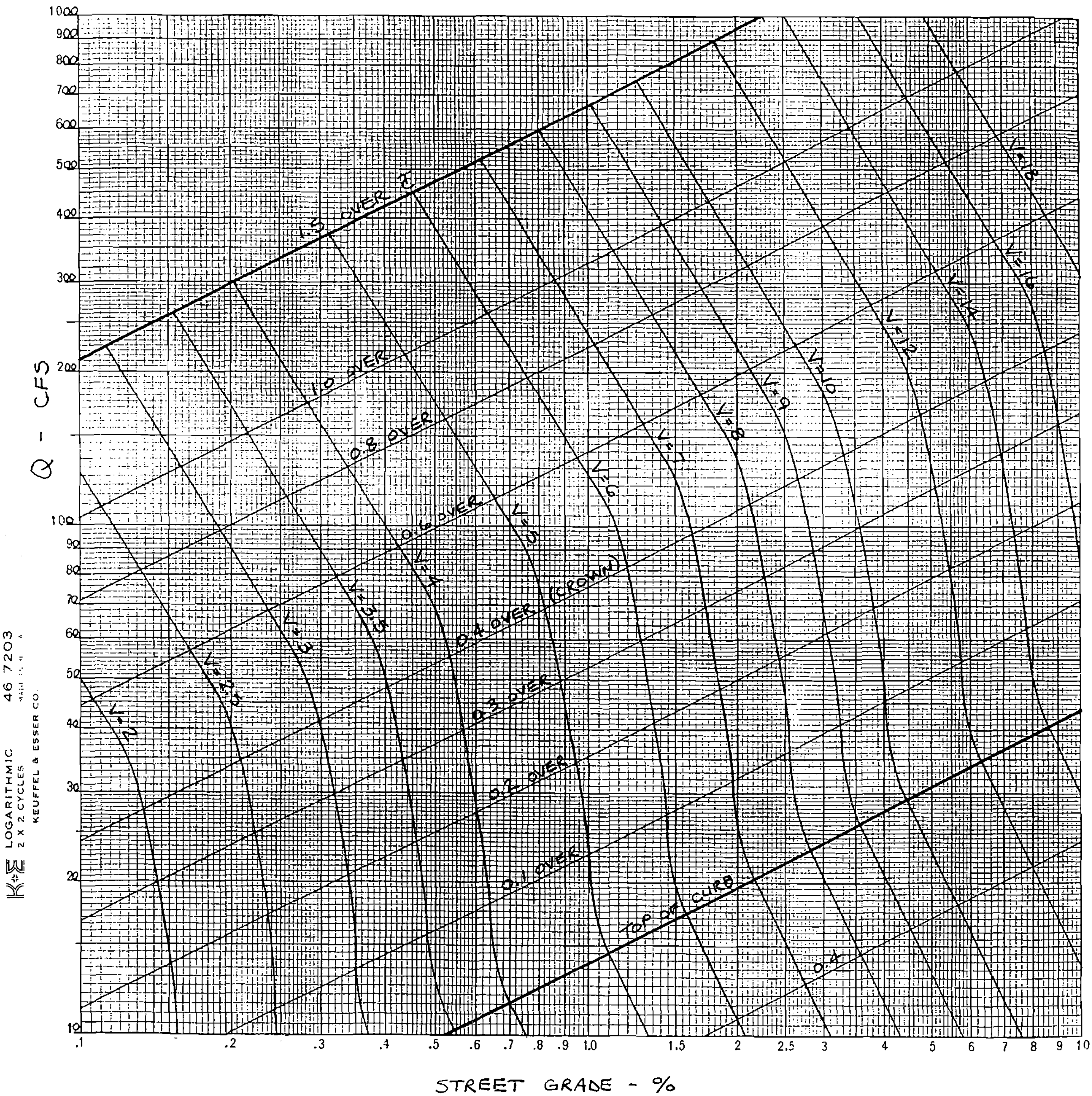


FLOW CAPACITY

1/2 STREET SECTION

65' B-B

4-1-94 MDK



FLOW CAPACITY

½ STREET SECTION

65' B-B

4-1-94 MDK

SECTION 4
STORM INLETS

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4.1 INTRODUCTION

The purpose of this section is to consider the significance of the hydraulic elements of storm inlets and their appurtenances to a storm drainage system. The hydraulic capacity of a gutter inlet depends upon its geometry and upon the characteristics of the gutter flow. The inlet capacity governs both the rate of water removed from the gutter and the amount of water that can enter the storm sewer system. Many costly storm sewers flow at less than design capacity because the storm runoff cannot get into the sewers. Inadequate inlet capacity or poor inlet location may cause flooding on the traveled way and create a safety hazard or interrupt traffic.

The inlet is frequently located in or near the path of vehicular traffic. Water-borne debris and trash may be deposited on the inlet causing complete or partial clogging. Often freedom from clogging and noninterference with traffic requires an inlet of a specific type rather than the most efficient inlet from a hydraulic point of view. For example, a curb-opening inlet might be used where a grate inlet would be more efficient.

4.2 INLET TYPES

Gutter inlets can be divided into the following three major classes, each with many variations: 1) curb-opening inlets; 2) gutter inlets, and; 3) combination inlets (see Figure 4-1).

Each type of inlet shall be installed with a depression of the gutter and may be a single or a multiple inlet (two or more closely spaced inlets acting as a unit). Two identical units placed end to end are called double inlets.

A brief description of the inlet types follow:

A. Curb-opening inlets. These inlets consist of a vertical opening in the curb through which the gutter flow passes.

B. Gutter inlets. These inlets consist of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe out along the longitudinal axis with a grate of spacer bars to form slot openings.

C. Combination inlets. These units consist of both a curb-opening and a grate inlet acting as a unit.

In mathematical form, efficiency, E, is defined by the following equation:

$$E = \frac{Q_i}{Q} \tag{4-1}$$

where:

E = efficiency of inlet

Q_i = intercepted flow by inlet, in cubic feet per second

Q = total gutter flow, in cubic feet per second

The discharge that bypasses the inlet, Q_c, is termed carry-over or bypass. The interception capacity of all inlet configurations increase with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates.

Factors affecting gutter flow also affect inlet interception capacity. The depth of water in the gutter immediately adjacent to the opening is the major factor in the interception capacity of both gutter inlets and curb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter.

Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and curb-opening length. Curb-opening inlet interception capacity and efficiency are increased by the use of a gutter depression at the curb-opening or a depressed gutter to increase the proportion of the total flow adjacent to the curb. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet.

Slotted inlets function in essentially the same manner as curb-opening inlets, i.e., as weirs with flow entering from the side. Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on the flow depth, inlet length and total gutter flow.

The interception capacity of a combination inlet consisting of a grate placed alongside a curb opening does not differ materially from that of a grate. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. A combination inlet, consisting of a curb-opening inlet placed upstream of a grate, has a capacity equal to that of the curb-opening length upstream of the grate plus that of the grate, taking into account the reduced spread and depth of flow over the grate because of the interception by the curb-opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

A combination inlet consisting of a slotted inlet upstream of a grate might appear to have advantages when 100 percent interception is necessary. However, grates intercept little more than frontal flow and would usually need to be more than 3 feet wide to contribute significantly to the interception capacity of the combination inlet. A more practical solution would be to use a slotted inlet of sufficient length to intercept total flow.

Most investigators have pointed out that the capacity of an inlet is greatly increased by allowing a small percentage of the flow to bypass the inlet. For a given gutter discharge, the catch of each additional increment of width (grate inlets) or length (curb-opening inlets) becomes rapidly less. Thus, the cost of catching the small amount of flow near the thin edge of the triangular flow channel approaches the cost of catching the gutter amount flowing nearer to the curb.

All types of inlets, including curb-opening inlets, are subject to clogging. Attempts to simulate clogging tendencies in the laboratory have not been successful, except to demonstrate the importance of parallel bar spacing in debris handling efficiency. Grates with wider spacings of longitudinal bars pass debris more efficiently. Problems with clogging are largely local since the amount of debris varies significantly from one neighborhood to another. Some neighborhoods may contend with only a small amount of debris while others experience extensive clogging of drainage inlets. Clogging should be considered in the design of storm inlets where debris is a problem. The total area of the inlet should be adjusted to account for clogging.

4.3 CURB-OPENING INLETS

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb-openings are relatively free of clogging tendencies and offer little interference to traffic operation.

The curb-opening inlet discussed in this section is illustrated in Figure 4-2. It has a depression beginning W feet away from the curb and dropping 1 inch per foot below the plane of the pavement. Transitions at the two ends extend W feet from the end of the opening. The equations given apply only if the cross section of the street has a uniform slope to the face of the curb.

A standard curb opening inlet in Amarillo is an inclined throat as shown in Figure 4-3.

Capacity of Curb-Opening Inlets on Grade

The ratio of frontal flow to total gutter flow, E_o, for a straight cross slope is expressed by Equation 4-2 or Figure 4-4 for either straight cross slopes or depressed gutter sections.

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67} \tag{4-2}$$

where:

E_o	=	ratio of frontal flow to total gutter flow, Figure 4-4
Q_w	=	flow in width W, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second
W	=	width of depressed gutter or grate, in feet
T	=	total spread of water in the gutter, in feet

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by Equation 4-3 or Figure 4-5:

$$L_T = 0.6\ Q^{0.42} S^{0.3} \left[\frac{1}{nS_x} \right]^{0.6} \qquad (4\text{-}3)$$

where:

L_T	=	curb-opening length required for 100% interception, in feet, Figure 4-5
Q	=	flow in gutter at inlet, in cubic feet per second
S	=	longitudinal gutter slope, in feet per foot
n	=	Manning's roughness coefficient for pavement
S_x	=	cross slope of pavement, in feet per foot

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by the following equation or Figure 4-6.

$$E = 1 - \left[1 - \frac{L_i}{L_t} \right]^{1.8} \qquad (4\text{-}4)$$

where:

E	=	efficiency of inlet or percentage of interception, Figure 4-6
L_i	=	curb-opening length, in feet
L_T	=	curb-opening length required for 100% interception, in feet, Figure 4-5

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in Equation 4-5.

$$S_e = S_x + S_w^{'} E_o \qquad (4\text{-}5)$$

where:

S_e	=	equivalent cross slope, in feet per foot
S_x	=	cross slope of pavement, in feet per foot
S'_w	=	cross slope of the gutter measured from the cross slope of the pavement, S_x
	=	(a/12W)

a	=	gutter depression, in inches
W	=	width of gutter, in feet
E_o	=	ratio of frontal flow to total gutter flow, Figure 4-6

It is apparent from examination of Figure 4-5 that the length of curb-opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e, Equation 4-3 becomes:

$$L_T = 0.6\, Q^{0.42}\, S^{0.3} \left[\frac{1}{nS_e}\right]^{0.6} \tag{4-6}$$

Equation 4-6 is applicable with either straight cross slopes or compound cross slopes. Figures 4-5 and 4-6 are applicable to depressed curb-opening inlets using S_e rather than S_x.

The interception of a standard length depressed curb-opening inlet is expressed by using Equation 4-1 and solving for Q_i.

The length of inlet required for a specific interception is expressed by rearranging Equation 4-4.

$$L_i = L_T - L_T\,(1 - E)^{0.55} \tag{4-7}$$

where:

L_i	=	curb-opening length required to intercept a specific % of the flow in the gutter, in feet
L_T	=	curb-opening length required for 100% interception, in feet, Figure 4-5
E	=	efficiency of inlet or percentage of interception, Figure 4-6

Example 1 Interception of a Curb-Opening Inlet

Given: Discharge in street of 5 cfs; pavement cross slope, S_x = 0.02 ft/ft; longitudinal slope, S = 0.005 ft/ft, pavement n = 0.016; gutter depression, a = 2 inches = 0.17 ft; and gutter width, W = 2 feet.

Find:

1) Q_i for a 10-ft curb-opening inlet
2) Q_i for a depressed 10-ft curb-opening inlet

Solution:

1) Q_i for a 10 ft curb-opening inlet

a) Find the depth from Figure 3-2 and calculate the spread

$y = 0.3$ ft
$T = Zy = 50\ (0.3) = 15$ ft

b) Find the curb-opening length for 100% interception from Figure 4-5

$L_T = 50$ ft

c) Find curb-opening efficiency from Figure 4-6

$L/L_T = 10/50 = 0.20$
$E = 0.33$

d) Using Equation 4-1, calculate the flow intercepted

$$\begin{aligned} Q_i &= EQ \\ &= 0.33(5) \\ &= 1.6 \text{ cfs} \end{aligned}$$

2) Q_i for a depressed 10 ft curb-opening inlet. Find the spread on the pavement. Use Figure 3-3.

a)

$$\begin{aligned} Qn &= 5(0.016) \\ &= 0.08 \text{ cfs} \end{aligned}$$

$$\begin{aligned} S_w &= S_x + a/w \\ &= 0.02 + 0.17/2 \\ &= 0.11 \\ S_w/S_x &= 0.11/0.02 = 5.5 \end{aligned}$$

From Figure 3-3:

$$\begin{aligned} T/W &= 4.9 \\ T &= 4.9W \\ &= 4.9(2.0) \\ &= 9.8 \text{ ft} \end{aligned}$$

b) Find the curb-opening length required for 100% interception
Find E_o, from Figure 4-4.

$$W/T = 2.0/9.8 = 0.20$$
$$S_e = S_x + S'_w E_o$$
$$= 0.02 + 0.083(0.62)$$
$$= 0.07$$

From Figure 4-5:

$$L_T = 30 \text{ ft}$$

c) Find the curb-opening efficiency

$$L/L_T = 10/30 = 0.33$$

From Figure 4-6:

$$E = 0.52$$

d) Calculate the flow intercepted by Equation 4-1

$$Q_i = QE$$
$$= 5.0(0.52)$$
$$= 2.6 \text{ cfs}$$

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb-opening and over 50 percent of the total flow.

Capacity of Curb-Opening Inlets in a Sump

The capacity of a curb-opening inlet in a sump depends on water depth at the curb, the curb-opening length, and the height of the curb-opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage. The flow in this range can be approximated by solving for the 1.0 and 1.4 values and interpolating.

Spread on the pavement is the usual criterion for judging the adequacy of pavement drainage inlet design. It is also convenient and practical in the laboratory to measure depth at the curb upstream of the inlet at the point of maximum spread on the pavement. Therefore, depth at the curb measurements from experiments coincide with the depth at curb of interest to designers. The weir coefficient for a curb-opening inlet is less than the usual weir coefficient for several reasons, the most obvious of which is that depth measurements from experimental tests were not taken at the weir, and drawdown occurs between the point where measurements were made and the weir.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb-opening. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb-opening, and its length is equal to that of the inlet. Limited experiments and extrapolation of the results of tests on depressed inlets indicate that the weir coefficient for curb-opening inlets without depression is approximately equal to that for a depressed curb-opening inlet.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = 2.3(L + 1.8W)\ d^{1.5} \qquad (4\text{-}8)$$

where:

Q_i	=	intercepted flow by inlet, in cubic feet per second
L	=	length of curb opening, in feet
W	=	lateral width of depression, in feet
d	=	depth at curb measured from the normal cross slope, in feet
	=	TS_x

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation 4-8 for a depressed curb-opening inlet is:

$$d \leq h + a/12$$

where:

d	=	depth at curb measured from the normal cross slope, in feet
h	=	height of curb-opening inlet, in feet
a	=	depth of depression, in inches

The weir equation for curb-opening inlets without a depression (W = 0) becomes Equation 4-9. The depth limitation for operation as a weir occurs when $d \leq h$. The terms of the equation are the same as identified with Equation 4-8.

$$Q_i = 2.3\ Ld^{1.5} \qquad (4\text{-}9)$$

Curb-opening inlets operate as orifices at depths greater than approximately 1.4h. The interception capacity can be computed by the following equation:

$$Q_i = 0.67hL(2gd_o)^{0.5} = 0.67A[2g(d_i - h/2)]^{0.5} \tag{4-10}$$

where:

Q_i	=	intercepted flow by inlet, in cubic feet per second, Figure 4-7, Figure 4-8 or 4-9
h	=	height of curb-opening inlet, in feet
d_o	=	effective head on the center of the orifice throat, in feet
A	=	clear area of opening, in square feet
d_i	=	depth of lip of curb-opening, in feet
h	=	height of curb-opening orifice, in feet
	=	$TS_x + a/12$

Equation 4-10 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression. Figure 4-7 is a graphical solution for a depressed curb-opening inlet, and Figure 4-8 provides solutions for undepressed curb-opening inlets. Figure 4-9 is used for curb-openings with nonvertical orifice throats.

The height of the orifice in Equation 4-10 assumes a vertical orifice opening. As illustrated in Figure 4-10, other orifice throat locations can change the effective depth on the orifice and the dimension (d_i - h/2). A limited throat width could reduce the capacity of the curb-opening inlet by causing the inlet to go into orifice flow at depths less than the height of the opening.

The orifice equation for curb-opening inlets with a horizontal or inclined throat is:

$$Q = 0.67hL(2gd_0)^{0.5} \tag{4-11}$$

where:

Q	=	flow in gutter at inlet, in cubic feet per second
h	=	orifice throat width, in feet
L	=	length of curb-opening, in feet

g = acceleration of gravity, 32.2 feet per second squared

d_o = effective head on the center of the orifice throat, in feet

Example 2 Curb Opening Inlet in a Sump Location

Given: Curb-opening inlet in a sump location, having a curb-opening length $L = 5.0$ ft; and an orifice throat $h = 5.0$ inches.

1) Undepressed curb-opening

S_x = 0.05 ft/ft
T = 8 ft

2) Depressed curb-opening

S_x = 0.05 ft/ft
a = 2.0 in = 0.167 ft
W = 2.0 ft
T = 8.0 ft

Find: Q_i

Solution:

1) a) Calculate the depth of flow at the curb

$d = TS_x = 8.0\ (0.05) = 0.4$ ft

b) Find the flow intercepted by the curb-opening inlet
$d < h$, from Figure 4-8.

$Q_i = 3.8$ cfs

2) a) Depth of flow is the same as (1) above

$d = 0.4 \text{ ft} < (h + a/12)$

b) Find the flow intercepted by the inlet

$P = L + 1.8W = 5.0 + 1.8(2.0) = 8.6$ ft

From Figure 4-7, Q_i= 5.0 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30 percent more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

Example 3 Curb Opening Inlet in a Sump location

Given: A curb-opening inlet in a sump having the following characteristics:

S_x = 0.03 ft/ft
n = 0.016
W = 1 ft
a = 1 in = 0.17 ft
h = 6 in = 0.5 ft

Discharge from both sides of inlet

Q_1 = 2 cfs, Q_2 = 8 cfs
Q_{tot} = 10 cfs

Find: Maximum depth of ponding, d_{max} for

L = 5 ft
L = 10 ft
L = 15 ft

Solution:

1) Use Figure 4-7 for the maximum ponding.
$P = L + 1.8W$; $A = hL$; $T = d_{max}/S_x$

L	15 ft	10 ft	5 ft
P	16.8 ft	11.8 ft	6.8 ft
d_{max}	0.40 ft	0.50 ft	0.70 ft
T	13.3 ft	16.7 ft	23.3 ft

2) The maximum depth of ponding at the curb opening may be exceeded in the approach gutter, particularly on low flows. The depth of ponding in the gutter can be checked by using Figure 4-7.

a) For L = 15 ft, Q_1 = 2 cfs, d_{max} = 0.40 ft (Step 1), and d = 0.14 ft

The gutter depth for Q_1 is less than the ponding depth at the inlet and water will back up in the gutter channel.

b) For Q_2 = 8 cfs, d_{max} = 0.40 ft (Step 1), and d = 0.38 ft (Figure 4-7).

The gutter depth for Q_2 is greater than the ponding depth at the inlet and the water profile tends to draw down on approaching the inlet.

c) For Q_2 = 8 cfs and L = 10 ft and 5 ft, d_{max} = 0.50 or 0.70 ft (Step 1), and d = 0.45 or 0.65 ft

The gutter depth for both Q_1 and Q_2 is less than the ponding depth for both 5- and 10-foot length inlets; therefore, water will back up in the gutter on both sides of the inlet.

In addition to illustrating the use of the sag curves, the example shows the necessity of picking up most of the gutter flow before it reaches the low point of the sag vertical curve. Spreads on the pavement (T) and depths at the curb (d_{max}) noted in Step 1 could not be tolerated on a high speed highway. The more common application of the sag curves would be in designing curb-opening inlets or their spacing to keep the depth of ponding and spread on the pavement within tolerable limits.

4.4 INLET LOCATION

In general, inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations, inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the crosswalk. Where pavement surfaces are warped, as at cross streets, ramps, or in transitions between superelevated and normal sections, gutter flow should be picked up before the change in the pavement begins to change in order to lessen water flowing across the roadway and to prevent icing.

In a sag vertical curve, three inlets are desirable. On major streets and arterials, three inlets along sag vertical curve with flat grades should be used; one at the low point and one on each side of this point where the grade elevation is at least 0.2 feet higher than that at the low point. The additional inlets furnish added capacity to allow for flow bypassing the upgrade inlets and provide a safety factor if the sag inlet becomes clogged. These inlets limit the deposition of sediment on the road in the sag and they also reduce flow arriving at the low point and thereby prevent ponding on the road.

Where a curbed roadway crosses a bridge, the gutter flow should be intercepted and not be permitted to flow onto the bridge.

4.4.1 Spacing of Inlets on Grade

Inlets should be spaced so as to limit the spread of the water on the pavement to the criterion outlined in Section 3.3.

With the maximum spread fixed and with a given pavement cross slope and longitudinal slope, the flow in the gutter is also fixed and can be calculated as explained in Section 3. The spacing of inlets is equal to the length of pavement needed to generate the discharge corresponding to the allowable spread on the pavement. The flow bypassing each inlet must be included in the flow arriving at the next inlet.

An example of the computations for inlet spacing for a curb opening inlet follows:

Example 4 Curb-Opening Inlets

Given: An arterial 4-lane section with 2' curb-and-gutter section (n = 0.016) (widths = 4(12) + 2(2) = 52'). The pavement cross slope, S_x = 0.02 ft/ft, longitudinal slope, S = 0.005 ft/ft; composite C for pavement and shoulder = 0.75. The contributing area is 105' to each side of the centerline. Design storm is the 2 year event, i = 2.0 in/hr. Flow spread must leave at least one lane in each duration free of water, T = 14 ft.

Find: Maximum design inlet spacing for a 10 foot curb-opening, depressed 2 inches from the normal cross slope in a 2 foot wide gutter.

Solution:

1) Q = CiA = 0.75 (2) (2)(105) x L/43,560 = 0.007L

2) Compute the discharge for the composite section from Figure 3-3.

T = 14 ft; W = 2 ft
T/W = 14/2 = 7
$S_w = S_x + a/W = 0.02 + 0.17/2 = 0.10$
$S_w/S_x = 0.10/0.02 = 5.0$

From Figure 3-3:

Qn = 0.09
Q = 0.09/0.016 = 5.6 cfs at initial inlet

3) Compute the location of the first inlet.

$$L = \frac{Q}{0.007} = \frac{5.6}{0.007} = 805 \text{ ft}$$

4) From Figure 4-4, find the ratio of gutter flow to total flow.

W/T = 2/14 = 0.14
E_o = 0.34

5) $S_e = S_x + S'_w E_o$
$= 0.02 + 0.083(0.34) = 0.05$

6) From Figure 4-5:

$L_T = 32$
$L/L_T = 10/32 = 0.32$

7) From Figure 4-6:

$E = 0.50$

8) $Q_i = 5.6(0.5)$
$= 2.8$ cfs

9) $Q_c = 5.6 - 2.8 = 2.8$ cfs

10) Assuming that the drainage area between inlets contributes runoff equal to the interception capacity of the inlet, calculate the spacing of successive inlets.

$L = 2.5/0.0207 = 121$ ft

4.4.2 Spacing of Inlets in a Sump

Three inlets should be placed in a sag vertical curve on all major streets, one at the low point and one on each side of this point, where the grade elevation is approximately 0.2 feet higher than that at the low point. The inlets should be spaced so as to limit the spread of water on the pavement to the criterion outlined in Section 3.3.

Sag vertical curves differ one from another in the potential for ponding, and criteria adopted for inlet spacing in sags should be applied only where traffic could be unduly disrupted if an inlet became clogged or runoff from the design storm were exceeded. Therefore, criteria adopted for inlet spacing in sag vertical curves are not applicable to the sag curve between two positive or two negative longitudinal slopes. Also, they should not be applied to locations where ponding depths could not exceed curb height and ponding widths would not be unduly disruptive, as in sag locations on embankment.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded. Table 4-1 shows the spacing required for various depth at curb criteria and vertical curve lengths defined by the following dimensionless coefficient:

$$K = \frac{L}{A} \tag{4-12}$$

where:

K	=	dimensionless coefficient
L	=	length of vertical curve, in feet
A	=	algebraic difference in approach grades (G_2 - G_1)

The AASHTO policy on geometrics specifies maximum K values for various design speeds, shown in Table 4-1.

TABLE 4-1 Distance* to Flanking Inlets in Sag Vertical Curve Locations

		Depth at Curb (inches)							
Speed (mph)	**"K" L/A**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**
20	20	20	28	35	40	45	49	53	57
25	30	24	35	42	49	55	60	65	69
30	40	28	40	49	57	63	69	75	80
35	50	32	45	55	63	71	77	84	89
40	70	37	53	65	75	84	92	99	106
45	90	42	60	73	85	95	104	112	120
50	110	47	66	81	94	105	115	124	133
55	130	51	72	88	102	114	125	135	144
60	160	57	80	98	113	126	139	150	160
	167**	58	82	100	116	129	142	153	163
65	180	60	85	104	120	134	147	159	170
70	220	66	94	115	133	148	162	176	188

Note: * $X = (200dK)^{0.5}$; where X = distance from the low point.
** Maximum drainage K = 167

Example 5 Spacing of Inlet in a Sag

Given: A sag vertical curve at an underpass on a 4-lane divided highway facility. Spread at design Q is not to exceed shoulder width of 10 ft.

$S_x = 0.02$; $K = 130$

Find: Location of flanking inlets if located: (1) in relief of the inlet at the low point when depth at the curb exceeds design depth, and (2) when depth at the curb is 0.1 ft less than depth at design spread.

Solution:

Depth at the curb at design spread,
$d = TS_x = 10\ (0.02) = 0.2$ ft

1) From Table 4-1 spacing to flanking inlet = 72 ft

2) From Table 4-1, d - 0.2 ft = 0.2 - 0.1 = 0.1 ft

Spacing to flanking inlets = 51 ft

The purpose in providing Table 4-1 is to facilitate the selection of criteria for the location of flanking inlets based on the ponding potential at the site, the potential for clogging of the inlet at the low point, design spread, design speeds, traffic volumes, and other considerations which may be peculiar to the site under consideration. A depth at curb criterion which does not vary with these considerations neglects consideration of cross slope and design spread and may be unduly conservative at some locations. Location of flanking inlets at a fixed slope rate on the vertical curve also neglects consideration of speed facilities and not at all conservative for high speed facilities.

Except where inlets become clogged, spread on low gradient approaches to the low point is a more stringent criterion for design that the interception capacity of the sag inlet. AASHTO recommends that a gradient of 0.3 percent be maintained within 50 feet of the level point in order to provide for adequate drainage. It is considered advisable to use spread on the pavement at a gradient comparable to that recommended by the AASHTO Committee on Design to evaluate the location and design of inlets upgrade of sag vertical curves. Standard inlet design and/or location may need adjustment to avoid excessive spread in the sag curve.

4.5 OTHER INLET TYPES

4.5.1 Grate Inlets

Grate inlets will intercept all of the gutter flow passing over the grate, or the frontal flow, if the grate is sufficiently long and the gutter flow velocity is low. Only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

Capacity of Grate Inlets on Grade

The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{4-13}$$

where:

Q_s	=	ratio of side flow, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second
Q_w	=	flow in width W, in cubic feet per second
E_o	=	ratio of frontal to total gutter flow, Figure 4-4

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by Equation 4-14 or Figure 4-11 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. This ratio is equivalent to frontal flow interception efficiency.

$$R_f = 1 - 0.09 \,(V - V_o) \tag{4-14}$$

where:

R_f	=	frontal flow interception efficiency, Figure 4-11
V	=	velocity of flow in the gutter, in feet per second
	=	total gutter flow divided by area of flow, Figure 3-4
V_o	=	gutter velocity where splash-over first occurs, in feet per second

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by the following equation or Figure 4-12.

$$R_s = 1/\left[1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}\right] \tag{4-15}$$

where:

R_s	=	side flow interception efficiency, Figure 4-12
V	=	velocity of flow in the gutter, in feet per second, Figure 3-4
S_x	=	cross slope, in feet per foot
L	=	length of the grate, in feet

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s (1 - E_o) \tag{4-16}$$

where:

E	=	grate efficiency
R_f	=	frontal flow interception efficiency, Figure 4-11
E_o	=	ratio of frontal flow to total gutter flow, Figure 4-4
R_s	=	side flow interception efficiency, Figure 4-12

The first term on the right side of Equation 4-16 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\,[R_f E_o + R_s (1 - E_o)] \tag{4-17}$$

where:

Q_i	=	flow intercepted, in cubic feet per second
E	=	grate efficiency
Q	=	total gutter flow, in cubic feet per second
R_f	=	frontal flow interception efficiency, Figure 4-11
E_o	=	ratio of frontal flow to total gutter flow, Figure 4-4
R_s	=	side flow interception efficiency, Figure 4-12

Example 6　Interception Capacity of Grate Inlets

Given: Data from Example 1 in Section 3, Right gutter

Find: Interception capacity of:

1) a 2-ft long curved vane grate, and
2) a reticuline grate 2-ft long and 2-ft wide

Solution:

From Example 1:

W =2 ft; Gutter depression = 0.1 in = 0.10 ft; T = 22 ft; S = 0.003; S_x = 0.02; S_w = 0.05; E_o = 0.25 (Figure 4-4); Q = 11.0 cfs; V = 2.2 fps (Figure 3-4)

Find frontal flow and side flow interception efficiency, R_f and R_s, from Figures 4-11 and 4-12.

1) Both grates: $R_f = 1.0$
   Both grates: $R_s = 0.137$

2) Both grates: From Equation 4-16
   $Q_i = 11.0[(1.0 \times 0.25) + 0.137(1-0.25)] = 3.9$ cfs

Capacity of Grate Inlets in a Sump

A grate inlet in a sump operates first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Weir operation continues to a depth (d) of about 0.4 feet above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 3.0\, Pd^{1.5} \tag{4-18}$$

where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second
P	=	perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb
d	=	depth of water at grate, in feet

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 0.67A_g\,(2gd)^{0.5} = 5.37A_g d^{0.5} \tag{4-19}$$

where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second

A_g = clear opening of the grate, in square feet

g = acceleration of gravity, 32.2 feet per second squared

d = depth of ponded water above top of grate, in feet

Equations 4-18 and 4-19 are solved graphically with Figure 4-13, neglecting clogging for various grate sizes. If clogging can occur, the clearing opening or perimeter of a grate should be greater than that required by the equation in order to remain below the design depth over the grate. A clogging factor of 0.6 should be used in the equations. Between depths of about 0.4 feet and about 1.4 feet (over the grate), the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by the above equations. The capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

AASHTO requires that the geometry in a sag vertical be adhered and checked. AASHTO geometric policy recommends a gradient of 0.3 percent within 50 feet of the level point in a sag vertical curve.

Example 7 Interception Capacity of Grate Inlet in a Sump

Given: Grate inlet, with one side against the curb, allow for clogging of the grate. Flow from right and left of Q_1 = 3.6 cfs and Q_2 = 4.4 cfs: therefore, Q_{tot} = 8 cfs.

Find: Grate inlet size for design flow and depth at curb. Cross slope of pavement 0.05 ft/ft.

Solution:

1) Calculate design discharge, Q_i, based on 50% clogging.

$$Q = 8(1.5) = 12.0 \text{ cfs}$$

From Figure 4-13, a grate must have a perimeter of 12 feet to intercept 12 cfs at a depth of 0.5 ft.

2) Therefore, a 4 x 4, 2 x 6, or a 3 x 5 grate would meet the requirements of a 12-ft perimeter, clogged grate. Assume that the design is a double 3 x 5 ft grate.

3) Calculate the effective perimeter of the grates, for clogged conditions.

$$P = 1.5 + 5 + 5 + 1.5 = 13 \text{ ft}$$

4) From Figure 4-13, find the depth of water at the grate inlet.
d = 0.46 ft, which is less than 0.5 ft (OK).

5) AASHTO geometric policy needs to be checked T at S = 0.003 ft/ft for the design flow. From Figure 3-2, Z/n = 1,250 at Q_1 = 3.6 cfs, d = 0.41 ft, T = 8.2 ft and at Q_2 = 4.4 cfs, d = 0.44 ft, T = 8.9 ft.

A clogged double 2 x 3 ft grate is adequate to intercept the design storm at a spread which does not exceed design spread and AASHTO requirements are met. The tendency of grate inlets to clog may warrant a combination inlet or curb-opening inlet on the low gradient approaches.

4.5.2 Slotted Inlets

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Capacity of Slotted Inlet on Grade

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from tests of slotted inlets with slot widths ≥ 1.75-in indicates that the length of slotted inlet required for total interception can be computed by Equation 4-3. Figure 4-5 is therefore applicable for both curb-opening inlets and slotted inlets and Figure 4-6 can be used to obtain the inlet efficiency for the selected length of inlet.

Use of Figures 4-5 and 4-6 for slotted inlets is identical to their use for curb-opening inlets. It is much less expensive to add length to a slotted inlet to increase interception capacity than it is to add length to a curb-opening inlet.

Capacity of Slotted Inlets in a Sump

Slotted inlets in sump locations perform as weirs to depths of about 0.2 ft, dependent on slot width and length. At depths greater than about 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by Equation 4-20:

$$Q_i = 0.8LW(2gd)^{0.5} \tag{4-20}$$

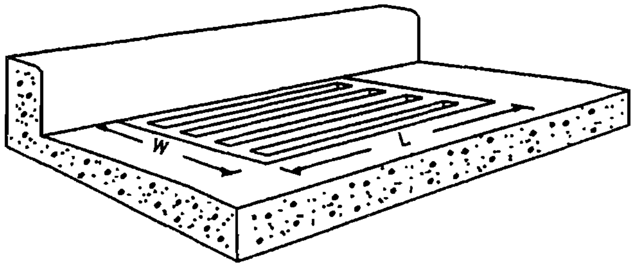
where:

Q_i	=	intercepted flow by inlet, in cubic feet per second
L	=	length of slot, in feet
W	=	width of slot, in feet
g	=	acceleration of gravity, 32.2 feet per second squared
d	=	depth of water at slot, in feet ≥ 0.4 ft

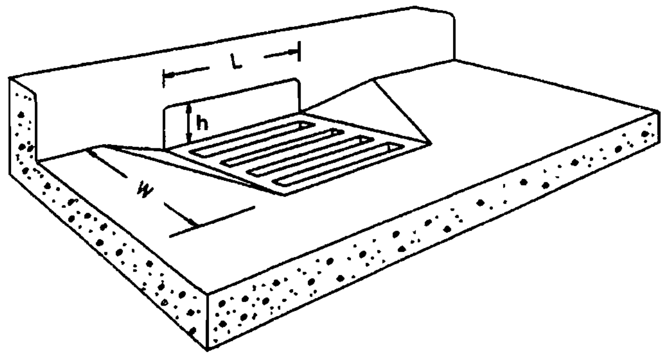
for a slot width of 1.75 in, Equation 4-20 becomes:

$$Q_i = 0.94Ld^{0.5} \tag{4-21}$$

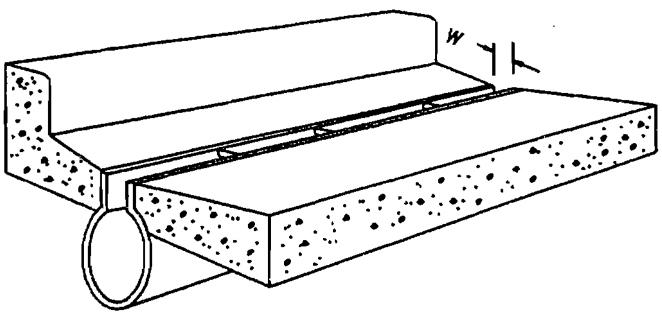
The interception capacity of slotted inlets at depths between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet. Figure 4-14 provides solutions for weir flow, Equation 4-21, and a plot representing data at depths between weir and orifice flow.



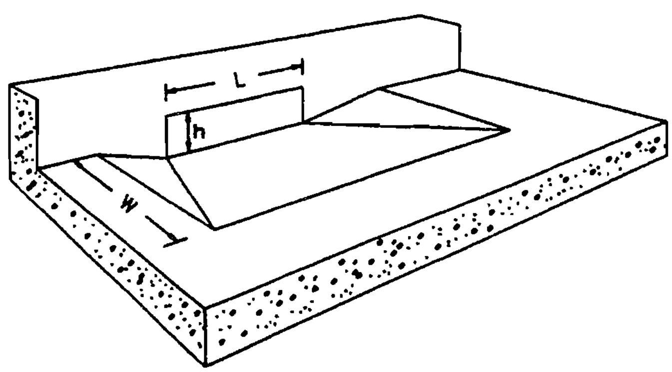
GRATE INLET



COMBINATION INLET



SLOTTED DRAIN INLET



CURB OPENING INLET

Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

Inlet Types

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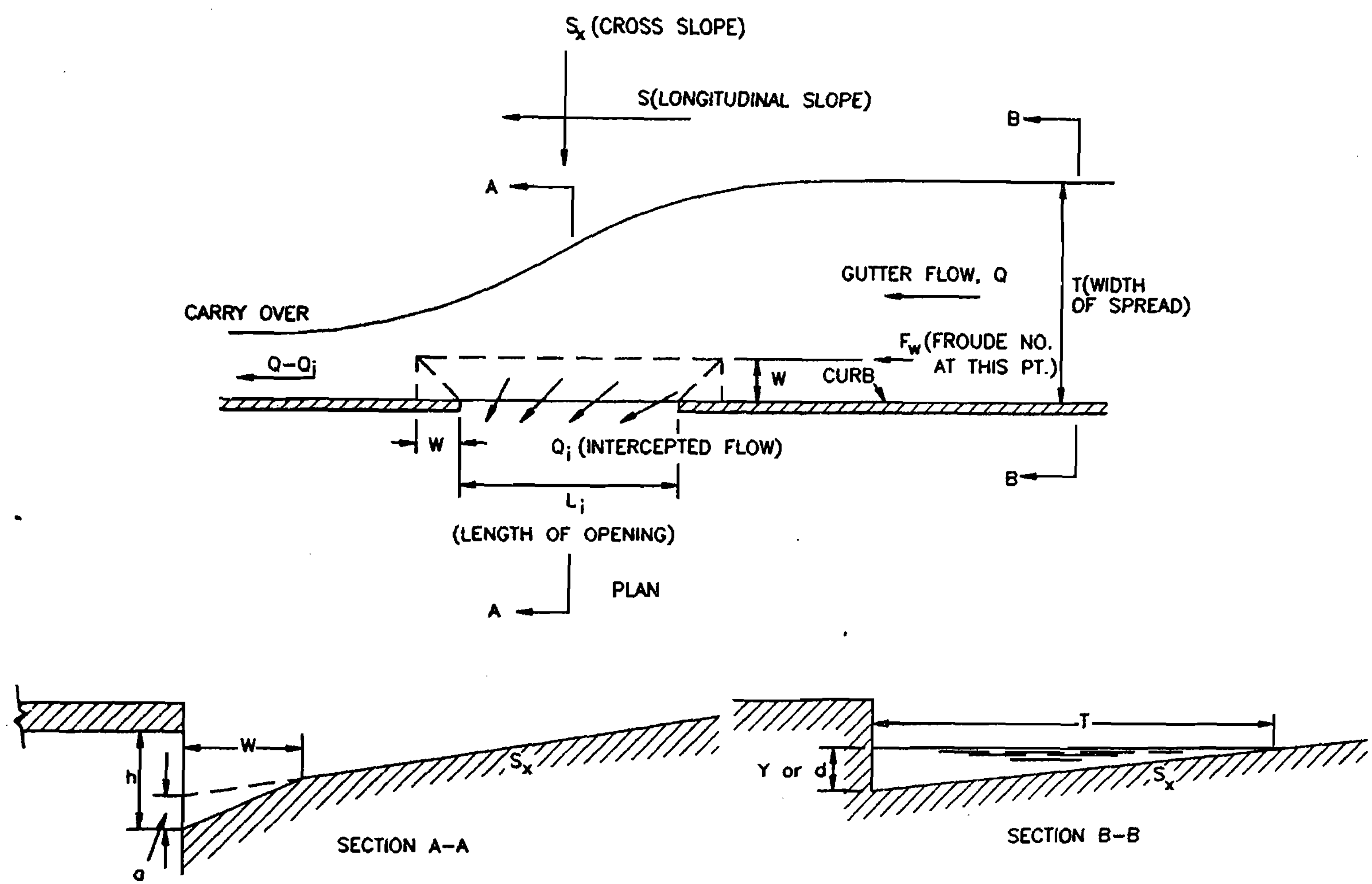
Figure
4-1



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Graphical Definition of Symbols

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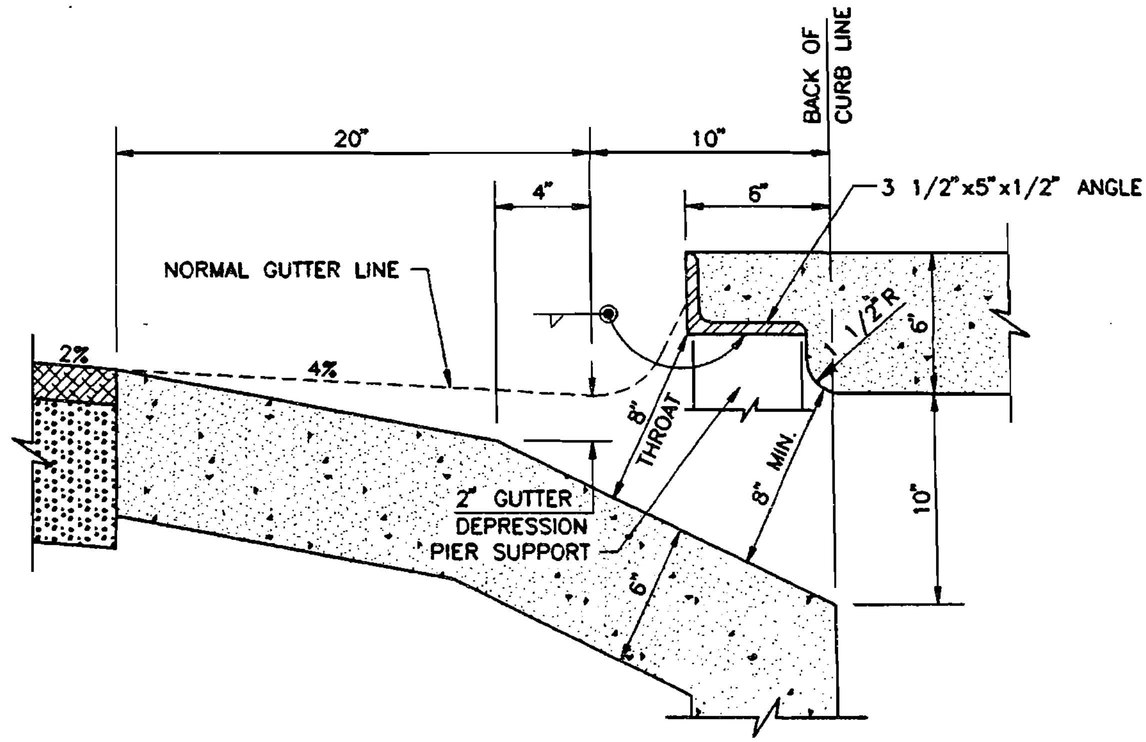
Figure 4-2



Source: City of Amarillo Public Works Division.

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Standard Curb-Opening Inlet
Amarillo, Texas

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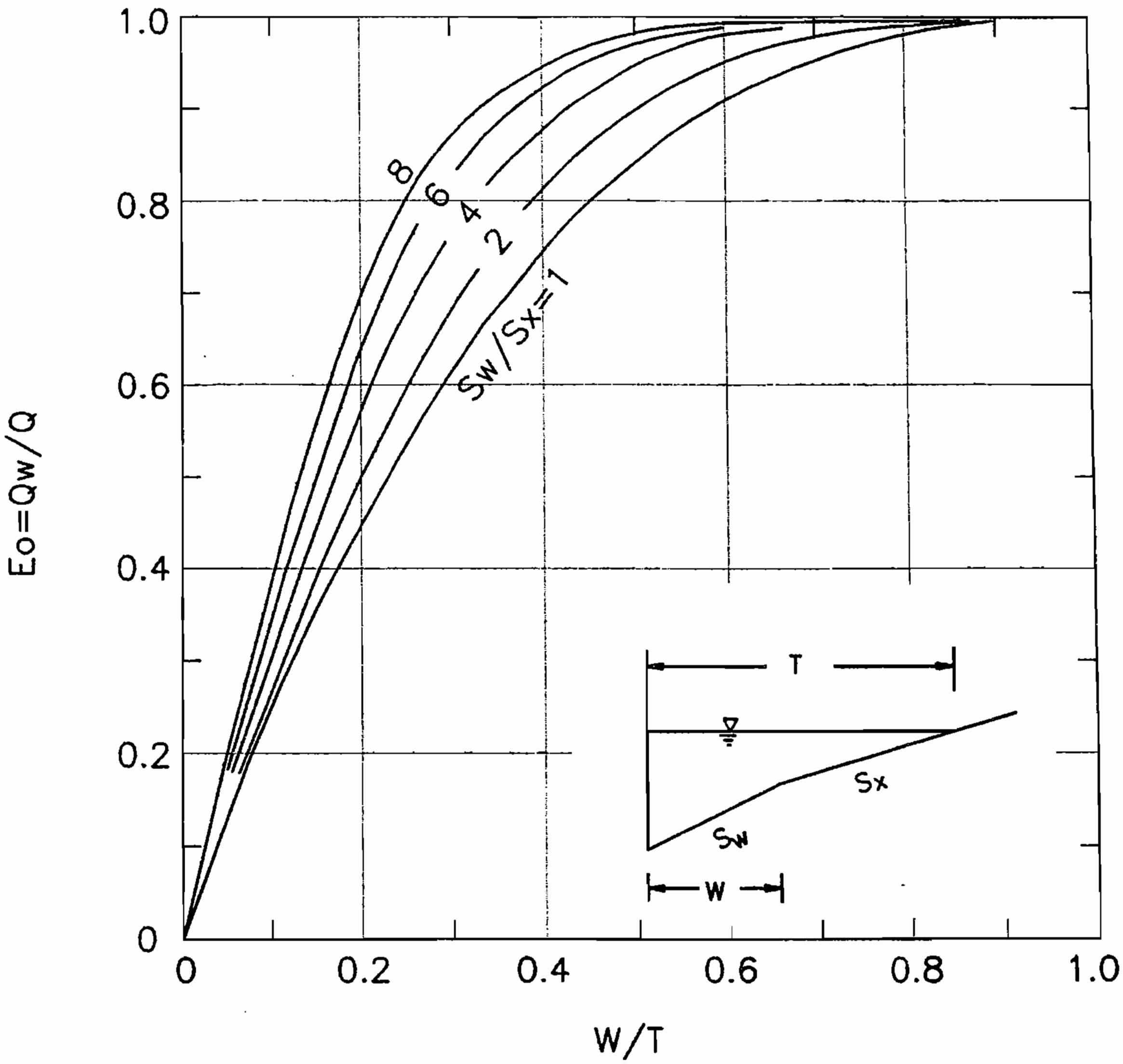
Figure	4-3



Source: *Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.*

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Ratio of Frontal Flow to Total Gutter Flow, E_o

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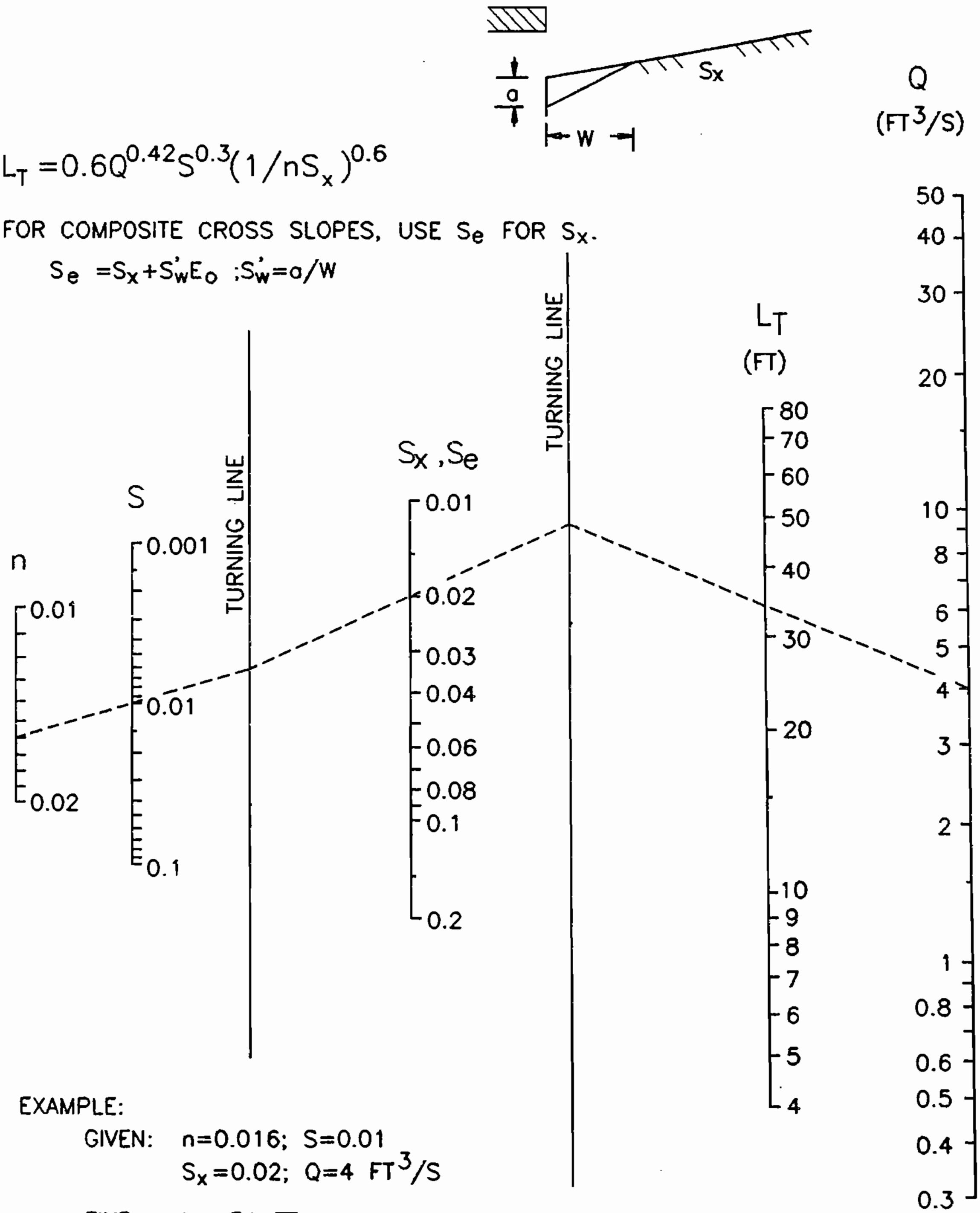
Figure 4-4

$$L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6}$$

FOR COMPOSITE CROSS SLOPES, USE S_e FOR S_x.

$S_e = S_x + S'_w E_o$; $S'_w = a/W$



EXAMPLE:

GIVEN: $n=0.016$; $S=0.01$
$S_x=0.02$; $Q=4$ FT^3/S

FIND: $L_T=34$ FT

Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

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Curb-Opening and Slotted Drain Inlet Length for Total Interception, L_t

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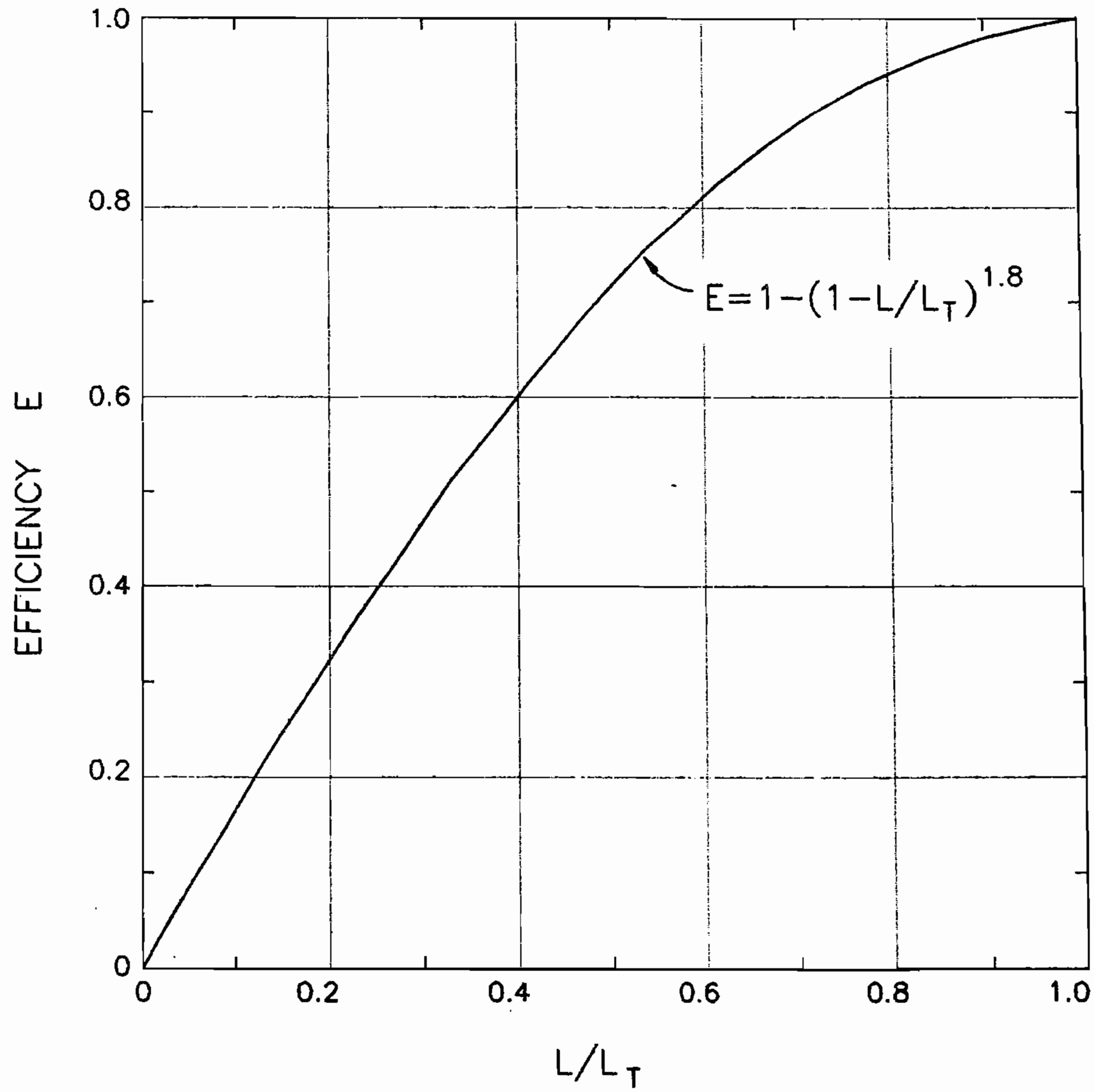
Figure
4-5



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

Curb-Opening and Slotted Drain Inlet Interception Efficiency, E

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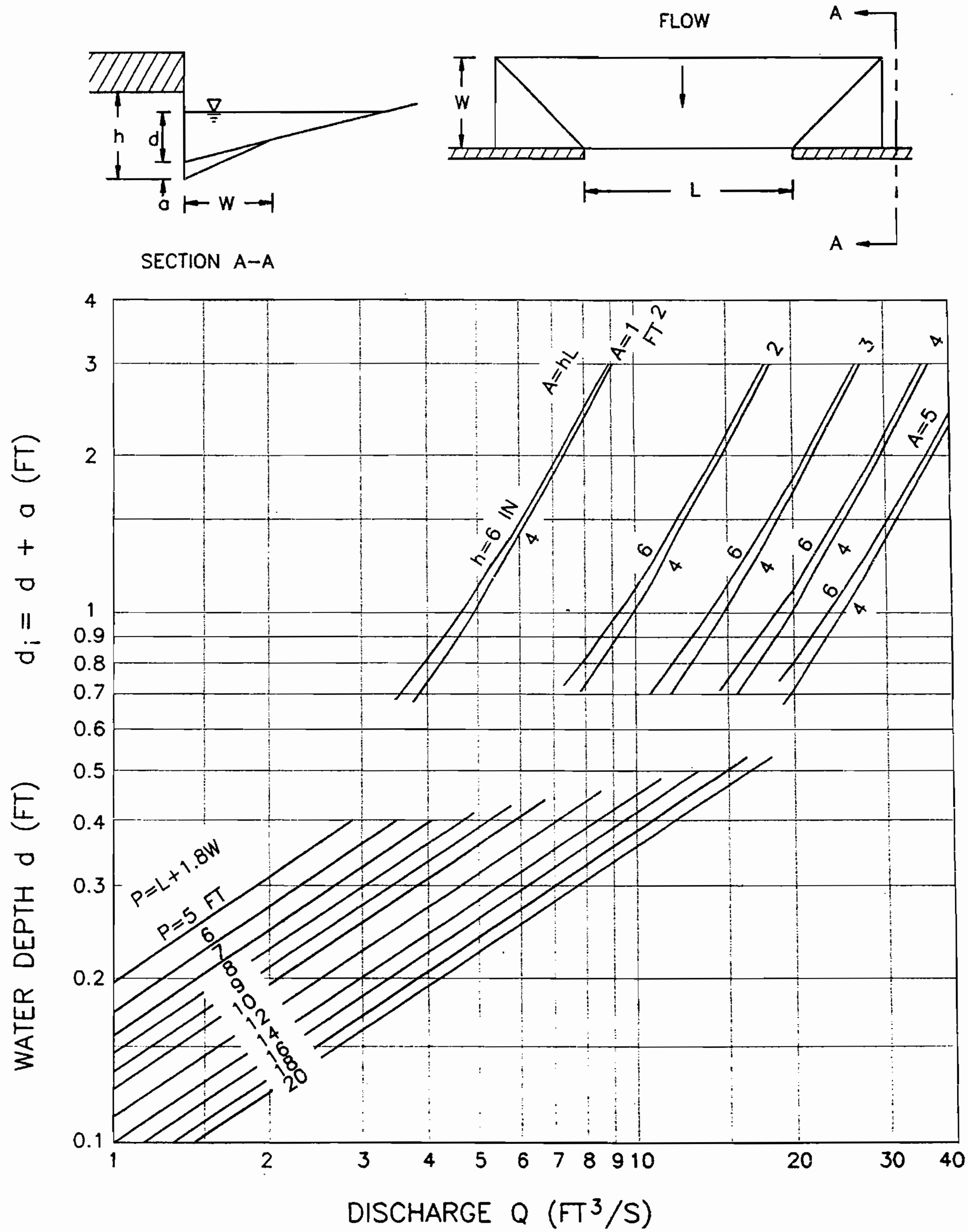
Figure
4-6



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

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Depressed Curb-Opening Inlet Capacity in Sump Locations

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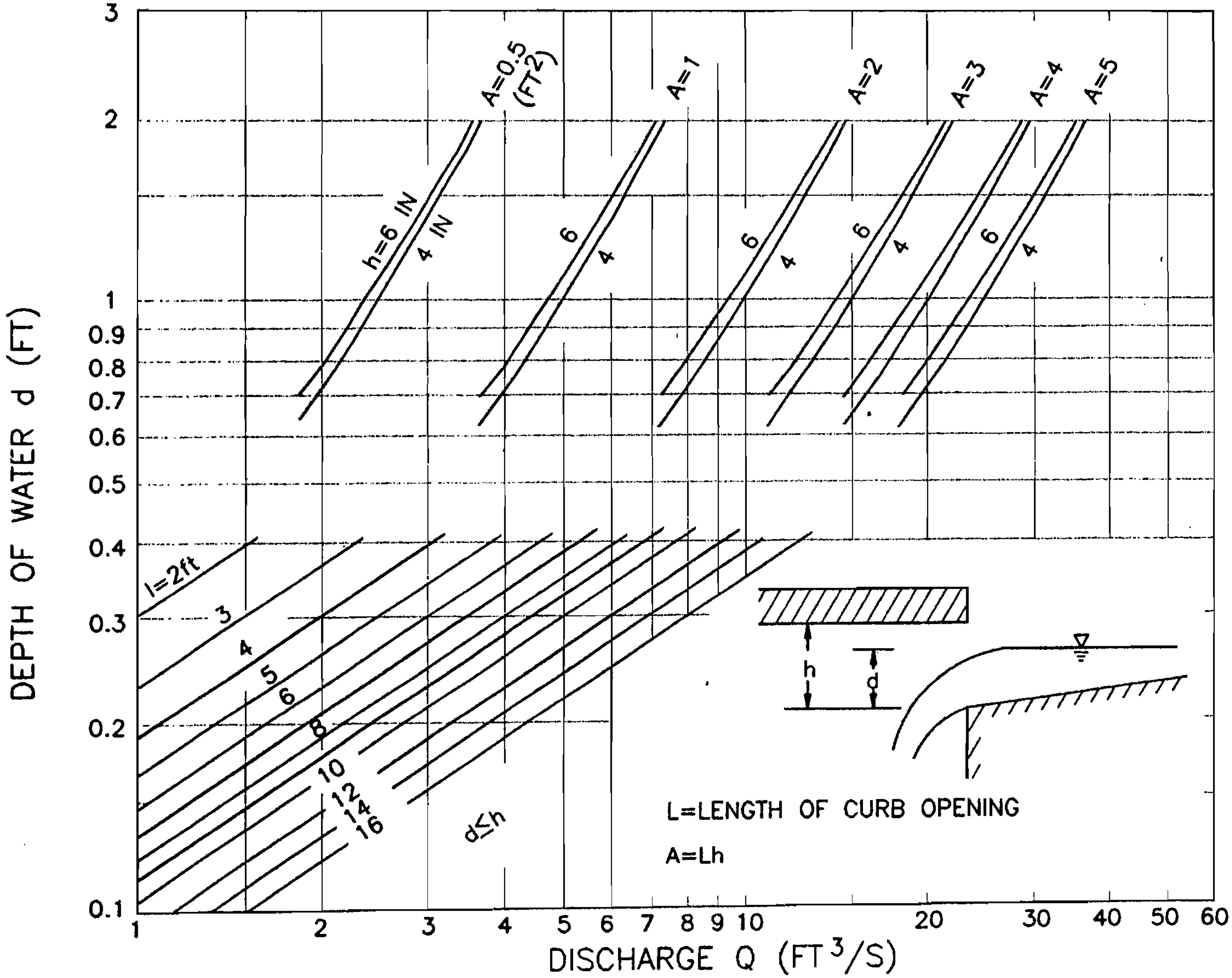
Figure 4-7



Source: Federal Highway Administration, HEC No. 12, <u>Drainage of Highway Pavements</u>.

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Curb-Opening Inlet Capacity in Sump Locations

S[illegible]m Water Management Criteria Manual

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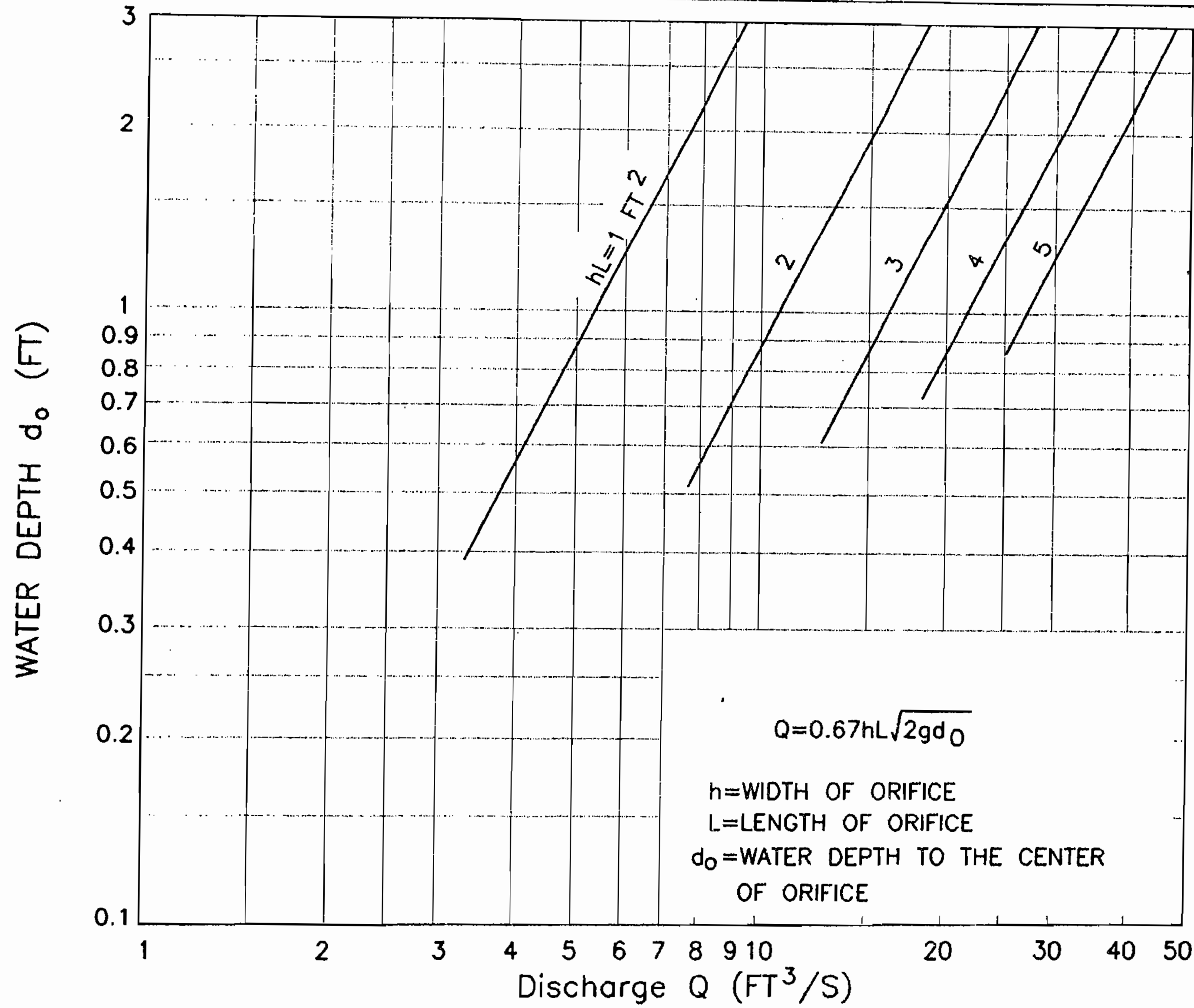
Figure 4-8



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

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Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

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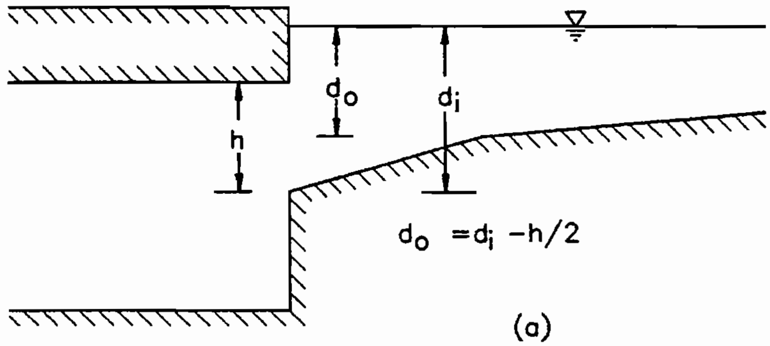
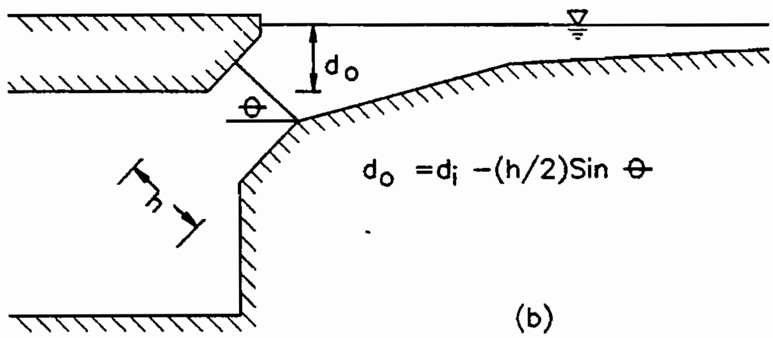
Figure: 4-9



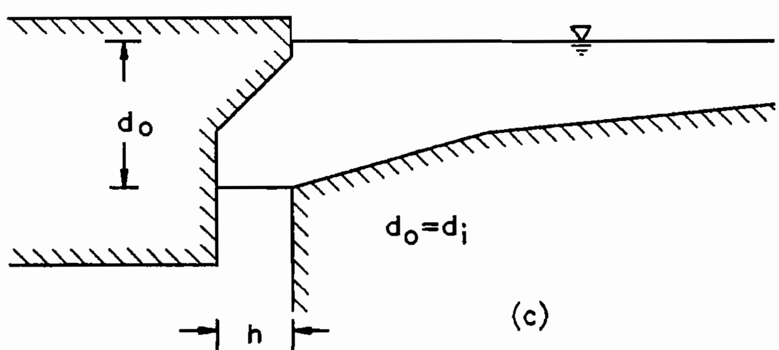
(a)
Horizontal throat

$Q = 0.67\ hL\sqrt{2gd_o}$

L=LENGTH OF OPENING



(b)
Inclined throat



(c)
Vertical throat

Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

Curb-Opening Inlets

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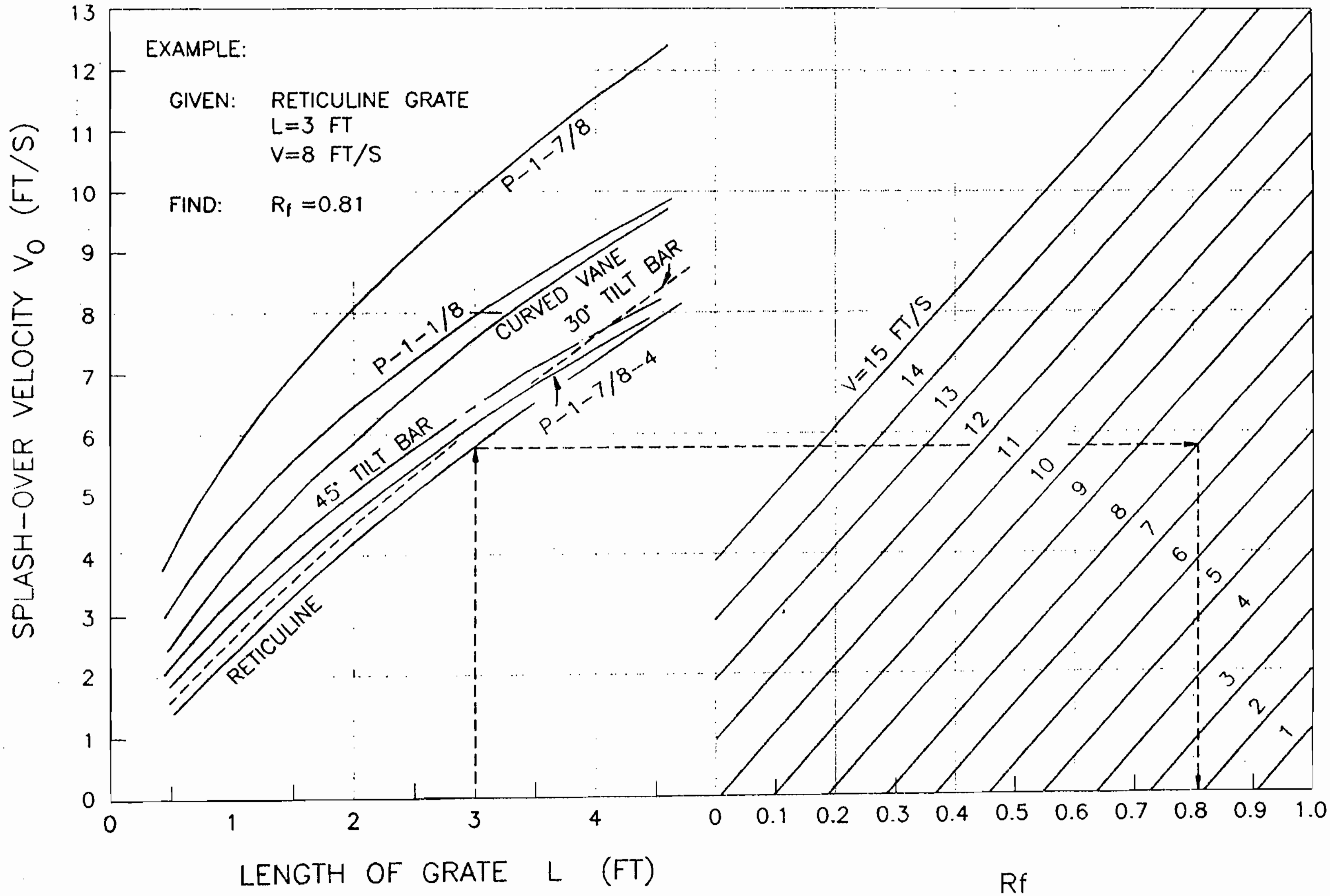
Figure
4-10



Source: Federal Highway Administration, HEC No. 12, <u>Drainage of Highway Pavements</u>.

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Grate Inlet Frontal Flow Interception Efficiency, R_f

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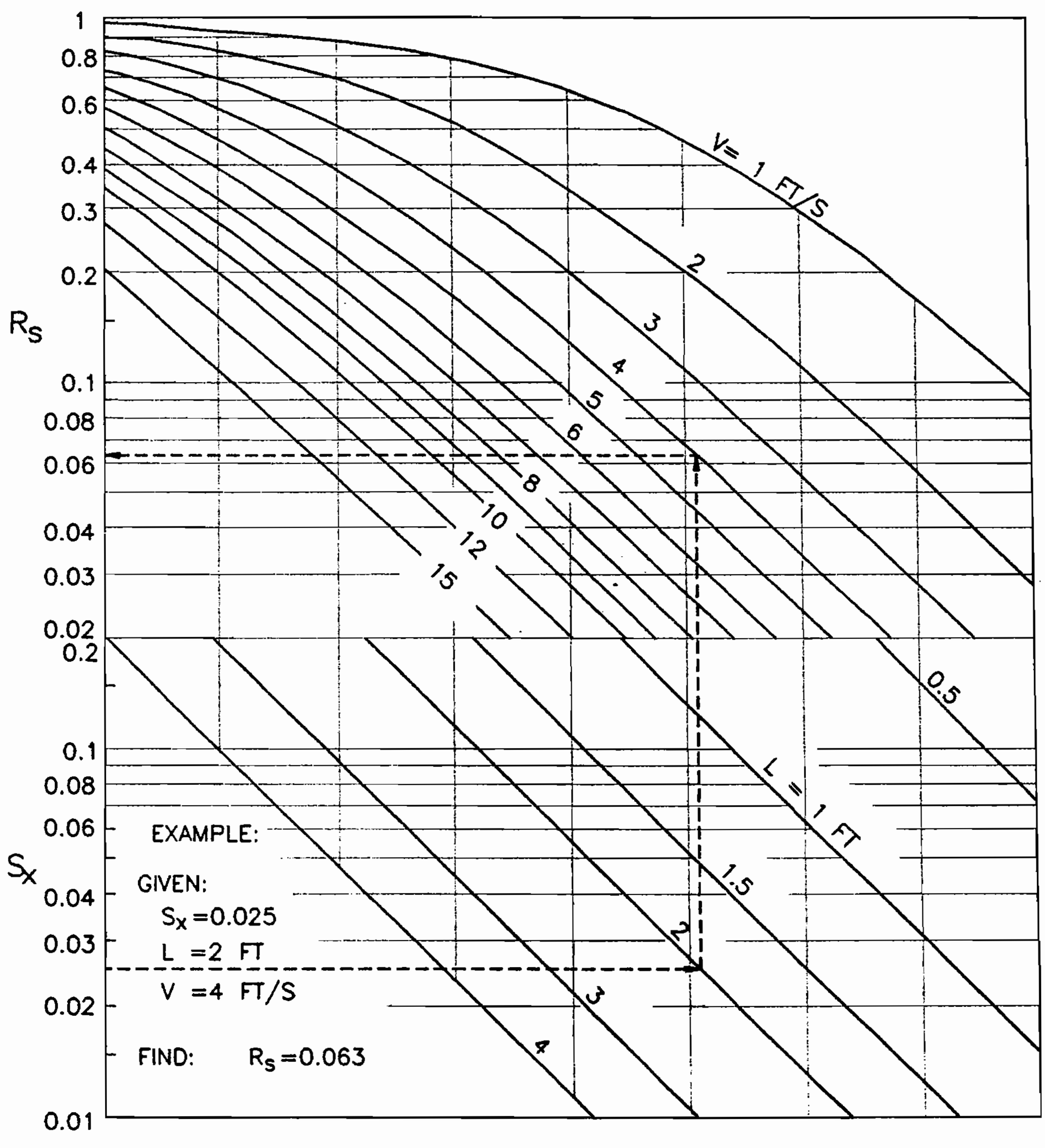
Figure 4-11



Source: *Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.*

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Grate Inlet Side Flow Interception Efficiency, R_s

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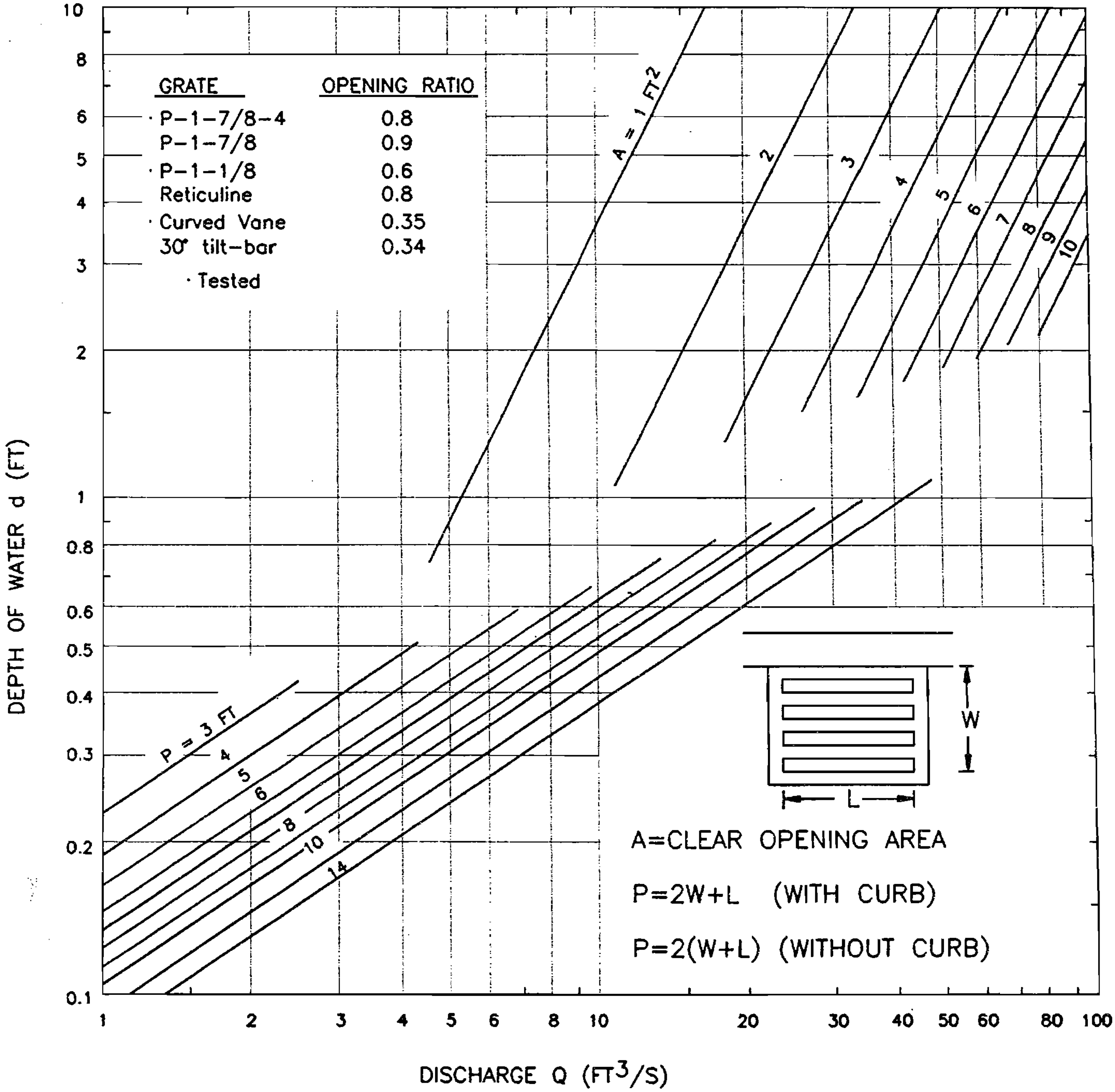
Figure 4-12



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

Grate Inlet Capacity in Sump Conditions

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Figure 4-13

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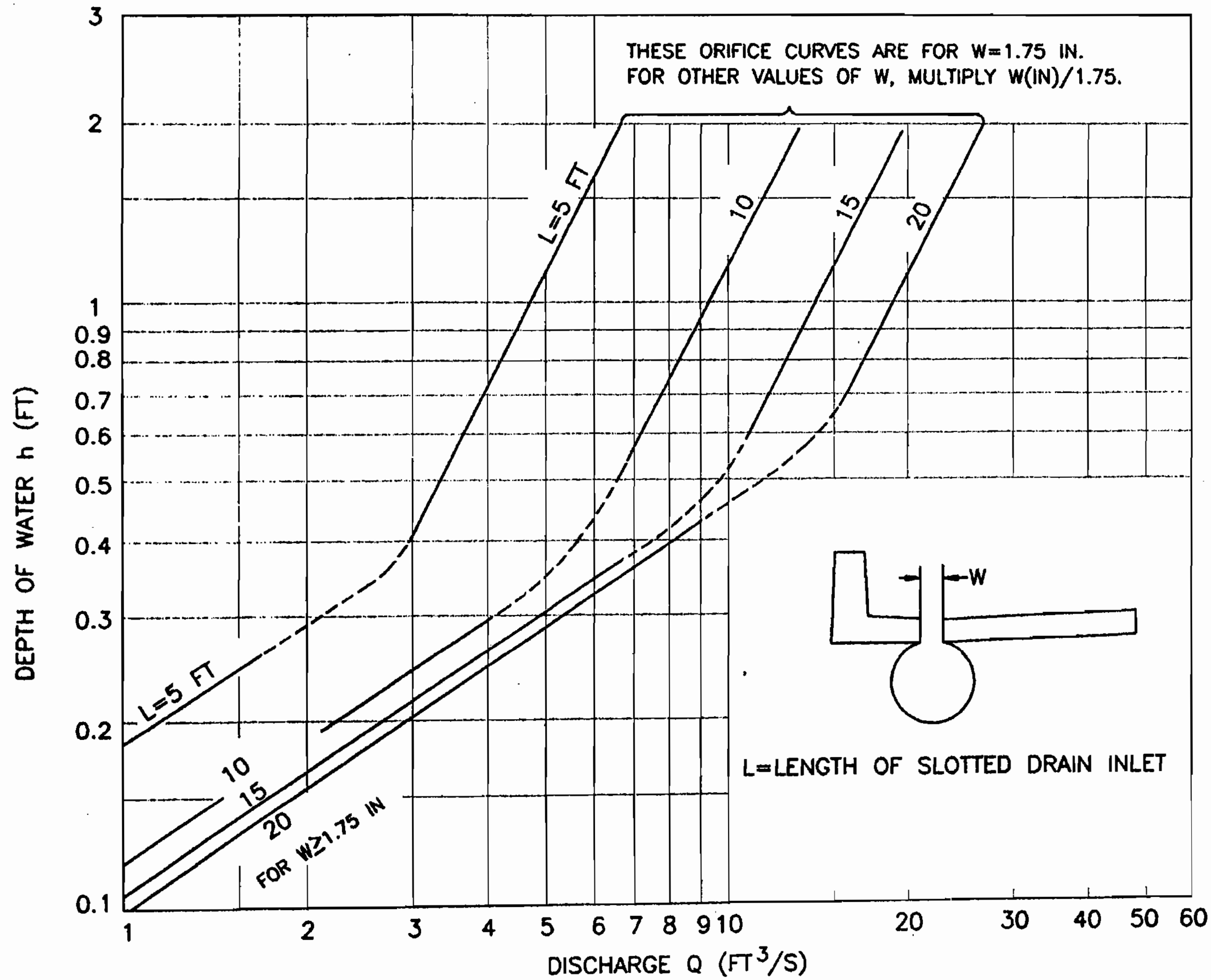
Storm Water Management Criteria Manual



Source: Federal Highway Administration, HEC No. 12, <u>Drainage of Highway Pavements</u>.

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Slotted Drain Inlet Capacity in Sump Locations

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Figure
4–14

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3. Bauer, W.J., and Woo, D.C., Hydraulic Design of Depressed Curb-Opening Inlets, Highway Research Board Record No. 58, Washington, DC, 1964, p. 61-80.

4. Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, Washington, DC, March 1984.

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LIST OF TABLES

LIST OF FIGURES

5.1 INTRODUCTION

It is the purpose of this section to consider the significance of the hydraulic elements of storm sewers and their appurtenances to a storm drainage system. Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exists. Storm sewers accordingly are designed for open-channel flow to satisfy as well as make possible, the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

All storm sewer systems shall be designed by the application of the Manning's Equation when flowing in open channel conditions. The hydraulic grade line shall be checked on storm sewer designs to determine if the open channel flow assumption is valid. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway in question.

The design of a storm drainage system should be governed by the following seven conditions:

A. The system must accommodate the minor surface runoff resulting from the selected design storm without serious damage to physical facilities or substantial interruption of normal traffic.

B. Runoff resulting from major storms must be anticipated and discharged with minimum damage to physical facilities and minimum interruption of normal traffic.

C. The storm drainage system must have a maximum reliability of operation.

D. The construction costs of the system must be reasonable with relationship to the importance of the facilities it protects.

E. The storm drainage system must require minimum maintenance and must be accessible for maintenance operations.

F. The storm drainage system must be adaptable to future expansion with minimum additional cost by the consideration of ultimate development on upstream or existing reaches.

G. Site design, swales and natural flow features should be utilized to reduce the need for extensive storm sewer systems whenever possible.

5.2 GENERAL CRITERIA

5.2.1 Frequency of Design Runoff

The frequency of design runoff is a function of operational and economic criteria with a special emphasis on public safety. As discussed in other sections of this Manual, some types of facilities do not require high levels of protection and periodic flooding is not objectionable. However, for all facilities, the designer must consider the impact of a 100-year flood and provide for its passage without the loss of life or major property damage.

Table 5-1 indicates the minimum acceptable frequencies of design runoff for storm sewers.

TABLE 5-1 Storm Sewer Design Storm Frequency

Facility	Storm Return Period (Frequency)
Streets and Gutters	2 years
Inlets	2 years
Storm Sewers	2 years
Major Drainage System	100 years

5.2.2 Velocities and Grades

Minimum Grades

Storm sewers should operate with flow velocities sufficient to prevent excessive deposition of solid material, resulting in objectionable clogging. The controlling velocity occurs near the bottom of the conduit and is considerably less than the mean velocity. Storm sewers shall be designed to have a minimum mean velocity flowing full of 2.0 fps, the lower limit of scouring velocity. Table 5-2 indicates the grades for both concrete pipe (n = 0.012) and corrugated metal pipe (n = 0.024) to produce a velocity of 2.0 fps. Any variance must be approved by the City Engineer. Outlets on pipes of minimum grade should be designed to avoid sedimentation at the outfall.

TABLE 5-2 Minimum Slope Required for Scouring Velocity*

Pipe Size (inches)	Concrete Pipe Slope (ft/ft)	Corrugated Metal Pipe Slope (ft/ft)
12	0.0016	0.0066
15	0.0012	0.0049
18	0.0010	0.0038
21	0.0008	0.0031
24	0.0007	0.0026
27	0.0006	0.0022
30	0.0005	0.0019
36	0.0004	0.0015
42	0.0003	0.0012
48	0.0003	0.0010
54	0.0002	0.0009
60	0.0002	0.0008
66	0.0002	0.0007
72	0.0002	0.0006
78	0.0001	0.0005
84	0.0001	0.0005
96	0.0001	0.0004

* Assume pipe flowing full

Maximum Velocities

Maximum velocities in conduits are important mainly because of the possibilities of excessive erosion on the storm sewer inverts. Table 5-3 shows the limits of maximum velocity.

TABLE 5-3 Maximum Velocity in Storm Sewers

Description	Maximum Permissible Velocity
Culverts (all types)	15 fps
Storm Sewers (collectors)	15 fps
Storm Sewers (mains)	12 fps

5.2.3 Pipe Sizes and Material Types

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 18 inches for gravity flow. If alternate shapes are required for utility clearance or special conditions, the designer must contact the City Engineer for approval. All reinforced concrete storm sewers shall meet, at a minimum, the requirement of AASHTO M170 Classes III-V. All pipe design and installation must meet the manufacturer's recommendation for minimum depth of cover.

A key consideration in selection of pipe material type involves the design life of the pipe. Pipe design life shall be a minimum 50 years as certified by the manufacturer. All manufacturer requirements for which the design life is based must be met by the engineer. For example, bedding requirements are critical to meeting the pipe design life.

In selecting a roughness coefficient, consideration shall be given to the average conditions during the useful life of the structure. An increased "n" value shall be used primarily in analyzing old conduits where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location and there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign materials, a roughness coefficient should be selected which, in the judgment of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe, or corrugated metal pipe, shall have the written approval of the City Engineer.

The coefficients of roughness listed in Table 5-4 are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

TABLE 5-4 Roughness Coefficients for Storm Sewers

Materials of Construction	Design Coefficient[1]
Concrete Pipe	0.012
Corrugated Metal	0.024

[1] Designer may select a single representative "n"

5.2.4 Manhole Location

Manholes shall be located at intervals not to exceed 800 feet for pipe 30 inches in diameter or smaller. Manholes shall be located at conduit junctions, changes in alignment, and ends of curved sections as necessary for maintenance equipment operation.

Manholes for pipe larger than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 1,000 feet.

5.2.5 Pipe Connections

Prefabricated wye and tee connections are recommended up to and including 24 inch x 24 inch. Connections larger than 24 inches will be made by field connections. This recommendation is based primarily on the fact that field connections are more easily fitted to a given alignment than are precast connections. Regardless of the amount of care exercised by the Contractor in laying the pipe, gain in footage invariably throws precast connections slightly out of alignment. This error increases in magnitude as the size of pipe increases.

5.2.6 Alignment

In general, storm sewer alignment between manholes shall be straight. Long radius curves may be allowed to conform to street alignment. Short radius curves may be used on larger pipes in order to reduce head losses at junctions. Curves may be produced by angling the joints or by fabricating beveled ends. Angled joints shall be kept at a minimum to maintain a tight joint. Pipe deflection shall not exceed manufacturers recommendations, unless precast or cast-in-place bends are specifically designed for deflection.

5.3 FLOW IN STORM SEWERS

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

$$Q = AV \tag{5-1}$$

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2} \tag{5-2}$$

where:

Q	=	pipe flow, in cubic feet per second
A	=	cross-sectional area of pipe, in square feet
V	=	velocity of flow, in feet per second
n	=	Manning's coefficient of roughness of pipe
R	=	hydraulic radius = A/WP, in feet
WP	=	wetted perimeter, in feet
S_f	=	friction slope of pipe, in feet per foot

There are several general rules to be observed when designing storm sewer sections. When followed, they will tend to alleviate or eliminate the common mistakes made in storm sewer design. These rules are as follows:

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 steeper slope.

C. At changes in pipe size from a smaller to a larger pipe, match the soffits (inside top surface) of the two pipes at the same level rather than matching the flow lines. (When necessary for minimal fall, match the 0.8 diameter point of each pipe.)

D. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

5.3.1 Pipe Flow Charts

Figures 5-1 through 5-3 are nomographs for determining uniform flow, critical depth, and velocity in circular conduits. Figures 5-4 and 5-5 are based on Manning's Equation for full flow and used to determine the discharge capacity for concrete pipe. The nomographs are based upon a value of "n" of 0.012 for concrete.

For values of "n" other than 0.012, the value of Q should be modified by using the formula below:

$$Q_c = \frac{Q_n(0.012)}{n_c} \tag{5-3}$$

where:

Q_c = flow based upon n_c, in cubic feet per second

Q_n = flow from nomograph based on n = 0.012, in cfs

n_c = value of "n" other than 0.012

This formula is used in two ways. If n_c = 0.015 and Q_c is unknown, use the known properties 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 known properties, you must 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 properties to find the unknown value on the nomograph.

Example 1 Calculate Design Discharge

Given: Slope = 0.005 ft/ft; depth of flow, d = 1.8 ft; diameter, D = 36"; n = 0.018

Find: Discharge, Q_c

Solution:

1) First determine d/D = 1.8 ft/3.0 ft = 0.6 ft

2) Enter Figure 5-1 to read Q_n = 34.0 cfs

3) Using Equation 5-3; Q_c = 34 (0.012/0.018) = 22.7 cfs

Example 2 Calculate Velocity of Flow

Given: Slope = 0.005 ft/ft; diameter D = 36"; Q_c = 22.7 cfs; n = 0.018

Find: Velocity of flow, V

Solution:

1) First convert Q_c to Q_n, so that the nomograph can be used.
Using Equation 5-3, Q_n = 22.7 (0.018/0.012) = 34.0 cfs

2) Enter Figure 5-1 to read d/D = 0.6 ft

3) Now enter Figure 5-3 to determine V = 7.5 fps

5.3.2 Bernoulli Equation

The law of conservation of energy as expressed by the Bernoulli Equation is the basic principle most often used in hydraulics. Energy cannot be lost, thus in a hydraulic system the sum of all energies is a constant. The total energy in mathematical form is Equation 5-4.

$$E = y + \frac{V^2}{2g} + \frac{P}{\gamma} = constant \qquad (5\text{-}4)$$

where:

E	=	total energy head, in feet
y	=	depth of water, in feet
V	=	mean velocity, in feet per second
P	=	pressure at given location, in pounds per square feet
g	=	acceleration of gravity, 32.2 feet per second squared
γ	=	specific weight of fluid, in pounds per cubic foot

The theorem states that the energy head at any cross-section must equal that in any other downstream section plus the intervening losses. In open channels, the flow is primarily controlled by the gravitational action of the moving fluid, which overcomes the hydraulic energy losses. The Bernoulli Equation defines the hydraulic principles in open channel flow.

$$H = y + \frac{V^2}{2g} + Z + h_f \tag{5-5}$$

where:

H	=	total energy head, in feet
y	=	depth of water, in feet
V	=	mean velocity, in feet per second
Z	=	height above datum, in feet
h_f	=	head loss, in feet
g	=	acceleration of gravity, 32.2 feet per second squared

The total energy at point one (1) is equal to the total energy at point two (2). The terms are defined as above.

$$y_1 + Z_1 + \frac{V_1^2}{2g} = y_2 + Z_2 + \frac{V_2^2}{2g} + h_f \tag{5-6}$$

The Bernoulli Equation is rewritten for pressure or closed conduit flow. The terms are defined as above.

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f \tag{5-7}$$

Figure 5-6 is a graphical representation of the energy in open channel flow and closed conduit flow. The following variables are used in Figure 5-4.

H	=	total energy head
y	=	depth of water
$V^2/2g$	=	velocity head

EGL	=	energy grade line
S_o	=	slope of bottom
h_f	=	head loss
V	=	mean velocity
Z	=	height above datum
HGL	=	hydraulic grade line
S_f	=	slope of EGL
S_w	=	slope of HGL
P/γ	=	pressure head

The sum of the pressure head, P/γ and the elevation head, y, is called the piezometric head. This is the height to which water would rise in a pipe with one of it's ends inserted into an arbitrary point in the flow field. The line connecting points of equal piezometric measurements along the path of flow is called the hydraulic grade line.

$$HGL = \frac{P}{\gamma} + y \tag{5-8}$$

where:

HGL	=	hydraulic grade line, in feet
P/γ	=	pressure head, in feet
y	=	elevation head, in feet

The energy grade line is equal to the hydraulic grade line plus the velocity head, $V^2/2g$.

$$EGL = \frac{P}{\gamma} + y + \frac{V^2}{2g} \tag{5-9}$$

where:

EGL	=	energy grade line, in feet
P/γ	=	pressure head, in feet
y	=	elevation head, in feet
$V^2/2g$	=	velocity head, in feet

5.4 ENERGY GRADIENT AND PROFILE OF STORM SEWERS

When using Bernoulli's Equation in the hydraulic design of storm sewers, all energy losses must be accounted for. These losses are commonly referred to as head losses, and are classified as either friction losses or minor losses. Friction losses are due to forces between the fluid and the boundary material, while minor losses are a result of the geometry of sewer appurtenances such as manholes, bends, and either expanding or contracting transition. Minor losses can constitute a major portion of the total head loss.

When storm sewer systems are designed for full flow, the designer shall establish the head losses caused by flow resistance in the conduit, changes of momentum and interference at junctions and structures. This information is then used to establish the design water surface elevation at each structure.

It is not necessary to compute the energy grade line of a conduit section if all three of the following conditions are satisfied;

A. The slope(s) and the pipe size(s) are chosen so that the slope is equal to or greater than friction slope.

B. The inside top surfaces (soffit) of successive pipes are lined up at changes in size.

C. The water surface at the point of discharge will not rise above the top of the outlet.

In such cases the pipe will not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the invert of the pipe.

In the absence of these conditions or when it is desired to check the system against a larger flood than that used in sizing the pipes, the hydraulic and energy grade lines shall be computed and plotted. The friction head loss shall be determined by direct application of Manning's Equation. Minor losses due to turbulence at structures shall be determined by the procedure described below. If the storm sewer system could be extended at some future date, present and future operation of the system must be considered.

The final hydraulic design of a system should be based on the procedures set forth in this Manual. The conduits are treated as either open channel flow or flowing full flow, as the case may be. For open channel flow, the energy grade line is used as a base for calculation, while the hydraulic grade line is used for flowing full flow. The following procedure is applicable to storm sewers flowing with a free water surface, or open channel flow. The basic approach to the design of open channel flow in storm sewers is to calculate the energy grade line along the system. It is assumed that the energy grade line is parallel to the pipe grade and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

5.4.1 Friction Head Loss

The pipe friction can be evaluated by modifying the Manning's Equation.

$$S_f = \left[\frac{Qn}{1.49AR^{2/3}}\right]^2 \qquad (5\text{-}10)$$

where:

S_f	=	slope of pipe, in feet per foot
Q	=	pipe flow, in cubic feet per second
n	=	Manning's roughness coefficient
A	=	cross-sectional area of pipe, in square feet
R	=	hydraulic radius, A/WP, in feet
WP	=	wetted perimeter, in feet

The pipe friction head loss is equal to the friction slope of the pipe multiplied by the length.

$$h_f = S_L L \qquad (5\text{-}11)$$

where:

h_f	=	pipe friction head loss, in feet
S_L	=	friction slope of pipe, in feet per foot
L	=	length of pipe, in feet

5.4.2 Minor Head Losses at Structures

The head losses at structures shall be determined for inlets, manholes, wye branches, or bends in the design of full flow closed conduits. Total energy losses at structures include minor losses, h_j, and the change in velocity head, h_v. See Figures 5-7 and 5-8 for details of each case. Minimum head loss used at any structure shall be 0.10 feet, unless otherwise approved.

Short radius bends may be used on 24 inch and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall be located at the end of such short radius bends if required for operation and maintenance.

The basic equations for minor head losses, where there is significant upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient, k_m, shown in Tables 5-5, 5-6 and 5-7.

$$h_j = k_m \left[\frac{V_2^2 - V_1^2}{2g} \right] \qquad (5\text{-}12)$$

where:

h_j	=	junction or structure minor head loss, in feet
k_m	=	junction or structure coefficient of loss, in feet
V_2	=	velocity in downstream pipe, in feet per second
V_1	=	velocity in upstream pipe, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

In the case where the initial velocity is negligible or when there is no velocity change, the basic equation for head loss becomes:

$$h_j = k_m \left[\frac{V_2^2}{2g} \right] \qquad (5\text{-}13)$$

The parameters are defined as above.

TABLE 5-5 Junction or Structure Minor Loss Coefficient, k_m

Case Number	Reference Figure	Description of Condition	Coefficient k_m
I	5-5	Inlet on Main Line	0.50
II	5-5	Inlet on Main Line with Branch Lateral	0.25
III	5-5	Manhole on Main Line with 45° Branch Lateral	0.25
IV	5-5	Manhole on Main Line with 90° Branch Lateral	0.25
V	5-6	45° Wye Connection or cut-in	0.75
VI	5-6	Inlet or Manhole at Beginning of Line	1.25
VII	5-6	Conduit on Curves for 90° *	
		Curve radius = diameter	0.50
		Curve radius = 2 to 8 diameters	0.40
		Curve radius = 8 to 20 diameters	0.25
VIII	5-6	Bends where Radius is Equal to Diameter	
		90° Bend	0.50
		60° Bend	0.43
		45° Bend	0.35
		22-1/2° Bend	0.20
		Manhole on Line with 60° Lateral	0.35
		Manhole on Line with 22-1/2° Lateral	0.75

* Where bends other than 90° are used, the 90° bend coefficient can be used with the following percentage factor applied:
60° Bend--85%; 45° Bend--70%; 22-1/2° Bend--40%

Obstructions

The values of the coefficient, k_m, for determining the loss of head due to obstructions in pipes are shown in Table 5-6, and the coefficients are used in the following equation to calculate the head loss at the obstruction:

$$h_j = k_m \left[\frac{V_2^2}{2g}\right] \tag{5-14}$$

where:

h_j = minor head loss, in feet

k_m = head loss coefficient

V_2 = velocity in smaller pipe, in feet per second

g = acceleration of gravity, 32.2 feet per second squared

TABLE 5-6 Head Loss Coefficients Due to Obstructions

$\frac{A^*}{A_o}$	k_m
1.05	0.10
1.10	0.21
1.20	0.50
1.40	1.15
1.60	2.40
1.80	4.00
2.00	5.55
2.20	7.05
2.50	9.70
3.00	15.00
4.00	27.30
5.00	42.00
6.00	57.00
7.00	72.50
8.00	88.00
9.00	104.00
10.00	121.00

* $\frac{A}{A_o}$ = Ratio of area of pipe to area of opening at obstruction

Expansions and Contractions

The values of the coefficient k_m for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in Table 5-7. These coefficients are used in the following equation to calculate the head loss at the change in section:

$$h_j = k_m \left[\frac{V_2^2}{2g} \right] \tag{5-15}$$

where:

h_j = minor head loss, in feet

k_m = head loss coefficient

V_2 = velocity in smaller pipe, in feet per second

g = acceleration of gravity, 32.2 feet per second squared

TABLE 5-7 Head Loss Coefficients for Expansions and Contractions

$\frac{D2}{D1}$*	Sudden Expansions k_m	Sudden Contractions k_m
1.2	0.10	0.08
1.4	0.23	0.18
1.6	0.35	0.25
1.8	0.44	0.33
2.0	0.52	0.36
2.5	0.65	0.40
3.0	0.72	0.42
4.0	0.80	0.44
5.0	0.84	0.45
10.0	0.89	0.46
	0.91	0.47

* $\frac{D2}{D1}$ = Ratio of larger to smaller diameter.

5.5 DESIGN PROCEDURE FOR STORM SEWER SYSTEMS

5.5.1 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 reconnaissance.

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

C. Outline the drainage area for each inlet in accordance with present and future street development.

D. Indicate on each drainage area a code identification number, the size of area, the direction of surface runoff by small arrows, and the coefficient of runoff for the area.

E. Show all existing underground utilities.

F. Establish design rainfall frequency.

G. Establish inlet time of concentration.

H. Establish the typical cross section of each street.

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

J. Include Steps A through I with plans submitted for review. The drainage map submitted shall be suitable for permanent filing with the appropriate agency and shall be a good quality reproducible copy.

5.5.2 Storm Sewer System

After the computation of the quantity of storm runoff entering each inlet, the storm sewer system required to carry the runoff is designed. It should be borne in mind that the quantity of flow to be carried by any particular section of the storm sewer system is not the sum of the inlet design quantities of all inlets above that section of the system, but is less than the straight total. This situation is due to the fact that as the time of concentration increases the rainfall intensity decreases.

Determining Type of Flow

Before treating conduit as open channel, checks must be made to determine the type of flow. To do this, calculations must proceed upstream, verifying progressively that the hydraulic grade line is below the crown of the pipe.

A. Discharge Point

The discharge point of the sewer usually establishes a starting point. If the discharge is submerged, as when the water level of the receiving water body is above the crown of the pipe, the exit loss should be added to the water level and calculations for head loss in the sewer started from this point. If the hydraulic grade line is above the pipe crown at the next upstream manhole, full flow calculations may proceed. If the hydraulic grade line is below the pipe crown at the upstream manhole, then open channel flow calculations must be used at the manhole.

When the discharge is not submerged, a flow depth must be determined at some control section to allow calculations to proceed upstream. If the tailwater depth is less than $(D + d_c)/2$, set the tailwater elevation equal to $(D + d_c)/2$, where D equals the pipe in diameter, and d_c equals the critical depth, both in feet, otherwise use the tailwater depth. The hydraulic grade line is then projected to the upstream manhole. Full flow calculations may be utilized at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines is not entirely correct, since backwater and drawdown exists, but should be accurate enough for the size pipes usually considered as storm sewers. If the designer feels that additional accuracy is justified, as with very large conduits or where the result will have a very significant effect on design, backwater and drawdown curves may actually be calculated.

B. Within System

At each manhole the same type of procedure as outlined for the discharge point must be repeated.

The water depth in each manhole must be calculated to verify that the water level is above the crown of all pipes. Whenever the level is below the crown of a pipe, open channel methods are applicable.

Storm Sewer Pipe

The ground-line profile is used in conjunction with the previous runoff calculations. When the initial energy gradient is established and the design discharge is determined, a Manning's flow chart may be used to determine the pipe size and velocity. (Figures 5-1 and 5-3 or Figures 5-4 and 5-5).

Velocities can be read directly from a Manning's flow chart based on a given discharge, pipe size and slope. (Figure 5-3).

Junctions, Inlets and Manholes

A. Determine the invert elevations at the upstream end and downstream end of the pipe section in question. The elevation of the invert of the upstream end of pipe is equal to the elevation of the downstream end of pipe (invert) plus the product of the length of pipe and the pipe gradient, S_o.

B. Determine the velocity of flow for incoming pipe (main line) at junction, inlet, or manhole at design point.

C. Determine the velocity of flow for outgoing pipe (main line) at junction, inlet, or manhole at design point.

D. Compute velocity head for outgoing velocity (main line) at junction, inlet, or manhole at design point.

E. Compute velocity head for incoming velocity (main line) at junction, inlet, or manhole at design point.

F. Determine head loss coefficient, k_m, at junction, inlet, or manhole at design point from Tables 5-5, 5-6, 5-7 or Figures 5-5 and 5-6.

G. Compute head loss at junction, inlet or manhole.

$$h_j = k_m \left[\frac{V_2^2 - V_1^2}{2g} \right]$$

H. Compute energy gradient at upstream end of junction as if junction were not there.

I. Add head loss to energy gradient elevation determined to obtain energy gradient elevation at upstream end of junction.

All information shall be recorded on the plans or in tabular form convenient for review.

Major Storm System

Check the proposed system for the 100-year major storm event. Modify the proposed system or provide additional flow capacity as required to accommodate the major storm runoff according to the requirements stated in Sections 2, 3 and 4.

5.5.3 Inlet System

Determining the size and location of inlets is largely a trial-and-error procedure. The following steps will serve as a guide for the procedure to be used.

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

B. 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.

C. 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 convenient for review.

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

E. Continue the above procedure for other subareas until a complete system of inlets has been established. Compare the time of concentration for a subarea to the time of concentration for the upstream contributing areas. Use the longer time of concentration to calculate the discharge at the inlet. Remember to account for carry-over from one inlet to the next. Add the carry-over to the calculated discharge to obtain the design discharge at the inlet. The difference between the inlet discharge and the design discharge is carry-over flow and is bypassed to the next downstream inlet.

F. 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.

G. Record information as in Steps C and D for all inlets.

H. After the inlets have been located and sized the inlet pipes can be designed.

I. Inlet pipes are sized to carry the volume of water intercepted by the inlet. Inlet pipe capacities may be controlled by the gradient available, or by entry conditions of the pipe at the inlet. Inlet pipe sizes should be determined for both inlet and outlet conditions and the larger size thus obtained.

5.5.4 Inlet Lateral Pipe Design

The design of an inlet lateral pipe to the trunk line is an iterative process. The designer must analyze the lateral for outlet and inlet control. Figure 5-9 is a definition sketch showing outlet and inlet control flow conditions. The figure assumes that the trunk line is flowing full and is not surcharged, or under pressure flow.

Outlet Control

The head or energy required to pass a given discharge through a lateral pipe flowing in outlet control and flowing full throughout its length, is comprised of three components. These include an exit loss h_o, an entrance loss h_e, and a friction loss h_f. The energy is expressed by the following equation.

$$h = h_o + h_e + h_f \tag{5-16}$$

where:

h = total head loss, in feet

h_o = exit loss, in feet

$= k_o \dfrac{V^2}{2g}$

k_o = exit coefficient, equal to 1.0

h_e = entrance loss, in feet

$$= k_e \frac{V^2}{2g}$$

k_e = entrance coefficient

h_f = friction loss, in feet

$$= \left[\frac{185n^2L}{d^{4/3}}\right]\frac{V^2}{2g}$$

n = Manning's roughness coefficient

L = length of pipe, in feet

d = diameter of lateral pipe, in feet

V = velocity of flow, in feet per second

g = acceleration of gravity, 32.2 feet per second squared

Simplifying Equation 5-4, for full flow, yields Equation 5-17

$$h = \left[1 + k_e + \frac{185n^2L}{d^{4/3}}\right]\frac{V^2}{2g} \qquad (5\text{-}17)$$

where the terms are as defined above.

The pipe discharge can be computed by the following equation for full flow:

$$Q = A\left[\frac{2gh}{1 + k_e + \frac{185n^2L}{d^{4/3}}}\right]^{0.5} \qquad (5\text{-}18)$$

where:

Q = discharge, in cubic feet per second

A = cross-sectional area, in square feet

Figures 5-10 through 5-18 have been developed for inlet lateral pipe design with outlet control for pipe diameters of 12 inch to 36 inch. An entrance coefficient, k_e, 0.5 and Manning's "n" value of 0.013 was assumed in the development of the figures. The head is defined as shown in Figure 5-9, with full flow in the trunk line and lateral.

If outlet control governs and the HW is higher than is acceptable, select a larger size until HW is acceptable for outlet control.

Inlet Control

The head required to pass a discharge through a pipe flowing in inlet control is controlled at the entrance by the depth of headwater (HW) and the entrance geometry. Figure 5-9 shows an outlet control sketch. The headwater, HW, is expressed by the following equation:

$$HW = H + \frac{d}{2} \tag{5-19}$$

where:

HW = headwater depth, in feet

H = hydraulic head, distance from the center of the lateral pipe to the water surface elevation, in feet

d = diameter of lateral pipe, in feet

The discharge may be computed by the following equation:

$$Q = 0.6A\sqrt{2gH} \tag{5-20}$$

where:

Q = discharge, in cubic feet per second

A = cross-sectional area of lateral pipe, in square feet

g = acceleration of gravity, 32.2 feet per square second

H = hydraulic head, in feet

If the HW is greater or less than allowable, try another pipe size until HW is acceptable for inlet control. Figure 5-19 can be used to determine the headwater for inlet lateral pipe design under inlet control. The head in Figure 5-19 is defined as the summation of the hydraulic head, pipe radius, and a curb height of 9 inches.

5.5.5 Storm Sewer System Design by the Rational Method

The Rational Method is the most commonly used method for storm sewer system design. Figure 5-20 is a computation form for designing storm sewer system by the Rational Method. Columns 1 through 15 of the computation sheet cover the tabulation for runoff computations. After the computation of the quantity of storm runoff entering each inlet, the size and gradient of pipe required to carry the design storm is determined. It should be kept in mind that the quantity of flow to be carried by any particular section of storm sewer is not the sum of the inlet design quantities of all inlets above that section of pipe, but is less than the total. This is due to the fact that as the time of concentration increases, the rainfall intensity decreases. Columns 16 through 29 of the computation sheet cover the minimum necessary hydraulic requirements to establish the hydraulic grade line for a storm sewer.

The following is an explanation for completing the storm sewer computation form by the Rational Method.

Column 1	From Design Point	Enter the storm sewer inlet point number. Design should start at the farthest upstream point.
Column 2	To Design Point	Enter the storm sewer inlet point number of inlet point immediately downstream. In numbering inlets and manholes, it is customary to start numbering inlets and manholes at the beginning of the storm drainage system, proceeding upstream.
Column 3	Distance	Enter the distance (in feet) between storm sewer inlet point shown in Column 1 and 2. Column 1 stationing minus Column 2 stationing.
Column 4	Drainage Area Number	Record the identification code number of each different drainage area to correspond to the numbers shown on the drainage area map.
Column 5	Drainage Area	Record the area in acres for each of the individual areas of Column 4.
Column 6	Total Drainage Area	Record the total drainage area in acres within the system corresponding to storm sewer inlet point shown in Column 1.
Column 7	Runoff Coefficient	Record the runoff coefficient "C" for each drainage area shown in Column 5.
Column 8	Incremental "CA"	Multiply Column 5 by Column 7 for each area.
Column 9	Total "CA"	Determine the total "CA" for the drainage system corresponding to the inlet or manhole shown in Column 1.
Column 10	t_c - Inlet Time	Determine inlet time of concentration.

Column 11	t_c - Sewer Time	Determine flow time in sewer in minutes. The flow time in sewer is equal to the length from Column 3 divided by 60 times the velocity of flow through the sewer.
Column 12	t_c - Total Time	Total time of concentration in minutes. Column 10 plus Column 11.
Column 13	Design Frequency	Design frequency.
Column 14	Intensity	Intensity of rainfall in inches per hour corresponding to time of concentration shown in Column 12. Figures 2-1 or 2-2 Intensity-Duration-Frequency Curves.
Column 15	Discharge	Design discharge in cfs. Column 9 times Column 14.
Column 16	Pipe Size	The size of pipe is chosen in such a manner that the pipe when, flowing full, will carry an amount of flow equal to or greater than the computed discharge with a desirable velocity.
Column 17	Frictional Gradient Slope, S_f	The slope of the frictional gradient (energy gradient) is chosen so that the pipe, when flowing full, will carry an amount of flow equal to or greater than the computed discharge. The pipe shall be constructed on a grade such that the inside crown of the pipe coincides with energy gradient or is below the developed energy gradient when flowing full.
Columns 18 and 19	Energy Gradient Elevation	Record the energy gradient elevations at the upstream end and downstream end of pipe section in question. The elevation of the energy gradient of the upstream end of pipe is equal to the elevation of the downstream (energy gradient) plus the product of Column 3 and Column 17.
Column 20	Inflow Velocity	Velocity of flow in incoming pipe (main line) at junction, inlet or manhole at design point. (Column 1).
Column 21	Outflow Velocity	Velocity of flow in outgoing pipe (main line) at junction, inlet or manhole at design point. (Column 1).
Column 22	Velocity Head	Velocity head for outgoing velocity (main line) at junction inlet or manhole at design point (Column 1).

Column 23	Velocity Head	Velocity head for incoming velocity (main line) at junction inlet or manhole at design point (Column 1).
Column 24	Head loss Coefficient	Head loss coefficient "k_m", at junction, inlet or manhole at design point from Tables 5-5 through 5-7 or Figures 5-7 and 5-8.
Column 25	Minor loss	Multiply Column 23 by Column 24.
Column 26	Head loss	Column 22 minus Column 25.
Column 27	Elevation of EGL	Column 18 plus Column 26.
Column 28	Inflow Elevation	Invert elevation at design point for incoming pipe.
Column 29	Outflow Elevation	Invert elevation at design point for outgoing pipe.

5.5.6 Storm Sewer System Design by the Amarillo Peak Flow Curve Method

The Amarillo Peak Flow Curve Method was introduced in Section 2 and can be used to design storm sewer systems. Figure 5-21 is a computation form similar to the Rational Method analysis, expect that the runoff computations are different. The following is an explanation of the computation form.

Column 1	From Design Point	Enter the storm sewer inlet point number. Design should start at the farthest upstream point.
Column 2	To Design Point	Enter the storm sewer inlet point number of inlet point immediately downstream. In numbering inlets and manholes, it is customary to start numbering inlets and manholes at the beginning of the storm drainage system, proceeding upstream.
Column 3	Distance	Enter the distance (in feet) between storm sewer inlet point shown in Column 1 and 2. Column 1 stationing minus Column 2 stationing.
Column 4	Drainage Area Number	Record the identification code number of each different drainage area to correspond to the numbers shown on the drainage area map.
Column 5	Drainage Area	Record the area in acres for each of the individual areas of Column 4.
Column 6	Total Drainage Area	Record the total drainage area in acres within the system corresponding to storm sewer inlet point shown in Column 1.

Column 7	Curve Number	Record the SCS Curve Number "CN" for each drainage area shown in Column 5.
Column 8	Upstream Elevation	Record the elevation of the highest upstream point.
Column 9	Downstream Elevation	Record the elevation of the lowest downstream point.
Column 10	Basin Length	Determine the straight line length from the highest to lowest point in the basin.
Column 11	Basin Slope	Calculate the basin slope. The slope is equal to Column 8 minus Column 9 divided by Column 10, in percent.
Column 12	Design Frequency	Design frequency.
Column 13	Unit Discharge	Unit discharge from peak flow curves, Figures 2-4 through 2-27, in cfs/acre.
Column 14	Design Discharge	Design discharge in cfs. Multiply Column 6 by Column 13.
Column 15	Pipe Size	The size of pipe is chosen in such a manner that the pipe when, flowing full, will carry an amount of flow equal to or greater than the computed discharge with a desirable velocity.
Column 16	Frictional Gradient Slope, S_f	The slope of the frictional gradient (energy gradient) is chosen so that the pipe when flowing full, will carry an amount of flow equal to or greater than the computed discharge. The pipe shall be constructed on a grade such that the inside crown of the pipe coincides with energy gradient or is below the developed energy gradient when flowing full.
Columns 17 and 18	Energy Gradient Elevation	Record the energy gradient elevations at the upstream end and downstream end of pipe section in question. The elevation of the energy gradient of the upstream end of pipe is equal to the elevation of the downstream (energy gradient) plus the product of Column 3 and Column 16.
Column 19	Inflow Velocity	Velocity of flow in incoming pipe (main line) at junction, inlet or manhole at design point. (Column 1).
Column 20	Outflow Velocity	Velocity of flow in outgoing pipe (main line) at junction, inlet or manhole at design point. (Column 1).

Column 21	Velocity Head	Velocity head for outgoing velocity (main line) at junction inlet or manhole at design point (Column 1).
Column 22	Velocity Head	Velocity head for incoming velocity (main line) at junction inlet or manhole at design point (Column 1).
Column 23	Head loss Coefficient	Head loss coefficient "k_m", at junction, inlet or manhole at design point from Tables 5-5 through 5-7 or Figures 5-7 and 5-8.
Column 24	Minor loss	Multiply Column 22 by Column 23.
Column 25	Head loss	Column 21 minus Column 24.
Column 26	Elevation of EGL	Column 17 plus Column 25.
Column 27	Inflow Elevation	Invert elevation at design point for incoming pipe.
Column 28	Outflow Elevation	Invert elevation at design point for outgoing pipe.

5.6 HYDRA MODEL DESCRIPTION

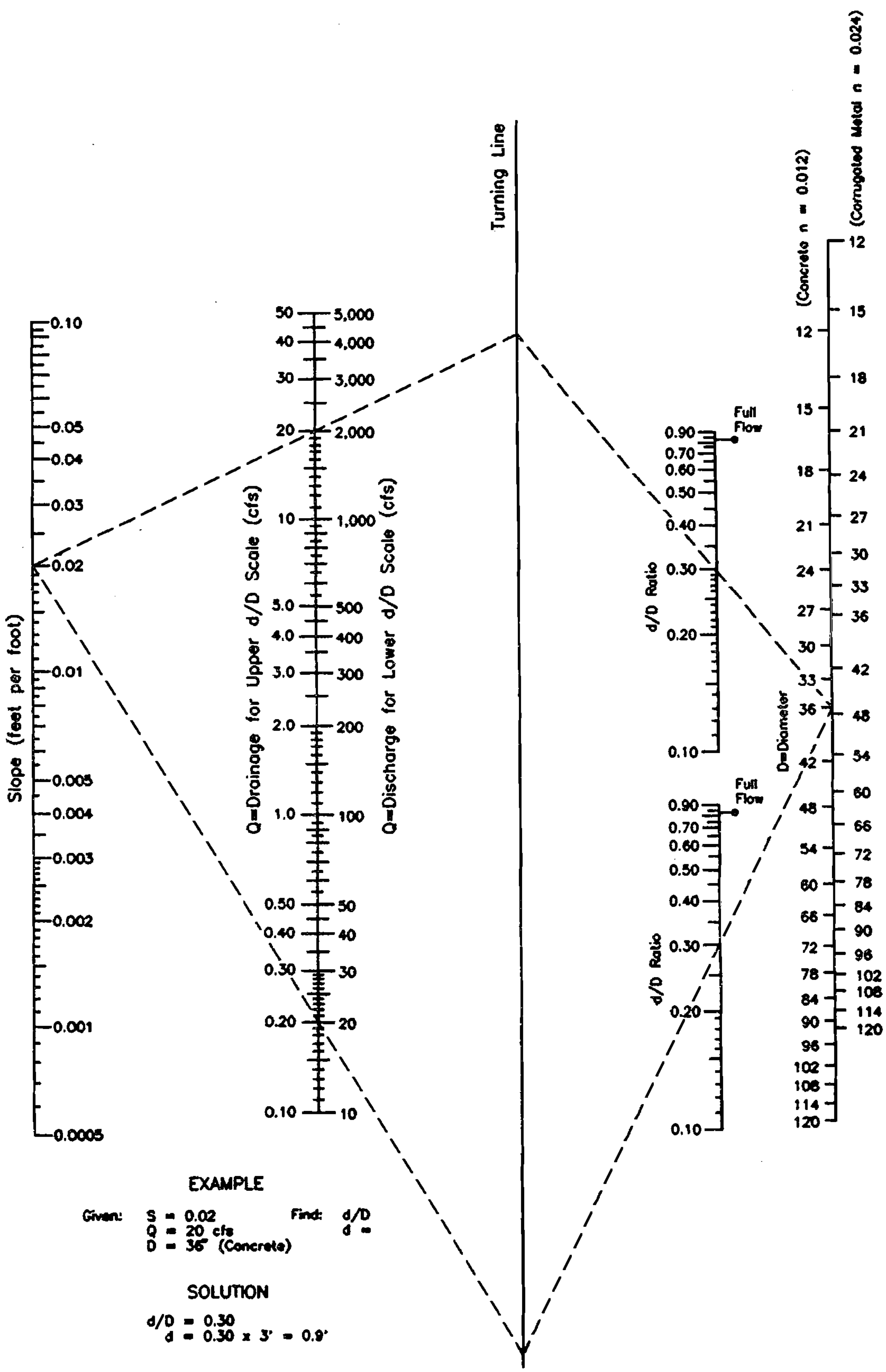
HYDRA is series of programs designed to analyze storm and sanitary sewer collection systems. The program was originally developed in 1973 and released commercially in 1975. The program has gone through about five releases since that time with upgrades and features added with each release. Features of the present program (Release 5.0) relating to storm flows, storm sewer analysis/design, and transport capabilities are:

A. Storm flows can be generated by continuous simulation, the SCS method, Rational Method, or directly input. The storm hydrograph can be viewed separately for each lateral or segment of a modelled storm sewer system. The size and characteristics of the storm cell can be defined including its starting position, tracking azimuth and velocity. Time steps may vary from 6 to 1,440 minutes, and the total number of 255 hydrographic steps.

B. There is no limit on the size of a system as to the number of sub-areas and/or hydrographs.

C. Routing of storm flows in a modelled system can include pumps and reservoirs including detention and wet well hydrographs. Hydrographs may be imported at any time into the system from other programs. Hydrographs are automatically converted to the time step chosen for use in the model.

D. Sanitary, infiltration, and industrial flows may be added or taken out at any time in the system.



EXAMPLE

Given: S = 0.02 Find: d/D
Q = 20 cfs d =
D = 36" (Concrete)

SOLUTION

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

Source: Texas Highway Department.

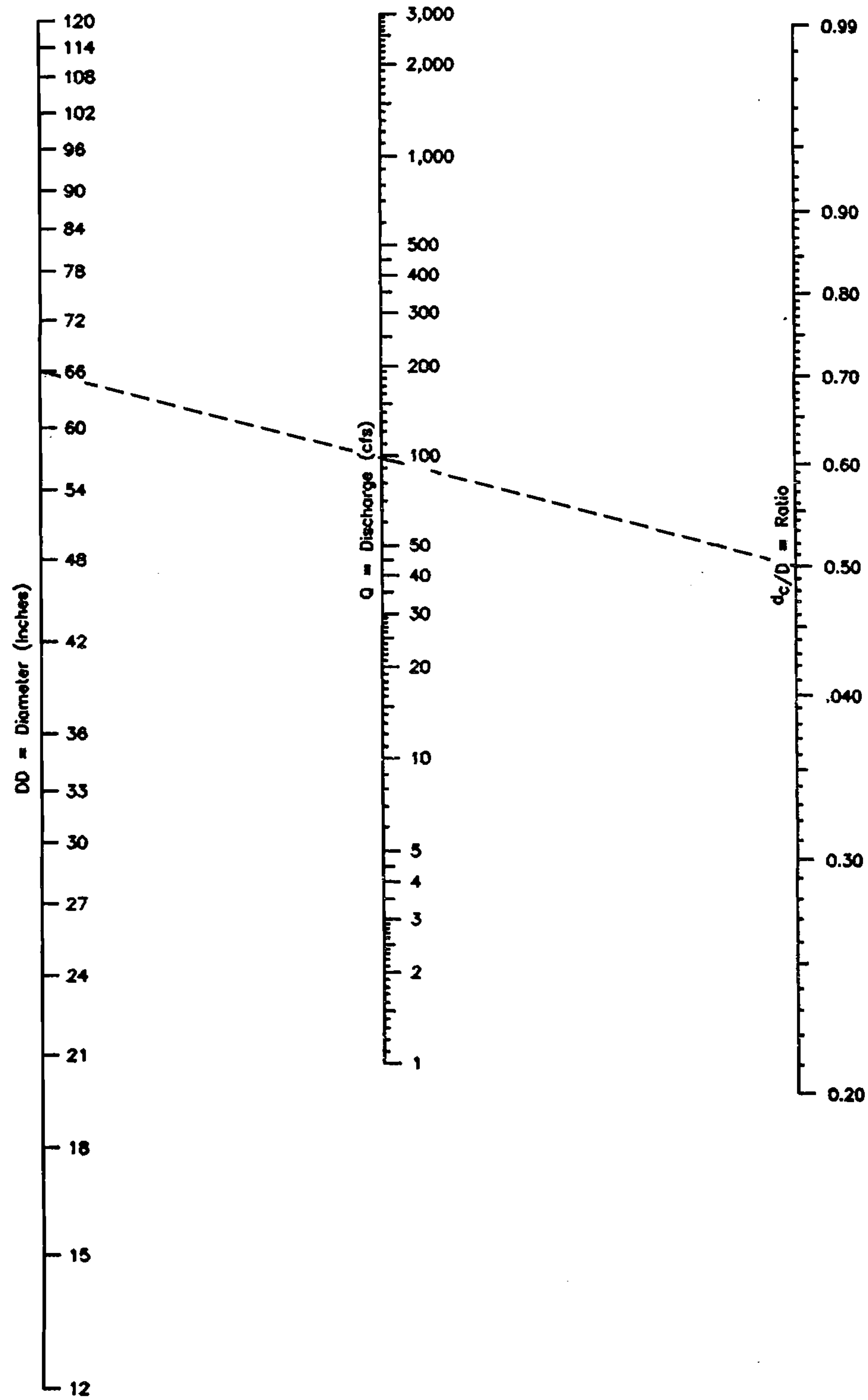
CITY OF AMARILLO

Uniform Flow for Circular Conduits
Concrete n = 0.012
Corrugated Metal n = 0.024

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
5-1



Source: Texas Highway Department.

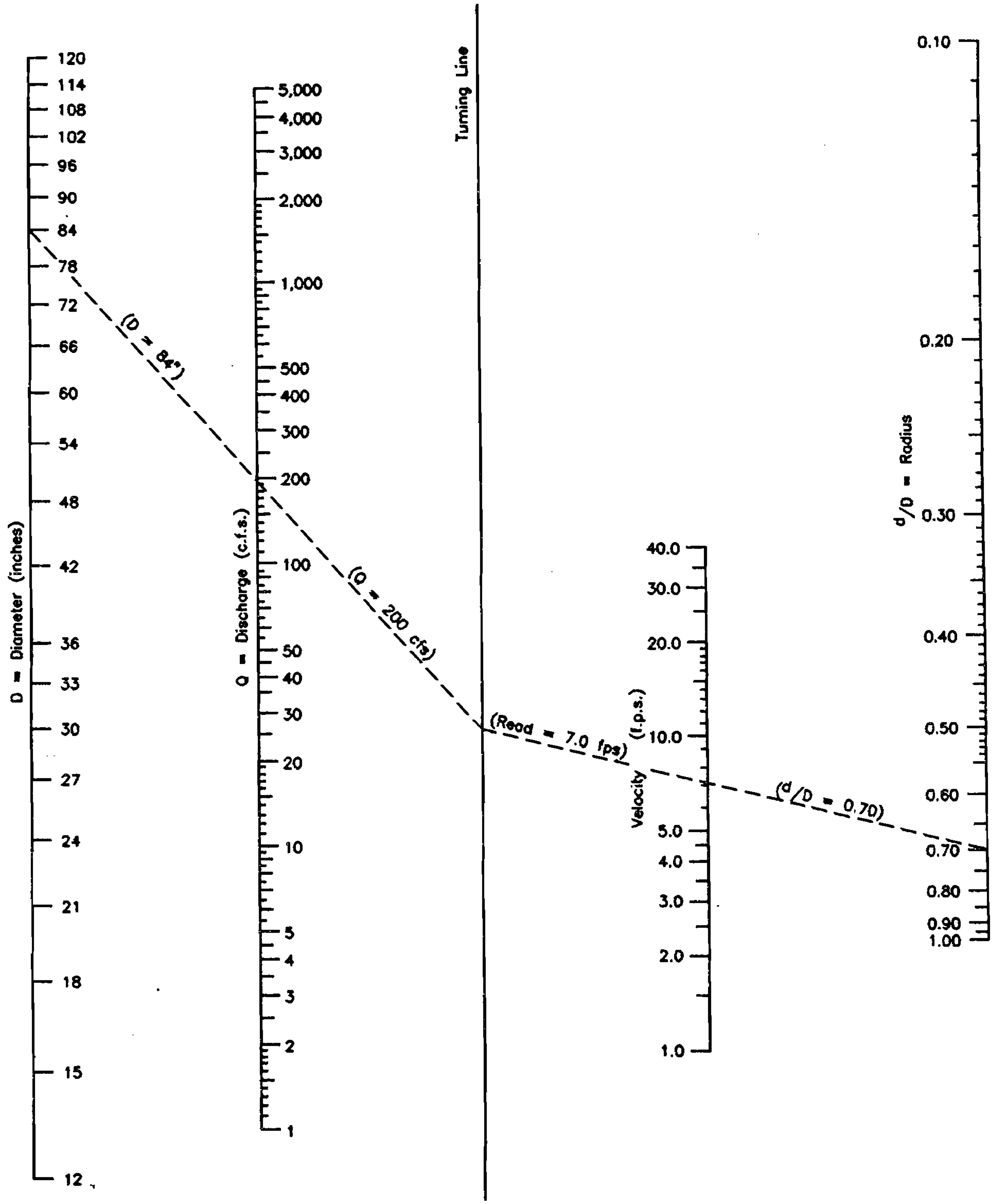
Critical Depth for Circular Conduits

CITY OF AMARILLO

Storm Water Management Criteria Manual

Date
NOV 19

Figure
5-2



Source: Texas Highway Department.

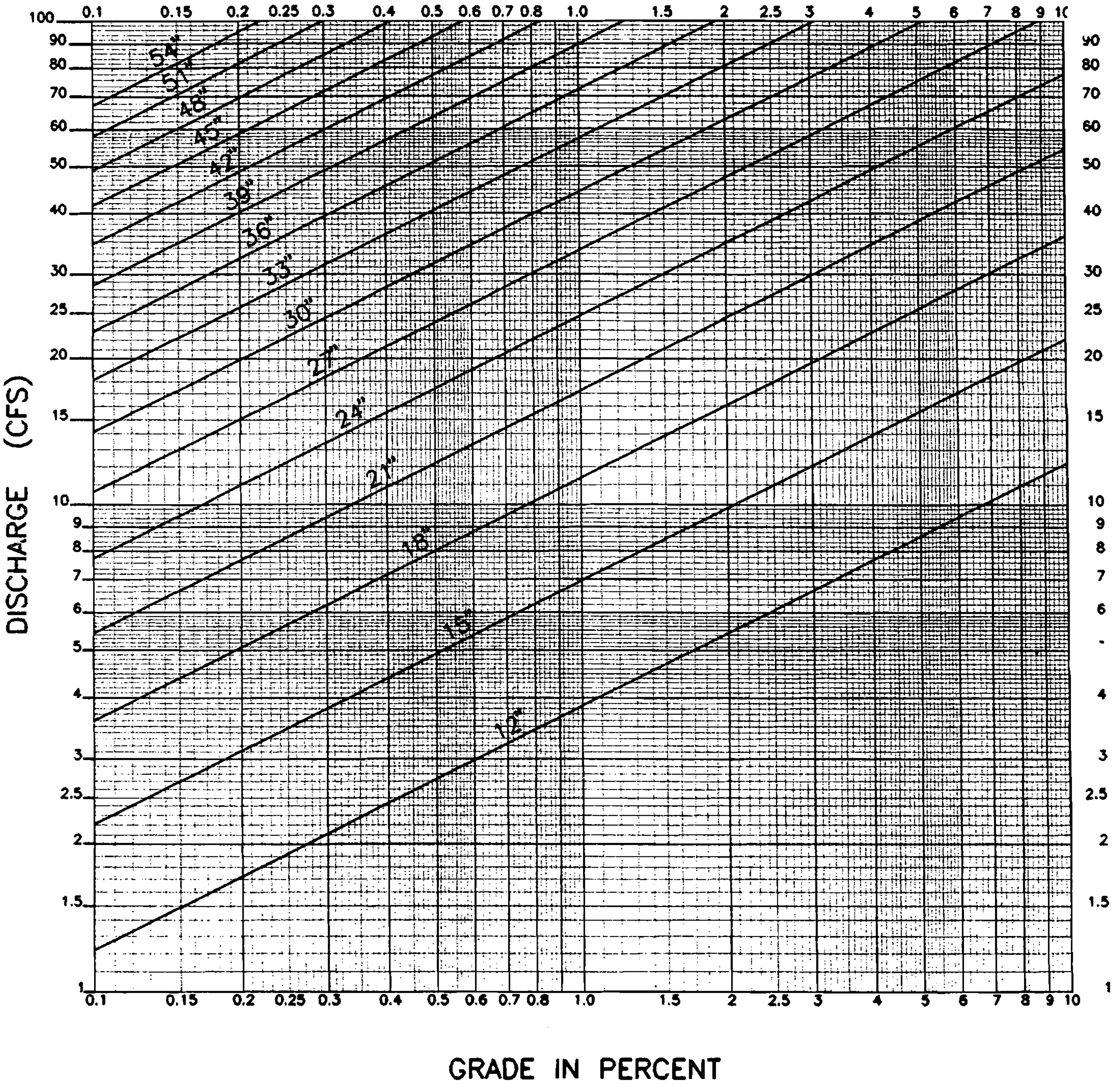
CITY OF AMARILLO

Velocity for Circular Conduits

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 5-3



Source: *City of Amarillo Public Works Division.*

CITY OF AMARILLO

Discharge Capacity for Concrete Pipe Flowing Full n = 0.012
Pipe Diameters 12" to 54"

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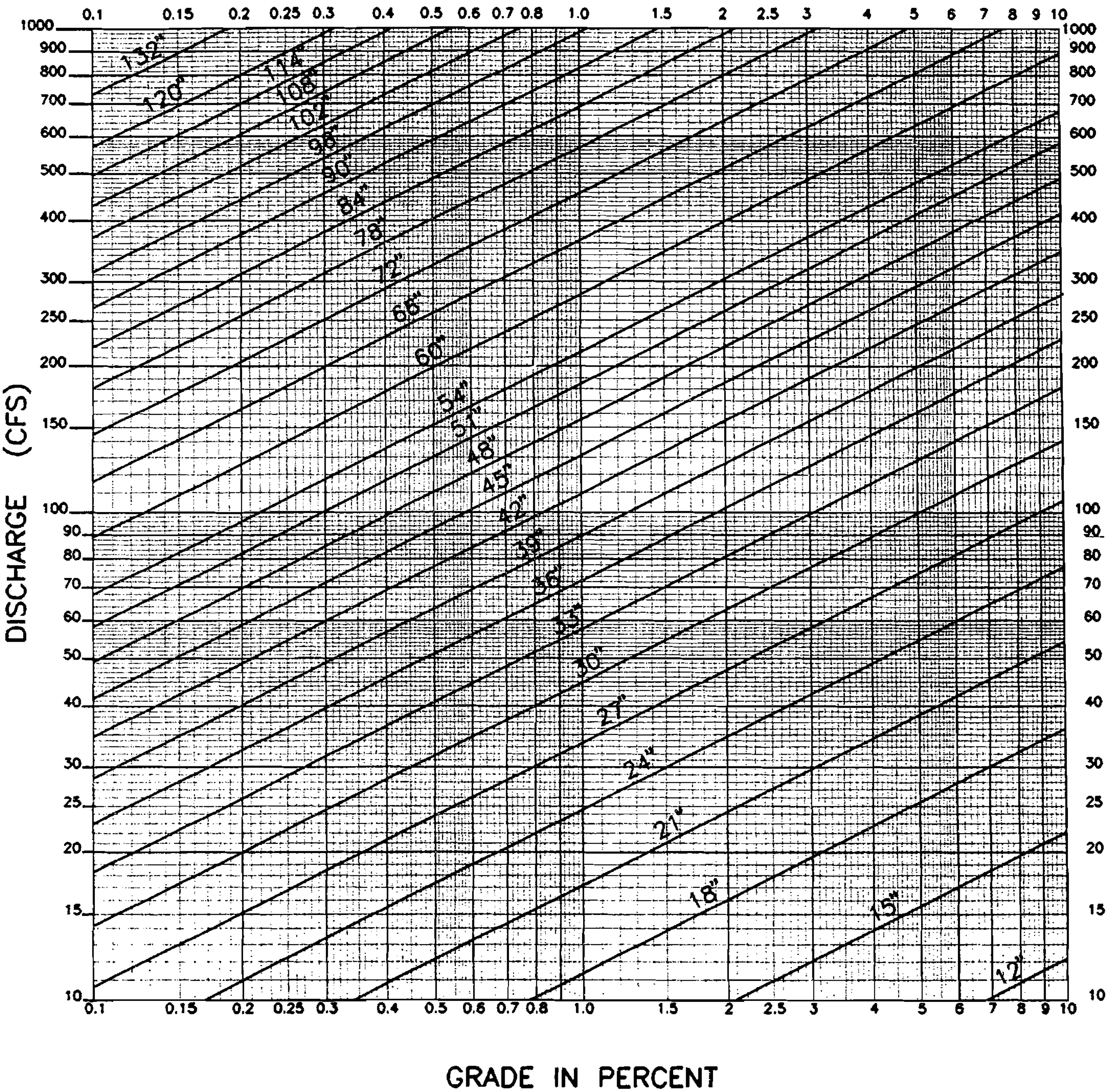
NOV 19__

Figure
5-4



Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Discharge Capacity for Concrete Pipe Flowing Full n = 0.012
Pipe Diameters 12" to 132"

Storm Water Management Criteria Manual

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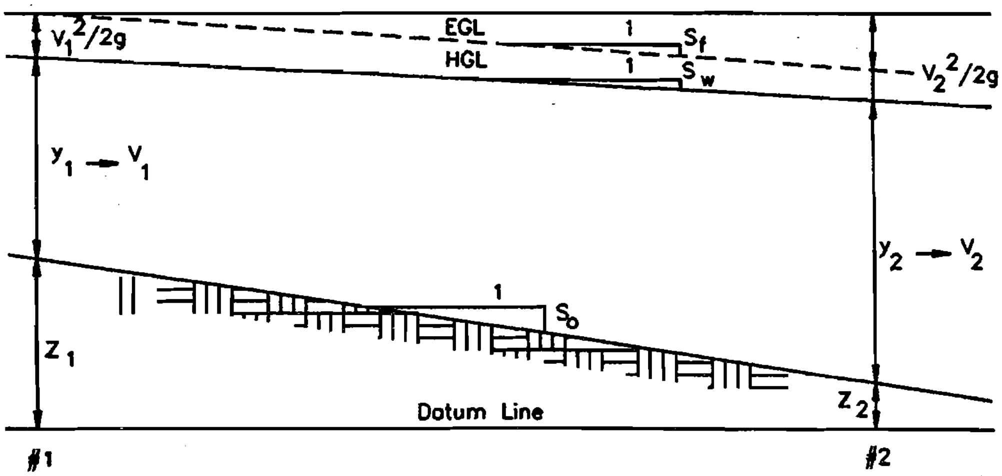
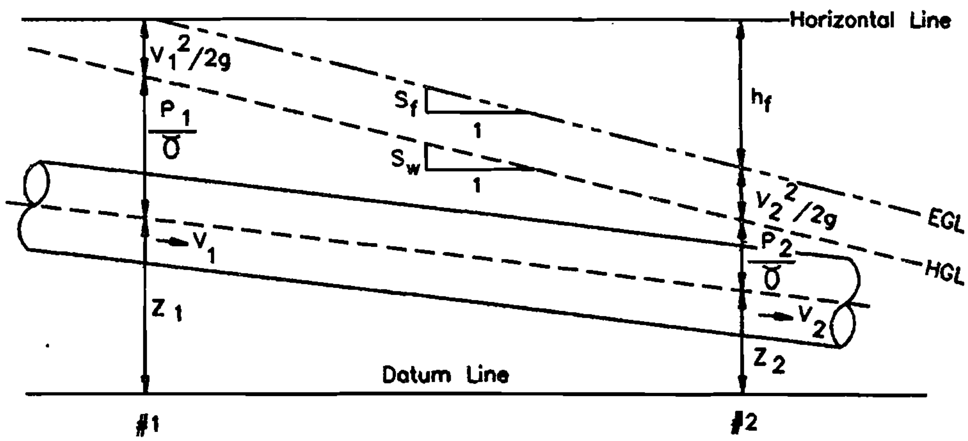
NOV 1992

Figure
5-5



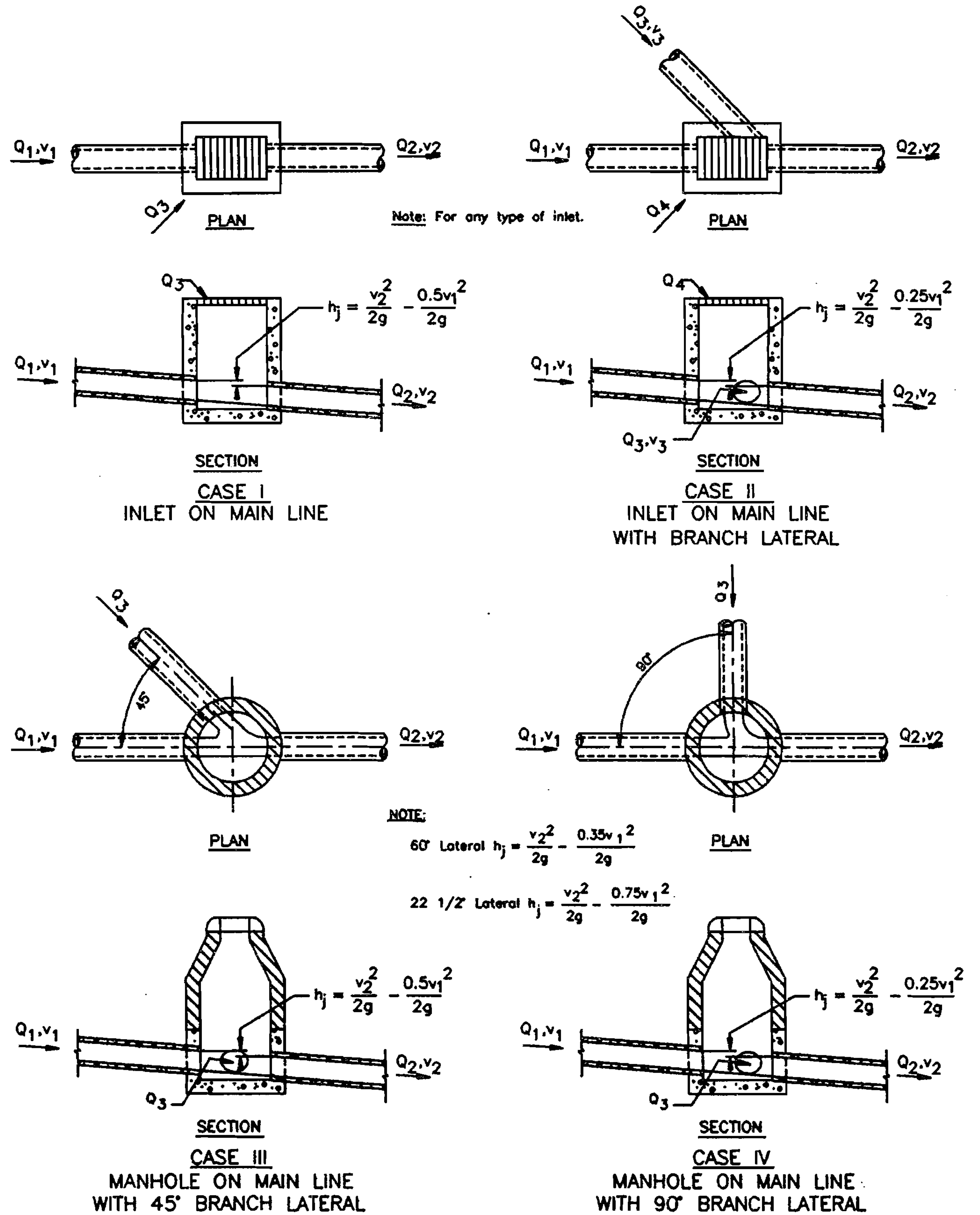
Energy in open–channel flow



Energy in closed–conduit flow

Source: American Iron and Steel Institute, Modern Sewer Design.

Energy in Open-Channel Flow and Closed Conduit Flow



Source: City of Waco, Texas, Storm Drainage Criteria Manual.

Minor Head Losses Due to Turbulence at Structures

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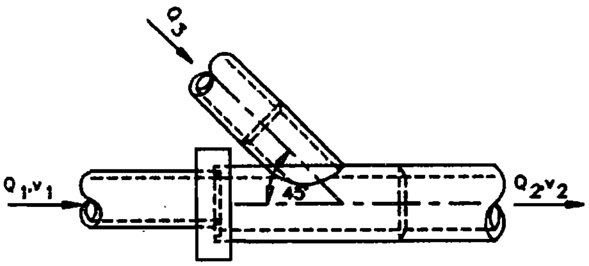
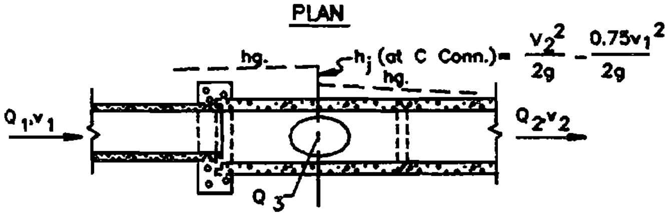
Figure 5-7

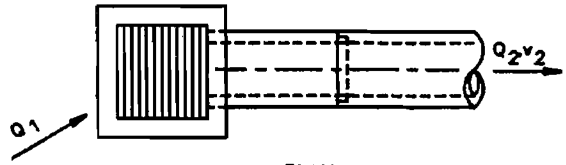


PLAN

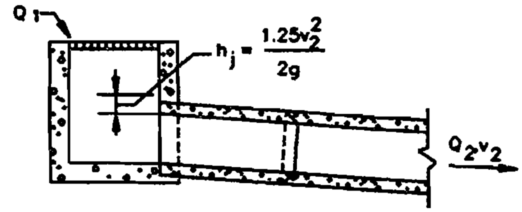


SECTION

CASE V
45° WYE CONNECTION
OR CUT IN

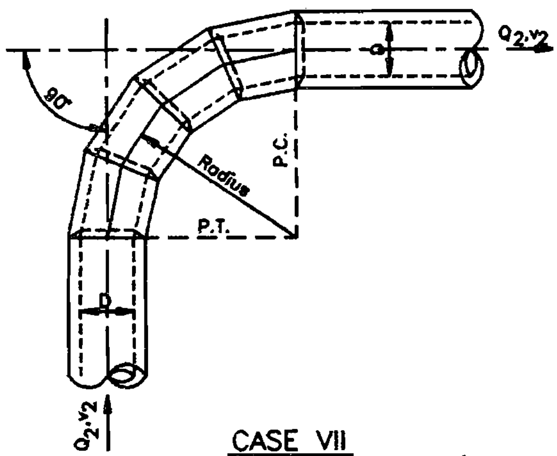


PLAN



SECTION

CASE VI
INLET OR MANHOLE AT
BEGINNING OF LINE



CASE VII
CONDUIT ON 90° CURVES*

NOTE: Head loss applied at P.C. for length of curve.

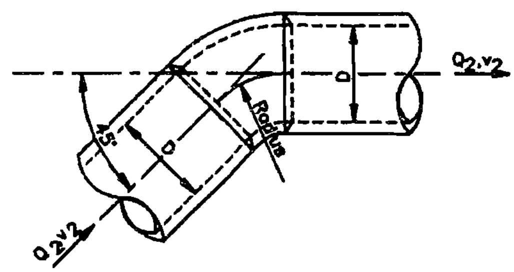
Radius = Dia. of pipe $h_j = 0.50 \frac{v_2^2}{2g}$

Radius = (2–8)Dia. of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$

Radius = (8–20)Dia. of Pipe $h_j = 0.40 \frac{v_2^2}{2g}$

Radius = Greater than 20 Dia. of Pipe $h_j = 0$

* When curves other than 90° are used, apply the following factors to 90° curves.
60° curve 85%
45° curve 70%
22½ curve 40%



CASE VIII
BEND WHERE RADIUS IS
EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at beginning of bend

90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$

60° Bend $h_j = 0.43 \frac{v_2^2}{2g}$

45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$

22½ Bend $h_j = 0.20 \frac{v_2^2}{2g}$

Source: City of Waco, Texas, Storm Drainage Criteria Manual.

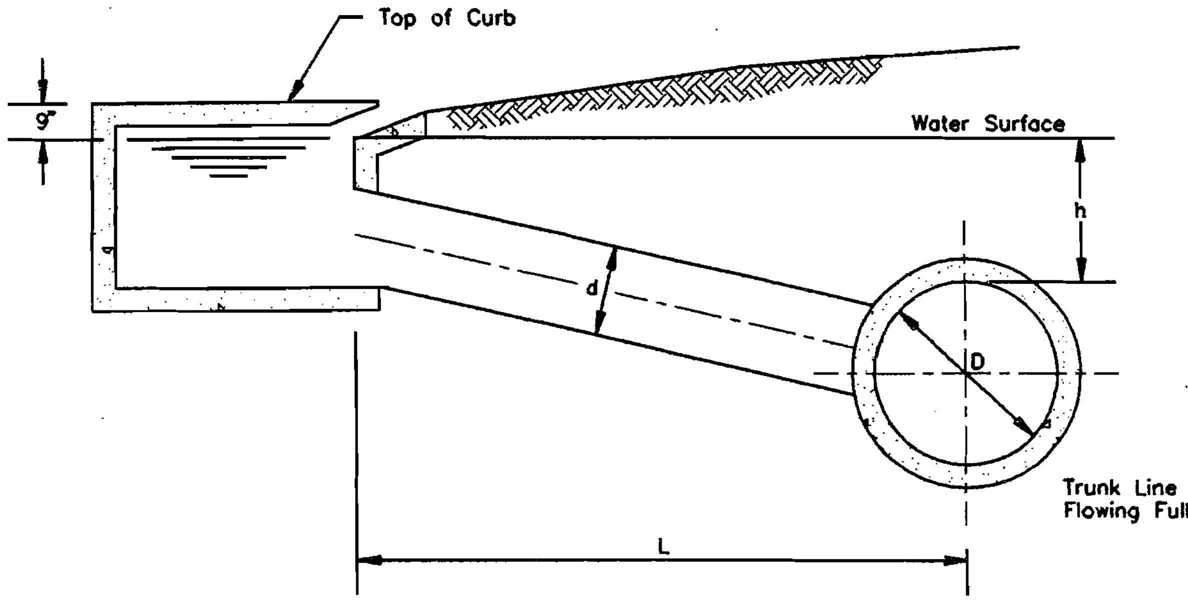
CITY OF AMARILLO

Minor Head Losses Due to Turbulence at Structures

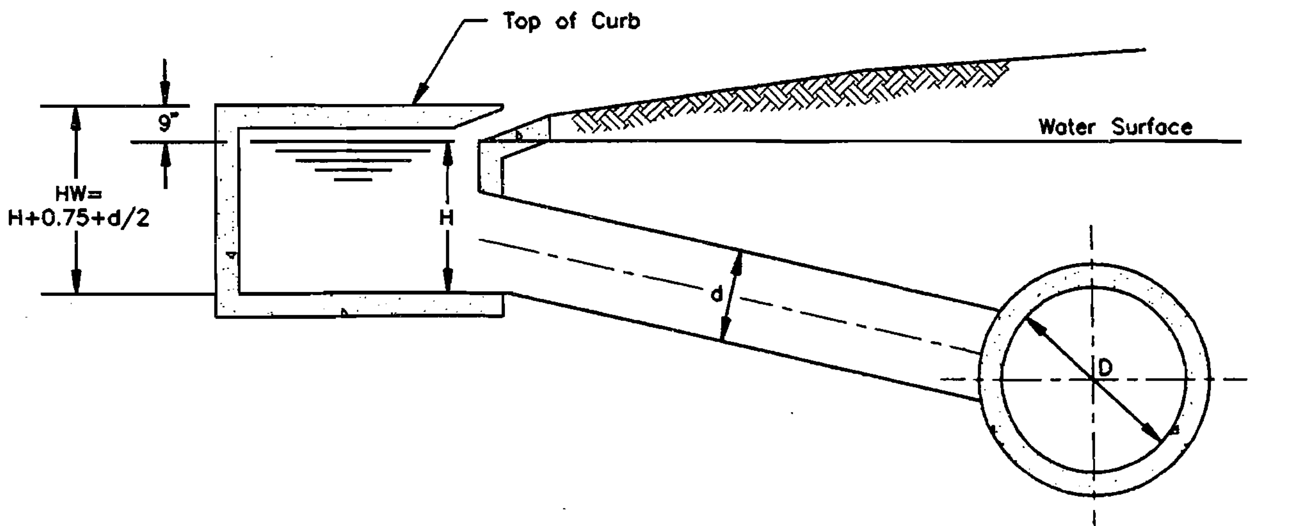
Storm Water Management Criteria Manual

Date NOV 19__

Figure 5-8



OUTLET CONTROL



INLET CONTROL

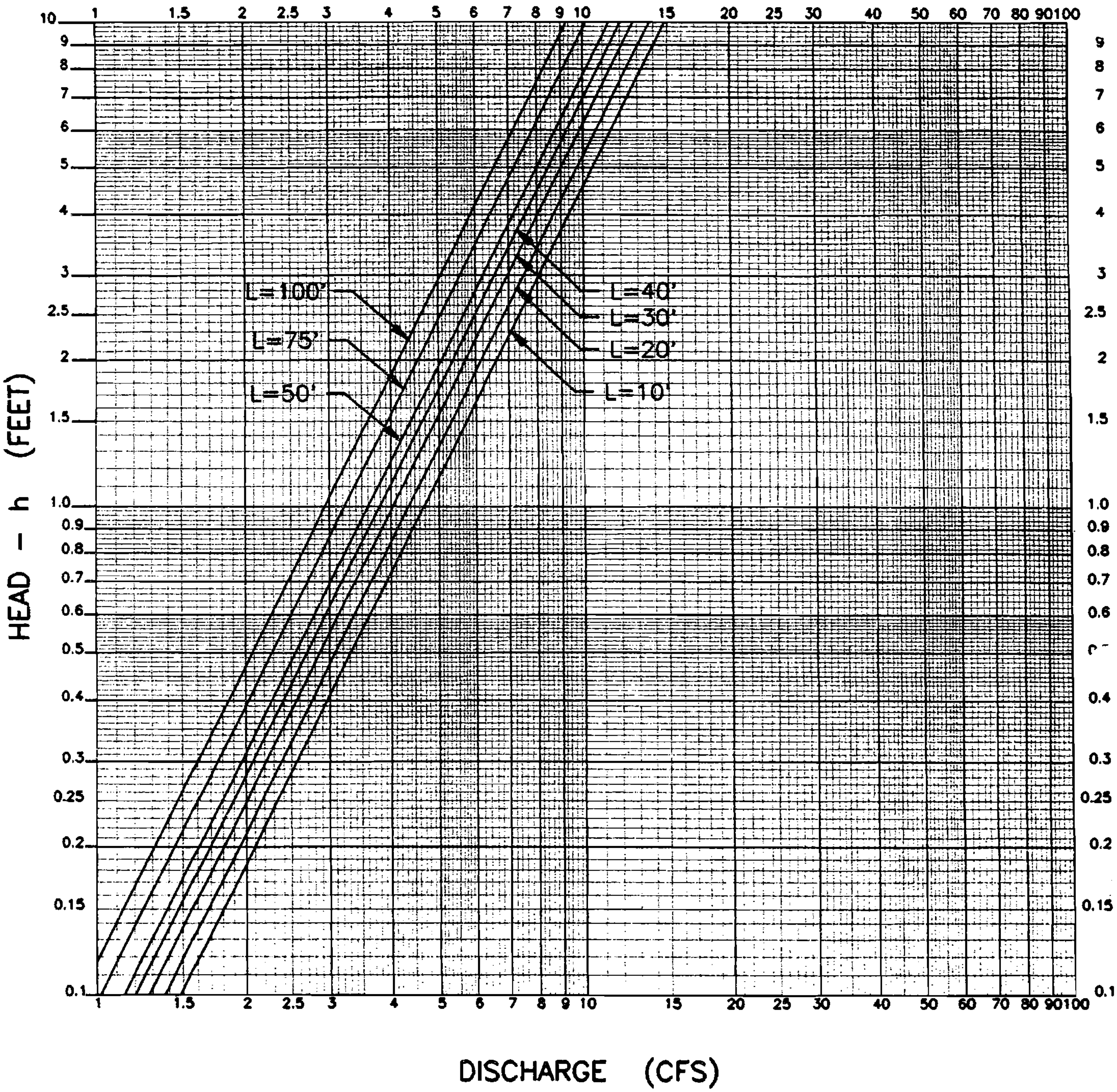
Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Inlet Lateral Definition Sketch
Flow Condition

Storm Water Management Criteria Manual

Date	NOV 1992
Figure	5-9



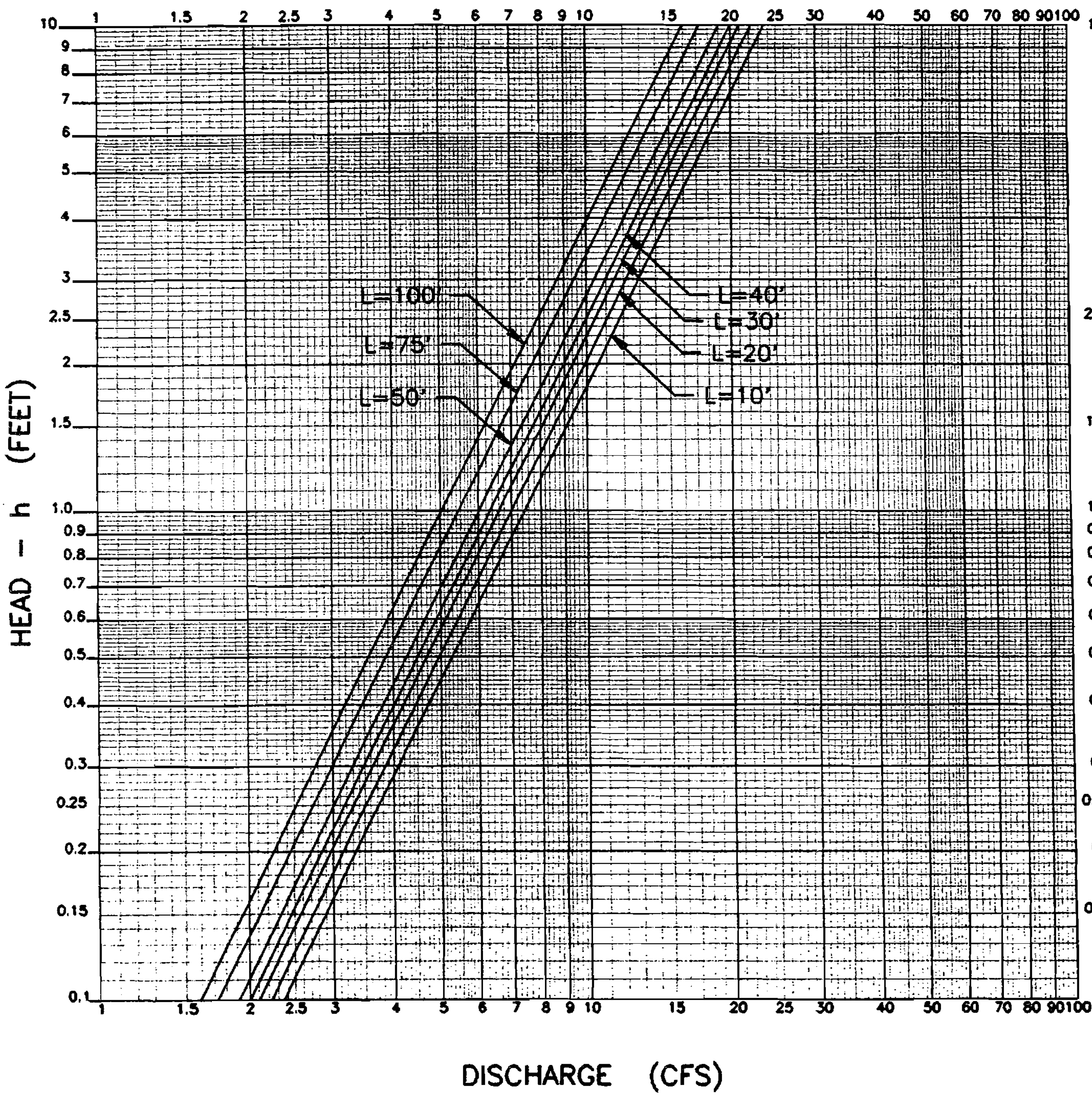
Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
12" Diameter Pipe

Storm Water Management Criteria Manual

Date	NOV 199_
Figure	5-10



Source: *City of Amarillo Public Works Division.*

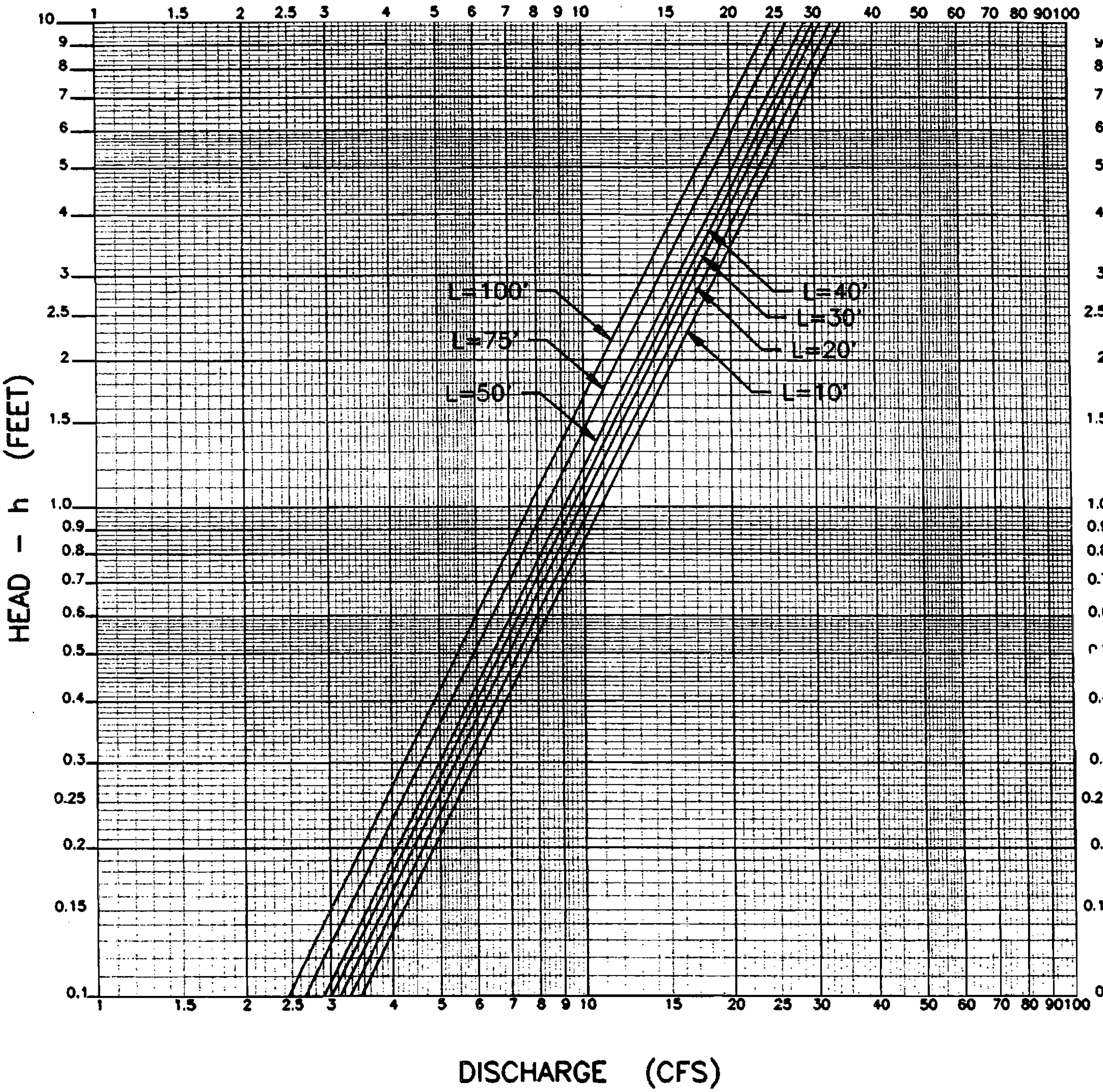
CITY OF AMARILLO

Inlet Lateral Pipe Design Outlet Control 15" Diameter Pipe

Storm Water Management Criteria Manual

Date NOV 1992

Figure 5-11



Source: City of Amarillo Public Works Division.

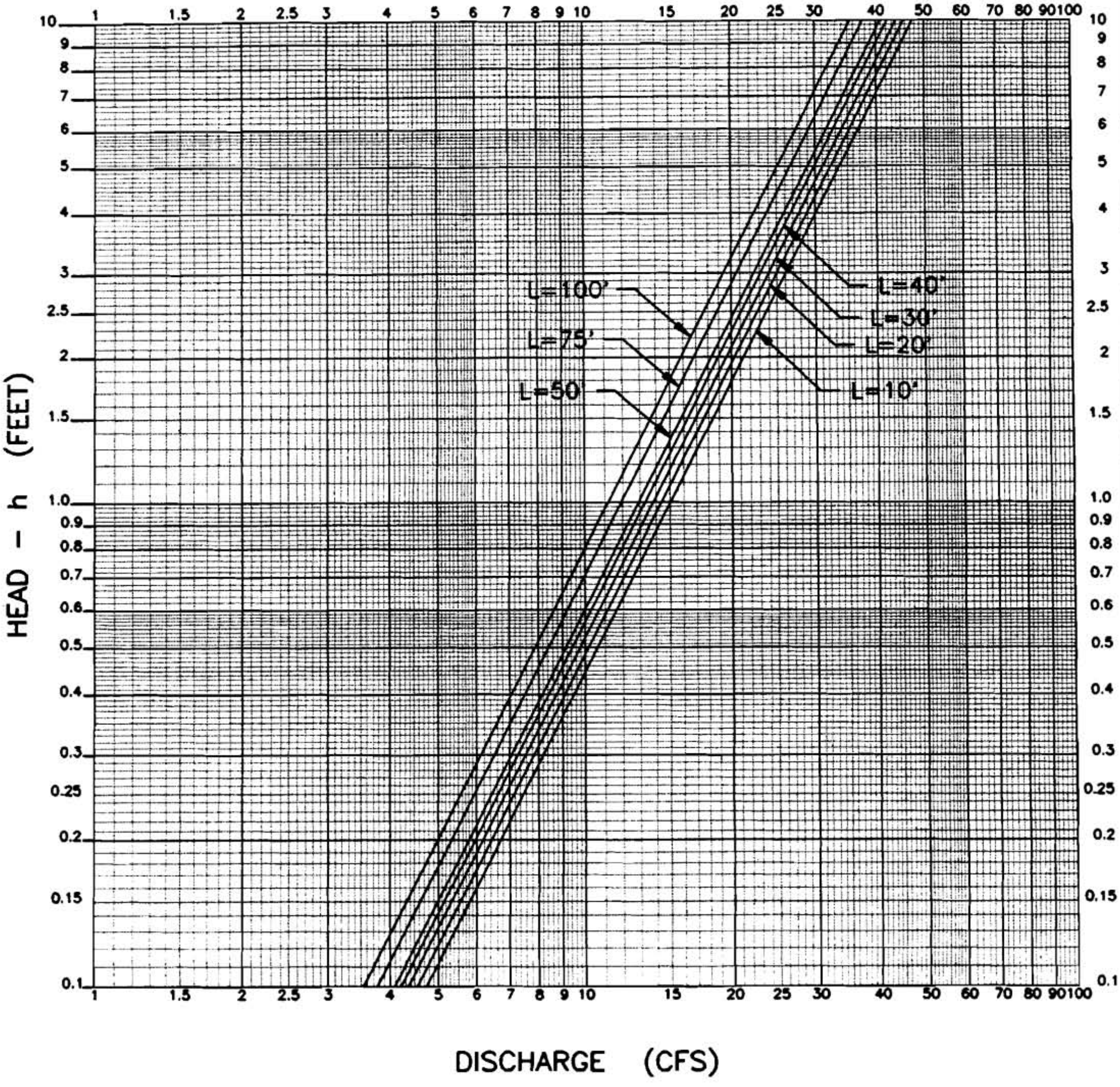
CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
18" Diameter Pipe

Storm Water Management Criteria Manual

Date
NOV 19

Figure
5-12



Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
21" Diameter Pipe

Storm Water Management Criteria Manual

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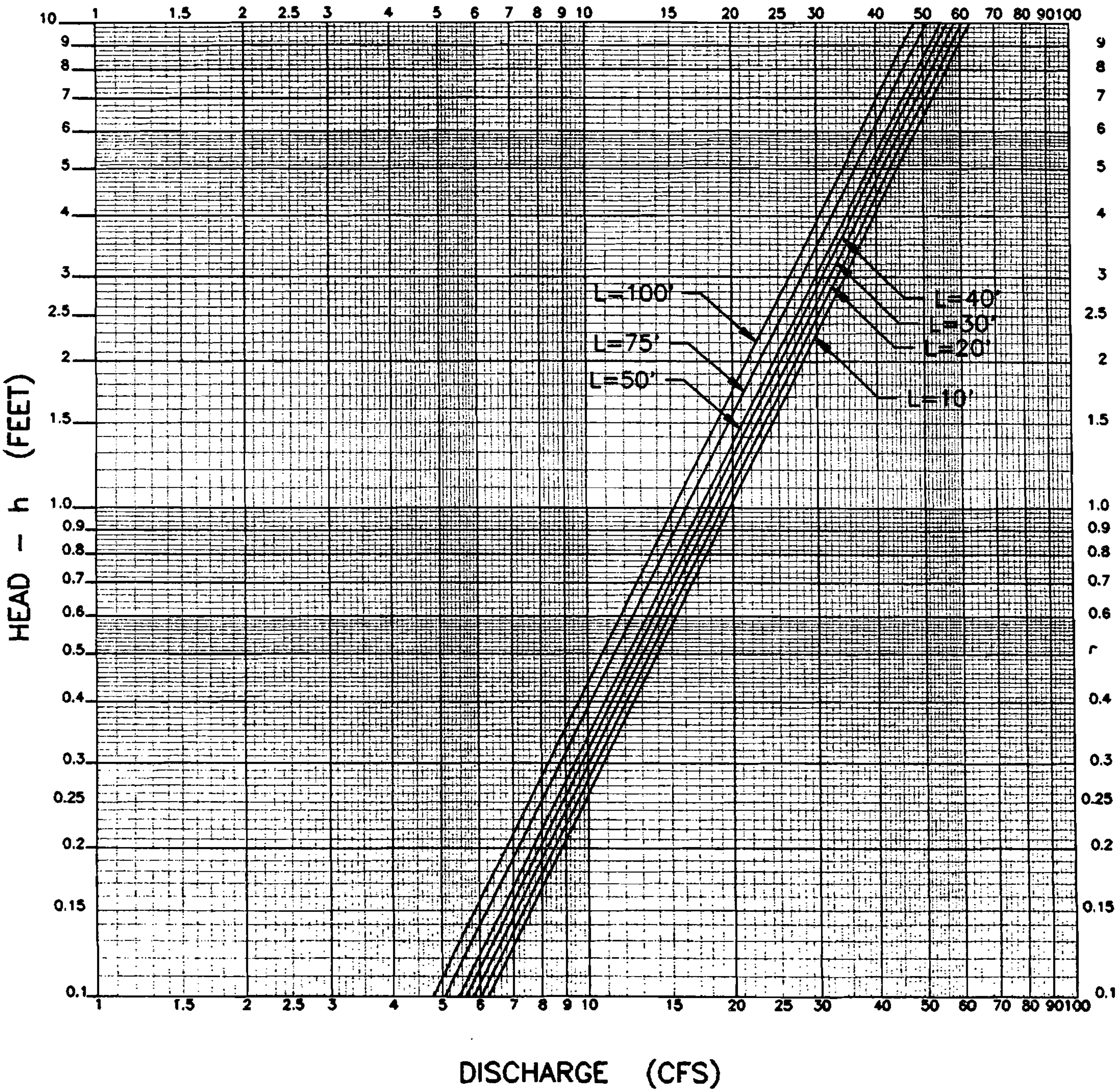
NOV 1992

Figure
5-13



Source: City of Amarillo Public Works Division.

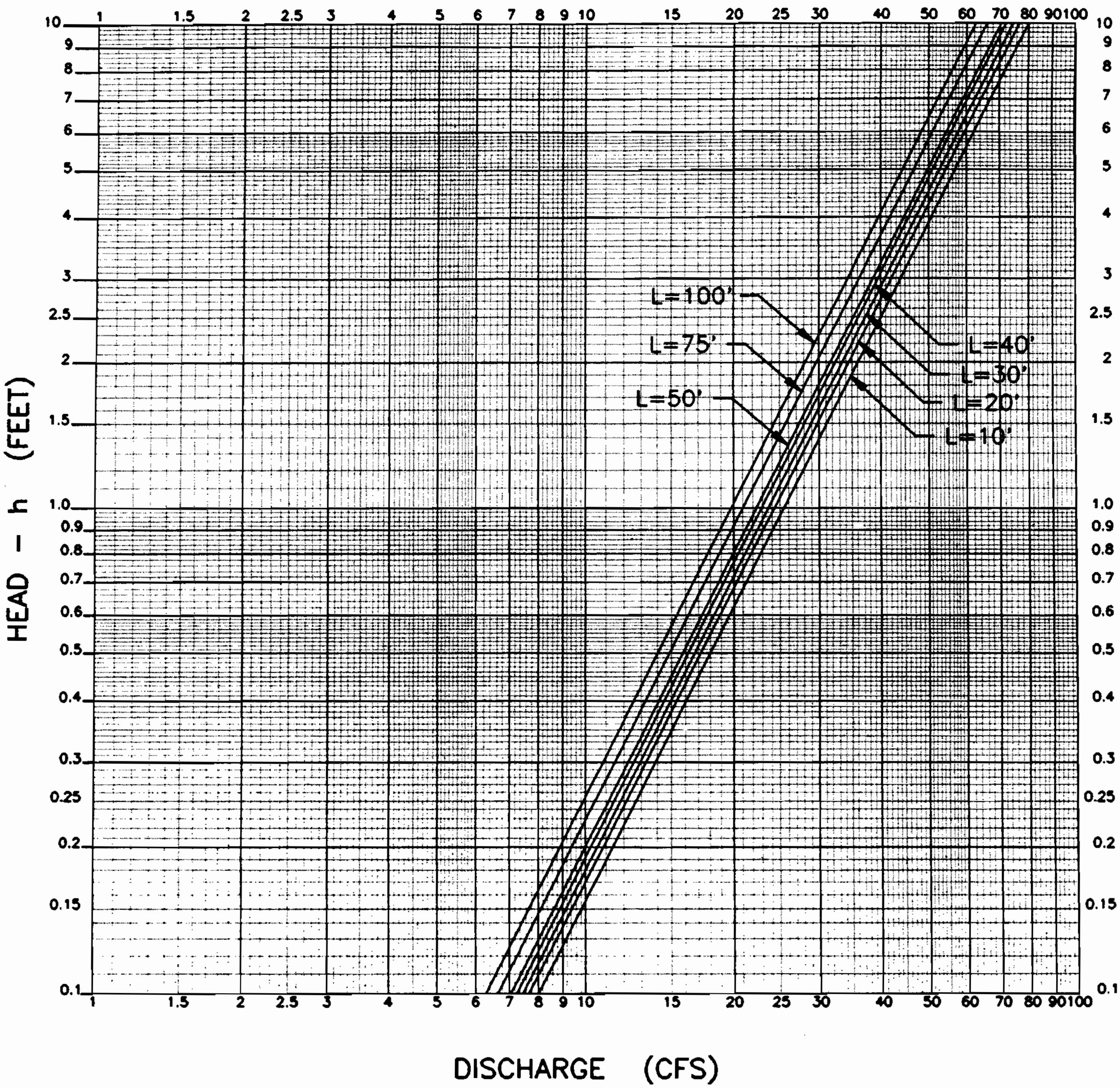
CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
24" Diameter Pipe

Storm Water Management Criteria Manual

Date: NOV 199_

Figure: 5-14



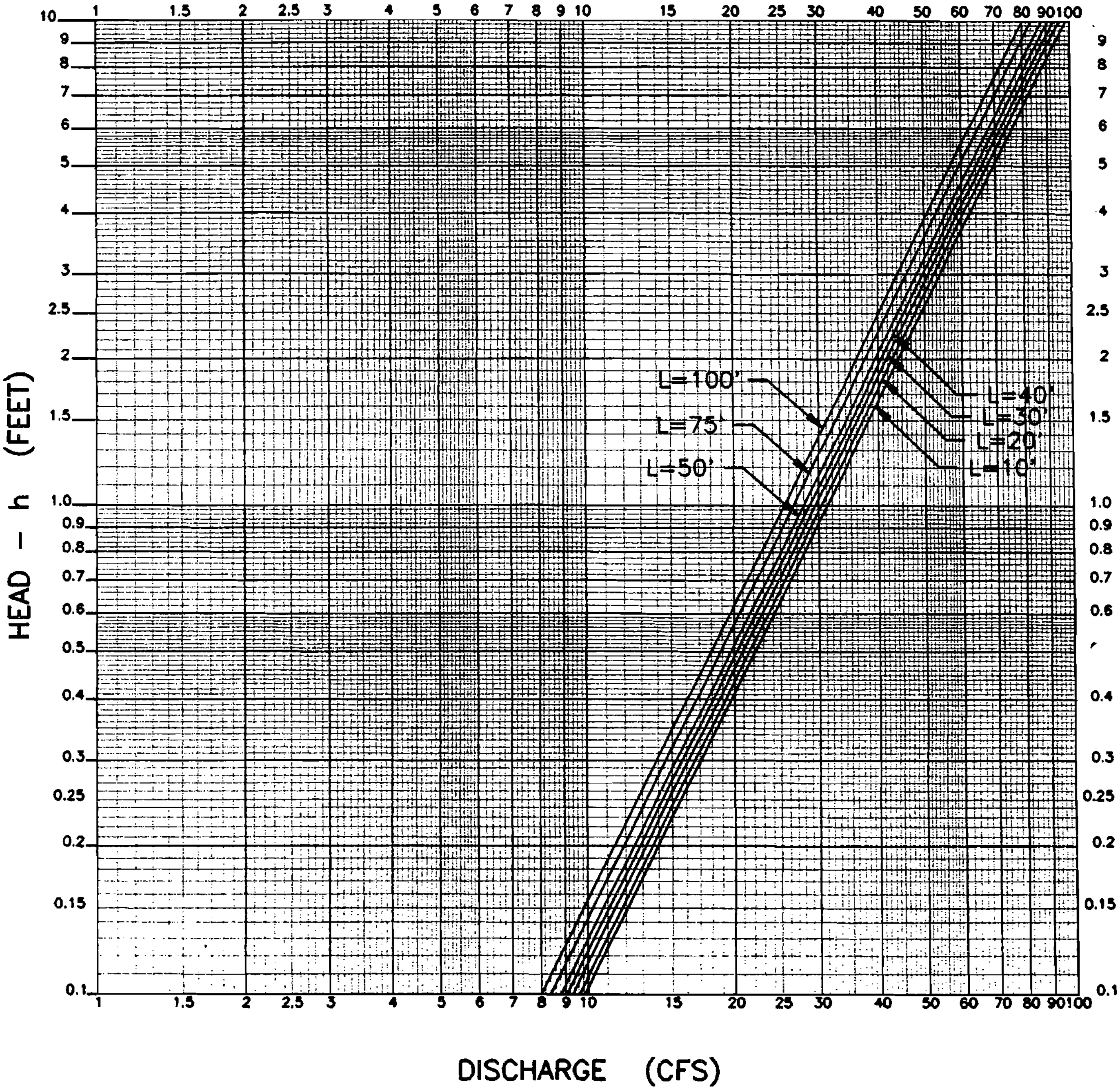
Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
27" Diameter Pipe

Storm Water Management Criteria Manual

Date	Figure
NOV 1992	5-15



Source: City of Amarillo Public Works Division.

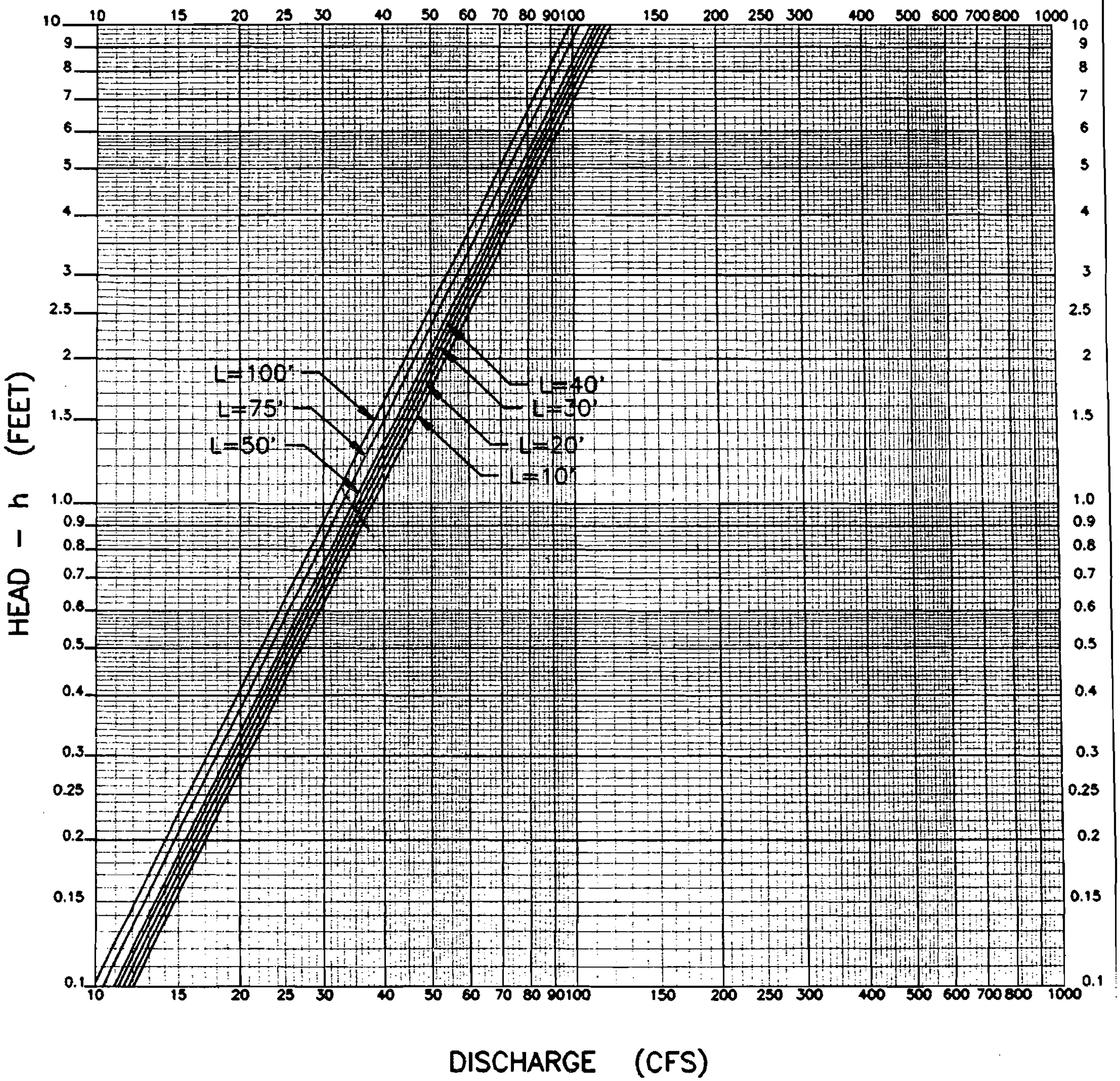
CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
30" Diameter Pipe

Storm Water Management Criteria Manual

Date
NOV 199_

Figure
5-16



Source: *City of Amarillo Public Works Division.*

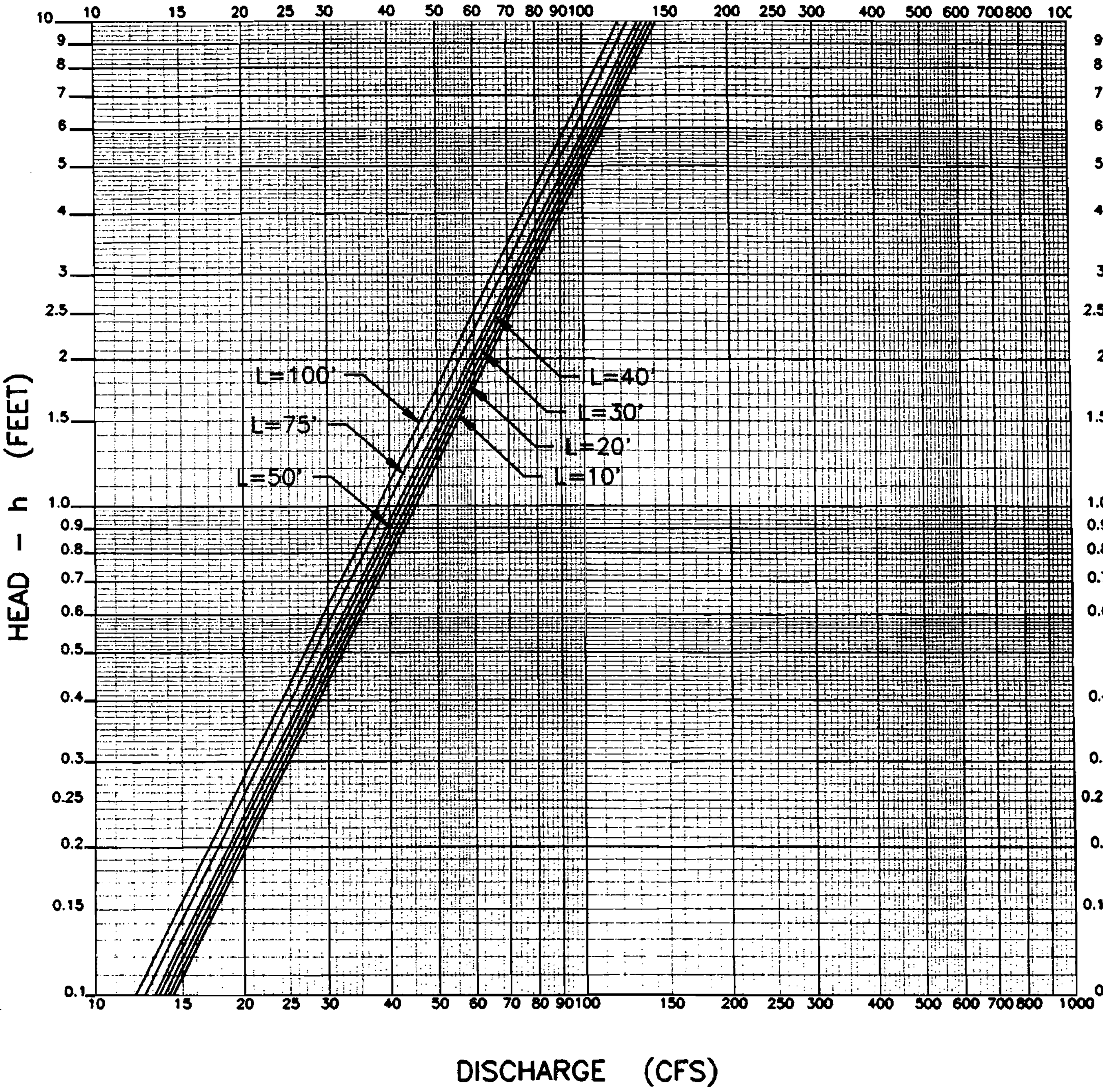
CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
33" Diameter Pipe

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 5-17



Source: City of Amarillo Public Works Division.

CITY OF AMARILLO

Inlet Lateral Pipe Design
Outlet Control
36" Diameter Pipe

Storm Water Management Criteria Manual

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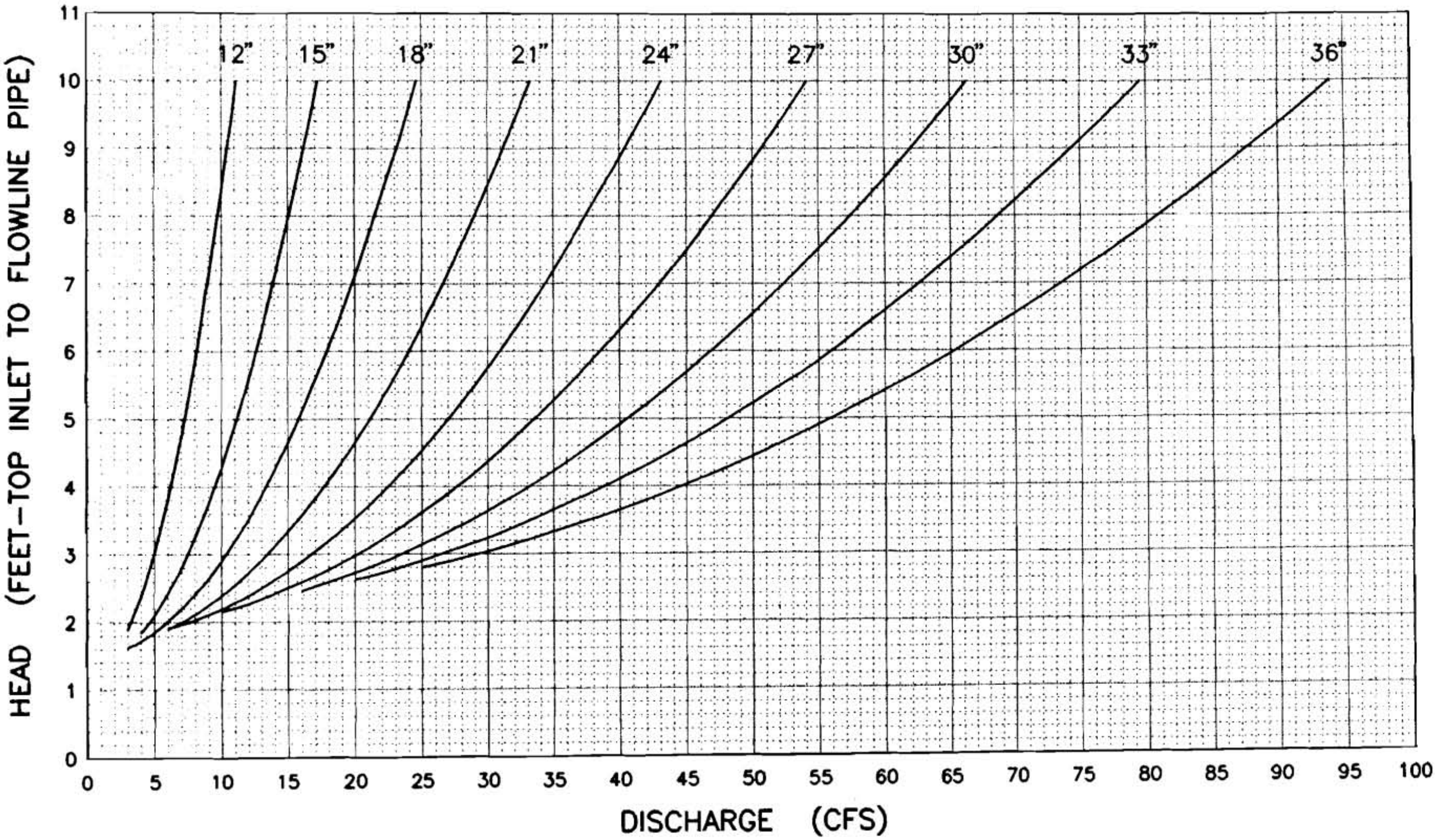
Figure: 5-18



Source: *City of Amarillo Public Works Division.*

CITY OF AMARILLO

Lateral Pipe Design
Inlet Control
12"-36" Diameter Pipes

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
5-19

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BY ______ DATE ______ CK'D ______ DATE ______

HYDROLOGIC AND HYDRAULIC COMPUTATIONS FOR STORM SEWERS
RATIONAL METHOD

LINE ______ "n" = ______

LOCATION ______ FROM ______ TO ______
MAJOR WATERSHED ______
CONTRACT OR FILE NO. ______

MANHOLE OR INLET		DIST. BETWEEN MANHOLE OR INLET	DRAINAGE AREA			RUNOFF COEFFICIENT "C"	INCREMENTAL "CA"	TOTAL "CA"	TIME OF CONCENTRATION			DESIGN FREQUENCY	INTENSITY	DISCHARGE "Q"	SELECTED SIZE OF PIPE	SLOPE OF FRICTIONAL GRADIENT "S_f"	ENERGY GRADIENT ELEVATIONS		HEAD LOSS AT CHANGE IN SECTION							ELEV. E.G. AT DESIGN POINT (COL. 18+COL. 26)	ELEV. OF INVERT AT DESIGN POINT (COL. 1)		NOTES
FROM / DESIGN POINT	TO		INCREMENT AREA NO.	INCREMENT AREA AREA	TOTAL AREA				INLET TIME	TIME IN SEWER	TOTAL TIME						UP STREAM	DOWN STREAM	V_1 INFLOW	V_2 OUTFLOW	$\frac{V_2^2}{2g}$	$\frac{V_1^2}{2g}$	K_m	$\frac{K_m V_1^2}{2g}$	h_j		IN COMING PIPE	OUT GOING PIPE	
		(FT)		(AC)	(AC)				(MIN)	(MIN)	(MIN)	(YRS)	(IN/HR)	(CFS)	(IN)	(FT/FT)	(FT MSL)		(FT/SEC)		(FT)		CONSTANT	(FT)		(FT MSL)	(FT MSL)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	

REMARKS, SKETCHES AND COMPUTATIONS:

Source: City of Waco, Texas, Storm Drainage Criteria Manual.

Sheet ___ of ___

CITY OF AMARILLO

Hydrologic and Hydraulic Computations for Storm Sewers Rational Method

Storm Water Management Criteria Manual

Date NOV 1992

Figure 5-20

BY ______ DATE ______ CK'D ______ DATE ______

HYDROLOGIC AND HYDRAULIC COMPUTATIONS FOR STORM SEWERS AMARILLO PEAK FLOW CURVE METHOD [1]

LINE ______ "n" = ______

LOCATION ______ FROM ______ TO ______
MAJOR WATERSHED ______
CONTRACT OR FILE NO. ______

MANHOLE OR INLET		DIST. BETWEEN MANHOLE OR INLET	DRAINAGE AREA			SCS CURVE NUMBER "CN"	ELEVATIONS		BASIN LENGTH	BASIN SLOPE	DESIGN FREQUENCY	DISCHARGE[1]		SELECTED SIZE OF PIPE	SLOPE OF FRICTIONAL GRADIENT "S_f"	ENERGY GRADIENT ELEVATIONS		HEAD LOSS AT CHANGE IN SECTION							ELEV. E.G. AT DESIGN POINT (COL. 17+COL. 25)	ELEV. OF INVERT AT DESIGN POINT (COL. 1)		NOTES
FROM / DESIGN POINT	TO		INCREMENT AREA NO.	INCREMENT AREA AREA	TOTAL AREA		UP STREAM	DOWN STREAM				UNIT	DESIGN			UP STREAM	DOWN STREAM	V_1 INFLOW	V_2 OUTFLOW	$\frac{V_2^2}{2g}$	$\frac{V_1^2}{2g}$	K_m	$\frac{K_m V_1^2}{2g}$	h_j		IN COMING PIPE	OUT GOING PIPE	
		(FT)		(AC)	(AC)		(FT MSL)		(FT)	(%)	(YRS)	(CFS/AC)[1]	(CFS)	(IN)	(FT/FT)	(FT MSL)		(FT/SEC)		(FT)		CONSTANT	(FT)		(FT MSL)	(FT MSL)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	

REMARKS, SKETCHES AND COMPUTATIONS:

1 From Amarillo, Texas Peak Flow Curves

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Page

LIST OF TABLES

LIST OF FIGURES

6.1 INTRODUCTION

Open channels designed for use in the major drainage system have significant advantages in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for instream storage and groundwater recharge. Disadvantages include potential right-of-way constraints and maintenance costs. Careful planning and design are needed to increase the benefits and to minimize the disadvantages.

Hydraulic structures include, stilling basins, channel drops, transitions, baffle chutes, and many other specific drainage works. Their shape, size, and other features vary widely depending upon the function to be served on a specific project. In general, a hydraulic structure is used to retain, regulate, or control the flow of water.

The ideal open channel is a stabilized water course developed by nature over time, characterized by stable bed and banks. The benefits of such a channel are:

A. Available channel storage can decrease peak flows.

B. Maintenance needs can be low when the channel is properly stabilized.

C. Natural subsurface infiltration of flows is provided.

D. Native vegetation and wildlife may not have to be disturbed.

E. The channel can provide a desirable green belt and recreational area adding significant social benefits.

Generally speaking, a stabilized natural channel, or the artificial, man-made channel which most nearly conforms to the character of a stabilized natural channel, is the most efficient and the most desirable.

Channel stability, particularly in unprotected alluvial materials, is a problem in urban hydrology because of the significant increase in low flow and peak storm runoff rates. A natural channel must be studied in sufficient detail to determine the measures needed to mitigate potential bottom scour and bank cutting. Erosion control measures can be provided at reasonable cost which will preserve the natural appearance without sacrificing hydraulic efficiency. This section provides the necessary criteria and methodology for selection and design of open channels.

6.2 DESIGN CRITERIA

6.2.1 Design Frequency

Open channels shall be designed such that residential dwellings or public, commercial and industrial buildings shall not be inundated at the lowest finished floor elevation for the 100-year storm event unless the building is flood-proofed.

6.2.2 Maintenance Easement

A dedicated maintenance easement shall be provided with all drainage channels. This easement shall provide a minimum access width of 20 feet from the channel bank on each side unless otherwise approved by the City Engineer. For some small channels, this easement may be provided on one side only. These dedicated maintenance easements shall be sufficiently cleared and graded to allow easy access by maintenance equipment.

6.3 TYPES OF CHANNELS

Channels are defined as natural or artificial. Natural channels include all water courses that have developed by the erosion process. Artificial channels are those constructed or significantly altered by human effort and include roadside ditches and grassed or improved channels.

6.3.1 Natural Channels

Many natural channels have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural channel is to be used for carrying storm runoff from an urbanizing area, the altered nature of the runoff peaks and volumes from urban development can and will cause scour and erosion. Hydraulic analyses will be required for natural channels in order to identify the erosion tendencies. Some on-site modification of the natural channel may be required to assure a stabilized condition.

The investigations necessary to assure that the natural channels will be adequate are different for every waterway. The engineer/designer must prepare cross sections of the channel, define the water surface profile for the minor and major design flood, investigate the bed and bank material to determine erosion tendencies, and study the bank slope stability of the channel under flow conditions. Supercritical flow does not normally occur in natural channels, but calculations must be made to assure that the results do not reflect supercritical flow.

6.3.2 Grass-Lined Channels

Grass-lined channels are the most desirable of the artificial channels. The grass will stabilize the body of the channel, consolidate the soil mass of the bed, check the erosion on the channel surface, and control the movement of soil particles along the channel bottom. The channel storage, the lower velocities, and the greenbelt multiple-use-benefits obtained create significant advantages over other artificial channels.

The presence of grass in channels creates turbulence which results in loss of energy and increased flow retardance. Therefore, the designer must give full consideration to sediment deposition and scour, as well as hydraulics.

6.3.3 Concrete-Lined Channels

Concrete linings must be designed to withstand the various forces and actions which tend to overtop the bank, deteriorate the lining, erode the soil beneath the lining, and erode unlined areas.

Maintenance responsibility should be a part of any drainage plan requiring concrete-lined channels.

If the project constraints dictate the use of a concrete channel, such use shall be allowed only upon approval by the City Engineer.

6.3.4 Rock-Lined Channels

Rock-lined channels are constructed from ordinary riprap or wire enclosed riprap (gabions). The rock lining increases the turbulence resulting in a loss of energy and increased flow retardance. The rock lining also permits a higher design velocity and therefore a steeper design slope than for grass-lined channels. Rock linings are also used for erosion control at culvert/storm drain outlets, at sharp channel bends, at channel confluences, and at locally steepened channel sections. Incorrectly designed rock-lined channels can result in excessive maintenance requirements. Correct sizing and bedding are essential to good performance. Maintenance responsibility should be a part of any drainage plan utilizing rock-lined channels.

If the project constraints dictate the use of a riprap or gabion lining, such use shall be allowed only upon approval of the City Engineer. Riprap for the purposes of local erosion control is permitted only if vegetation is unsuitable.

6.4 CHANNEL DISCHARGE

Understanding the basic concepts of open channel flow is necessary to properly design channels. In open channel flow, the water surface is not confined. Surface configuration, flow pattern and pressure distribution within the flow depends on gravity. In rigid-boundary open channel flow, no deformation or movement of the bed and banks is assumed, whereas in mobile-boundary hydraulics, bed configuration is considered a function of the flow. Discussions in this section pertain primarily to rigid-boundary open-channel flow, since they are most appropriate to open channel design in the Amarillo area.

Designing a stable alluvial channel (one without a channel lining) or a stable lined channel under dynamic channel conditions requires an understanding of sediment transport and stream channel response. For example, unlined channels must be designed to minimize excessive scour while lined channels must be designed to prevent deposition of sediments. Unlined channels are most successful when designed under the concept of dynamic equilibrium. These topics and other related design considerations are discussed in detail in basic fluid mechanics and sediment transport textbooks.

All variables used in fluid mechanics and hydraulics fall into one of three classes: those describing the boundary geometry, those describing the flow, and those describing the fluid.

Various combinations of these variables define parameters that describe the state of flow in open channels.

6.4.1 Manning's Equation

Careful attention must be given to the design of drainage channels to provide adequate capacity and allow for minimum maintenance. The hydraulic characteristics of open channels shall be determined by using Manning's Equation, commonly expressed as:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \tag{6-1}$$

where:

Q	=	channel discharge, in cubic feet per second
A	=	cross-sectional area of channel flow, in square feet
R	=	hydraulic radius of channel, A/WP , in feet
WP	=	wetted perimeter of channel flow, in feet
S	=	slope of the energy gradient, in feet per foot
n	=	Manning's coefficient of channel roughness

A graphical solution for the Manning's Equation is presented on Figure 6-1.

6.4.2 Uniform Flow

Manning's Equation is an accurate representation of flow conditions only when the rate of flow and channel characteristics (roughness, cross section geometry and slope) remain relatively constant, hence, uniform flow. For a channel of given roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow. This depth is commonly expressed as the normal depth. The corresponding discharge is expressed as the normal discharge. Under uniform flow conditions, the water surface profile is assumed parallel to both the energy grade line and the bottom of the channel.

Uniform flow is most often considered a theoretical abstraction. A channel is commonly designed on the assumption it will convey uniform flow at normal depth, but it is difficult, if not impossible, to evaluate. The actual flow depth can differ from the theoretical uniform flow depth.

Normal depth computations are made so frequently that it is convenient to use nomographs for various types of open channel cross sections to eliminate the need for trial and error solutions. A nomograph for estimating uniform flow for trapezoidal channels is shown in Figure 6-2.

6.4.3 Critical Flow

Flowing water contains potential and kinetic energy. The relative values of the potential and kinetic energy are important in the analysis of open channel flow. The potential energy is represented by the depth of water plus the elevation of the channel bottom above a datum. The kinetic energy is represented by the velocity head, $V^2/2g$. The specific energy or specific head is equal to the depth of water plus the velocity head.

$$H = d + \frac{V^2}{2g} \tag{6-2}$$

where:

H	=	specific energy head, in feet
d	=	depth of flow, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

When depth of flow is plotted against specific energy for a given channel discharge at a section, the resulting curve shows that, at a given specific energy, there are two possible flow depths (see Figure 6-3). At minimum energy, only one depth of flow exists. This is known as the critical depth. At critical depth, the following relationship applies for rectangular sections:

$$d_c = \frac{V^2}{g} \tag{6-3}$$

where:

d_c	=	critical depth, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

The effect of gravity upon the state of flow is represented by a ratio of the inertial forces to gravity forces. This ratio is known as the Froude Number, Fr, and is used to categorize the flow. The Froude Number is defined by Equation 6-4 for a rectangular section.

$$Fr = \frac{V}{(gd)^{0.5}} \tag{6-4}$$

where:

Fr	=	Froude Number
V	=	average channel flow velocity, in feet per second

g	=	acceleration of gravity, 32.2 feet per second squared
d	=	depth of flow, in feet

The critical state of flow through a rectangular channel is characterized by several important conditions:

A. The specific energy is a minimum for a given discharge.

B. The discharge is a maximum for a given specific energy.

C. The specific force is a minimum for a given discharge.

D. The velocity head is equal to half the hydraulic depth in a channel of small slope.

E. The Froude Number is equal to 1.0.

If the critical state of flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope, S_c. A flow at or near the critical state is unstable, because minor changes in specific energy, such as from channel debris, will cause a major change in depth.

In the analysis of nonrectangular channels, the Froude Number equation is rewritten. The depth of flow is defined as the cross sectional area divided by the top width.

$$Fr = \left[\frac{Q^2 B}{g A^3}\right]^{0.5} \tag{6-5}$$

where:

Fr	=	Froude Number
Q	=	discharge in channel, in cubic feet per second
B	=	top width of channel, in feet
g	=	acceleration of gravity, 32.2 per second squared
A	=	cross-sectional area, in square feet

It can be shown that Fr = 1 for critical flow. If the Froude Number is greater than 1, the flow is supercritical, but when the Froude Number is less than 1, the flow is subcritical.

6.4.4 Gradually Varied Flow

Gradually varied flow is used to describe a type of steady nonuniform flow. The change in the depth and velocity occur gradually over a considerable length of channel and the nonuniformity of the flow is not pronounced. The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile must be computed using backwater techniques.

6.4.5 Rapidly Varied Flow

Rapidly varied flow is characterized by very pronounced curvature of the streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Whereas there are several mathematical solutions to some cases of rapidly varied flow, the practical hydraulician has generally relied on empirical solutions of specific problems. The two cases of rapidly varied flow (weir flow and hydraulic jump) occurring commonly in storm drainage will be discussed in this section.

Weir Flow

The common use of weirs in storm drainage analysis is for spillway outlets in detention ponds. The general form of the equation for horizontal crested weirs is:

$$Q = CLH^{3/2} \tag{6-6}$$

where:

Q	=	channel discharge, in cubic feet per second
C	=	weir coefficient
L	=	horizontal length, in feet
H	=	total energy head, in feet

Another common weir is the V-notch, whose equation is as follows:

$$Q = C \tan(\theta/2) H^{5/2} \tag{6-7}$$

where:

Q	=	channel discharge, in cubic feet per second
C	=	weir coefficient, usually 2.50
Θ	=	angle of the notch at the apex, in degrees
H	=	total energy head, in feet

The weir coefficient is a function of various hydraulic properties and dimensional characteristics of a weir. Experiments have been conducted on various types of weir configurations and formulas have been developed to determine the "C" value. Available empirical formulas are numerous and the designer is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King[2] and use engineering judgement. When designing or evaluating weir flow, the effects of submergence must be considered. A simple check on submergence can be made by comparing the tailwater to the headwater elevations.

Hydraulic Jump

In urban hydraulics, a hydraulic jump may occur at grade control structures (i.e., check drops), inside of storm drains or concrete box culverts, or at the outlet of an emergency spillway for detention ponds. The evaluation of hydraulic jumps is important since there is a loss of energy and erosive forces associated with a jump. For hard-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, the erosive forces must be controlled to prevent serious damages. The control is usually obtained by check drops or grade control structures which confine the erosive forces to a protected area.

The analysis of the jump inside of storm drains is approximate due to the lack of data for circular, elliptical or arch sections. The jump can be approximately located by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are: 1) if the pipe can withstand the forces which may separate the joints or damage the pipe wall, and 2) if the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line taking into account the energy lost by the jump. In general, for a Froude Number less than 2.0, the loss of energy is less than 10 percent.

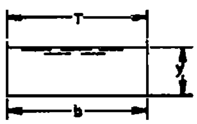
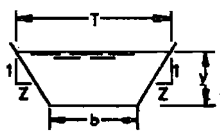
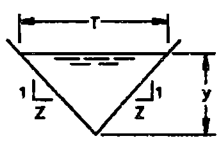
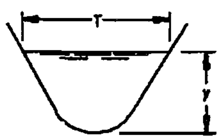
For long box culverts with a concrete bottom, the concerns of the jump are the same as for storm drains. However, the jump can be adequately defined for box culverts/drains and for spillways using the jump characteristics of rectangular sections. A detailed evaluation of the hydraulic jump is beyond the scope of this Manual and the user is referred to other texts for discussion of this subject. The calculations are to be included with the required submittals.

6.5 DESIGN CONSIDERATIONS

Typical channel cross sections are triangular, trapezoidal and parabolic in shape. A triangular channel is a special type of trapezoidal section with a bottom width of zero. Due to the difficulty of maintenance, their application is generally not feasible. Trapezoidal channels of varying bottom widths and side slopes are the most commonly constructed channels. Parabolic channels are generally used only when a vegetated lining is required, although different sections may be selected. Formulas used in channel size design for typical cross section geometrics are presented in Table 6-1.

Man-made open channels are commonly designed to have trapezoidal sections of adequate cross sections to incorporate ease of maintenance, uncertainties in runoff estimates, changes in channel roughness coefficients, channel obstructions and sediment accumulations. Figure 6-4 shows several typical cross sections used for design of grass-lined open channels in urban areas, including channels with alternative trickle channel designs. These channel configurations may be necessary where there are limited right-of-way constraints and where hard lined channels are required.

TABLE 6-1 Geometric Elements of Channel Sections

Section	Area A	Wetted Perimeter WP	Hydraulic Radius R	Top Width T
 Rectangular	by	$b + 2y$	$\dfrac{by}{b + 2y}$	b
 Trapezoidal	$(b + Zy)y$	$b + 2y\sqrt{1 + Z^2}$	$\dfrac{(b + Zy)y}{b + 2y\sqrt{1 + Z^2}}$	$b + 2Zy$
 Triangular	zy^2	$2y{+}\sqrt{1 + Z^2}$	$\dfrac{Zy}{2\sqrt{1 + Z^2}}$	$2Zy$
 Parabolic	$\frac{2}{3}Ty$	$T + \dfrac{8y^2}{3T}$	$\dfrac{2T^2y}{3T^2 + 8y^2}$	$\dfrac{3A}{2y}$

Source: Chow, Ven Te, 1959; Open-Channel Hydraulics

Determination of a representative Manning's "n" value is critical in the analysis of the hydraulic characteristics of an open channel. The "n" value for each channel reach should be based on the individual channel characteristics. Table 6-2 presents a method of determining the composite roughness coefficient for an unlined channel reach based on actual channel conditions. Typical minimum, normal and maximum roughness coefficients for various types of open channels are presented in Table 6-3.

Typical roughness coefficients for a straight channel without shrubbery or trees have been developed by the Soil Conservation Service. Table 6-4 presents a variable "n" value dependent on the depth of flow in the channel. It is required that the Manning's "n" in Tables 6-2 and 6-3 be used to describe a straight channel roughness for nonlinear channels. Experience and judgement should also be used in selecting the proper "n" value for a channel. When working with a detailed hydraulic model such as HEC-2, "n" values should be calibrated, whenever possible, to known water surface conditions. The designer should expect to use higher values than those listed. The higher values are often required to account for losses due to channel blockage, meander and many other factors not included in the tables.

TABLE 6-2 Composite Roughness Coefficients for Unlined Open Channels

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m_5 \tag{6-8}$$

Coefficients	Channel Conditions	Value
Material Type	Earth	0.020
n_o	Rock cut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity	Smooth	0.000
n_1	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel	Gradual	0.000
Cross Section	Alternating Occasionally	0.005
n_2	Alternating Frequently	0.010 - 0.015
Relative Effect	Negligible	0.000
of Obstructions	Minor	0.010 - 0.015
n_3	Appreciable	0.020 - 0.030
	Severe	0.040 - 0.060
Vegetation	Low	0.005 - 0.010
n_4	Medium	0.010 - 0.025
	High	0.025 - 0.050
	Very High	0.050 - 0.100
Degree of Meandering	Minor	1.000
m_5	Appreciable	1.150
	Severe	1.3

Source: Chow, Ven Te, 1959; Open-Channel Hydraulics

TABLE 6-3 Typical Roughness Coefficients for Open Channels

Type of Channel and Description		Minimum	Normal	Maximum
EXCAVATED OR DREDGED				
a.	Earth, straight and uniform:			
	1. Clean, recently constructed	0.016	0.018	0.020
	2. Clean, after weathering	0.018	0.022	0.025
	3. Gravel, uniform section, clean	0.022	0.025	0.030
	4. With short grass, few weeds	0.022	0.027	0.033
b.	Earth, winding and sluggish:			
	1. No vegetation	0.023	0.025	0.030
	2. Grass, some weeds	0.025	0.030	0.033
	3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
	4. Earth bottom and rubble sides	0.028	0.030	0.035
	5. Stony bottom and weedy banks	0.025	0.035	0.040
	6. Cobble bottom and clean sides	0.030	0.040	0.050
c.	Dragline-excavated or dredged:			
	1. No vegetation	0.025	0.028	0.033
	2. Light brush on banks	0.035	0.050	0.060
d.	Rock cuts:			
	1. Smooth and uniform	0.025	0.035	0.040
	2. Jagged and irregular	0.035	0.040	0.050
e.	Channels not maintained, weeds and brush uncut:			
	1. Dense weeds, high as flow depth	0.050	0.080	0.120
	2. Clean bottom, brush on sides	0.040	0.050	0.080
	3. Same, highest stage of flow	0.045	0.070	0.110
	4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS				
Minor streams (top width at flood stage < 100 feet)				
a.	Streams on plain			
	1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
	2. Same as above, but more 0.030 stones and weeds	0.035	0.040	
	3. Clean, winding, some pools and shoals	0.033	0.040	0.045

TABLE 6-3 Typical Roughness Coefficients for Open Channels (Cont'd)

Type of Channel and Description			Minimum	Normal	Maximum
NATURAL STREAMS (Cont'd)					
	4.	Same as above, but some 0.035 weeds and stones	0.045	0.050	
	5.	Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
	6.	Same as 4, but more stones	0.045	0.050	0.060
	7.	Sluggish reaches, weedy, 0.050 deep pools	0.070	0.080	
	8.	Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
LINED OR BUILT-UP CHANNELS					
a.	Corrugated Metal		0.021	0.025	0.030
b.	Concrete:				
	1.	Trowel finish	0.011	0.013	0.015
	2.	Float finish	0.013	0.015	0.016
	3.	Finished, with gravel on bottom	0.015	0.017	0.020
	4.	Unfinished	0.014	0.017	0.020
	5.	Gunite, good section	0.016	0.019	0.023
	6.	Gunite, wavy section	0.018	0.022	0.025
	7.	On good excavated rock 0.017	0.020		
	8.	On irregular excavated rock	0.022	0.027	
c.	Concrete bottom, float finished with sides of:				
	1.	Dressed stone in mortar 0.015	0.017	0.020	
	2.	Random stone in mortar 0.017	0.020	0.024	
	3.	Cement rubble masonry, 0.016 plastered	0.020	0.024	
	4.	Cement rubble masonry	0.020	0.025	0.030
	5.	Dry rubble or riprap	0.020	0.030	0.035

TABLE 6-3 Typical Roughness Coefficients for Open Channels (Cont'd)

Type of Channel and Description				Minimum	Normal	Maximum
LINED OR BUILT-UP CHANNELS (Cont'd)						
d.	Gravel bottom with sides of:					
	1.	Formed concrete	0.017	0.020	0.025	
	2.	Random stone in mortar	0.020	0.023	0.026	
	3.	Dry rubble or riprap		0.023	0.033	0.036
e.	Asphalt:					
	1.	Smooth	0.013	0.013		
	2.	Rough		0.016	0.016	
f.	Rock-lined:					
	1.	Riprap		0.023	0.033	0.036
	2.	Grouted riprap		0.020	0.023	0.026
	3.	Gabions		0.025		0.033

Source: Chow, Ven Te, 1959; Open-Channel Hydraulics.

TABLE 6-4 Manning's Roughness Coefficients for Straight Channels Without Shrubbery or Trees

Grass Condition	Depth of Flow of 0.7 to 1.5 feet	Depth of Flow greater than 3.0 feet
Bermudagrass, Buffalograss, Kentucky Bluegrass:		
a. Mowed to 2 inches	0.035	0.030
b. Length 4-6 inches	0.040	0.030
Good stand, any grass:		
a. Length of 12 inches	0.070	0.035
b. Length of 24 inches	0.100	0.035
Fair stand, any grass:		
a. Length of 12 inches	0.060	0.035
b. Length of 24 inches	0.070	0.035

TABLE 6-5 Maximum Permissible Design Open Channel Flow Velocities in Earth*

Soil Types	Permissible Mean Channel Velocity (fps)
Fine Sand (noncolloidal)	2.0
Coarse Sand (noncolloidal)	4.0
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Silty Clay	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (colloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Hard Shales and Hard Pans	6.0
Soft Shales	3.5
Soft Sandstone	8.0
Sound rock (igneous or hard metamorphic)	20.0

* These velocities shall be used in conjunction with scour calculations and as approved by the City Engineer.
Source: Chow, Ven Te, 1959: Open Channel Hydraulics.

Where applicable, unlined open channels of a given soil type should have sufficient gradient to provide self-cleaning flow velocities but not be so great as to create excessive erosion. Maximum permissible design flow velocities for earth channels are presented in Table 6-5. Table 6-6 presents maximum permissible velocities for earth channels with varied grass linings and sloping configurations. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from high channel flow velocities. Overall, the design of open channels, including stable, alluvial channel systems, is tied closely to the criteria for erosion and sediment control.

6.6 DESIGN STANDARDS

The design standards for open channels cannot be presented in a step-by-step fashion because of the wide range of options available to the designer. Certain planning and conceptual criteria are particularly useful in the preliminary design of a channel. These criteria, which have the greatest effect on the performance and cost of the channel, are discussed below. Design submittals shall be in a clear and concise format convenient for review and shall include, but not be limited to, 1) storm runoff computations and mapping, 2) hydraulic design computations, assumptions, references, sketches and drawings, 3) floodplain mapping, 4) and all other pertinent data.

TABLE 6-6 Maximum Permissible Velocities for Earth Channels with Varied Grass Linings and Slopes

Channel Slope	Lining	Permissible Mean Channel Velocity*(fps)
0 - 2%	Small grains (temporary)	2.5
1 - 5%	Western Wheatgrass/Buffalograss	5.0
	Western Wheatgrass/Tall Fescue	5.0
	Western Wheatgrass	5.0
	Bermudagrass	6.0
	Buffalograss/Bluegrama	4.0
5 - 10%	Bermudagrass	6.0
	Western Wheatgrass	5.0

* For highly erodible soils, decrease permissible velocities by 25%.
* Grass lined channels are dependent upon assurances of continuous growth and maintenance of grass.

6.6.1 Natural Channels

The design criteria and evaluation techniques for natural channels are:

A. The channel and overbank areas shall have adequate capacity for major storm runoff.

B. Natural channel segments which have a Froude Number greater than 0.95 for any flow shall be protected from erosion.

C. The water surface profiles shall be defined so that the major storm floodplain can be mapped.

D. Filling of the flood fringe reduces valuable floodplain storage capacity and tends to increase downstream runoff peaks. Filling of the flood fringe is subject to the restriction of floodplain regulations.

E. Roughness factors "n", which are representative of unmaintained or "in need of maintenance" conditions, shall be used for the analysis of water surface profiles.

F. Roughness factors "n", which are representative of maintained channel conditions, shall be used to determine velocity limitations.

G. Erosion control structures, such as riprap, check drops or check dams, may be required to control flow velocities, including the initial storm runoff.

H. Plan and profile drawings of the major storm floodplain, including flooded limits, shall be prepared. Appropriate allowances for future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis.

With most natural waterways, grade control structures should be constructed at regular intervals to decrease the slope and control erosion. However, these channels should be left in as near natural condition as possible. For that reason, extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream.

The usual rules of freeboard depth, curvature, and other guidelines which are applicable to artificial channels do not necessarily apply to natural channels. There are significant advantages which may occur if the designer incorporates into his/her planning the overtopping of the channel and localized flooding of adjacent areas which are laid out and developed for the purpose of being inundated during the major storm runoff. The freeboard criteria can be used to an advantage in gaging the adequacy of a natural channel for future changes in runoff.

6.6.2 Grass-Lined Channels

Key parameters in grass-lined channel design include velocity, slopes, roughness coefficients, depth, freeboard, curvature, cross section shape, and lining materials. Other factors such as water surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

A. Flow Velocity and Capacity

The maximum normal depth velocity should not exceed 7.0 feet per second for grass-lined channels, except in sandy soil where the maximum velocity should not exceed 5.0 feet per second. The Froude Number (turbulence factor) shall be less than 0.8 for grass-lined channels. Grass-lined channels having a Froude Number greater than 0.8 shall not be permitted. The minimum velocity should be greater than 2.0 feet per second for self-cleansing.

B. Longitudinal Channel Slopes

Grass-lined channels normally will have slopes of 0.2 percent to 0.5 percent. Where the natural topography is steeper than desirable, drop structures should be utilized to maintain design velocities.

C. Freeboard for Major Drainageways

Except where localized overflow in certain areas is desirable for additional ponding benefits or other reasons, the freeboard should be:

$$H_{FB} = 1.0 + \frac{V^2}{2g} \tag{6-9}$$

where:

H_{FB}	=	freeboard height, in feet
V	=	average channel flow velocity, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared

The minimum freeboard should be 1.0 feet above the computed water surface elevation. Freeboard should not be obtained by the construction of levees.

An approximation of the superelevation of the water surface at a curve can be obtained from the following equation:

$$h = \frac{V^2 T}{g r_c} \tag{6-10}$$

where:

h	=	superelevation, in feet
V	=	average channel flow velocity, in feet per second
T	=	top width of channel, in feet
g	=	acceleration of gravity, 32.2 feet per second squared
r_c	=	centerline radius of curvature, in feet

The freeboard shall be measured above the superelevation water surface.

D. Curvature

The centerline curvature should have a radius twice the top width of the design flow, but not less than 100 feet.

E. Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often, the shape can be chosen to suit open space and recreational needs. However, limitations within which the design must fall for the major storm design flow include:

1. Trickle Channel

The base flow should be carried in a trickle channel. The minimum capacity should be 1.0 percent to 3.0 percent of the 100-year flow, but not less than 1 cfs. Trickle channels shall be constructed of materials to minimize erosion, to facilitate maintenance and to aesthetically blend with the adjacent vegetation and soils.

2. Bottom Width

The minimum bottom width shall be consistent with the maximum depth and velocity criteria. The minimum width should be 4 feet to accommodate the trickle channel.

3. Maintenance Easements

A maintenance easement should be provided for all channels. Channels of less than 20 feet top width shall be provided with a 20 feet easement accessible to maintenance equipment on one side. Channels of 20 feet and greater top widths shall have 20 feet easements on both sides. No permanent structures, fences or barriers to access shall be placed within the maintenance easement.

4. Side Slopes

Side slopes should be 4H:1V or flatter. Steeper slopes may be used in existing developed areas subject to additional erosion protection and approval from the City Engineer.

5. Grass

The grass species chosen must be sturdy, drought resistant, easy to establish and able to spread. A thick root structure is necessary to control weed growth and erosion. The USDA Soil Conservation Service can provide assistance in selecting grass mixtures which have been successful, as well as recommending seeding and maintenance methods.

Newly constructed channels need a protective cover consisting of mulch and grass seeding immediately after completion. If possible, seed the disturbed areas with permanent grass seed mix. To provide quick ground cover the seed mix shall include a perennial ryegrass. The perennial ryegrass germinates quickly and will not compete with the sod-forming grasses later on. When immediate seeding of permanent grass is not practical, an annual crop may be planted with the perennial grass seeded later in the stubble or residue. Rye, oats, or ryegrass gives a fair temporary protection for waterways, though the crop should be clipped before it matures to seed.

6.6.3 Concrete-Lined Channels

The criteria for the design and construction of concrete lined channels is presented below:

A. Freeboard

Adequate channel freeboard above the designed water surface shall be provided and should be not less than that determined by Equation 6-11:

$$H_{FB} = 2.0 + 0.025V\,(d)^{1/3} \tag{6-11}$$

where:

H_{FB} = freeboard height, in feet

V = average channel flow velocity, in feet per second

d = depth of flow, in feet

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

B. Superelevation

Superelevation of the water surface shall be determined at all horizontal curves and design of the channel section adjusted accordingly.

C. Velocities

Flow velocities should not exceed 8 feet per second or result in a Froude Number greater than 0.9 for non-reinforced linings. Flow velocities should not exceed 18 feet per second for reinforced linings.

6.6.4 Rock-Lined Channels

Channel linings constructed from ordinary riprap, grouted riprap, or wire encased rock (gabions) to control channel erosion have been found to be cost effective. Situations for which riprap linings might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values (5 feet per second for sandy soil conditions and 7 feet per second in erosion resistant soils); 2) where channel side slopes must be steeper than 3H:1V; 3) for low flow channels, and; 4) where rapid changes in channel geometry occur, such as at channel bends and transitions.

A. Ordinary and Grouted Riprap Channel Linings

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rocks, the shape of the stones, the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and the slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action and waves.

Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than dumped riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning the projecting rocks with a wet broom. Grouted riprap should meet all the requirements for ordinary riprap except that the smallest rock fraction (smaller than the 10% size) should be eliminated from the gradation. A reduction of riprap size by 3 inches - 6 inches is permitted for grouted rock.

1. Roughness Coefficient

The Manning's roughness coefficient for ordinary riprap and grouted riprap should be selected using Table 6-3. The "n" value is dependent on the predominant rock size.

2. Rock Size

The design should refer to the Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets, and Bridges[21] for gradation requirements for riprap.

3. Toe Protection

Where only the channel sides are to be lined, additional riprap is needed to provide for long term stability of the lining. In this case, the riprap lining should extend at least three feet below the existing channel bed and the thickness of the blanket below the existing channel bed increased to at least three (3) times d_{50} to accommodate possible channel scour during floods. D_{50} is the rock size for which 50% of the sample is finer and can be determined by a sieve analysis from a material sample.

4. Channel Bends

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in channels which otherwise would not need protection.

The minimum allowable radius for a riprap lined bend is 1.2 times the top width of the design flow water surface and in no case less than 50 feet. The riprap protection should be placed along the outside of the bank and should extend downstream from the bend a distance equal to the length of the bend.

Where the mean channel velocity exceeds the allowable non-eroding velocity so that riprap protection is required for straight channel sections, increase the rock size by three (3) to six (6) inches around bends having a radius less than the greater of the following: two times the top width, or 100 feet. The minimum allowable radius for a riprap lined bend in this case is also 1.2 times the top width of the design flow water surface.

5. Transitions

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such as transitions and bridges. Riprap protection for subcritical transitions (Froude Number 0.8 or less) is selected by increasing the channel velocity by twenty percent (20%). Since the channel velocity varies through a transition, the maximum velocity in the transition should be used in selecting riprap size after it has been increased by 20%. Protection should extend upstream from the transition entrance a minimum of five (5) feet or 2 times the width of the structure and extend downstream from the transition exit a minimum of ten (10) feet or 4 times the width of the structure.

B. Wire Enclosed Rock (Gabions)

Wire enclosed rock refers to rocks that are bound together in a wire basket so that they act as a single unit, usually referred to as a gabion. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for ordinary riprap. Another advantage is the versatility that results from the regular geometric shapes of wire enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. Plastic coated wire should be specified. The designer should be aware that if the flow contains coarse material, sand or gravel, it may abrade and break the wire basket, enabling the smaller rocks within the gabions to be transported downstream.

C. Bedding Requirements for Rock-Lined Channels

Long term stability of riprap and gabion erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures. A properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent piping of channel particles through the voids in the riprap. Two types of bedding are in common use: 1) a granular bedding filter and 2) filter fabric.

1. Granular Bedding

A bedding of mineral aggregate is adequate for most ordinary riprap, grouted riprap or wire encased riprap applications. The Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets and Bridges[21] should be used for granular bedding.

2. Filter Fabric

Filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides an adequate bedding of channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

Filter fabric is not a complete substitute for granular bedding. Filter fabric usually provides filtering action only perpendicular to the fabric and usually has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, filter fabric is restricted to slopes no steeper than 2.5H:1V. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is usually not recommended and care must be exercised during construction.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric might be reduced by folding the edge of the fabric vertically downward about two feet (similar to a cutoff wall) at approximately 12 foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric should be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay may clog the openings in the filter fabric, preventing free drainage and increasing failure potential due to uplift. For this reason, a granular filter is recommended for fine silt and clay channel beds.

D. Riprap Channel Linings

Design procedures for the design of ordinary and grouted riprap channel linings are presented in the Federal Highway Administration's Hydraulic Engineering Circulars Numbers 11 (HEC-11)[10] and 15 (HEC-15)[11]. For discharges less than 50 cfs, HEC-15 Design of Roadside Channels with Flexible Linings[11] should be used. HEC-11, Design of Riprap Revetments[10] provides the procedures for the design of riprap revetments to be used as channel bank protection and channel linings on larger streams and rivers (i.e. generally greater than 50 cfs).

6.6.5 Other Channel Linings

The criteria for the design of channels with linings other than grass, rock, or concrete will be dependent on the manufacturers recommendations for the specific product. The designer will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the City Engineer to verify assumptions or design criteria. The following minimum criteria will also apply:

A. Flow Velocity

The maximum normal depth velocity will be dependent on the construction material utilized. The Froude number shall be less than 0.8.

B. Freeboard

Same as for grass-lined channels, adjust for horizontal curvature.

C. Curvature

The center line curvature shall have a minimum radius twice the top width of the design flow but not less than 100 feet.

D. Roughness Coefficient

A Manning's "n" value range shall be established by the manufacturers data with the high value used to determine depth/capacity requirements and the low value used to determine Froude Number and velocity restrictions.

E. Cross Sections

Same as for grass-lined channels.

6.7 WATER SURFACE PROFILE ANALYSIS

For final design, water-surface profiles must be computed for all major channels. Computation of the water-surface profile should utilize standard backwater analysis, and should consider all losses due to changes in channel velocity, drops, curves, bridge openings, and other obstructions. Computations begin at a known point, and extend in an upstream direction for subcritical flow.

Backwater computation can be made using the standard step method presented in Open-Channel Hydraulics, by Chow[6]. Many computer programs are available for computation of backwater curves. The most general and widely used program is HEC-2, Water-Surface Profiles, developed by the US Army Corps of Engineers. This program will compute water-surface profiles for natural and manmade channels.

WSPRO, a program developed for the Federal Highway Administration can also be used to analyze one-dimensional gradually varied steady flow in open channels. WSPRO can analyze flow through bridges and culverts, embankment overflow, and multiple-opening stream crossings.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method. For an irregular non-uniform channel, the Standard Step Method is used, which is a more tedious iterative process. The use of HEC-2 or WSPRO is recommended for non-uniform channel analysis.

The effects of superelevation and energy losses due to resistance in bends in open channels must be considered in backwater computations. In addition to superelevation on bends, flow separation in the bend creates a backwater effect that must also be considered. More detail on determining these effects may be found in Chow[6].

6.8 SUPERCRITICAL FLOW

Supercritical flow in an open channel creates certain hazards which the designer must take into consideration. From a practical standpoint, it is generally not possible to have any curvature in such a channel. Careful attention must be taken to insure against excessive oscillatory waves resulting from minor obstructions upstream which may extend down the entire length of the channel. Imperfections at joints of lined channels may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can rapidly occur. In addition, high-velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining. It is evident that, when designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

6.9 FLOOD PROOFING

The National Flood Insurance Program was created to reduce flood losses by promoting a wiser use of the flood plain. In return for making subsidized flood insurance available for existing structures, the participating community agrees to regulate new development in the flood plains. These regulations are adopted by a community in the form of a flood plain ordinance. The ordinance requires all new structures in the flood plain to be protected to a base flood elevation determined either by the Federal Emergency Management Agency (FEMA) or other sources. FEMA's elevations must be used if they are more restrictive. Improvements to existing structures in a flood plain can be elevated or flood proofed to reduce flood damages, at the option and cost of the property owner.

FEMA conducts a Flood Insurance Study (FIS) to determine the base flood elevations and, if appropriate, the floodway boundaries. The base flood is generally defined as the 100-year event, although reference should be made to the City's Flood Hazard Ordinance for Amarillo's interpretation of the base flood. The 100-year floodplain and corresponding elevations of the 100-year flood can be determined by reviewing the Flood Hazard Boundary Maps and Flood Insurance Rate Maps published by the Federal Insurance Administration and are available for review in the office of the City Engineer.

Flood proofing is defined by Federal Insurance Administration as any combination of structural and nonstructural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property and sanitary facilities, structures and their contents. The Federal Insurance Administration has published several references to provide detailed criteria and design procedures for flood proofing structures. It is beyond the scope of this Manual to describe flood proofing alternatives and their designs.

6.10 ENERGY DISSIPATORS

Hydraulic structures include energy dissipators, channel drops, transitions, baffle chutes, riprap and many other specific drainage works. Their shape, size, and other features vary widely depending upon the function to be served.

It is not the intent of this Section to describe all types of hydraulic structures, rather typical hydraulic structures are presented. Additional information can be obtained from technical references, some of which are listed in the bibliography.

Energy dissipators are often necessary at the end of outfall sewers, culverts or channels. Stilling basins, a type of energy dissipator, are useful at locations where the flow changes from supercritical to subcritical. Stilling basins can reduce or limit potential erosion downstream from a high-velocity channel or conduit.

6.10.1 Impact-Type Stilling Basins

When the energy of flow must be dissipated, the impact-stilling basin is an effective structure for reducing the exit velocity to a tranquil state. A hydraulic jump occurs and energy is dissipated by the stilling basin. The Bureau of Reclamation has developed various types of impact-stilling basins. Figure 6-5 shows Type II, III and IV basins. A Type II basin is typically used for Froude numbers above 4.5 and is used for high dam and earth dam spillways and large canal structures. Basin III is developed for smaller structures having low or moderate (less than 60 fps) velocities and discharges per foot of width less than 200 cfs. A Type IV Basin is used for Froude Numbers between 2.5 and 4.5 and is used for stilling-basin design, outlet works and division dams.

Impact-type stilling basins are subject to local flow turbulences and large dynamic forces which must be considered in the structural design. The structure must be made sufficiently stable to resist sliding due to the impact load on the baffle wall. The entire structure must also resist the severe vibrations inherent in this type of device and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

6.10.2 Plunge Pools

A plunge pool is formed by scour when a free-falling overflow drops vertically into a pool. It will be scoured to a depth relative to the height of fall, depth of tailwater and the concentration of flow. Since the depth of scour is influenced by the erodibility of the streambed material, the pool must be heavily protected with large riprap or reinforced concrete. A plunge pool may only be used in a channel with a continuous low flow because of the health and safety hazards which could be created by a stagnant pool.

6.10.3 Drop Structures

The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from its fall. In a channel steep enough to cause severe erosion in earth channels or disruptive flow in lined channels, the water can be conveyed through a drop structure designed to safely dissipate the excess energy. Different kinds of drop structures used are vertical, baffled apron, rectangular inclined, and pipe drops.

Vertical drops are often the most economical for drops of less than three (3) feet. They can consist of a simple weir above a vertical retaining wall and a splash-pool-type energy dissipator that are combined in a single structure. These structures can be constructed from steel sheet pile, riprap, gabion retaining walls and channel mats, soil cement or reinforced concrete.

Baffled apron drops may be used for nearly any decrease in water-surface elevation where the horizontal distance for a grade drop is relatively short. They are particularly adaptable to the situation where the downstream water-surface elevation may vary because of channel degradation or an uncontrolled water surface. A further discussion on baffled aprons may be found in the Bureau of Reclamation publication <u>Hydraulic Design of Stilling Basins and Energy Dissipators</u>[5] and the Federal Highway Administration publication <u>Hydraulic Design of Energy Dissipators for Culverts and Channels</u>[13].

Rectangular inclined (RI) drops (Figure 6-6) and pipe drops (Figure 6-7) are used where the decrease in elevation is in the range of 3 to 15 feet over a relatively short distance. They convey water as well as dissipate the excess energy upon reaching the lower elevation.

Pipe drops are easily designed, built and operated. The inlets can be readily adapted to either an earth or a lined waterway and the outlets can be easily adapted to an earth or a lined canal or to a waterway where there is no downstream water surface control. The inlets can be made to incorporate a control notch, a check or a weir. If there is a control or a check inlet, there should be side overflow walls for emergency flows. A pipe drop can easily be taken under another waterway or a roadway.

Pipe drops are economical, especially for small discharges. Pipe drops require very little maintenance, provided they are constructed of durable pipe having good rubber gasket joints and the bends are properly made. It is important to provide adequate gravel or riprap protection at outlets discharging into unlined waterways. The maximum fall for any one pipe drop is about 15 feet.

A sumped pipe drop should not be used if there is a strong possibility of the pipe becoming clogged with sediment. There is also the possibility of the drop becoming clogged with weeds or debris. To prevent this, the drop may have a weed screen on the inlet or the pipe may be sufficiently sized to discharge this material if it should get into the pipe drop.

Rectangular inclined drops are easily designed, built and operated. The inlets and outlets can be easily adapted to either an earth or a lined waterway. The inlets can be made to incorporate a control notch, a check or a weir. If there is a control or check inlet, side overflow walls for emergency situations should be included. It is important to provide adequate gravel or rock protection of outlets discharging into unlined waterways. The maximum fall in water surface for any one RI drop is about 15 feet.

The rectangular inclined drop should have adequate percolation path and sufficient resistance to sliding. The standard RI drop structure in Figures 6-6 and 6-7 are designed to provide this stability. However, if unusual foundation conditions are encountered, the percolation and sliding resistance should be checked, and additional stability may be obtained by increasing the length.

6.10.4 Chute Structures

Chute structures are commonly used where the drop in elevation is greater than 15 feet. A chute structure usually consists of an inlet, a chute section, an energy dissipator, and an outlet transition. Figure 6-8 shows the relationship of the different parts of the structure. Chutes are similar to drops except that they carry the water over longer distances, over flatter slopes, and through greater changes in grade. The inlet portion of the structure transitions the flow from the channel upstream of the structure to the chute structure. The chute section generally follows the original ground surface and connects to an energy dissipator at the lower end. Stilling pools or baffled outlets are used as energy dissipators on chute structures. An outlet transition is used when it is necessary to transition the flow between the energy dissipator and the downstream channel.

In a pipe chute, Figure 6-9, the open section is replaced by a pipe. Pipe chutes may be designed to provide a crossing.

The decision as to whether to use a chute structure or a series of smaller drops should be based upon a hydraulic and economic study of the alternatives. Drops should not be so closely spaced as to possibly preclude uniform flow between outlet and inlet ends of consecutive structures, particularly where checks or control notches are not used at the inlets. Sufficient tailwater depths may not exist between the structures to produce hydraulic jumps in the pools, and thus "shooting flow" may develop through the series of drops and possibly damage the channel. Also, with drops too closely spaced on a steep slope, problems of excavation and backfill may make construction undesirable or prohibitive. About 200 feet of channel should be the minimum distance between the inlet and outlet ends of consecutive drop structures. The economic study should compare costs of a series of drops and a single chute structure considering advantages and disadvantages pertinent to the specific conditions. Since the maintenance costs for a series of drops is usually considerably more than for a single chute structure performing the same hydraulic function, it is sometimes economically feasible to spend considerably more in initial costs for a chute structure than for a series of drops. More complete discussions on chute structures are presented in Bureau of Reclamation, Design of Small Canal Structures[3].

6.11 FLOW TRANSITIONS

A flow transition structure is a change of channel cross section designed to allow for a minimum amount of flow disturbance. Several types of transitions are shown on Figure 6-10. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations. Section 7 includes a discussion on culvert design with improved inlets.

Outlet transitions (expansions) must be considered in the design of all culverts, energy dissipators and channel protection. The standard wingwall-apron combinations and expansions downstream of dissipator basins are most common.

6.12 RIPRAP

Placement of riprap is used for preventing or limiting channel bed and bank erosion damage caused by excessive channel flow or surges from energy dissipators. Placement of riprap on the channel bottom and banks downstream of an energy dissipator structure is required for alleviating possible undermining of the structure due to scour.

Experience has shown that a primary reason for riprap failure is placement of undersized individual stones in the maximum size range. Failure has also occurred because of improper engineering design for gradation of riprap, seepage control and/or bedding filter requirements.

Design of riprap should take into account the following parameters: 1) stone durability; 2) stone density; 3) stone size; 4) stone shape; 5) stone gradation; 6) velocity of flow against the stone; 7) filter bed requirements; 8) channel side slopes; and 9) Froude Number.

A well-graded riprap layer contains about 40 percent of the rock pieces smaller than the required size as stable, or more stable, than a single stone of the required size. Most of the mixture should consist of stones having length, width, and thickness dimensions as nearly equal as practical and should not be flat slabs. The riprap layer should be a minimum of 1-1/2 times or more, as thick as the dimension of the large stones (curve size), and should be placed over a gravel or reverse filter layer.

6.13 SCOUR

Basically, scour is the net result of an imbalance between the capacity of the flow to transport sediment out of an area and the rate of supply of sediment to that area. At a bridge crossing, for instance, the area of interest is the immediate vicinity of the bridge foundation, the piers and abutments. The imbalance of this capacity and supply can arise from a variety of causes which can be generally categorized as 1) those characteristics of the stream itself, and 2) those due to the modification of the flow by the bridge piers and abutments.

Because of the overall complexity of the hydrodynamic forces existing in a natural stream channel, the detailed flow pattern in an unobstructed stream cannot be predicted over time with great accuracy. Reasonable estimates can be made based on observations along reaches of similar streams, and in some cases, actual records and measurements for the particular reach of the stream under investigation can be performed. The designer is encouraged to use the method and procedures in the Federal Highway Administration's publication HEC-18, Evaluating Scour of Bridges[12] to evaluate scour.

Scour, which occurs because of modification of the flow patterns by a bridge crossing, can be further divided into two distinct types of scour depending upon whether or not sediment is supplied to the scour hole. Equilibrium is attained when a scour hole is enlarged to a size where the capacity to remove material from the scour hole is balanced by the rate at which sediment is supplied to the scour hole. During floods, a scour hole located in the main channel will be supplied with sediment at a rate characteristic of the stream. Ignoring the complexities of material stratification that may exist below the stream bed, the material supplied will be essentially the same as the material removed.

If no sediment is supplied to the scour hole, equilibrium is not attained until the configuration of the bed is such that the scouring capacity of the flow is zero. This condition is most likely to occur in overbank areas where vegetation reduces flow velocities, causing the coarser material to drop out of suspension, resulting in a greater degree of scour in overbank areas than would otherwise occur in the main channel.

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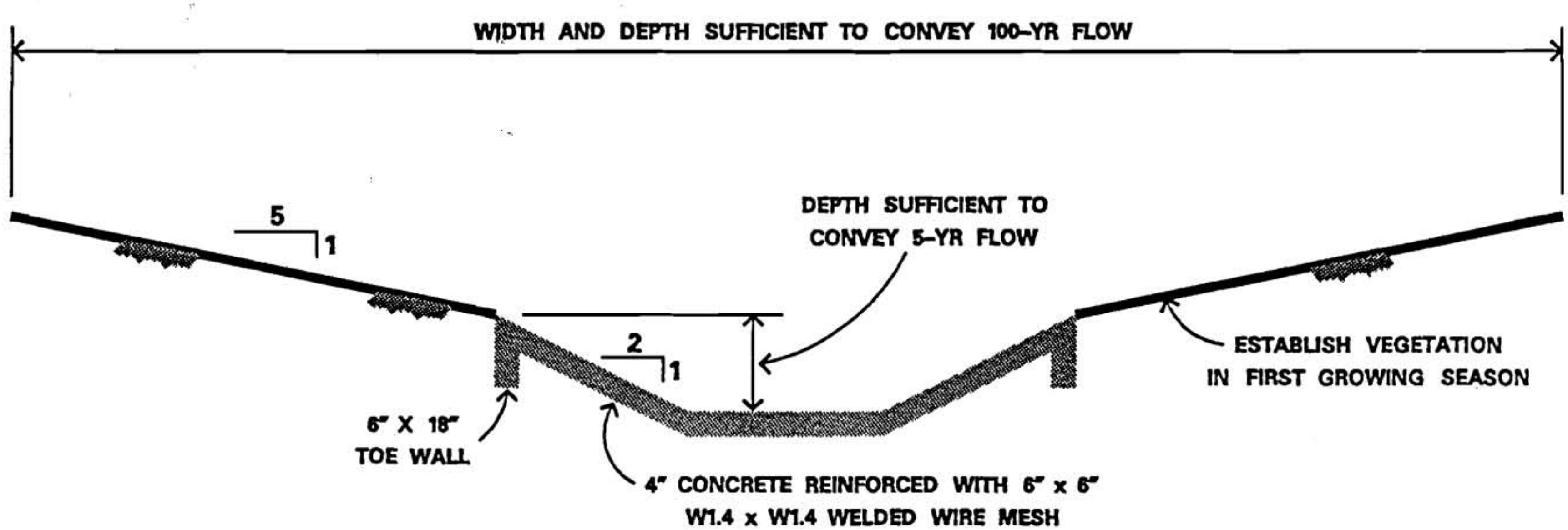
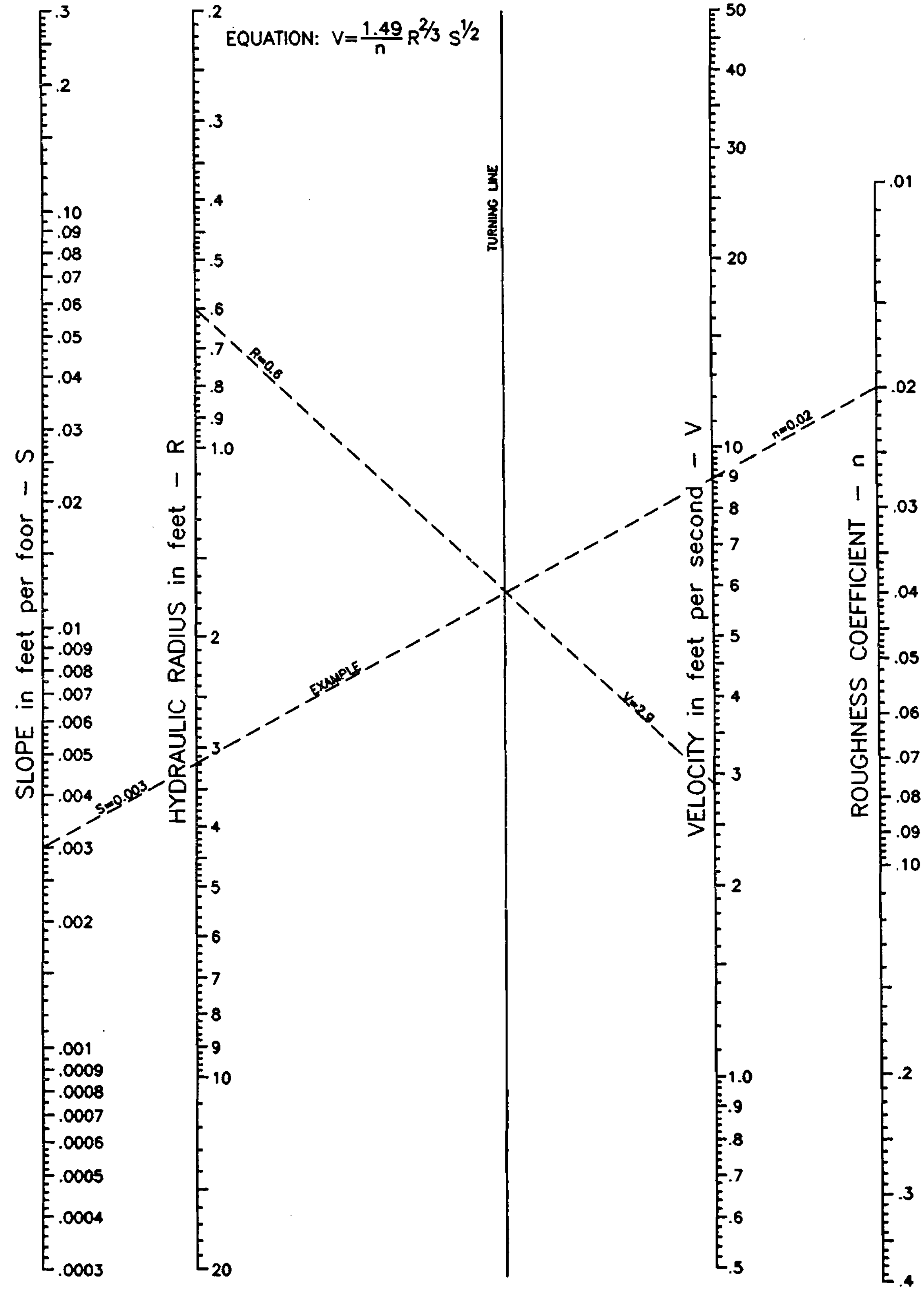


MINIMUM STANDARD FOR OPEN CHANNELS



Source: Federal Highway Administration, HDS No. 3, Design Charts for Open-Channel Flow.

Nomograph for Solution of Manning's Equation

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Figure
6-1

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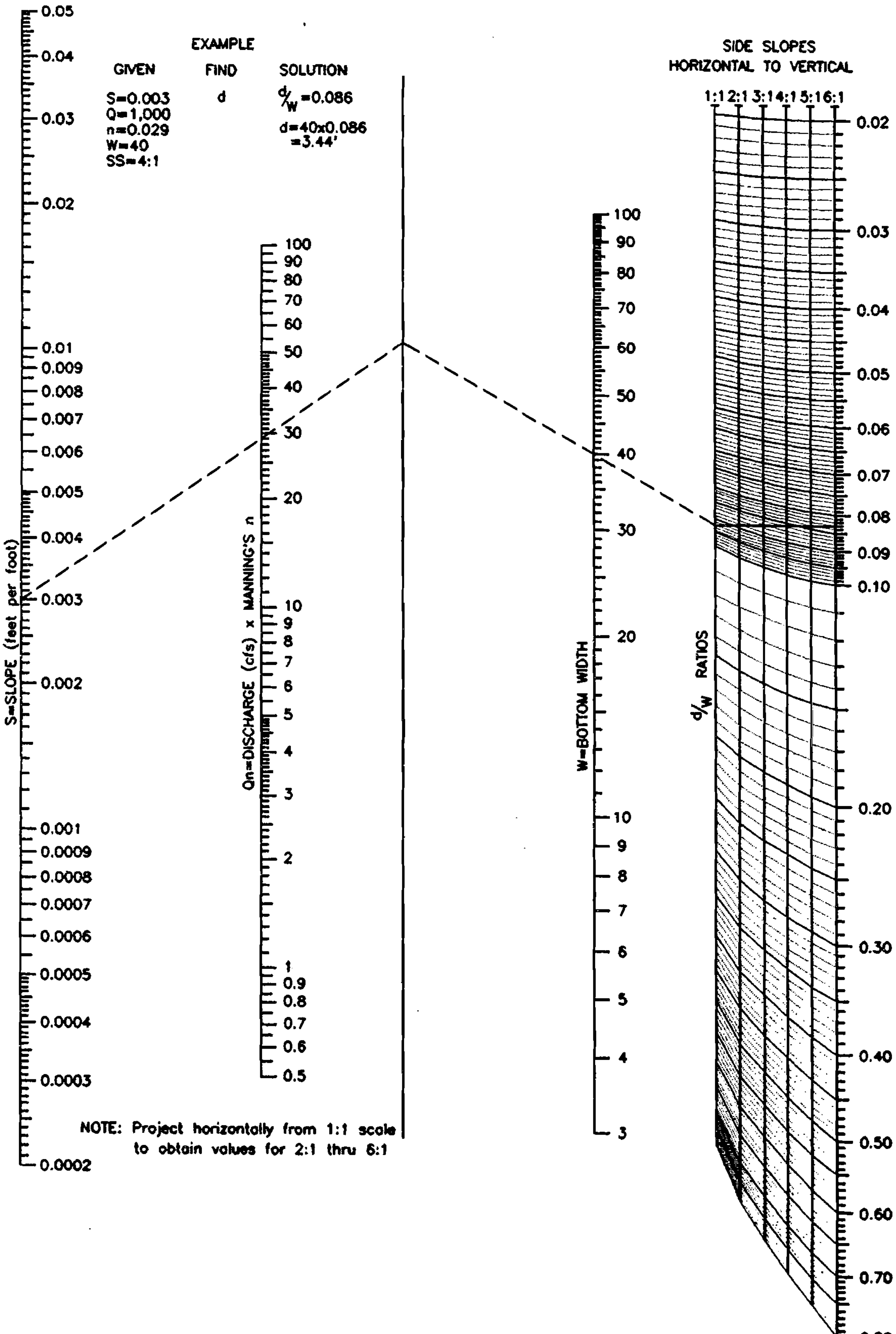

Source: Texas Highway Department.

Uniform Flow for Trapezoidal Channels

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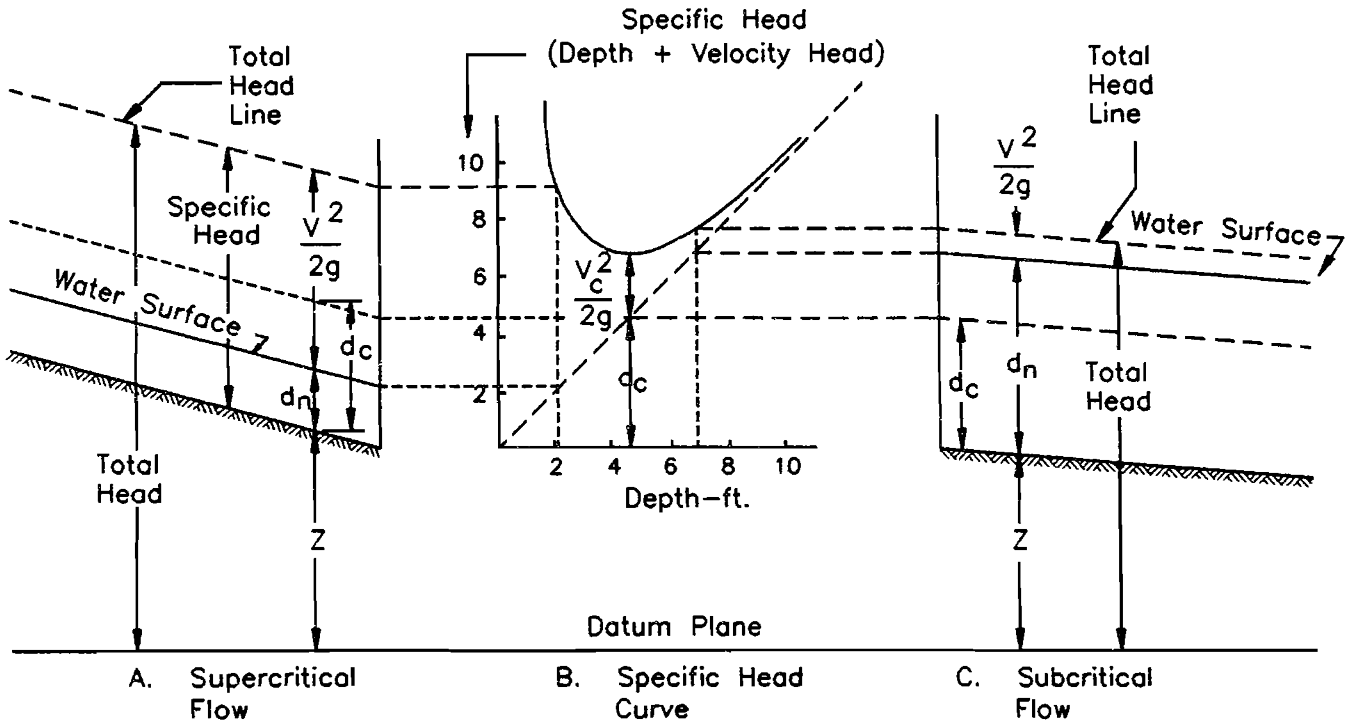
Figure
6-2



Source: Federal Highway Administration, HDS No. 3, Design Charts for Open–Channel Flow.

Definition Sketch of Specific Head

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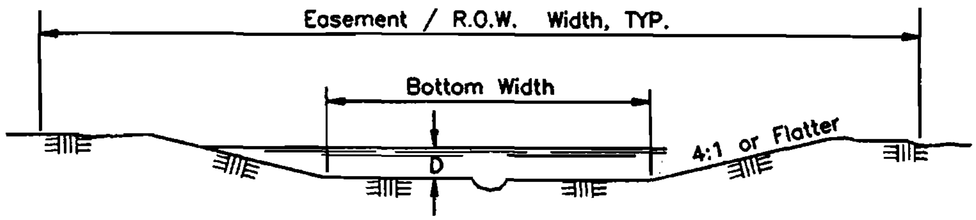
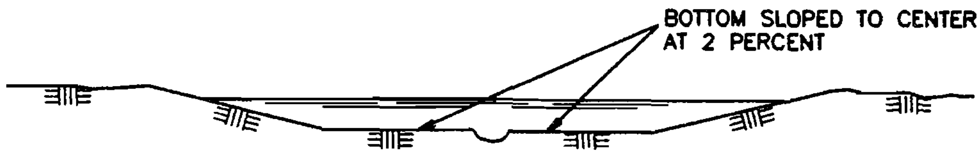
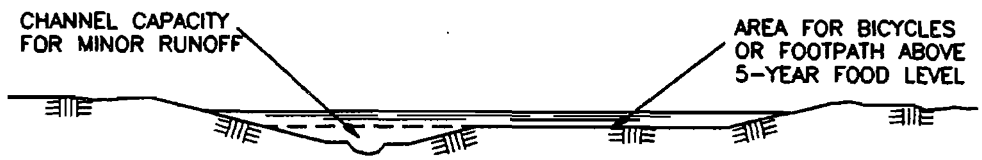
Figure
6–3



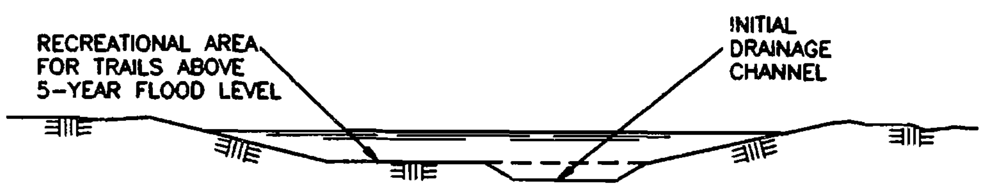
FLAT BOTTOM WITH TRICKLE CHANNEL



OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL



MINOR DRAINAGE CHANNEL WITH OVERFLOW AREA FOR MAJOR DRAINAGE RUNOFF



RIPRAPPED MINOR DRAINAGE CHANNEL WITH OVERFLOW FOR MAJOR DRAINAGE RUNOFF

NOTE: Provisions for erosion protection shall be included for the entire Easement / R.O.W.

Source: Urban Drainage and Flood Control District.

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Typical Grassed-Lined Channels

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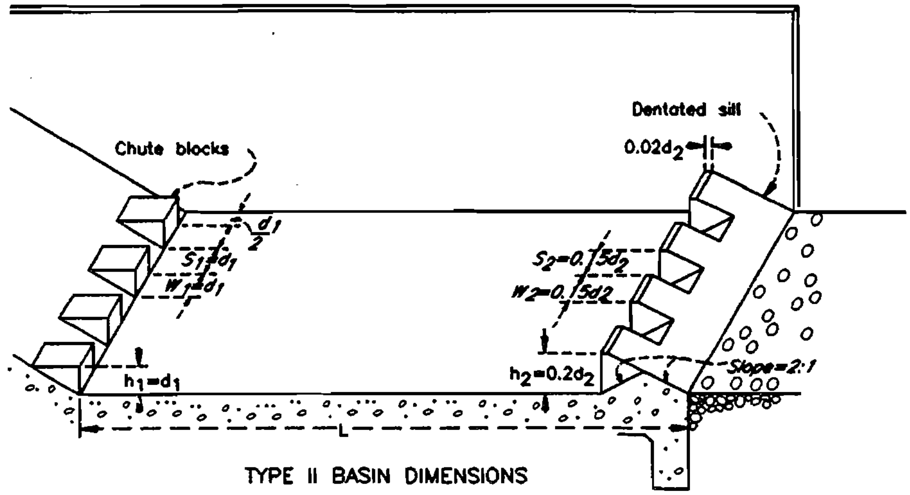
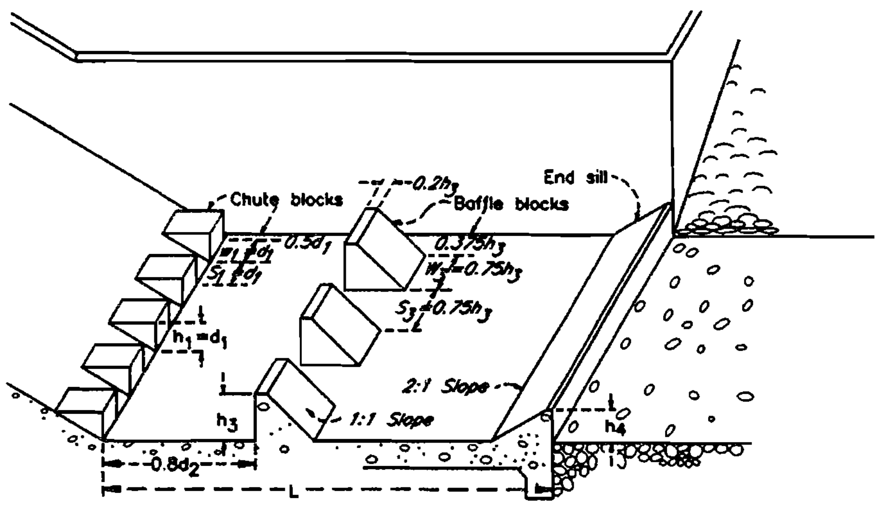
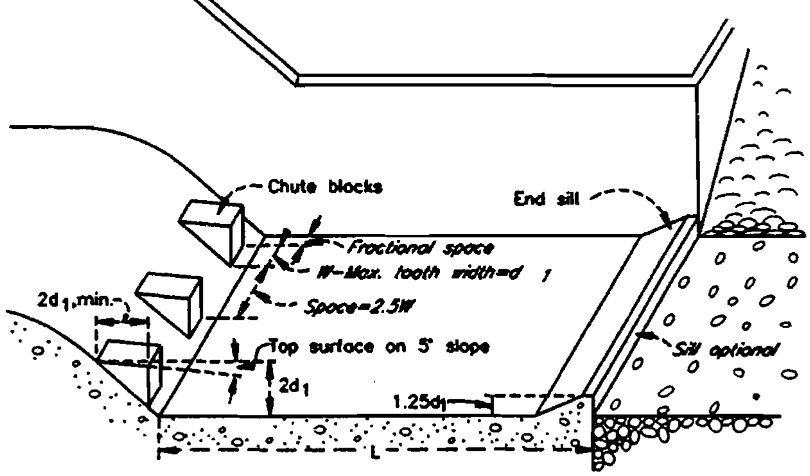
Figure
6-4



TYPE II BASIN DIMENSIONS
Stilling basin for Froude numbers above 4.5.



TYPE III BASIN DIMENSIONS
Stilling basin for Froude numbers above 4.5
where incoming velocity, V ≤ 60ft/s.



TYPE IV BASIN DIMENSIONS
Stilling basin for Froude numbers between 2.5 and 4.5.

Source: US Bureau of Reclamation, Design of Small Dams.

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Impact-Type Energy Dissipators

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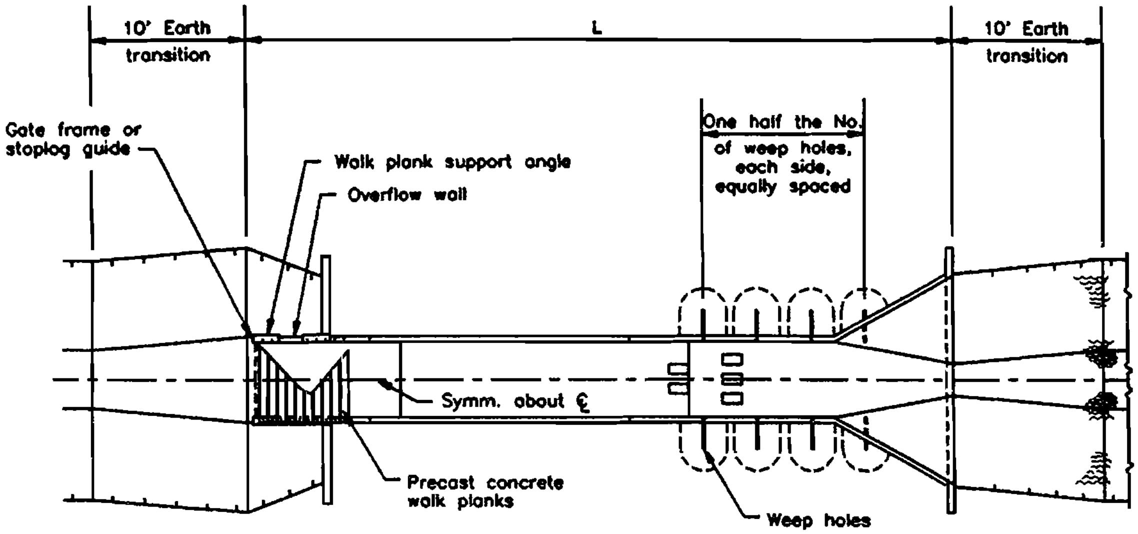
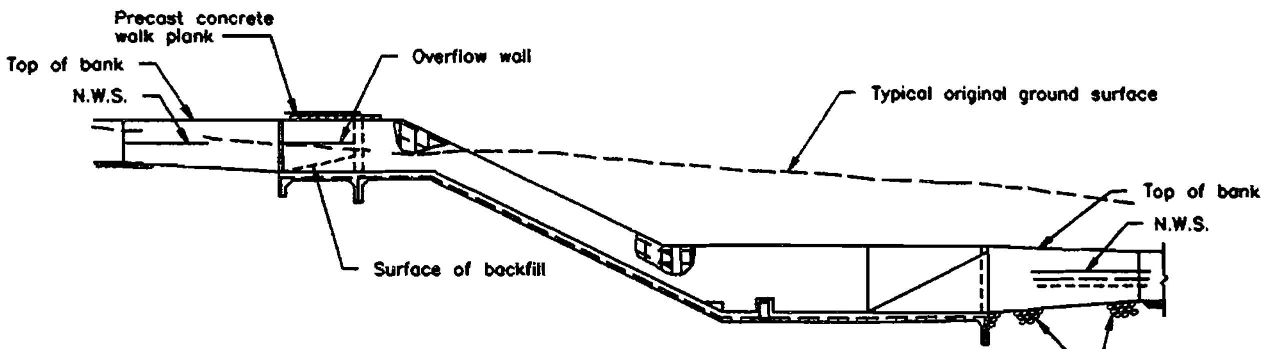
Figure
6–5

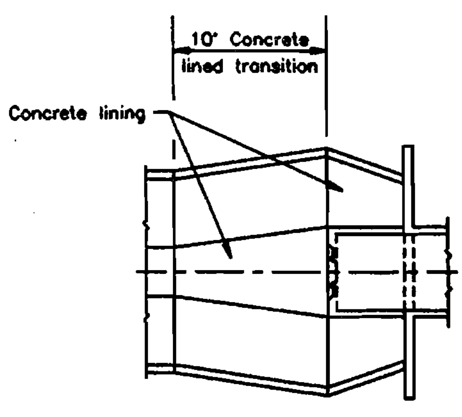


USBR Type 1

PLAN – OUTLET TO EARTH CANAL OR LATERAL



LONGITUDINAL SECTION



CONTROL INLET FROM CONCRETE LINED CANAL

Source: Bureau of Reclamation, Design of Small Canal Structures.

Rectangular Inclined Drop

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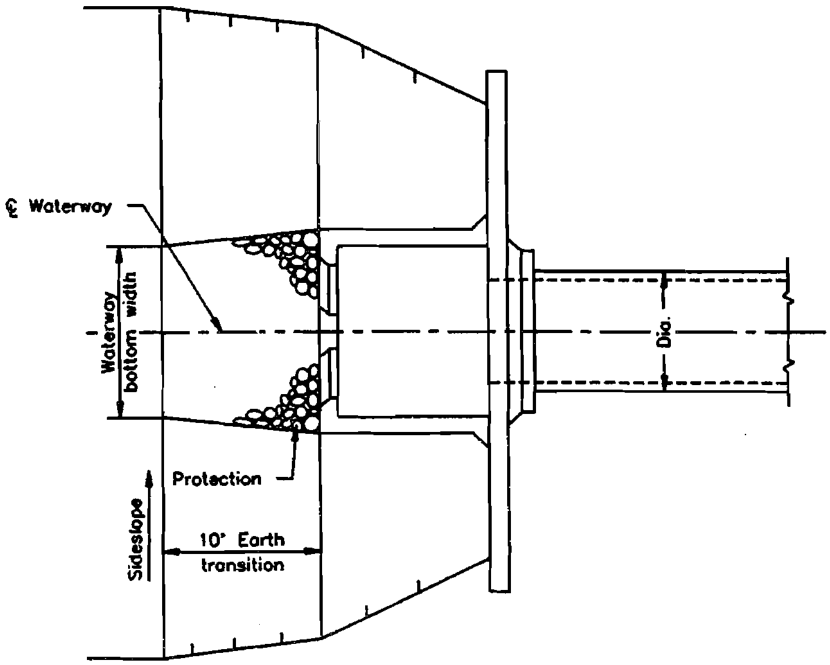
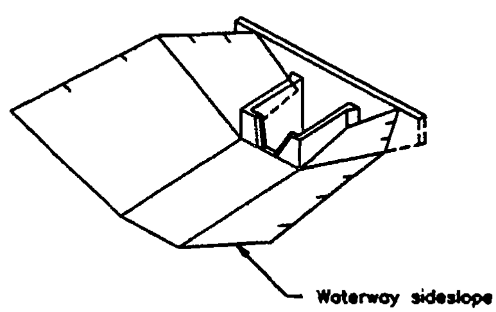
Figure 6-6



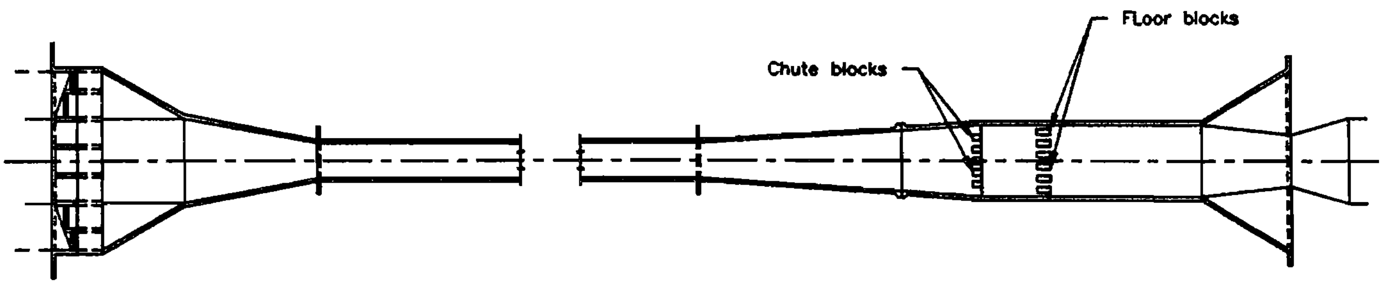
PLAN



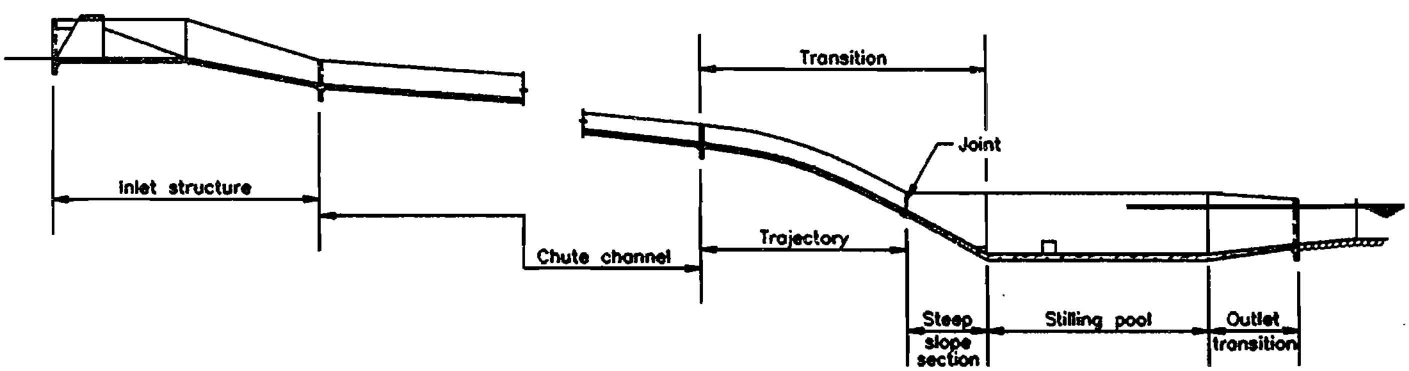
ISOMETRIC PROJECTION

Source: Bureau of Reclamation, Design of Small Canal Structures.

Control and Pipe Inlet



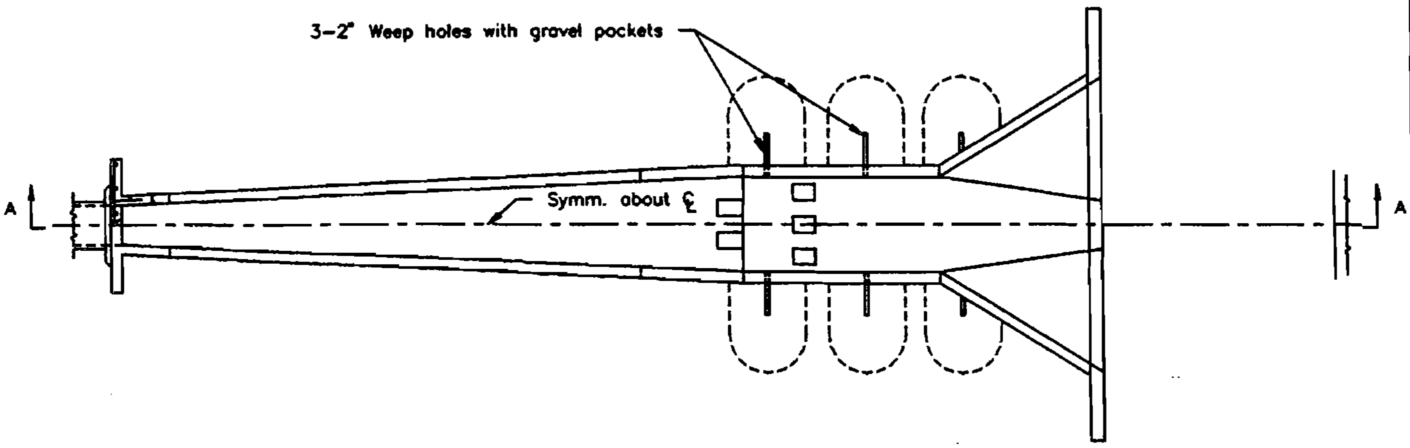
PLAN



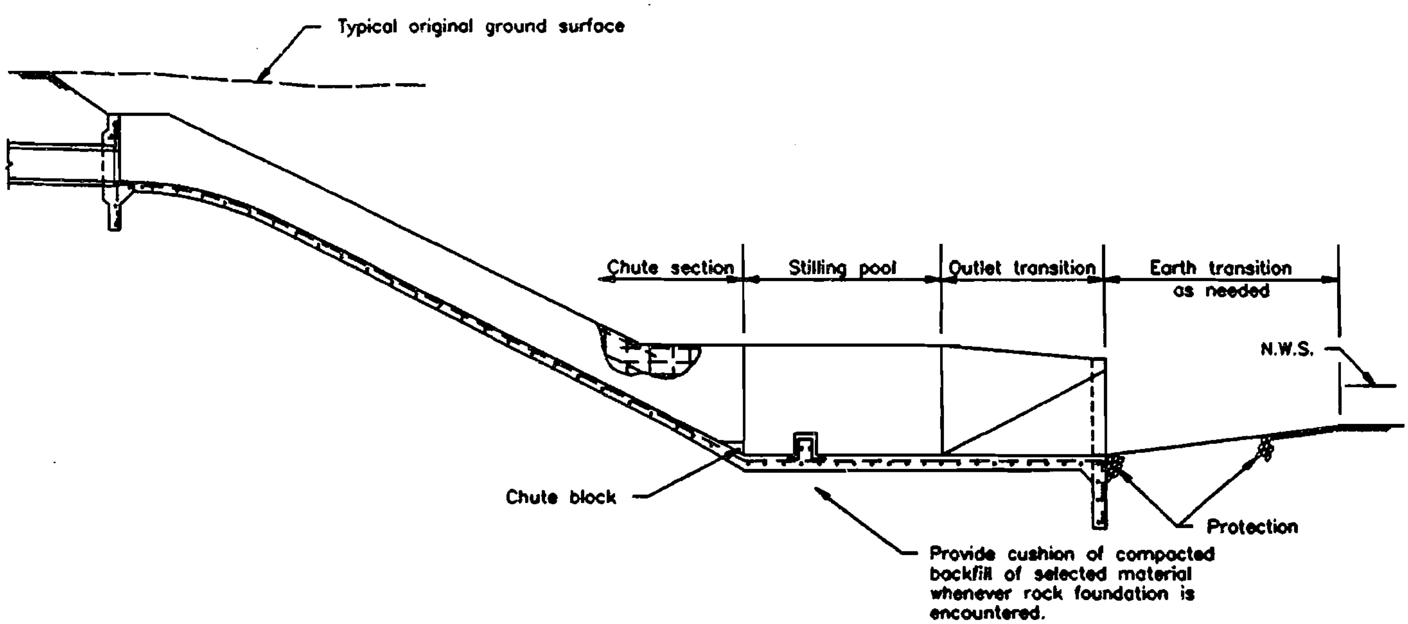
LONGITUDINAL SECTION

Source: Bureau of Reclamation, Design of Small Canal Structures.

Typical Rectangular Chutes



TYPICAL PLAN



SECTION A-A

Source: Bureau of Reclamation, Design of Small Canal Structures.

Stilling Pool on Pipe Chute

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Figure
6-9

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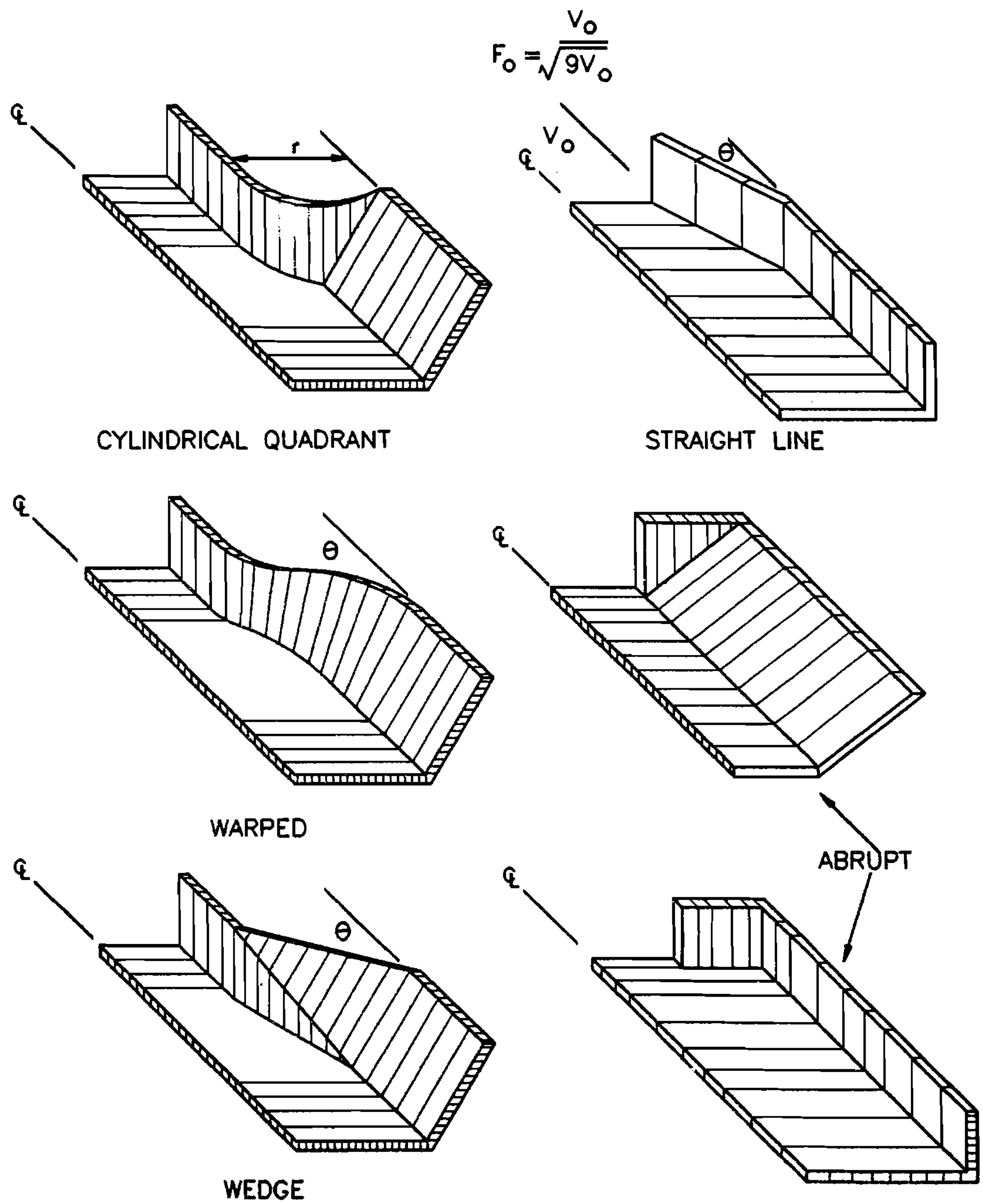

Source: Federal Highway Administration, HEC No. 14, Hydraulic Design of Energy Dissipators for Culverts and Channels.

Channel Transition Types

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Figure 6-10

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7.1 INTRODUCTION

The function of a drainage culvert or bridge is to pass the design storm flow under a roadway, railroad, or other feature without causing excessive backwater and without creating excessive downstream velocities.

7.2 CULVERT AND BRIDGE DESIGN CRITERIA

7.2.1 Design Frequency

Culverts and bridges shall be designed such that there is no overtopping of the associated roadway for the 25-year storm event. For the 100-year storm event, overtopping of local and collector streets shall not exceed 18 inches and overtopping of arterial streets shall not exceed 12 inches. In addition, for the 100-year storm event, residential dwellings or public, commercial and industrial buildings shall not be inundated at the lowest finished floor elevation unless the building is flood proofed.

7.2.2 Culvert Discharge Velocities

The velocity of discharge from culverts should have consideration given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and roadway embankment. The maximum permissible velocity released from culverts into downstream channels shall be based on criteria discussed in Section 6.

It is recommended that a minimum velocity of 3.0 feet per second for the design flow be maintained in all drainage structures to prevent siltation. Where doubt exists concerning silt or scour, protection commensurate with the value of the structure and surrounding property shall be installed to insure that damage to or failure of the structure will not occur.

7.2.3 Culvert Material Types

Material for culverts shall consist of either concrete or corrugated metal. Structural plate products are also acceptable. It is recommended that culverts under arterials be made of concrete. All corrugated metal culverts shall be designed with headwall and endwall treatments, with the exception of driveway crossings.

7.2.4 Bridge Openings

Bridge openings should be designed to have as little effect on the flow characteristics as is possible, consistent with good bridge design and economics. However, in regard to supercritical flow with a lined channel, the bridge should not affect the flow at all. That is, there should be no projections into the design water prism.

7.3 CULVERT TYPES

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. There are two major types of culvert flow; inlet and outlet control. For each type of control, different factors and equations are used to compute the hydraulic capacity of a culvert.

A culvert barrel may flow full or partly full. Full flow throughout the length of pipe is rare, and generally at least part of the barrel flows partly full. Water is flowing under pressure in a full flow condition and the capacity of the culvert is affected by the upstream and downstream conditions and the hydraulic characteristics of the culvert. Partly full or free surface flow can be categorized as subcritical, critical, or supercritical.

A dimensionless parameter known as the Froude Number (Fr) is calculated to categorize the flow. The Froude Number is discussed in Section 6. The different flow regimes are summarized in Table 7-1.

TABLE 7-1 Flow Categories

Flow Type	Depth	Velocity	Froude Number
Subcritical	$y > y_c$	$V < V_c$	$Fr < 1$
Critical	$y = y_c$	$V = V_c$	$Fr = 1$
Supercritical	$y < y_c$	$V > V_c$	$Fr > 1$

7.3.1 Inlet Control

Under inlet control, only the headwater and the inlet configuration affect the hydraulic performance. The headwater depth is measured from the invert of the culvert entrance to the water surface at the culvert entrance. This water surface elevation at the culvert entrance supplies the energy necessary to force flow through a culvert. The inlet face area is the same as the barrel area for nonimproved inlets. The inlet edge configuration is an important factor in the hydraulic efficiency of culverts. Types of configurations include projected, mitered, square edges in a headwall, and beveled edge. Figure 7-1 shows several types of inlet control.

In a condition where neither the inlet nor the outlet end of the culvert are submerged, the flow passes through critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partly full over its length, and the flow approaches normal depth at the outlet end.

If critical flow occurs near the inlet, the culvert is operating under inlet control. The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet is governed by the following equation when the approach velocity is considered zero (Figure 7-1A)

$$HW = d_c + \frac{V_2^2}{2g} + h_e \tag{7-1}$$

where:

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D

d_c = critical depth of flow, in feet

V_2 = critical velocity at entrance of culvert, in feet per second

g = acceleration of gravity, 32.2 feet per second squared

h_e = entrance head loss, in feet

= $k_e \left[\frac{V_2^2}{2g}\right]$

k_e = entrance loss coefficient (Table 7-2)

The submergence of the outlet end of the culvert does not assure outlet control as shown in Figure 7-1B. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

Figure 7-1C is a more typical design situation. The inlet end is submerged and the outlet end flows freely, the flow is supercritical and the barrel flows partly full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert.

Figure 7-1D illustrates the fact that submergence of both the inlet and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet is required to ventilate the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.

TABLE 7-2 Entrance Loss Coefficients

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete:	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square-cut end	0.5
Headwall or headwall and wingwalls:	
Socket end of pipe (groove end)	0.2
Square-edged	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
* End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal:	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
* End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete:	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End sections conforming to fill slope" are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have superior hydraulic performance.

Source: Federal Highway Administration, Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts.

7.3.2 Outlet Control

All the factors affecting the hydraulic performance of a culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length and slope) and the tailwater elevation affect culvert performance in outlet control.

The barrel roughness is a function of the culvert material and is represented by Manning's "n" coefficient. The barrel length is the total length extending from the entrance to the exit of the culvert. The barrel slope is the actual slope of the culvert and is often equivalent to the slope of the stream. The tailwater elevation is based upon the downstream water surface elevation measured from the outlet invert. Backwater calculations or normal depth approximations, when appropriate, are two methods used to determine the tailwater elevation. Figure 7-2 depicts several examples of outlet control.

Figure 7-2A represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

In Figure 7-2B, the outlet is submerged with the inlet unsubmerged. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Most culverts flow with free outlet, but, depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installations. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_o L \qquad (7\text{-}2)$$

where:

HW	=	headwater depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.
H	=	head for culvert flowing full, in feet
TW	=	tailwater depth, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

Figure 7-2-C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition.

Figure 7-2-D is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partly full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.

The capacity of a culvert flowing full over at least part of its length with a submerged entrance ($HW \geq 1.2$ D) is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if tailwater depth is less than critical depth ($TW < d_c$). If tailwater depth is greater than critical depth ($TW > d_c$), outlet velocity is based on tailwater depth.

$$HW = H + P - S_o L \qquad (7\text{-}3)$$

where:

HW	=	headwater depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.
H	=	head for culverts flowing full, in feet
P	=	pressure line height, in feet
	=	$(d_c + D)/2$
d_c	=	critical depth, in feet
D	=	diameter or height of structure, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

In the condition where neither the inlet nor the outlet end of the culvert are submerged, the barrel flows partly full over its entire length, and the flow profile can be subcritical or supercritical. The tailwater depth can be above or below critical flow.

If the headwater pool elevation does not submerge the culvert inlet ($HW < 1.2D$), the slope at design discharge is subcritical ($S_o < S_c$), the tailwater depth is above critical depth ($TW \geq d_c$), and the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_o L \tag{7-4}$$

where:

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D

TW = tailwater depth above the invert of the downstream end of the culvert, in feet

V_{TW} = culvert discharge velocity, at tailwater depth, in feet per second

h_e = entrance head loss, in feet

= $k_e \left[\frac{V_{TW}^2}{2g} \right]$

k_e = entrance loss coefficient (Table 7-2)

g = acceleration of gravity, 32.2 feet per second squared

h_f = friction head loss, in feet

= $\frac{29n^2L}{R^{1.33}} \left[\frac{V^2}{2g} \right]$

n = Manning's roughness coefficient

L = length of culvert barrel, in feet

V = average pipe velocity, in feet per second

= Q/A

Q = discharge, in cubic feet per second

A = cross sectional area of flow, in square feet

R	=	hydraulic radius, in feet
	=	A/WP
WP	=	wetted perimeter, in feet
S_o	=	slope of culvert, in feet per foot

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth ($TW \leq d_c$) is governed by the following equation when the approach velocity is considered zero. The entrance is unsubmerged (HW < 1.2D), and the design discharge is subcritical ($S_o < S_c$).

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_o L \tag{7-5}$$

where:

HW	=	headwater depth above the invert of the upstream end of the culvert, in feet. Headwater must be equal to or less than 1.2D, or entrance is submerged.
d_c	=	critical depth, in feet
V_c	=	critical velocity occurring at critical depth, in feet per second
h_e	=	entrance head loss, in feet
	=	$k_e \left[\frac{V_c^2}{2g}\right]$
k_e	=	entrance loss coefficient (Table 7-2)
g	=	acceleration of gravity, 32.2 feet per second squared
h_f	=	friction head loss, in feet
	=	$\frac{29n^2L}{R^{1.33}}\left[\frac{V^2}{2g}\right]$
V	=	average pipe velocity, in feet per second
	=	Q/A
Q	=	discharge, in cubic feet per second
A	=	cross sectional area of flow, in square feet

n = Manning's roughness coefficient

L = length of culvert barrel, in feet

R = hydraulic radius, in feet

= A/WP

WP = wetted perimeter, in feet

S_o = slope of culvert, in feet per foot

7.4 CULVERT END TREATMENTS

The normal functions of properly designed headwalls and endwalls for culverts are to provide gradual flow transition, to anchor the culvert preventing movement, to control erosion and scour resulting from excessive flow 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 headwalls, flared headwalls, or warped headwalls with or without aprons, as may be required by local site conditions. All corrugated metal culverts shall have headwall and endwalls with the exception of driveway crossings unless otherwise directed by the City Engineer.

7.4.1 Entrance Conditions

It is important to recognize that the operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts must involve consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

$$h_e = k_e \left[\frac{V^2}{2g}\right] \qquad (7\text{-}6)$$

where:

h_e = entrance head loss, in feet

k_e = entrance loss coefficient (Table 7-2)

V = velocity of flow in culvert, in feet per second

g = acceleration of gravity, 32.2 feet per second squared

7.4.2 Headwall/Endwall Treatment

In general, the following guidelines should be used in the selection of the type of headwall or endwalls:

Parallel Headwall and Endwall

A. Approach velocities are low (below 6 feet per second).

B. Backwater pools may be permitted.

C. Approach channel is undefined.

D. Sufficient right-of-way or easement is available.

E. Downstream channel protection is not required.

Flared Headwall and Endwall

A. Channel is well defined.

B. Approach velocities are between 6 and 10 feet per second.

C. Medium amounts of debris exist.

Warped (twisted) Headwall and Endwall

A. Channel is well defined and concrete lined.

B. Approach velocities are between 8 and 20 feet per second.

C. Medium amounts of debris exist.

Warped headwalls are effective with drop-down aprons to accelerate flow through the culvert, and they are effective endwalls for transitional flow from closed conduit flow to open-channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

7.4.3 Improved Inlets

Several types of improved inlets have been developed. The use of these inlets may provide substantial savings by a reduction in the barrel size of the proposed structure. The use of these inlets is optional and should be based on an economic analysis by the designer. For box culverts, reinforced concrete structures, and structures using headwalls, the use of beveled inlets or tapered inlets is strongly recommended. For more information, the designer is referred to Hydraulic

Design of Highway Culverts[4], Hydraulic Design Series No. 5 (HDS-5), by the Federal Highway Administration (FHWA).

7.5 CULVERT DESIGN WITH STANDARD INLETS

The information and publications necessary to design culverts according to the procedure presented in this Section can be found in Hydraulic Design of Highway Culverts[4], Hydraulic Design Series No. 5. Several charts and nomographs, from this publication, covering the more common requirements, are located at the end of this section. For special cases and larger structure sizes, the FHWA publication should be used. In addition, an IBM PC compatible computer program (HY-8) is available from the FHWA to perform these calculations.

7.5.1 Culvert Sizing

Figures 7-3 through 7-10 contain a series of curves which show the discharge capacity per barrel for each of several sizes of similar type culverts for various headwater depths above the culvert invert at the inlet. The invert of the culvert is defined as the low point of its cross-section.

Each culvert size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to the slope, $100S_o$, in percent. The dashed lines represent the maximum $L/(100S_o)$ ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of $L/(100S_o)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100S_o)$ value given on the solid line. The solid-line inlet control curves are plotted from model test data. The dashed-line outlet control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed (tailwater depth is assumed to not influence the culvert performance).

For culverts flowing under outlet control, the head loss at the entrance was computed, and the hydraulic roughness of the various culvert materials was taken into account in computing resistance loss for full or partly full flow. The Manning's "n" values used for each culvert type ranged from 0.012 to 0.032. Table 7-3 lists typical roughness coefficients for various types of culverts.

Except for large pipe sizes, headwater depths on the charts extend to three times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are generally used in areas of low fill. The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line, the headwater elevation should be checked with the standard inlet nomographs found in Figures 7-11 through 7-18.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head, that is depth plus velocity head, for flow in the approach channel. In most cases, the water surface upstream from the inlet is close to this level and the chart determination may be used as headwater depth for practical design purposes.

Where the approach velocity is in excess of 3.0 feet per second, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

Proper use of the capacity charts, Figures 7-3 through 7-10, will minimize problems of scour or sedimentation. The procedure for sizing the culvert is summarized below.

TABLE 7-3 Manning's Roughness Coefficients "n" for Culverts

Construction Materials	Design Coefficient[1]					
Concrete Pipe	0.012					
Corrugated-Metal Pipe*	2-2/3"x1/2" Corrugations			3"x1" Corrugations		
Unpaved	0.024			0.027		
25% Paved	0.021			0.023		
	Diameter					
Structural Plate Pipe*	5 ft	7 ft		10 ft		15 ft
Unpaved	0.033	0.032		0.030		0.028
25% Paved	0.028	0.027		0.026		0.024
Helically Corrugated Pipe*	2-2/3"x1/2" Corrugations					
	Diameter					
	12"	18"	24"	36"	48"	60" and Larger
Unpaved	0.011	0.014	0.016	0.019	0.020	0.021
25% Paved	--	--	0.015	0.017	0.020	0.019
	3"x1" Corrugations					
	Diameter					
	48"	54"	60"	66"	72"	78" and Larger
Unpaved	0.023	0.023	0.024	0.025	0.026	0.027
25% Paved	0.020	0.020	0.021	0.022	0.022	0.023

[1] Designer may select a single representative "n" for design purposes.

* Fully Paved All Types 0.012

Source: American Iron and Steel Institute, Modern Sewer Design

A. List design data: Q (cfs), L (ft), allowable HW (ft), S_o (ft/ft), type of culvert barrel and entrance.

B. Compute $L/(100S_o)$.

C. Enter the appropriate capacity chart with the design discharge, Q.

D. Find the $L/(100S_o)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figures 7-3 through 7-10, use the nomographs to check headwater conditions.

E. If the computed $L/(100S_o)$ is less than the value of $L/(100S_o)$ given for the solid line, then the value of HW is the value obtained from the solid line curve. If the computed L/(100S) is larger than the value for the dashed outlet control curve, then special measures must be taken, and the reader is referred to the Federal Highway Administration publications listed in the bibliography. Check the HW value obtained with the allowable HW. If the indicated HW is greater than the allowable HW, then try the next larger pipe size.

The advantage of the capacity charts (Figures 7-3 through 7-10) over the standard inlet nomographs (Figure 7-11 through 7-18) is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical at the outlet. Critical depth for various culvert sections can be determined using the appropriate curves on Figures 7-19 and 7-20. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used. However, both methods will provide the same results where either of the two methods are applicable.

Inlet Control

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration in inlet control. The approach velocity head may be included as part of the headwater if desired. The inlet control nomograph for standard inlets located at the end of this section (Figures 7-11 through 7-13) is used in the design process. The proper culvert selection procedure is outlined by Steps A through F.

A. Locate the selected culvert size and flow rate on the appropriate scales of the inlet control nomograph. (Note that for box culverts, the flow rate per foot of barrel width is used.)

B. Using a straightedge, carefully extend a straight line from the culvert size through the flow rate and mark a point on the first headwater/culvert height (HW/D) scale. The first HW/D scale is also a turning line.

C. If another HW/D scale is required, extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.

D. Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. If the approach velocity is neglected, HW equals the required headwater depth (HW_i). If the approach velocity is included in the calculations, deduct the approach velocity head from HW to determine HW_i.

E. Calculate the required depression (FALL) of the inlet control section below the stream bed as follows:

$$HW_d = EL_{hd} - EL_{sf} \tag{7-7}$$

$$FALL = HW_i - HW_d \tag{7-8}$$

where:

HW_d = design headwater depth, in feet

EL_{hd} = design headwater elevation, in feet

EL_{sf} = elevation of the stream bed at the face, in feet

FALL = required depression below the stream bed, in feet

HW_i = required headwater depth, in feet

Possible results and consequences of this calculation are:

1. If the FALL is negative or zero, set FALL equal to zero and proceed to step F.

2. If the FALL is positive, the inlet control section invert must be depressed below the stream bed at the face by that amount. If the FALL is acceptable, proceed to step F.

3. If the FALL is positive and greater than is judged to be acceptable, select another culvert configuration and begin again at step A.

F. Calculate the inlet control section invert elevation as follows:

$$EL_i = EL_{sf} - FALL \tag{7-9}$$

where:

EL_i = the invert elevation at the face of the culvert (EL_f) or at the throat of the culvert with a tapered inlet (EL_t), in feet

EL_{sf} = elevation of the stream bed at the face, in feet

FALL = required depression below the stream bed, in feet

Outlet Control

Outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The approach and downstream velocities may be included in the design process, if desired. The outlet control nomographs and critical depth charts located at the end of this section (Figures 7-14 through 7-20) are used in the design process. The proper procedure for outlet control is outlined below.

A. Determine the tailwater depth (TW) above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations, or from field observations.

B. Enter the appropriate critical depth chart (Figure 7-19 or Figure 7-20) with the flow rate and read the critical depth (d_c). Remember that d_c cannot exceed D. The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for $d_c > 0.9D$, consult the Handbook of Hydraulics[1], by Brater and King or other hydraulic references.

C. Calculate $(d_c + D)/2$

D. Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o).

h_o = TW or $(d_c + D)/2$, whichever is larger.

E. From Table 7-2, obtain the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration.

F. Determine the losses through the culvert barrel, H, using the outlet control nomograph.

1. If the Manning's "n" value given in the outlet control nomograph is different than the Manning's "n" for the culvert, adjust the culvert length using the formula:

$$L_1 = L\left[\frac{n_1}{n}\right]^2 \qquad (7\text{-}10)$$

where:

L_1	=	adjusted culvert length, in feet
L	=	actual culvert length, in feet
n_1	=	desired Manning's "n" value

n = Manning's "n" value from the outlet control chart

Then, use L_1 rather than the actual culvert length when using the outlet control nomograph.

2. Using a straightedge, connect the culvert size with the culvert length on the appropriate k_e scale. This defines a point on the turning line.

3. Again using the straightedge, extend a line from the discharge through the point on the turning line to the Head Loss (H) scale. Read H. H is the energy loss through the culvert, including entrance, friction, and outlet losses.

   Note: Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph.

G. Calculate the required outlet control headwater elevation.

$$EL_{ho} = EL_o + H + h_o \tag{7-11}$$

where:

EL_{ho} = required outlet control headwater elevation, in feet

EL_o = invert elevation at the outlet, in feet

H = total head loss, in feet

h_o = depth from the culvert outlet invert to the hydraulic grade line, in feet

= TW or $(d_c+D)/2$, whichever is larger

If it is desired to include the approach and downstream velocities in the calculations, add the downstream velocity head and subtract the approach velocity head from the right side of the equation.

H. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

If the controlling headwater is based on inlet or outlet control determine the area of flow at the outlet based on the barrel geometry and the following:

A. Critical depth if the tailwater is at or below critical depth.

B. Tailwater depth if the tailwater is between critical depth and the top of the barrel.

C. The height of the barrel if the tailwater is above the top of the barrel.

Table 7-4 can be used to determine the cross sectional area of a circular pipe flowing part full. The ratio of d/D is the depth of water to the diameter of the pipe. The cross sectional area can be calculated with Equation 7-12.

$$A = C_a D^2 \tag{7-12}$$

where:

A	=	cross sectional area of pipe, in square feet
C_a	=	cross sectional area coefficient, Table 7-4
D	=	diameter of pipe, in feet

Repeat the design process until an acceptable culvert configuration is determined. Once the barrel is selected, it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wingwalls must be dimensioned.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partly full though its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, the approximate method should not be used.

TABLE 7-4 Cross Sectional Area Coefficients, C_a, for a Circular Conduit Flowing Part Full

$\frac{d^*}{D}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

$*\frac{d}{D} = \frac{\text{depth of water}}{\text{diameter of pipe}}$

Source: Brater and King, Handbook of Hydraulics.

If the selected culvert will not fit the site, return to the culvert design process and select another culvert. If neither tapered inlets nor flow routing are to be applied, document the design. An acceptable design should always be accompanied by a performance curve which displays culvert behavior over a range of discharges.

7.5.2 Design Procedure

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The procedure is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to ensure that some special problem is not overlooked.

Design Computation Forms

The Culvert Design Form, Figure 7-21, has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data of the form are also included. At the top right is a small sketch of the culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected.

The first step in the design process is to summarize all known data for the culvert at the top of the appropriate design form. This information will have been collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. When considering ponded and non-ponded inlets, scour is not likely to occur in an artificial channel when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. The flow velocity in the channel upstream from the culvert should be investigated to determine if scour will occur.

Culvert Diameter

After the invert elevations have been assumed and using the design computation forms, the capacity charts, and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. The smallest diameter that appears in the nomographs and capacity charts is 12 inches. Since the pipe roughness influences the culvert diameter, both concrete and corrugated metal pipe should be considered in design if both will satisfy the headwater requirements.

Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the above.

If the inlet invert is lowered, special consideration must be given to scour. The use of gabions or concrete drop structures, riprap, and headwalls with aprons and toe walls should be investigated and compared to obtain the proper design.

Culvert Outlet

The outlet velocity must be checked to determine if excessive scour will occur downstream. If scour is indicated, then riprap, an expanding end section, or a more sophisticated energy dissipating structure should be used.

Minimum Slope

To prevent sediment from plugging the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity of 3.0 feet per second for design flow. The slope should be checked for each design, and, if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these used.

Example 1 Pipe Culvert

Given: Design Discharge, Q_{100} = 200 cfs

Allowable Headwater Elevation = 108.0 ft

Shoulder Elevation = 111 ft
Elevation Inlet Invert = 100 ft

Culvert Length, L_a = 200 ft

Downstream channel approximates a 5-foot wide trapezoidal channel with 1.5H:1V side slopes, a Manning's "n" of 0.04, and S_o = 0.01 ft/ft

Find: Design a circular corrugated metal pipe with standard 2-2/3 by 1/2 inch corrugations and a concrete pipe with a groove end. Set the inlet invert at the material stream bed elevation.

Note: Use Figures 7-12, 7-13, 7-15, 7-16 and 7-19 and Design Form, Figure 7-21.

PROJECT: EXAMPLE PROBLEM No. 1 CHAPTER III, HDS No. 5	STATION: 1+00 SHEET 1 OF 1	**CULVERT DESIGN FORM** DESIGNER / DATE: LAC, 7/18 REVIEWER / DATE: DES, 7/19

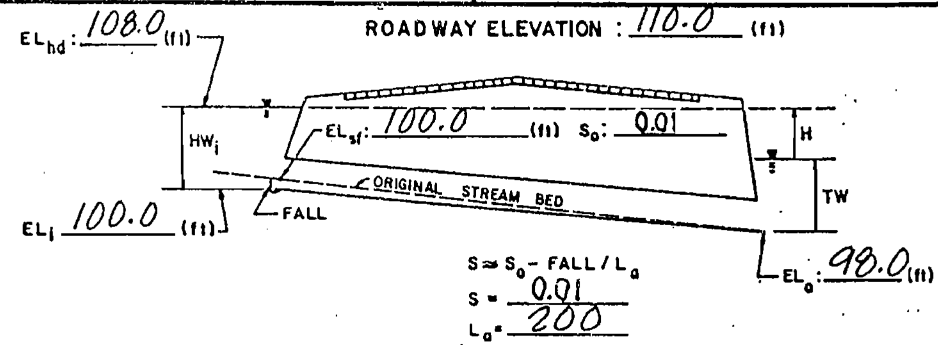
SEE ADD'L. SHTS.

HYDROLOGICAL DATA

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 125AC. ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAP, bw=5', n=0.04, 1.5H:1V
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	200	3.5



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
C.M.P.-CIRC.-72IN.-SQUARE EDGE	200	200	1.01	6.1	-	106.1	3.5	3.9	4.9	4.9	0.02	2.6	105.5	106.1	10.3	TRY 60" C.M.P.
" - " - 60IN.- " "			1.50	6.50	-	106.5		4.0	4.6	4.6		6.3	108.9	108.9	11.9	TRY 60" CONC.
CONC.- " - 60IN.- GROOVE END			1.36	6.8	-	106.8			4.6	4.6		2.9	105.5	106.8	11.9	TRY 54" CONC.
" - " - 54IN.- "			1.77	7.97	-	108.0			4.3	4.3		4.7	107.0	108.0	13.4	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_f/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2 / 2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

HIGH OUTLET VELOCITY - OUTLET PROTECTION OR LARGER CONDUIT MAY BE NECESSARY

CULVERT BARREL SELECTED:

- SIZE: 54 IN.
- SHAPE: CIRCULAR
- MATERIAL: CONC. n 0.012
- ENTRANCE: GROOVE END

7.6 OTHER INLET TYPES

7.6.1 Beveled Inlets

The hydraulics of a culvert can be improved by altering the inlet configuration. Beveled edges at the top of a culvert inlet will enhance the flow characteristics. Figures 7-22 through 7-25 are used to calculate the headwater for inlet control for beveled inlet edges.

Example 2 **Box Culvert**

Given: Design Discharge, Q_{100} = 300 cfs

Slope of stream bed, S_o = 0.02 ft/ft

Allowable Headwater Elevation = 110.0 ft

Shoulder Elevation = 113.5 ft

Elevation Inlet Invert = 100.0 ft

Culvert Length, L_a = 250 ft

Downstream channel approximates a 6-foot wide trapezoidal channel with 1H:1V side slopes and a Manning's "n" of 0.05.

Find: Design a reinforced concrete box culvert. Try both square and 45° beveled edges in a headwall. Do not depress the inlet (no FALL).

Note: Use Figures 7-11, 7-14, 7-20, and 7-23 and Design Form, Figure 7-21 for the solution.

PROJECT: EXAMPLE PROBLEM No. 2 CHAPTER III, HDS No. 5

STATION: 1+00

SHEET 1 **OF** 1

CULVERT DESIGN FORM

DESIGNER / DATE: LAC / 7/18

REVIEWER / DATE: GAB / 7/19

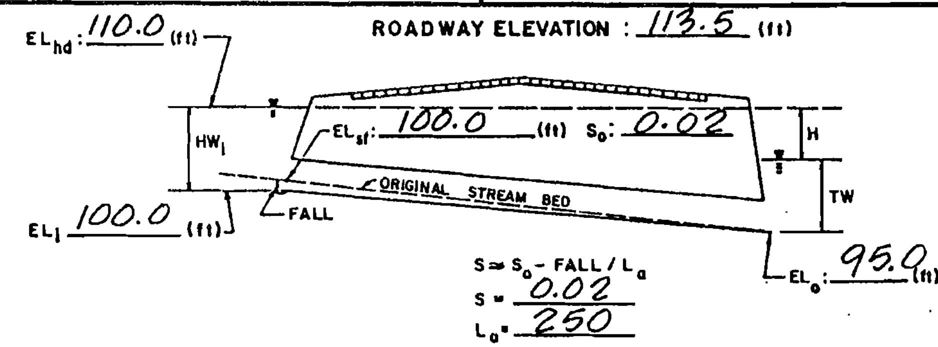
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: SCS
- ☐ DRAINAGE AREA: 200 AC. ☐ STREAM SLOPE: 2.0%
- ☐ CHANNEL SHAPE: TRAP. 1H:1V, Bw = 614, n = 0.05
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	300	4.0



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
CONC.-BOX-6'x5'-SQ. EDGE	300	50	1.57	7.9	–	107.9	4.0	4.2	4.6	4.6	0.5	3.55	103.2	107.9	21.7	OK. TRY SMALL BOX
" - " -5'x5'- "	300	60	1.91	9.6	–	109.6	4.0	4.8	4.9	4.9	0.5	5.2	105.1	109.6	20.8	CHECK BEVEL
" - " - " - 45° BEVEL	300	60	1.71	8.55	–	108.6	4.0	4.8	4.9	4.9	0.2	4.6	104.5	108.6	20.8	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

5'x5' BOX WILL WORK WITH OR WITHOUT BEVELS. BEVELS PROVIDE ADDTIONAL FLOW CAPACITY.

CULVERT BARREL SELECTED:

SIZE: 5'x5'

SHAPE: RECTANGULAR

MATERIAL: CONC. n 0.012

ENTRANCE: SQ.EDGE-90° HEADWALL

7.6.2 Performance Curves

The objective of this design procedure is the hydraulic design of culverts using improved inlets. This design procedure hinges on the selection of a culvert barrel based on its outlet control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustment of its elevation in order to achieve inlet-control performance compatible with the outlet control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

Performance curves are an integral part of the culvert design process and can be used to optimize the selected culvert design, particularly when using tapered inlets and/or upstream depressions. This optimization may involve further reduction in the barrel size required to pass the design flow at the design headwater, provision of a factor of safety against damages, or a more balanced design. The visualization of culvert performance provided by performance curves may lead to a further reduction in the size of the culvert barrel.

The procedures described here enable the designer to maximize the performance of the selected culvert or to optimize the design in accordance with an evaluation of site constraints, design parameters, and costs for construction and maintenance.

Outlet Control Performance Curves

Outlet control calculations are performed before inlet control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control (HW_o) exceeding the allowable headwater elevation (AHW EL). For use in this procedure, the equation for headwater is in terms of elevation.

The full-flow outlet control performance curve for a given culvert (size, inlet edge, shape, or material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet control curve is shifted to the right, indicating a higher capacity for a given head (see Figure 7-26). Also, it is generally true that increasing the barrel size will flatten the slope of the outlet control curve.

The outlet control curve passing closest to and below the AHW EL, for the design Q on the performance-curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design), or the use of such a small barrel may not be practical. The proper culvert selection procedure to define the outlet control performance curves is:

A. Calculate headwater elevations, HW_o, at design discharge for trial culvert sizes, entrance conditions, shapes and materials.

B. Calculate headwater elevations at two additional discharge values one above and one below the design Q, in order to define outlet-control performance.

C. Plot outlet control performance curves for trial culvert sizes.

D. Select barrel size, shape and material.

This selection should not be based solely on calculations which indicate that the required headwater at the design discharge is near the AHW EL, but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characteristics, and the possible consequences of a flood occurrence in excess of the estimated design flood. A sharply rising outlet control performance curve may be sufficient reason to select a culvert of different size, shape or material. A typical series of outlet control performance curves is shown in Figure 7-27.

In order to calculate the barrel size required in outlet control, the applicable outlet control for standard inlets (Figures 7-14 through 7-18) nomograph may be used as follows:

A. Intersect the "Turning Line" with a line drawn between Discharge and Head, H. To estimate H, use the following equation:

$$H = AHW\ EL - Outlet\ Invert\ Elevation - h_o \tag{7-13}$$

where h_o may be selected as the elevation at culvert height. Accuracy is not critical at this point.

B. Using the point on the "Turning Line", k_e, and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Inlet Control Performance Curves

Inlet control performance curves should be drawn for the inlet edge configurations selected. These edges may include square edges, beveled edges, or the throat section of a tapered inlet. A FALL may be incorporated upstream of the inlet control section to lower the inlet control headwater elevation. To construct the inlet control performance curves, perform the following steps:

A. Calculate the inlet control headwater depth, H_f, at the culvert face at design discharge for the culvert selected based on outlet control.

B. Determine the required face invert elevation to pass the design discharge by subtracting H_f from the AHW EL.

1. If this invert elevation is above the stream bed elevation at the face, the invert would generally be placed on the stream bed and the culvert will

then have a capacity greater than the design Q with headwater at the AHW EL.

2. If this required invert elevation is below the stream bed elevation at the face, the invert must be depressed using a FALL.

3. Add H_f to the invert elevation to determine HW_f. If HW_f is lower than HW_o, the barrel operates in outlet control at the design Q.

4. If the required FALL is excessive from the stand-point of aesthetics, economy and other engineering reasons, the inlet geometry must be improved or a larger barrel must be used. If square edges were assumed, repeat with beveled edges, and if beveled edges were assumed, try a side-tapered inlet.

5. If the FALL is acceptable, determine the inlet control performance by calculating the required headwater elevation using the discharges from the Outlet Control Performance curves, and the FALL determined above. $HW_f = H_f +$ EL face invert.

6. Plot the inlet control performance curve with the outlet-control performance curve.

Figure 7-28 depicts a series of inlet edge configurations, along with the outlet control performance curve for the selected barrel. Note that an inlet with square edges and no FALL will not meet the design conditions. Either square edges with a FALL or beveled edges with no FALL satisfy the design criteria.

Analyze the Effect of Additional FALL

It is apparent from Figure 7-28 that either additional FALL or inlet improvements such as a tapered-inlet would increase the culvert capacity in inlet control by moving the inlet control performance curve to the right toward the outlet control performance curve. If the outlet control performance curve of the selected culvert passes below the point defined by the AHW EL, and the design Q, there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW EL or to pass the design discharge at the lowest possible headwater elevation.

Some possibilities are illustrated on Figure 7-29. The minimum inlet control performance which will meet the selected design criteria is illustrated by Curve A. In this design, the cost for inlet improvements and/or FALL is at a minimum and the inlet will pass a flood in excess of the design Q before performance is governed by outlet control. This performance is adequate in many locations, including those locations where headwaters in excess of the AHW EL would be tolerable.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW EL. A safety factor in capacity is thereby incorporated by geometric improvements at the inlet, by a FALL, or by a combination of the two. Additional inlet improvement and/or FALL will not increase the capacity at or above the AHW EL.

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are insufficient to cause the AHW EL to be set at a lower elevation. The maximum possible reduction in headwater at the design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at the design Q.

The water surface elevation in the natural stream may be a limiting factor in design, i.e., it is not productive to design for headwater at a lower elevation than natural stream flow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, curve D gives the maximum reduction in headwater elevation at the design Q. Flow depths calculated by assuming normal depth in the stream channel may be used to estimate natural water surface elevations in the stream at the culvert inlet.

Curve A has been previously established for the inlet control section. To define any other inlet control performance curve such as B, C or D for the same control section:

A. Select the point of interest on the outlet control performance curve.

B. Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established. For example, the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

Tapered Inlet Face Control Performance Curves

The face is the entrance of the barrel perpendicular to the flowline. The throat lies within the barrel section and is the intersection of the transition section and the barrel. The face or throat control is that section which governs the hydraulics of the structure.

Side-tapered inlets have an enlarged face area with a transition to the culvert barrel accomplished by tapering the sidewalls. The inlet has equal barrel and face heights except for the addition of a top bevel at the face. Slope-tapered inlets provide a greater head at the control section. The capacity depends largely upon the amount of FALL between the invert at the face and the invert at the throat section.

Either a side- or slope-tapered inlet with an upstream sump or slope-tapered inlet design may be used if a FALL is required at the throat control section of a tapered inlet. The minimum face design for the tapered inlet is one with a performance curve which does not exceed the AHW EL at the design Q. However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater gained through use of various FALLS. This suggests a face-control curve which intersects the throat-control curve either: 1) at the AHW EL; 2) at the design Q; 3) at its intersection with the outlet control curve, or; 4) at other points selected by the designer.

These options are illustrated on Figure 7-30 by points "a" through "e" representing the intersections of face-control curves with the throat-control curves. The options are explained as follows:

A. Intersection of face- and throat-control curves at the AHW EL (Point a or b). For the minimum acceptable throat-control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW EL at the design Q (Point a). For throat-control performance greater than minimum but equal to or less than Curve B, this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW EL (Point b).

B. Intersection of face- and throat-control curves at the design Q (Points a, c or d). This option makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than design Q.

C. Intersection of the face-control curve with throat-control curve at its intersection with the outlet control curve (Points b or e) This option is the minimum face size which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW EL at the design Q; e.g., with the minimum acceptable throat-control curve.

D. Variations in the above options are available to the designer. For example, the culvert face can be designed so that culvert performance will change from face control to throat control at any discharge at which inlet control governs. Options (A) through (C), however, fulfill most design objectives. Generally the design objective will be to design either the minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than the design Q.

Figure 7-31 illustrates some of the possible tapered-inlet designs for a specific design situation. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the side-tapered inlet nearly parallels the performance of the throat, and an increase in headwater on the throat by virtue of an increased FALL results in an almost equal increase in headwater on the face. Depressing the throat of a culvert with a side-tapered inlet requires additional barrel length.

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat. On the other hand, use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

7.6.3 Tapered Inlet Design

The following steps outline the design process for culverts with tapered inlets. Steps A and B are the same for all culverts, with and without tapered inlets.

A. Preliminary Culvert Sizing

Estimate the culvert barrel size to begin calculations.

B. Culvert Barrel Design

Complete the Culvert Design Form. (Figure 7-21) These calculations yield the required FALL at the culvert entrance. For the inlet control calculations, the appropriate inlet control nomograph is used for the tapered inlet throat. The required FALL is upstream of the inlet face section for side-tapered inlets and is between the face section and throat section for slope tapered inlets. Figure 7-21 should be completed for all barrels of interest. Plot outlet control performance curves for the barrels of interest and inlet control performance curves for the faces of culverts with nonenlarged inlets and for the throats of tapered inlets.

C. Tapered Inlet Design

Use the Tapered Inlet Design Form (Figure 7-32) for selecting the type of tapered inlet to be used and determining its dimensions. If a slope-tapered inlet with mitered face is selected, use the special design form shown in Figure 7-33. A separate form is provided for the mitered inlet because of its dimensional complexity.

To use the Tapered Inlet Design Form (Figure 7-32) or the design form for a slope-tapered inlet with mitered face (Figure 7-33), perform the following steps:

A. Complete Design Data

Fill in the required design data on the top of the form.

1. Flow, Q, is the selected design flow rate, from Figure 7-21.

2. EL_{hi} is the inlet control headwater elevation.

3. The elevation of the throat invert (EL_t) is the inlet invert elevation (EL_i) from Figure 7-21.

4. The elevation of the stream bed at the face (EL_{sf}), the stream slope (S_0), and the slope of the barrel (S) are from Figure 7-21. For the slope-tapered inlet with mitered face, estimate the elevation of the stream bed at the crest. This point is located upstream of the face sections.

5. The FALL is the difference between the stream bed elevation at the face and the throat invert elevation.

6. Select a side taper (TAPER) between 4H:1V and 6H:1V and a fall slope (S_f) between 2H:1V and 3H:1V. The TAPER may be modified during the calculations.

7. Enter the barrel shape and material, the size, and the inlet edge configuration from Figure 7-21. Note that for tapered inlets, the inlet edge configuration is designated the "tapered inlet throat."

B. Calculate the Face Width

1. Enter the flow rate, the inlet control headwater elevation (EL_{hi}), and the throat invert elevation on the design forms. For the slope-tapered inlet with mitered face, the face section is downstream of the crest. Calculate the vertical difference between the stream bed at the crest and the face invert (y). Y includes part of the total inlet FALL.

2. Perform the calculations resulting in the face width (B_f). Figures 7-34, 7-35, and 7-36 are face control nomographs. Figures 7-37 and 7-38 are used for throat control.

C. Calculated Tapered-Inlet Dimensions

If the FALL is less than D/4 (D/2 for a slope-tapered inlet with a mitered face), a side-tapered inlet must be used. Otherwise, either a side-tapered with a depression upstream of the face or a slope-tapered inlet may be used.

1. For a slope-tapered inlet with a vertical face, calculate L_2, L_3, and the TAPER. For the slope-tapered inlet with a mitered face, calculate the horizontal distance between the crest and the face section invert L_4. These dimensions are shown on the small sketches in the top center of Figure 7-32 and 7-33.

2. Calculate the overall tapered inlet length, L_1.

3. For a side-tapered inlet, check to assure that the FALL between the face section and the throat section is one foot or less. If not, return to step B with a revised face invert elevation.

D. Calculate the Minimum Crest Width

For a side-tapered inlet with FALL upstream of the face or for a slope-tapered inlet with a mitered face, calculate the minimum crest width and check it against the proposed crest width. In order to obtain the necessary crest length for a depressed side-tapered inlet, it may be necessary to increase the flare angle of the wingwalls, or to increase the length of crest on the sump. For a slope-tapered inlet with a mitered face, reduce the TAPER to increase crest width. Note that the TAPER must be greater than 4H:1V.

E. Fit the Design into the Embankment Section

Using a sketch based on the derived dimensions, and a sketch of the roadway section to the same scale, assure that the culvert design fits the site. Adjust inlet dimensions as necessary but do not reduce dimensions below the minimum dimensions on the design form.

F. Prepare Performance Curves

Using additional flow rate values and the appropriate nomographs, calculate a performance curve for the selected face section. Do not adjust inlet dimensions at this step in the design process. Plot the face control performance curve on the same sheet as the throat control and the outlet control performance curves.

G. Enter Design Dimensions

If the design is satisfactory, enter the dimensions at the lower right of Figure 7-32 or Figure 7-33. Otherwise, calculate another alternative design by returning to step A.

7.6.4 Dimensional Limitations

The following dimensional limitations must be observed when designing tapered inlets using the design figures located at the end of the section.

Side-Tapered Inlets

A. 4H:1V $\leq$ Taper $\leq$ 6H:1V - Tapers greater than 6H:1V may be used but performance will be underestimated.

B. Wingwall flare angle range from 15° to 26° with top edge beveled or from 26° to 90° with or without bevels.

C. If FALL is used upstream of face, extend barrel slope upstream from face a distance of D/2 before sloping upward more steeply. The maximum slope of the apron is 2H:1V.

D. For pipe culverts, these additional requirements apply:

1. $D \leq E \leq 1.1D$

2. Square to circular transition length $\geq 0.5D$

3. Square-throat dimension must equal the barrel diameter

Slope-Tapered Inlets

A. 4H:1V $\leq$ Taper $\leq$ 6H:1V - Tapers greater than 6H:1V may be used, but performance will be underestimated.

B. 3H:1V $\geq S_f \geq$ 2H:1V - If $S_f >$ 3H:1V, use side-tapered design

C. Minimum $L_3 = 0.5B$

D. $D/4 \leq FALL \leq 1.5D$

For FALL < D/4, use side-tapered design

For Fall < D/2, do not use the slope-tapered inlet with mitered face

For FALL > 1.5D, estimate friction losses between face and throat and add additional losses to HW_t

$$H_i = \left[\frac{29n^2L_i}{R^{1.33}}\right]\frac{Q^2}{2gA^2} \tag{7-14}$$

where:

H_i	=	friction head loss in the tapered inlet, in feet
n	=	Manning's roughness coefficient for the tapered inlet
L_i	=	length of tapered inlet, in feet
Q	=	flow rate, in cubic feet per second
R	=	average hydraulic radius of the tapered inlet, in feet
	=	$\frac{A_f + A_t}{P_f + P_t}$
A_f	=	area at face, in square feet
A_t	=	area at throat, in square feet
P_f	=	perimeter at face, in feet
P_t	=	perimeter at throat, in feet
g	=	acceleration of gravity, 32.2 feet per second squared
A	=	average cross sectional area of tapered inlet, in square feet
	=	$(A_f + A_t)/2$

E. Wingwall flare angle range from 15° to 26° with top edge beveled or from 26° to 90° with or without bevels.

F. For pipe culvert, these additional requirements apply:

1. Square to circular transition length $\geq 0.5D$

2. Square-throat dimension must equal the barrel diameter. It is not necessary to check square-throat performance.

Example 3 **Box Culvert**

Given: Design Discharge, Q_{100} = 400 cfs

Elevation of Outlet Invert = 172.5 ft

Allowable Headwater Elevation = 195 ft

Elevation of Shoulder = 198 ft

Streambed Slope = 0.05 ft/ft

Culvert Length; L_a = 300 ft

The tailwater variation is as follows:

Flow (cfs)	T.W. (ft)
300	4.4
400	4.9
500	5.3

Find: Design the smallest possible barrel to pass the peak flow rate without exceeding the AHW EL. The culvert will be located in a rural area with a low risk of damage. Underground utilities limit the available FALL to 2.5 ft below the standard stream bed elevation at the inlet. Use a reinforced concrete box culvert with n = 0.012.

Note: Use Figure 7-14, 7-20, 7-29, 7-34, 7-35 and 7-37 and Design Forms, Figures 7-21 and 7-32 in this solution.

PROJECT: EXAMPLE PROBLEM NO. 3
CHAPTER IV, HDS No. 5

STATION: 5+00
SHEET 1 **OF** 4

CULVERT DESIGN FORM
DESIGNER/DATE: LAC / 7/16
REVIEWER/DATE: LAC / 7/17

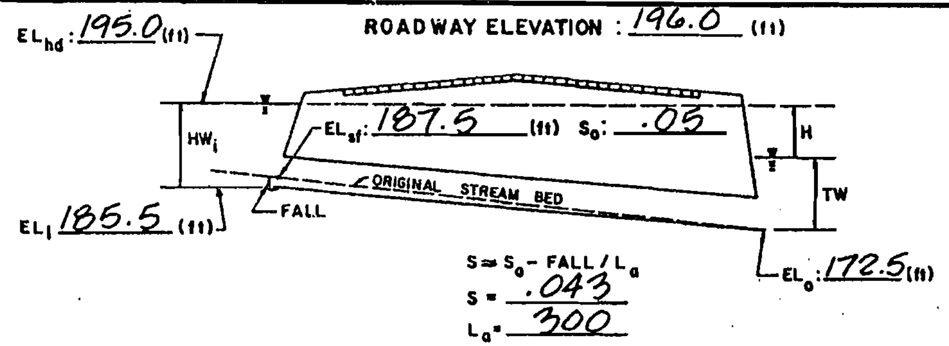
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 150AC. ☐ STREAM SLOPE: 5.0%
- ☐ CHANNEL SHAPE: TRAP.
- ☐ ROUTING: N/A ☐ OTHER: –

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	400	4.9



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
CONC.-BOX-5'x5'-45° BEVEL	400	80	2.4	12.0	4.5	195.0	4.5	≥5	5.0	5.0	0.2	8.9	186.4	195.0	–	EXCESSIVE FALL REQ'D.
" - " - " - TAPERED INLET THROAT	400	80	1.9	9.5	2.0	195.0	"	"	"	5.0		"	"	195.0	29.1	INLET AND FALL O.K.
" - " - " - "	300	60	1.48	7.4		192.9	4.4	4.8	4.9	4.9		5.0	182.4	192.9	–	PERFORM CURVE DATA
" - " - " - "	500	100	2.4	12.0		197.5	5.3	≥5	5.0	5.0		14.1	191.6	197.5	–	

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

USE 5'x5' CONCRETE BOX WITH TAPERED INLET. CHECK SIDE-&-SLOPE-TAPERED ENTRANCES

CULVERT BARREL SELECTED:

SIZE: 5'x5'
SHAPE: RECTANGULAR
MATERIAL: CONC. n .012
ENTRANCE: TAPERED INLET

TAPERED INLET DESIGN FORM

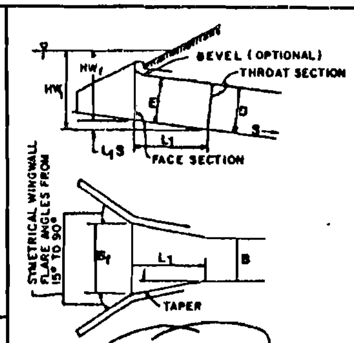
PROJECT: EXAMPLE PROBLEM NO. 3
CHAPTER IV, HDS NO. 5

STATION: 5+00
SHEET 2 OF 4

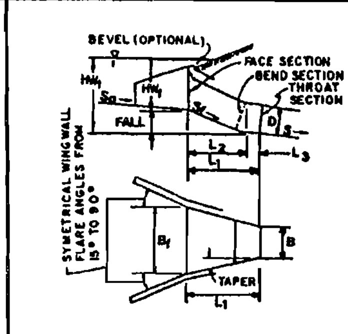
DESIGNER / DATE: LAC, 7/16
REVIEWER / DATE: LAC, 7/17

DESIGN DATA:

Q_{100} = 400 cfs ; EL_{hi} 195.0 ft
EL. THROAT INVERT 185.5 ft
EL. STREAM BED AT FACE 187.5 ft
FALL 2.0 ft TAPER 4 : 1 (4:1 TO 6:1)
STREAM SLOPE, S_o = .05 ft/ft
SLOPE OF BARREL, S = .043 ft/ft
S_f — : 1 (2:1 TO 3:1)
BARREL SHAPE AND MATERIAL: BOX-CONC.
N = 1 , B = 5 FT. , D = 5 FT.
INLET EDGE DESCRIPTION TAPERED INLET THROAT



SIDE-TAPERED



SLOPE-TAPERED

COMMENTS

USE 26° TO 90° WINGWALLS WITH NO BEVEL ON A SIDE-TAPERED INLET.

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f/E (3)	Q/B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)
400	195.0	185.5	186.5	8.5	1.7	47	8.51	9.0						8.0	188.5	6.5	8.5
400	193.8		185.8	7.95	1.59	44	←	9.0									
300	192.1			6.25	1.25	33	←	9.0	FACE CONTROL PERFORMANCE DATA								
500	196.0			10.2	2.04	56	←	9.0									

(1) SIDE-TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE-TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE
(2) $HW_f = EL_{hi}$ - EL. FACE INVERT
(3) $1.1D \geq E \geq D$
(4) FROM DESIGN CHARTS
(5) MIN. $B_f = Q/(Q/B_f)$
(6) MIN. $L_3 = 0.5NB$
(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f
(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_3$
(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_2$
(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$
(11) SIDE-TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER
SLOPE-TAPERED: $L_1 = L_2 + L_3$
(12) $HW_c = EL_{hi}$ - EL. CREST INVERT
(13) MIN. $W = 0.35Q/HW_c^{1.5}$

SELECTED DESIGN

B_f 8.5
L_1 8.0
L_2 ____
L_3 ____
BEVELS ANGLE 0°
b = 0 in ; d = 0 in
TAPER 4 : 1
S_f = — : 1

*ACTUAL EL. FACE INVERT = $EL_t + L_1S = 185.5 + (8.0)(.043) = 185.8$ FT.

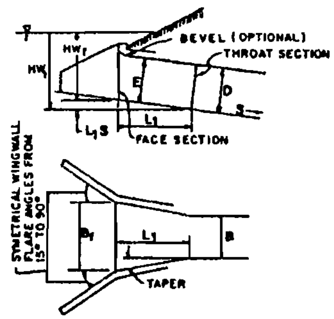
PROJECT: EXAMPLE PROBLEM NO. 3
CHAPTER IV, HDS No. 5

STATION: 5 + 00
SHEET 3 OF 4

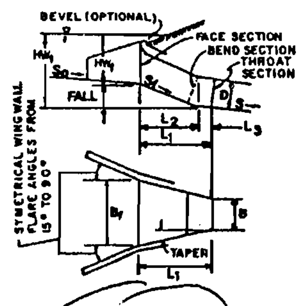
TAPERED INLET DESIGN FORM

DESIGNER/DATE: LAC / 7/16
REVIEWER/DATE: BLB / 7/17

DESIGN DATA:
Q 100 - 400 cfs ; EL_{hi} 195.0 ft
EL. THROAT INVERT 185.5 ft
EL. STREAM BED AT FACE 187.5 ft
FALL 2.0 ft TAPER 4 :1 (4:1 TO 6:1)
STREAM SLOPE, So = 0.05 ft/ft
SLOPE OF BARREL, S = 0.044 ft/ft
S_f 2 :1 (2:1 TO 3:1)
BARREL SHAPE AND MATERIAL: BOX-CONC.
N = 1, B = 5FT. D = 5FT.
INLET EDGE DESCRIPTION TAPERED INLET THROAT



SIDE-TAPERED



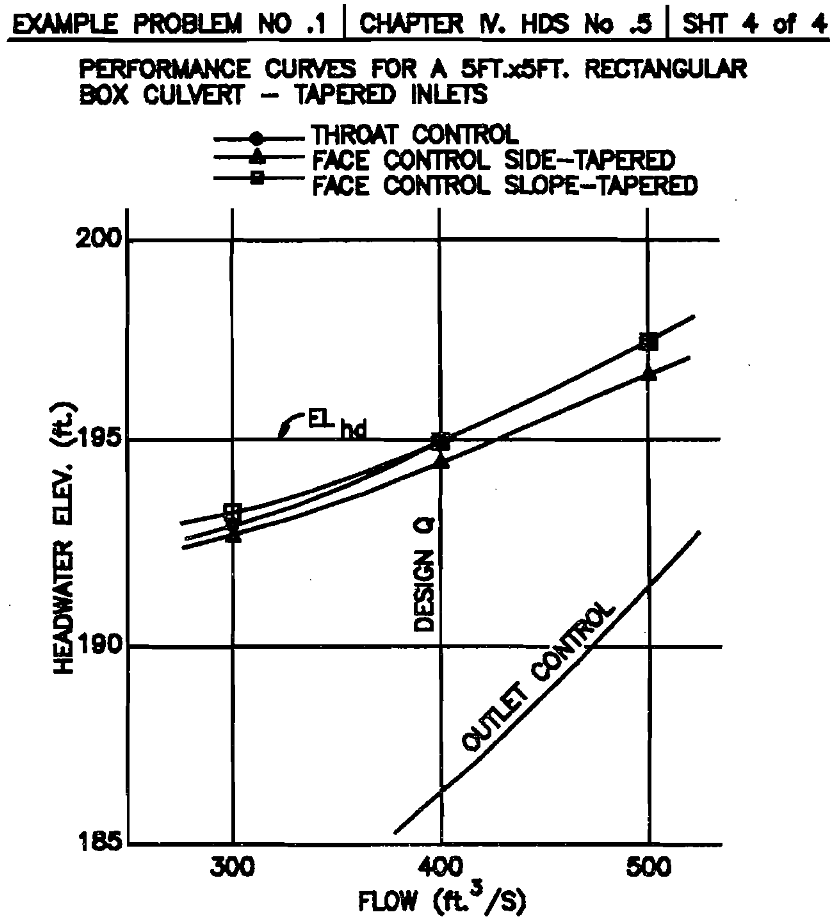
SLOPE-TAPERED

COMMENTS
USE 26° TO 90° WINGWALLS FLARES WITH NO BEVELS ON A SLOPE TAPERED INLET.

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f/E (3)	Q/B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)
400	195.0	185.5	187.5	7.5	1.5	50	8.0	8.0	2.5	4.0	3.5	—	4.33	6.5			
300	193.2			5.7	1.14	37.5	←	8.0	FACE CONTROL PERFORMANCE DATA								
500	197.5			10.0	2.0	62.5	←	8.0									

(1) SIDE-TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE-TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE
(2) $HW_f = EL_{hi}$ - EL. FACE INVERT
(3) $1.1D \geq E \geq D$
(4) FROM DESIGN CHARTS
(5) MIN. $B_f = Q/(Q/B_f)$
(6) MIN. $L_3 = 0.5NB$
(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f
(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_3$
(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_2$
(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$
(11) SIDE-TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$
SLOPE-TAPERED: $L_1 = L_2 + L_3$
(12) $HW_c = EL_{hi}$ - EL. CREST INVERT
(13) MIN. $W = 0.35Q/HW_c^{1.5}$

SELECTED DESIGN
B_f 8.0
L_1 6.5
L_2 4.0
L_3 2.5
BEVELS ANGLE 0°
b = 0 in; d = 0 in
TAPER 4.33 :1
S_f = 2.0 :1



(Example 3 Continued)

Use a slope-tapered inlet with a vertical face since it is the smallest inlet in this case. Note that since the FALL is less than D/2, a slope-tapered inlet with a mitered face cannot be used at this site.

<u>Dimensions:</u>

B	=	5 ft	D	=	5 ft	B_f	=	8 ft
TAPER	=	4.33H:1V	S_f	=	2H:1V			
L_1	=	6.5 ft	L_2	=	4.0 ft	L_3	=	2.5 ft

Entrance: 26° to 90° wingwalls with no bevels

7.7 BRIDGE HYDRAULIC DESIGN

Bridges are required across nearly all open urban channels sooner or later and therefore, sizing the bridge openings is of paramount importance. When large culverts are used in lieu of bridges, the design approach often differs. Open channels with improperly designed bridges will either have excessive scour, or deposition, or not be able to carry the design flow.

7.7.1 Design Approach

The method of planning for bridge openings must include water surface profile and hydraulic gradient analysis of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined.

Scour is the result of the erosive action of running water, excavating, and carrying away material from the bed of banks of streams. Local scour involves the removal of material from the channel bed or bank and is restricted to a minor part of the width of a channel. This scour occurs around piers, abutments, spurs, and embankments and is caused by the acceleration of the flow and the development of vortex systems induced by the obstruction to the flow. It is beyond the scope of this Manual to fully outline the procedure. The FHWA publication, Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18)[7] outlines the procedure to evaluate scour at bridges.

7.7.2 Bridge Opening Freeboard

The distance between the design flow water surface and the bottom of the bridge deck will vary from case to case. However, the debris which may be expected must receive full consideration in setting the freeboard. Freeboard may vary from several feet to minus several feet. There are no general hard and fast rules. Each case must be studied separately.

In certain unusual cases, the designer might properly choose to intentionally cause ponding upstream from bridges to reduce downstream peaks during the storms creating flow greater than the major design runoff. This is sometimes done when downstream areas are highly developed, and the upstream areas have adjacent open space and park areas next to the channel. In these cases, there normally would be no freeboard allowed between the design water surface and the bridge deck bottom.

7.8 BRIDGE HYDRAULIC ANALYSIS

Bridge waterway design usually requires determination of the amount and extent of backwater caused by an encroachment in the flood plain. The bridge-affected water surface elevations will be higher than water surface elevations for unconstricted flow (natural profile). The standard step method can be used to compute the water surface profile on the basis of energy losses. The computations begin at one end of the study reach and proceed cross section by cross section to the other end of the river. The standard step method involves the solution of the dynamic equation of gradually varied flow. This method is discussed in Open Channel Hydraulics by

Chow[2]. At bridge crossings where the flow hydraulics is more complicated, momentum and other equations may be used to compute the water surface elevation changes.

Many computer programs are used for the computation of water surface profiles. Two widely used programs are HEC-2[11] and WSPRO[9]. HEC-2, Water Surface Profiles was developed by the U.S. Army Corps of Engineers. Water Surface Profile Computations (WSPRO) was developed by the U.S. Geological Survey for the Federal Highway Administration. The use of HEC-2 or WSPRO is recommended for bridge hydraulic analysis.

7.8.1 HEC-2 Model

HEC-2, Water Surface Profiles computer program used to analyze backwater effects from bridge waterways. The methodology incorporated into HEC-2, is based on several simplifying assumptions, but the model produces satisfactory results in many applications. The assumptions are as follows: 1) steady flow; 2) gradually varied flow; 3) one-dimensional flow with correction for horizontal velocity distribution; 4) small channel slope; 5) friction slope (averaged) constant between two adjacent cross sections, and; 6) rigid boundary conditions.

HEC-2 is intended for calculating water surface profiles for steady gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs, and structures in the flood plain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's Equation. HEC-2 is also designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the effects of channel improvements and levees on water surface profiles.

HEC-2 computes energy losses caused by structures such as bridges and culverts in two parts. One part consists of the losses that occur in reaches immediately upstream and downstream from the bridge where contraction and expansion of the flow is taking place. The second part consists of losses at the structure itself and is calculated with either the normal bridge method, special bridge method, or the special culvert option.

The normal bridge method handles the cross section at the bridge just as it would any river cross section with the exception that the area of the bridge below the water surface is subtracted from the total area and the wetted perimeter is increased where the water surface elevation exceeds the low chord. The normal bridge method is particularly applicable for bridges without piers, bridges under high submergence, and for low flow through oval and arch culverts. Whenever flow crosses critical depth in a structure, the special bridge method should be used. The normal bridge method is automatically used by the computer, even though data was prepared for the special bridge method, for bridges without piers and under low flow control.

The special bridge method can be used for any bridge, but should be used for bridges with piers where low flow controls, for pressure flow, and whenever flow passes through critical depth when going through the structure. The special bridge method computes losses through the structure for low flow, weir flow and pressure flow or for any combination of these.

The culvert option is a new feature in Version 4.5. The special culvert method is similar to the special bridge method, except that the Federal Highway Administration (FHWA) standard equations for culvert hydraulics are used to compute losses through the structure.

7.8.2 WSPRO Model

The FHWA contracted with the U.S. Geologic Survey to develop an improved water surface profile computation program. WSPRO is a digital model for water surface profile computations for open-channel flow and is compatible with conventional techniques used in existing step-backwater analysis models. WSPRO incorporates several desirable features from existing models. Profile computations for free-surface flow through bridges are based on relatively recent developments in bridge backwater analysis and recognize the influence of bridge geometry variations. Pressure flow situations (girders partially or fully inundated) are computed using existing Federal Highway Administration techniques. Embankment overtopping flows, in conjunction with either free-surface or pressure flow through the bridge, can be computed. WSPRO is also capable of computing profiles at stream crossings with multiple openings (including culverts).

Although specifically oriented towards hydraulic design of stream highway crossings, WSPRO is equally suitable for water surface profile computations unrelated to highway design.

7.9 LIST OF SYMBOLS

The following is a list of symbols used in Section 7 of this Manual, their corresponding units and a brief description of the symbol.

Symbol	Units	Description
A_b	sq ft	Area of bend section of slope-tapered inlets
A_f	sq ft	Area of inlet face section
A_t	sq ft	Area of inlet throat section
AHW EL	ft	Allowable headwater elevation at culvert entrance
B	ft	Width of culvert barrel or diameter of pipe culvert
b	in	Dimension of side bevel
B_b	ft	Width of bend section of slope-tapered inlets
B_f	ft	Width of face section of improved inlets
C_d		Discharge coefficient based on bend section control

C_f		Discharge coefficient based on face section control
C_t		Discharge coefficient based on throat section control
D	ft	Height of box culvert or diameter of pipe culvert
d	in	Dimension of top bevel
d_c	ft	Critical depth of flow
E	ft	Height of side-tapered pipe-culvert face section, excluding bevel dimension
EL_{hi}		Elevation of the inlet control headwater of improved inlet
EL_i		Elevation of the inlet invert of improved inlets
EL_{sf}		Elevation of the stream bed at the face of improved inlets
EL_t		Elevation of the throat invert of improved inlets
FALL	ft	Approximate depression of control section below the streambed
g	ft/sec^2	Acceleration of gravity = 32.2
H	ft	Head or energy required to pass a given quantity of water through a culvert flowing in outlet control
H_b	ft	Depth of pool, or head, above the bend section invert
H_c	ft	Depth of pool, or head, above the crest
H_f	ft	Depth of pool, or head, above the face section invert
H_t	ft	Depth of pool, or head, above the throat section invert
HG Line	ft	Hydraulic grade line
HW	ft	Headwater elevation; subscript indicates control section (HW, as used in HDS 5, is a depth and is equivalent to H_f)
HW_c	ft	Headwater elevation required for flow to pass crest in crest control
HW_f	ft	Headwater elevation required for flow to pass face section in face control

HW_o	ft	Headwater elevation required for culvert to pass flow in outlet control
HW_t	ft	Headwater elevation required for flow to pass throat section in throat control
h_e	ft	Entrance head loss
h_f	ft	Friction head loss
h_o	ft	Elevation of equivalent hydraulic grade line referenced to the outlet invert
k_e		Entrance energy loss coefficient
k_b		A dimensionless effective pressure term for bend section control
k_t		A dimensionless effective pressure term for inlet throat control
L_a	ft	Approximate total length of culvert, including inlet face section control
L_1, L_2, L_3, L_4	ft	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning's roughness coefficient
P	ft	Length of depression
Q	cfs	Volume rate of flow
R	ft	Hydraulic radius = Area/Wetted Perimeter
S	ft/ft	Slope of culvert barrel
S_c	ft/ft	Slope of natural channel producing critical discharge
S_e	ft/ft	Slope of embankment
S_f	ft/ft	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)
S_n	ft/ft	Friction slope
S_o	ft/ft	Slope of natural channel
T	ft	Depth of the depression

Taper	ft/ft	Sidewall flare angle (also expressed as the cotangent of the flare angle)
TW	ft	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft/sec	Mean velocity of flow
V_c	ft/sec	Critical velocity
W	ft	Width of weir crest for slope-tapered inlet with mitered face
W	ft	Top width of depression
y	ft	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face

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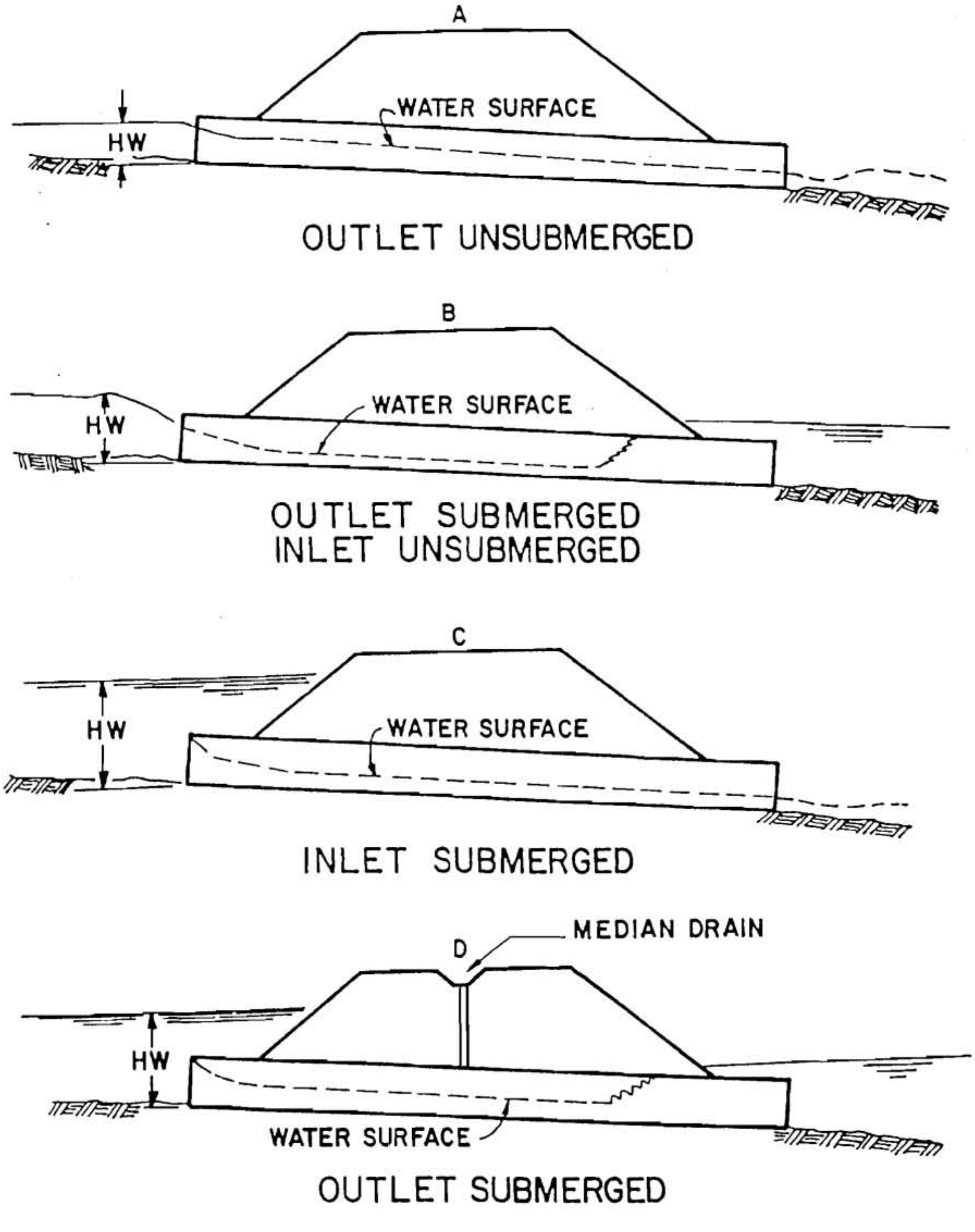
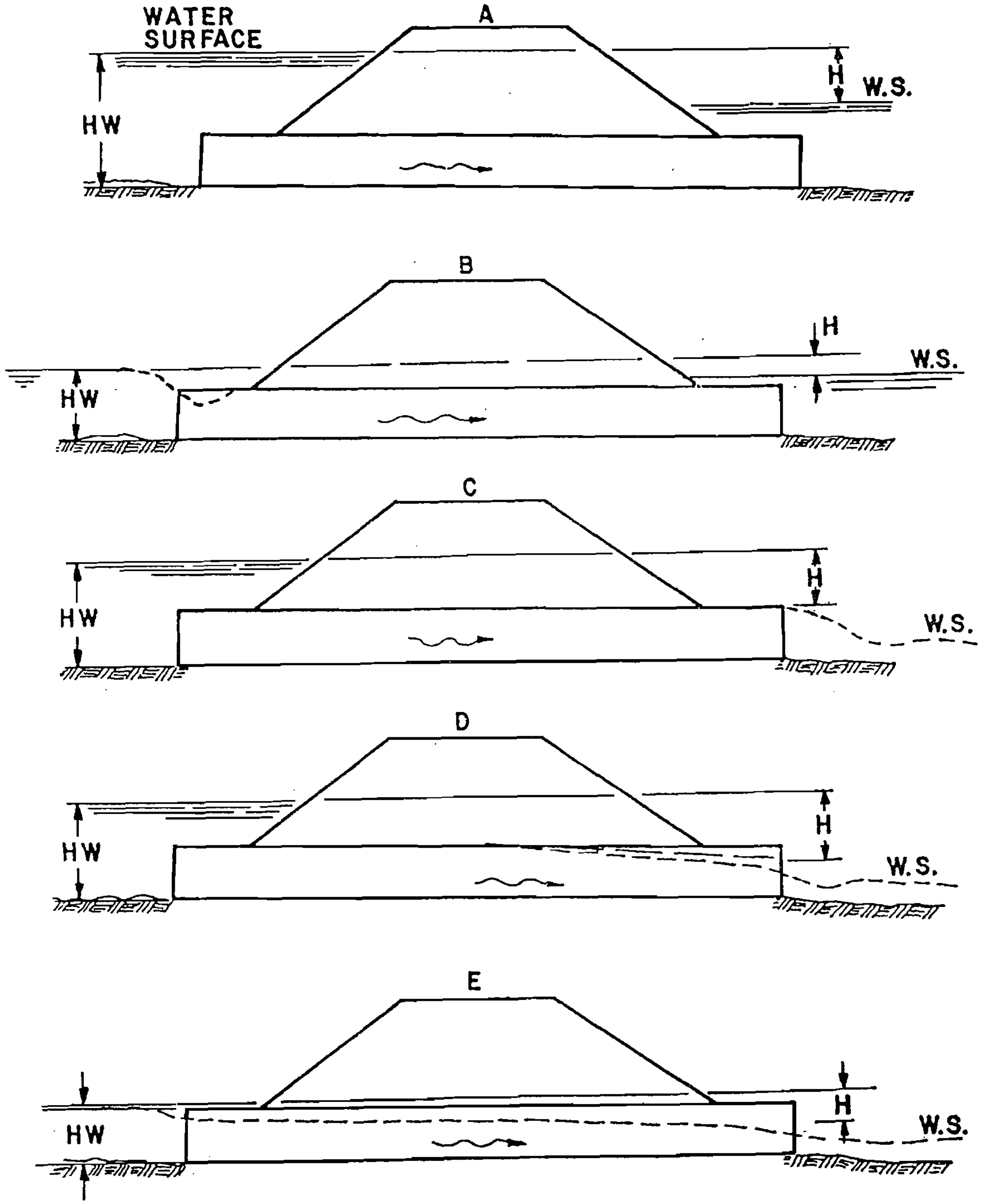


Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Types of Inlet Control

Date NOV 1992

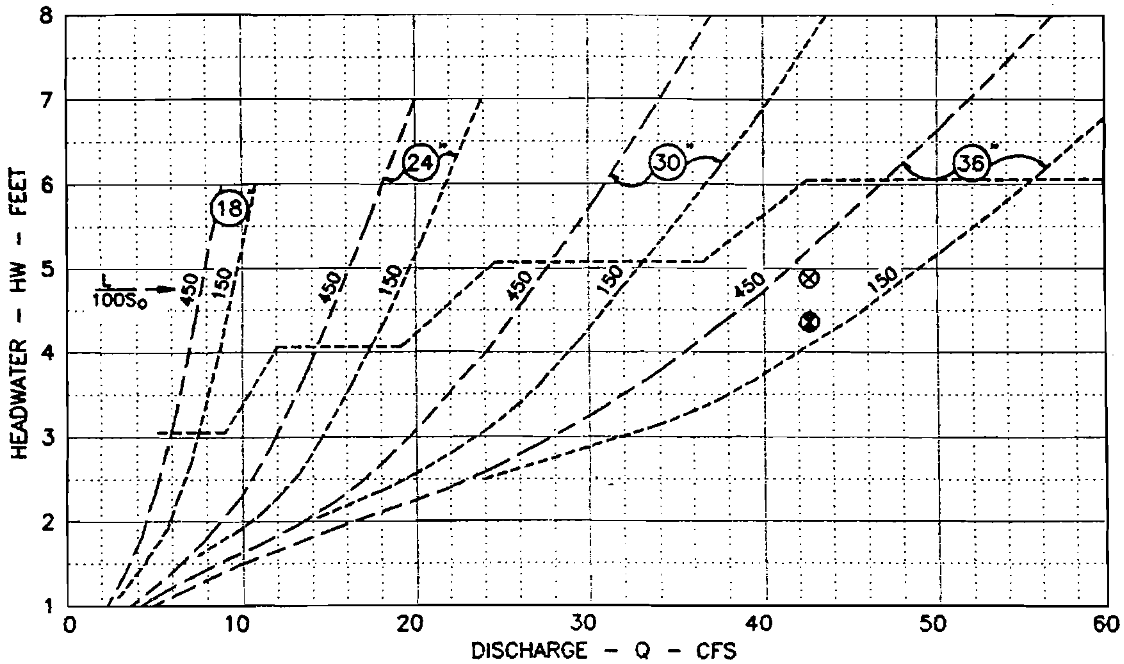
Figure 7-1

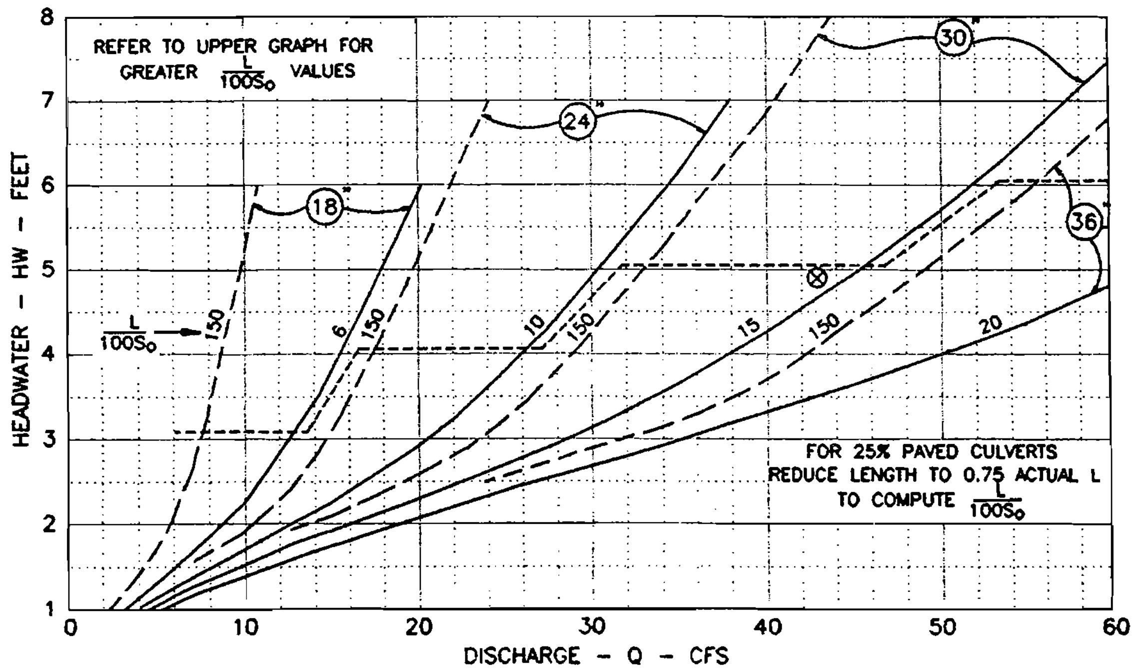


Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

Types of Outlet Control

Date	NOV 1992
Figure	7-2





EXAMPLE

⊗ GIVEN
43 CFS, AHW=4.9 FT
L=72 FT, S_o=0.003

⊗ SELECT 36" UNPAVED
HW=4.4 FT

Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

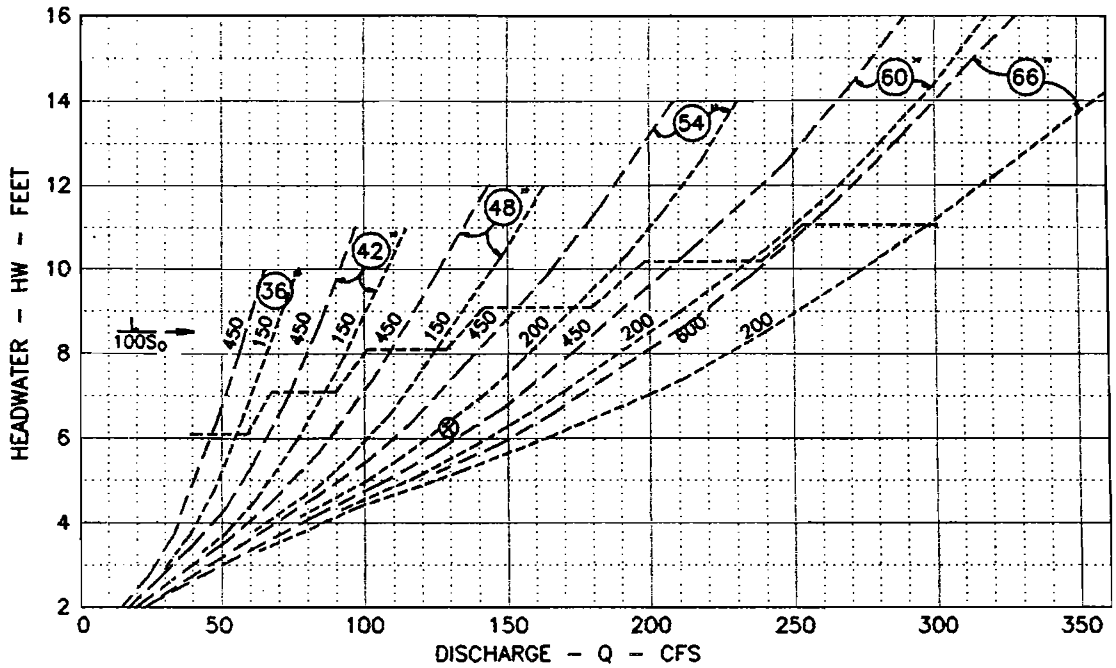
CITY OF AMARILLO

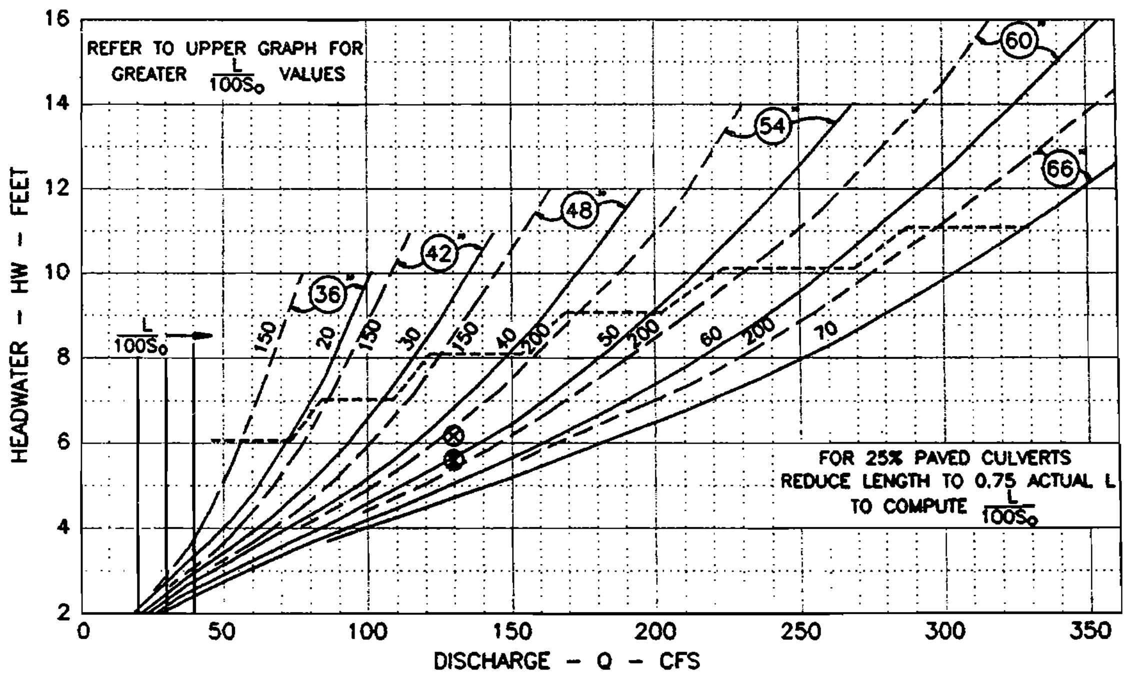
Culvert Capacity Standard Circular CMP, Headwall Entrance 18" to 36"

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 7-3





EXAMPLE

⊗ GIVEN
130 CFS, AHW=6.2 FT
L=120 FT, S_o=0.025

⊗ SELECT 54" UNPAVED
HW=5.6 FT

Source: *Federal Highway Administration, HEC No. 10,* *Capacity Charts for the Hydraulic Design of Highway Culverts*.

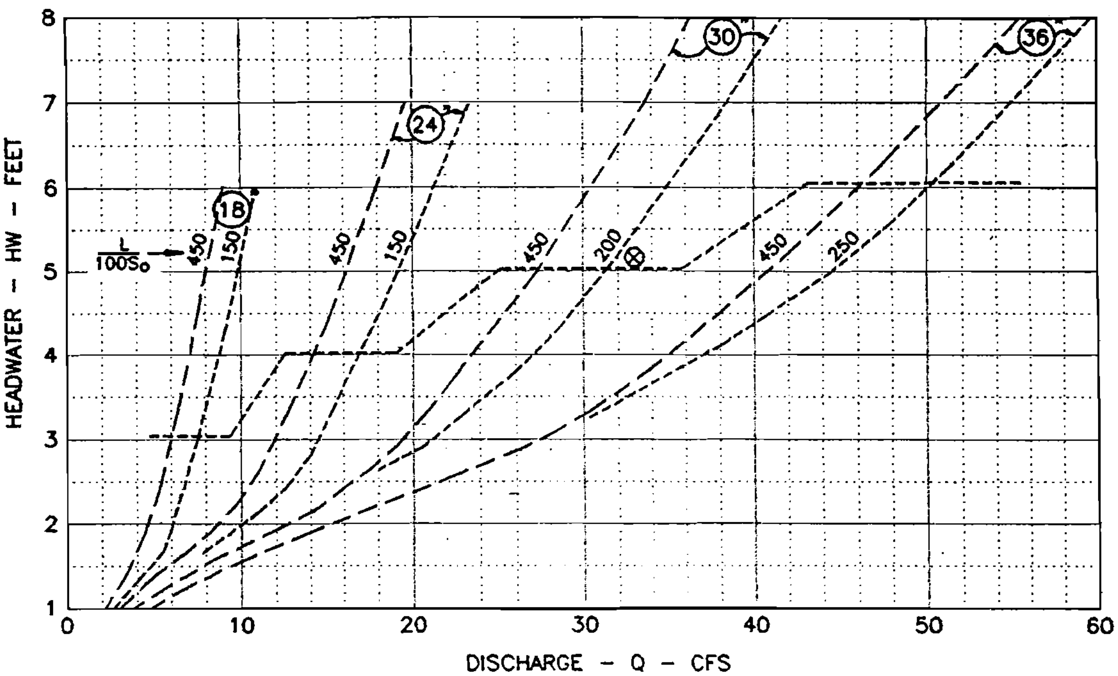
CITY OF AMARILLO

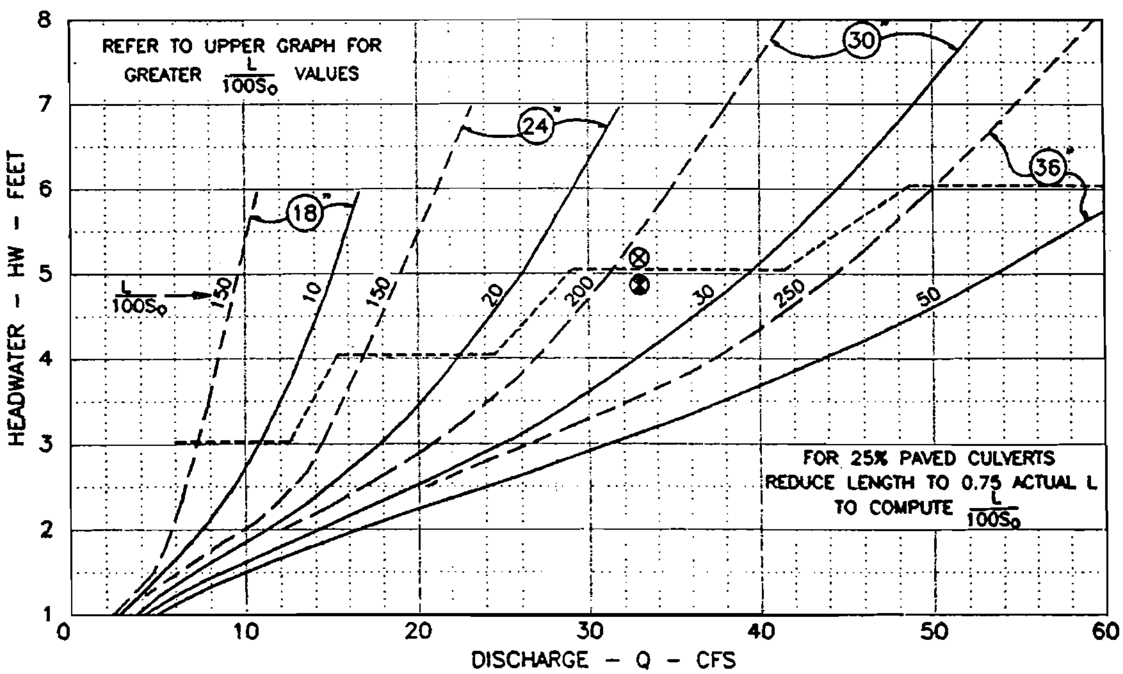
Culvert Capacity Standard Circular CMP, Headwall Entrance 36" to 66"

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
7-4





EXAMPLE

⊗ GIVEN
43 CFS, AHW=5.2 FT
L=70 FT, S_o=0.005

● SELECT 30" UNPAVED
HW=4.9 FT

Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

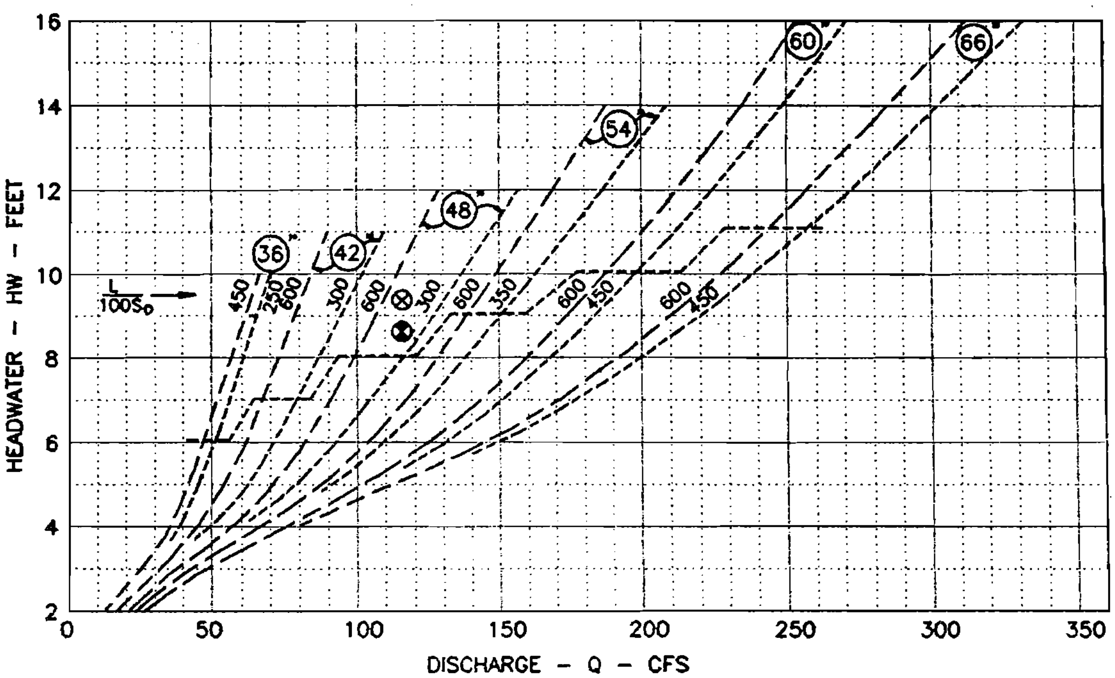
Culvert Capacity Standard Circular CMP, Projecting Entrance 18" to 36"

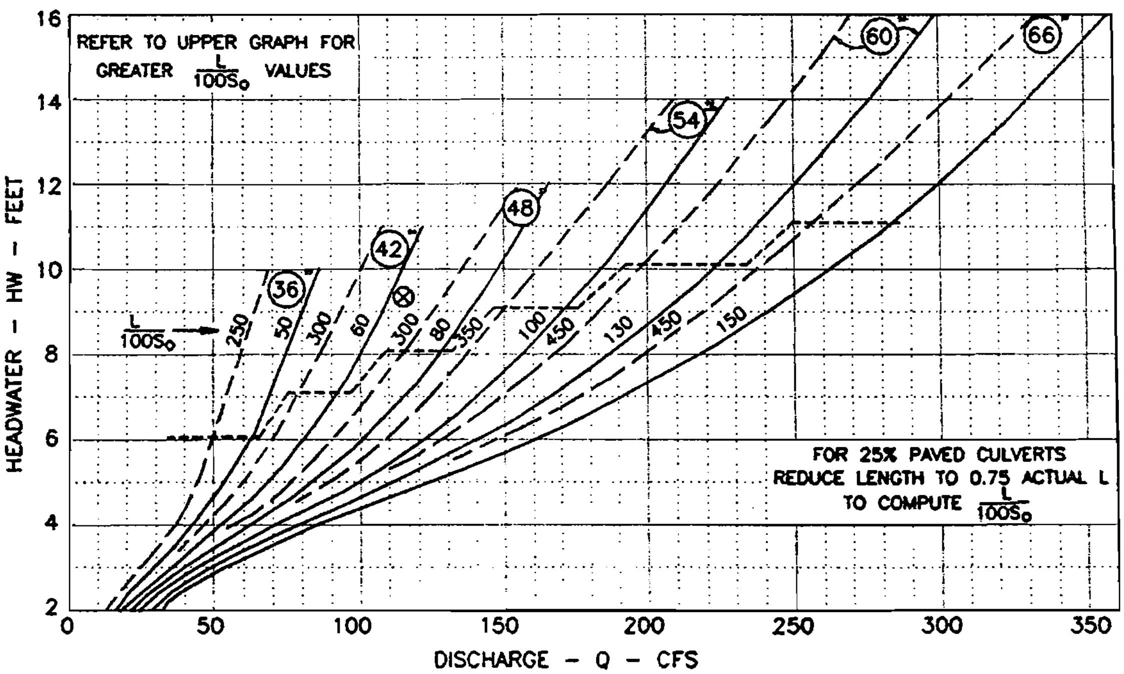
Storm Water Management Criteria Manual

CITY OF AMARILLO

Date NOV 1992

Figure 7-5





EXAMPLE

⊗ GIVEN
115 CFS, AHW=9.4 FT
L=135 FT, S_o=0.0034

⊗ SELECT 48" UNPAVED
HW=8.6 FT

Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

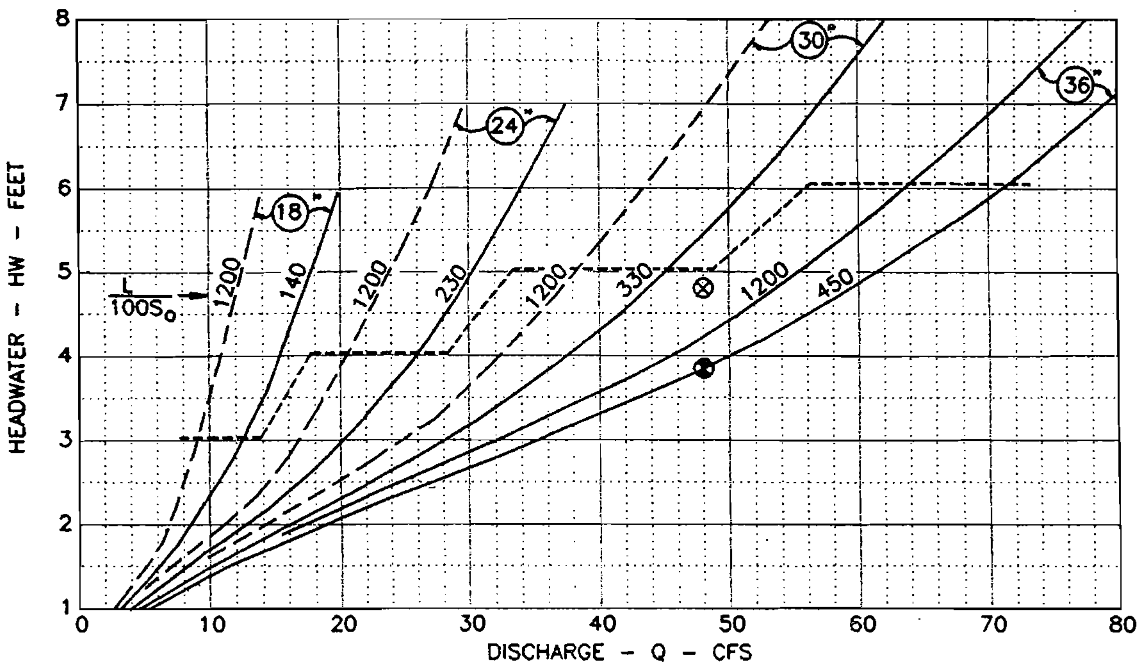
CITY OF AMARILLO

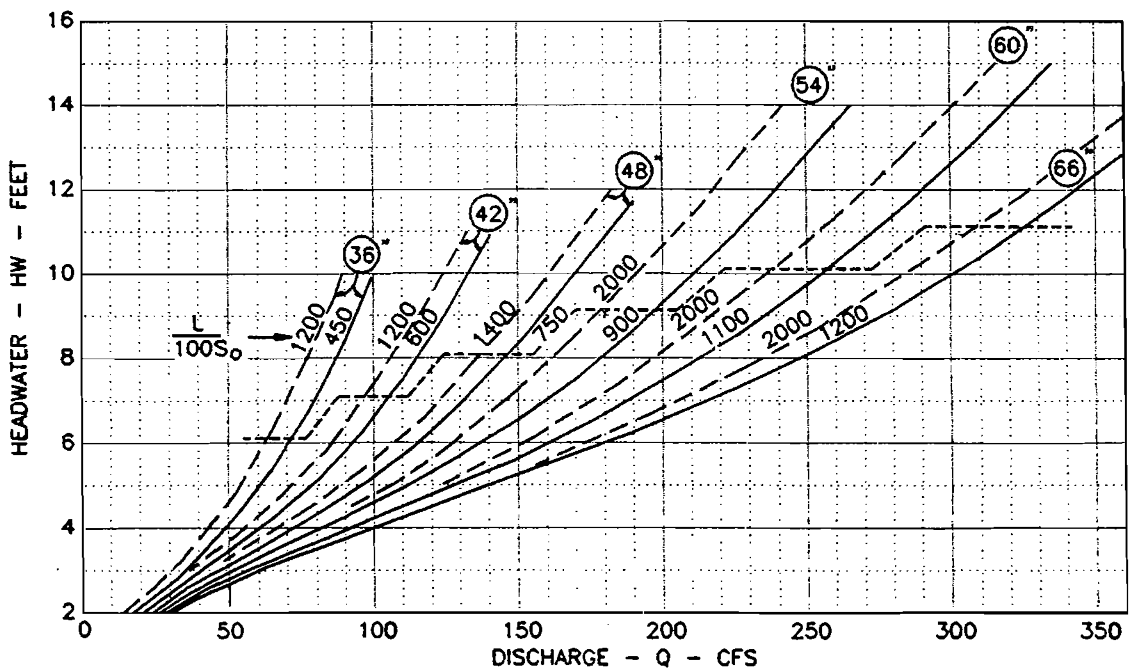
Culvert Capacity Standard Circular CMP, Projecting Entrance 36" to 66"

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
7-6





EXAMPLE

⊗ GIVEN
48 CFS, AHW=4.8 FT
L=60 FT, S_o=0.003

SELECT 36"
HW=3.9 FT

Source: Federal Highway Administration, HEC No. 10, <u>Capacity Charts for the Hydraulic Design of Highway Culverts</u>.

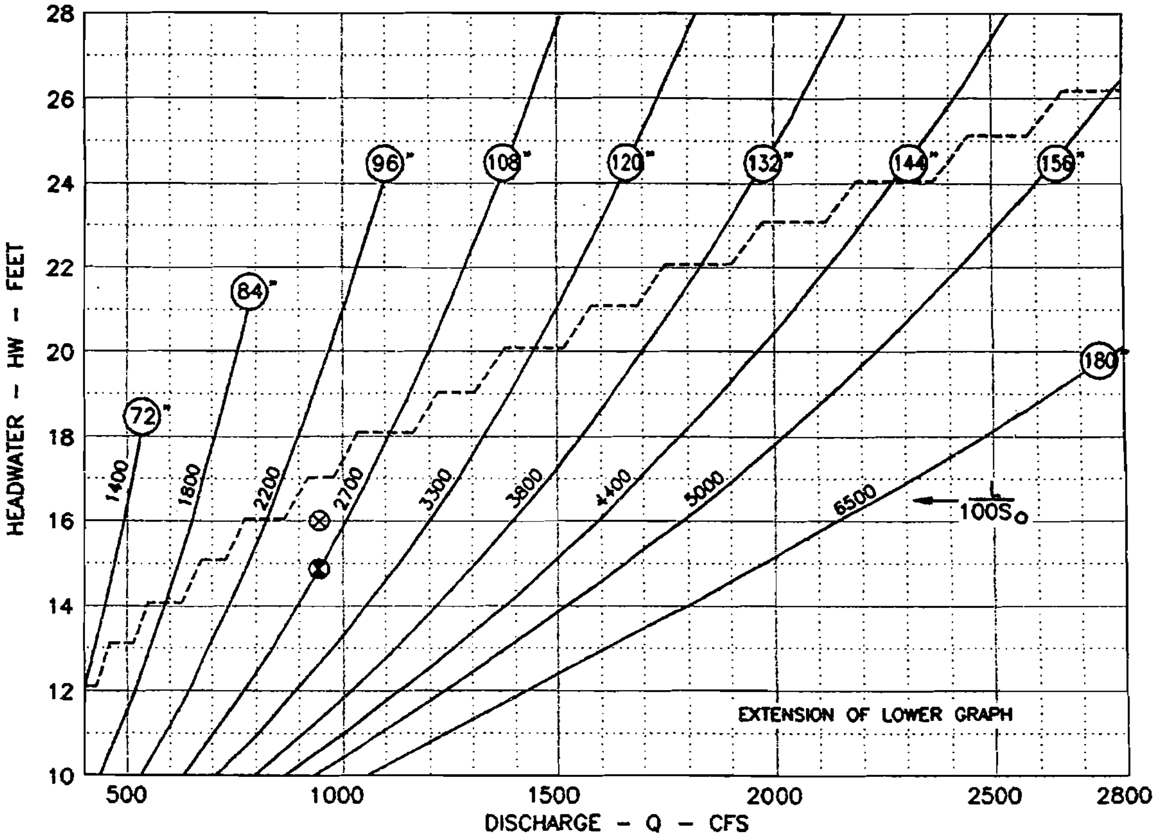
CITY OF AMARILLO

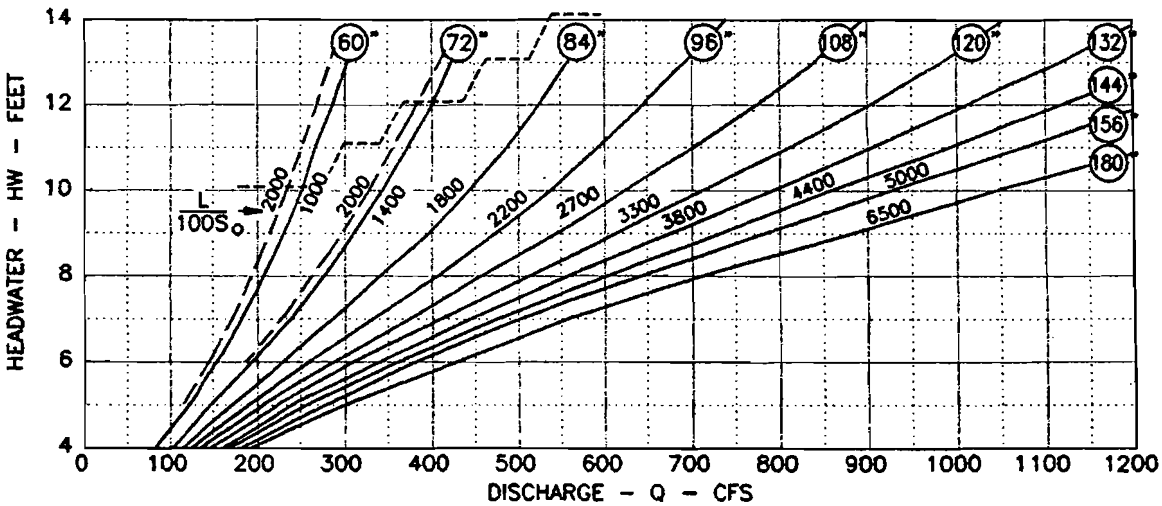
Culvert Capacity Standard Circular RCP, Square Entrance 18" to 66"

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
7-7





EXAMPLE

⊗ GIVEN
950 CFS, AHW=16 FT
L=480 FT, S_o=0.040

⊗ SELECT 108"
HW=15.0 FT

Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

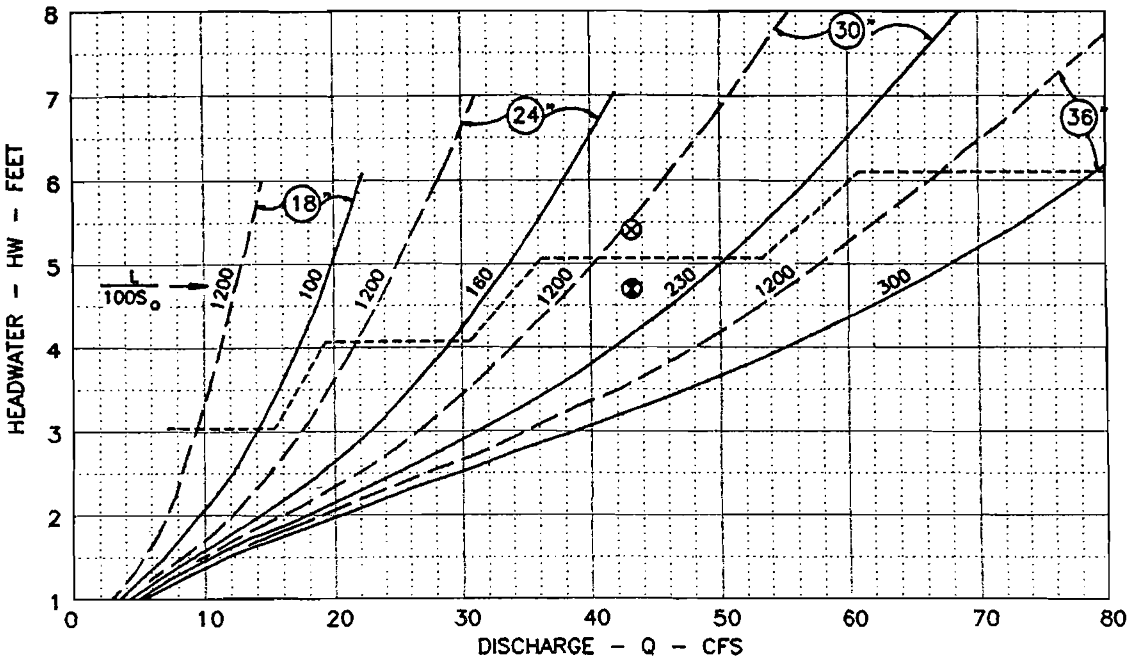
CITY OF AMARILLO

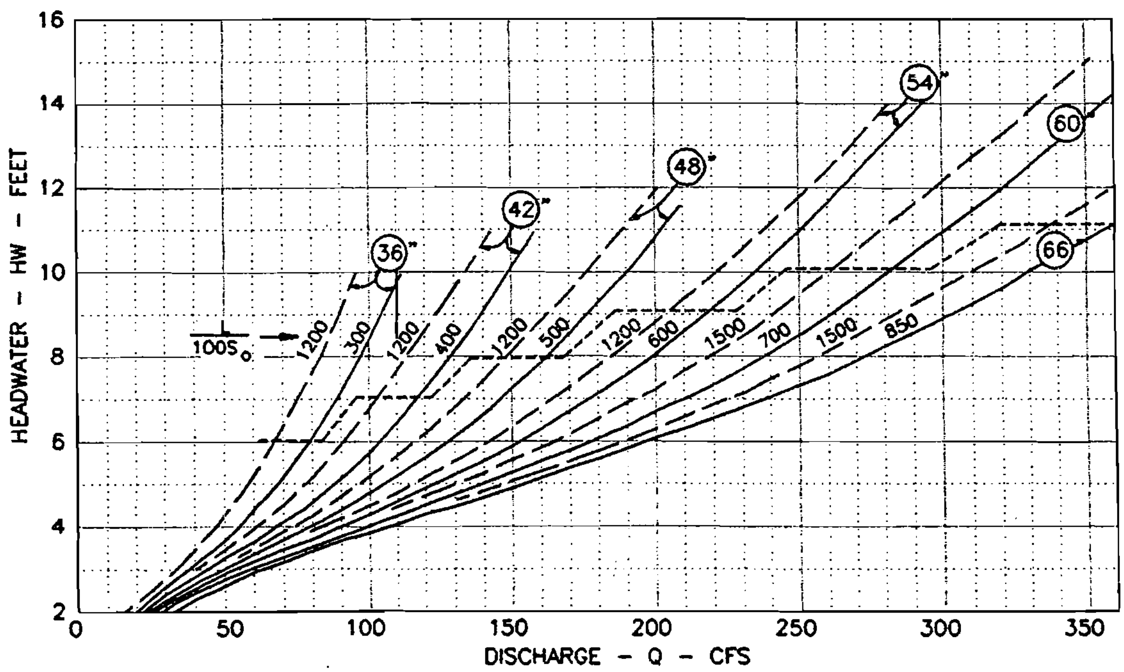
Culvert Capacity Standard Circular RCP, Square Edged Entrance 60" to 180"

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
7-8





EXAMPLE

⊗ GIVEN
43 CFS, AHW=5.4 FT
L=120 FT, S_o=0.002

SELECT 30"
HW=4.7 FT

Source: *Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.*

CITY OF AMARILLO

Culvert Capacity Circular RCP Groove-Edged Entrance 18" to 66"

Storm Water Management Criteria Manual

Date
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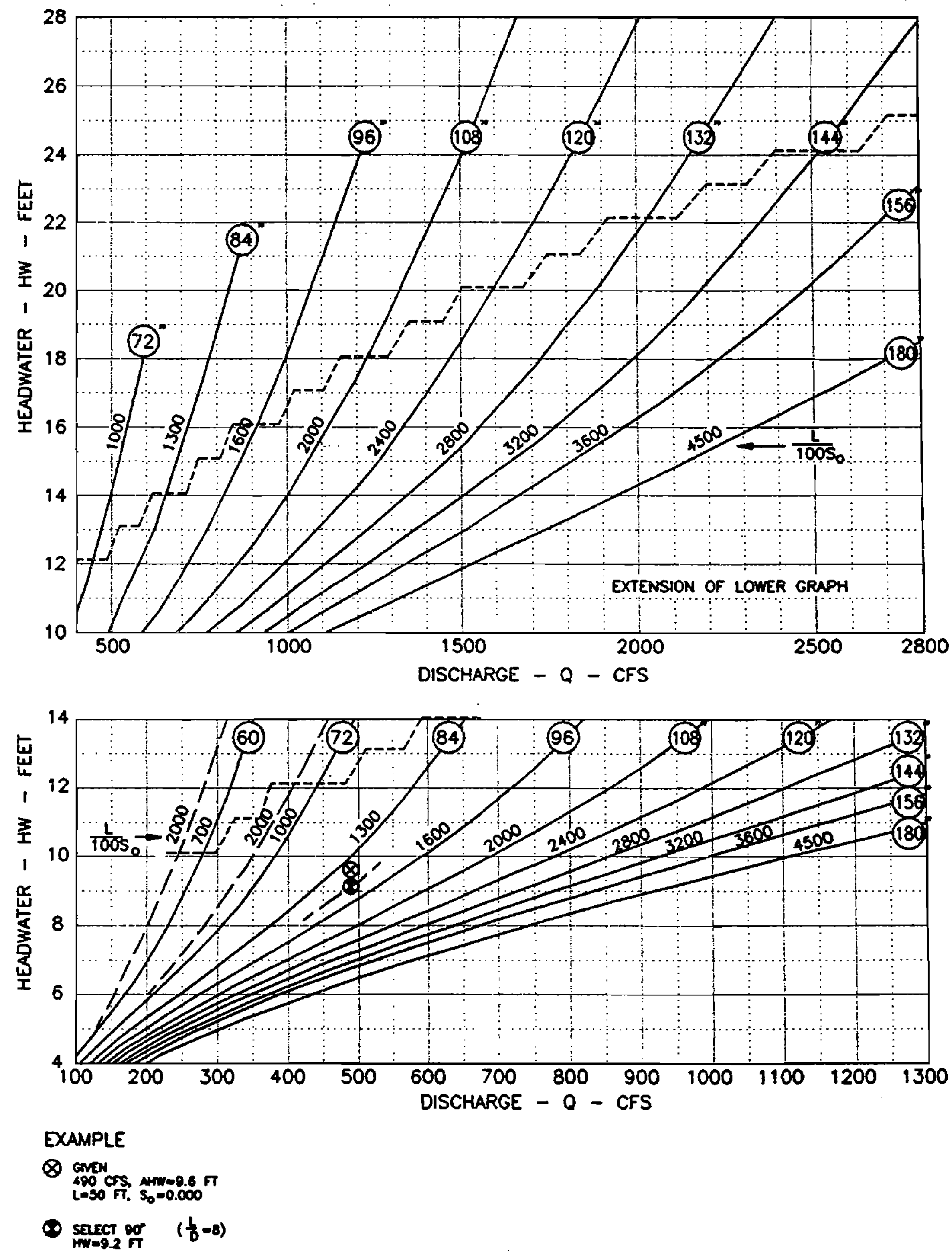
Figure
7-9



Source: Federal Highway Administration, HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts.

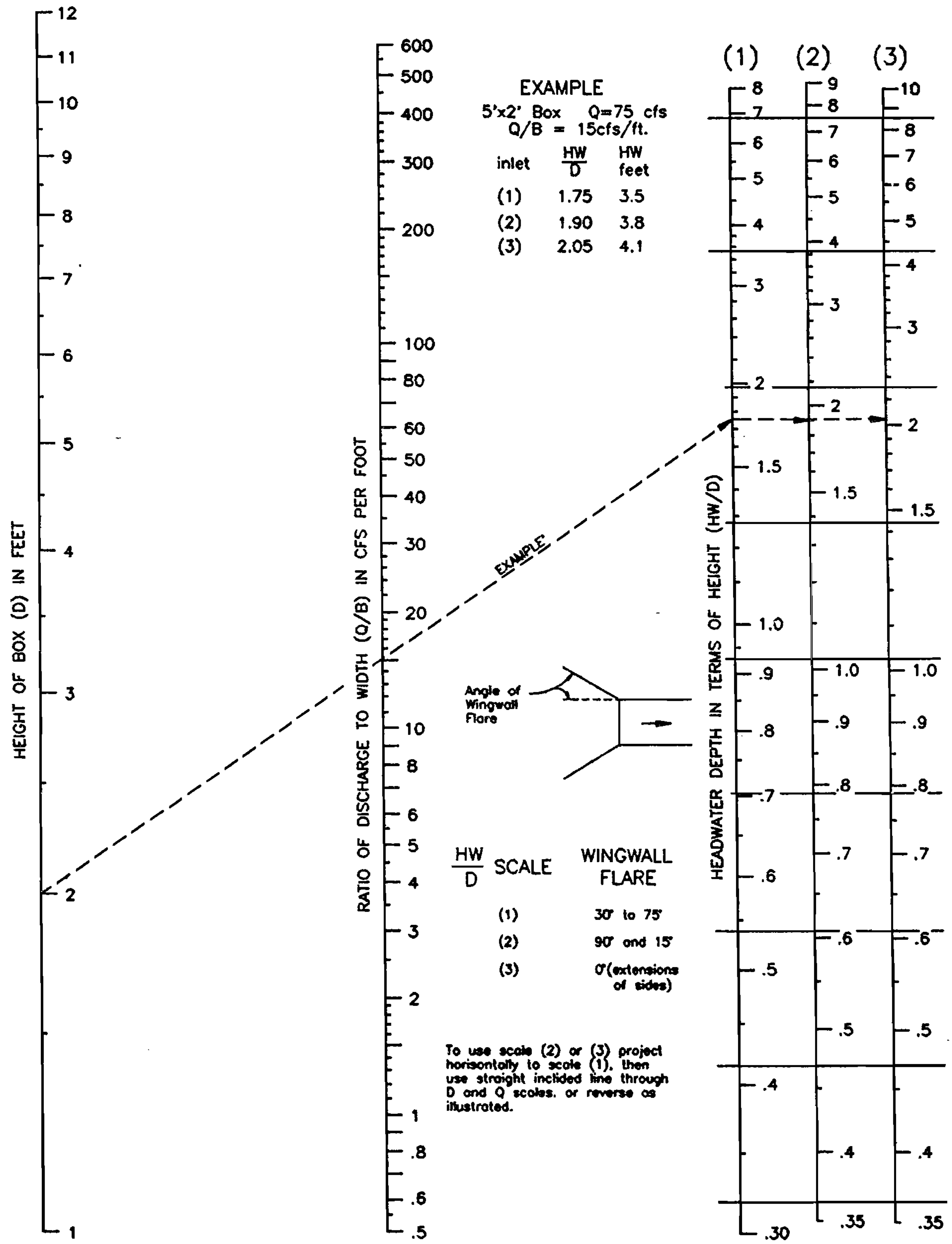
Culvert Capacity Circular RCP Groove-Edged Entrance 60" to 180"

Date
NOV 1992

Figure
7-10

CITY OF AMARILLO

Storm Water Management Criteria Manual

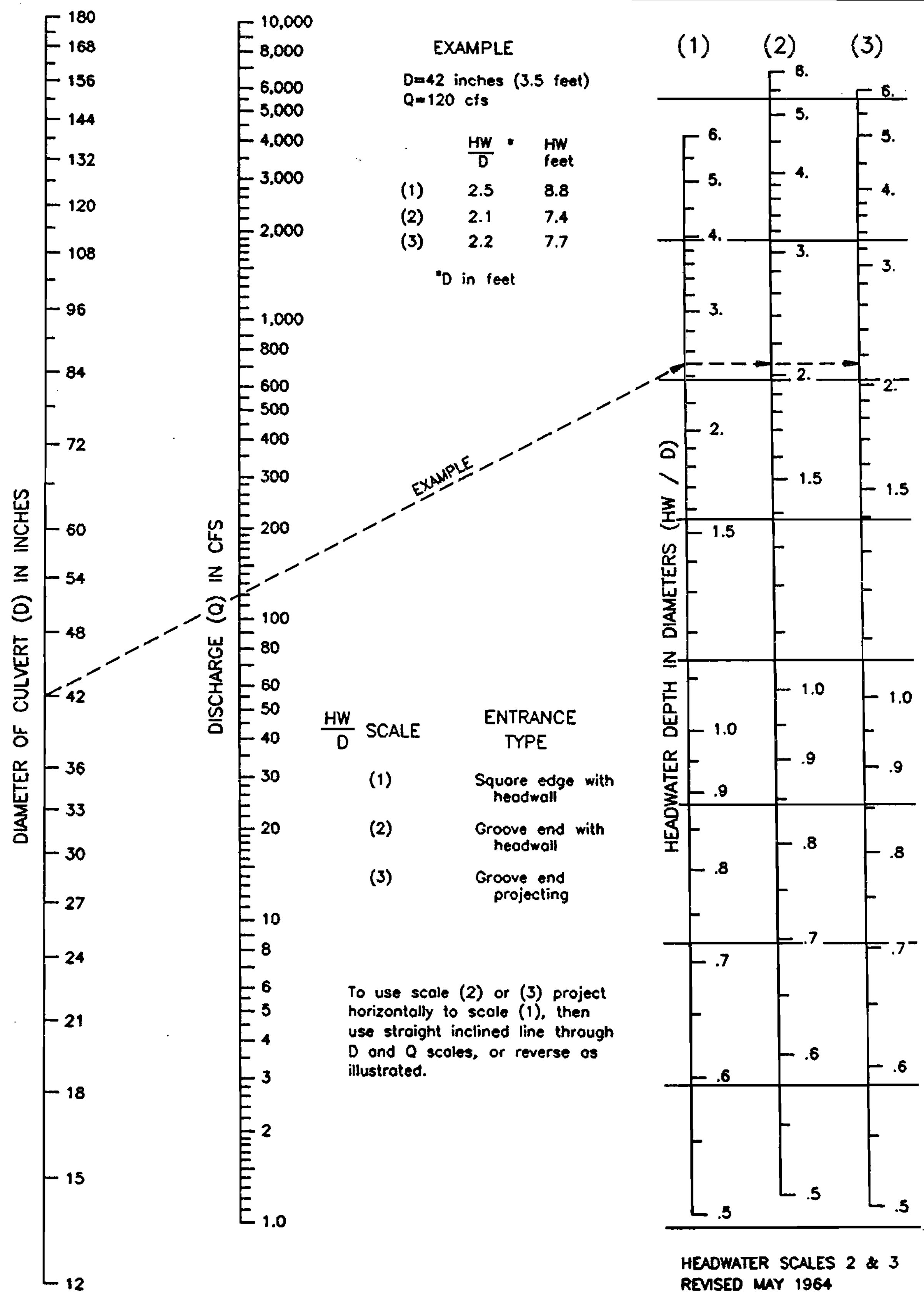


Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Headwater Depth for Box Culverts with Inlet Control

Date: NOV 1992

Figure 7-11



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

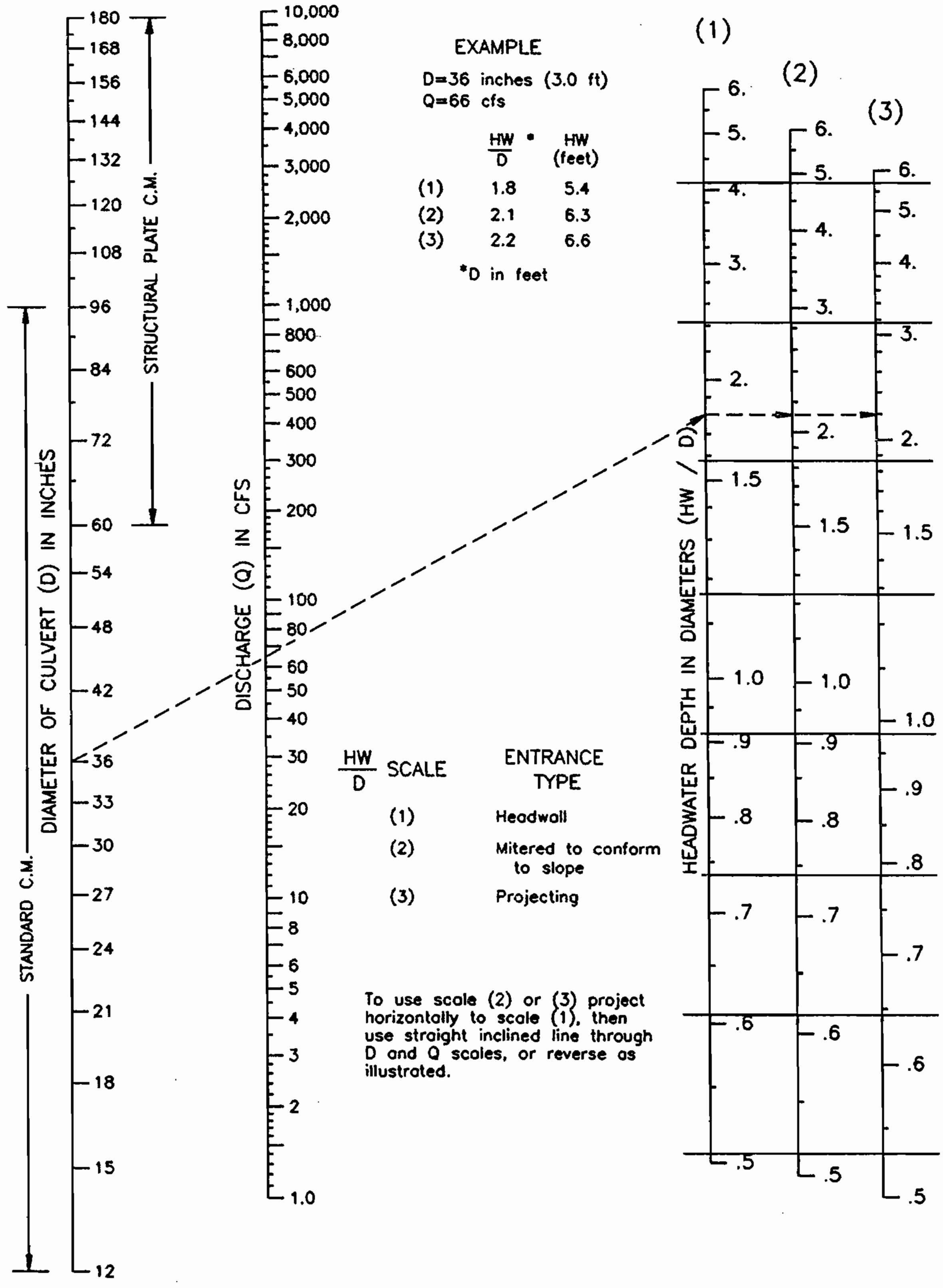
Headwater Depth for Concrete Pipe Culverts with Inlet Control

CITY OF AMARILLO

Storm Water Management Criteria Manual

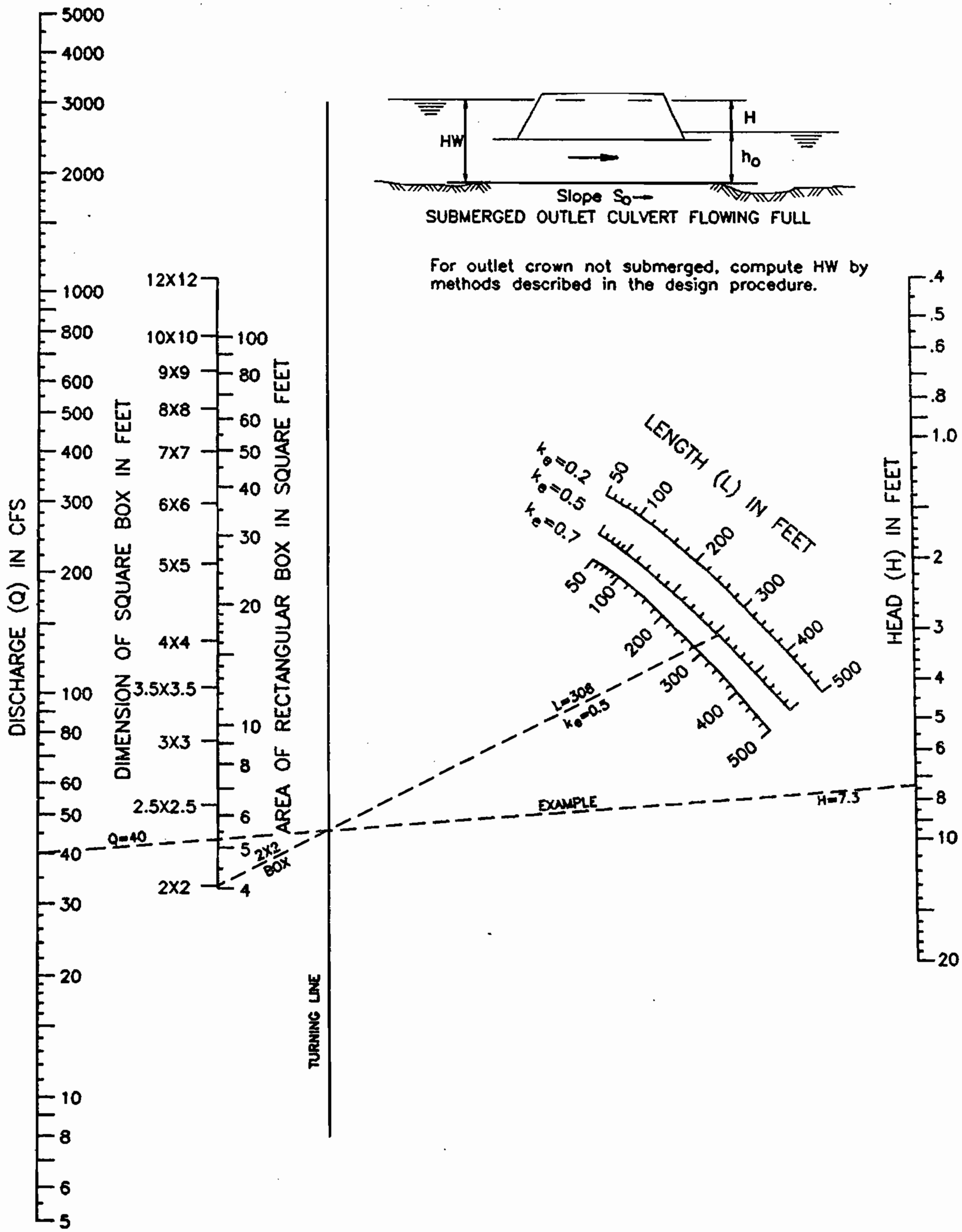
Date: NOV 1992

Figure 7-12



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO	**Headwater Depth for Corrugated Metal Pipe Culverts with Inlet Control**	Date NOV 1992
	Storm Water Management Criteria Manual	Figure 7-13



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

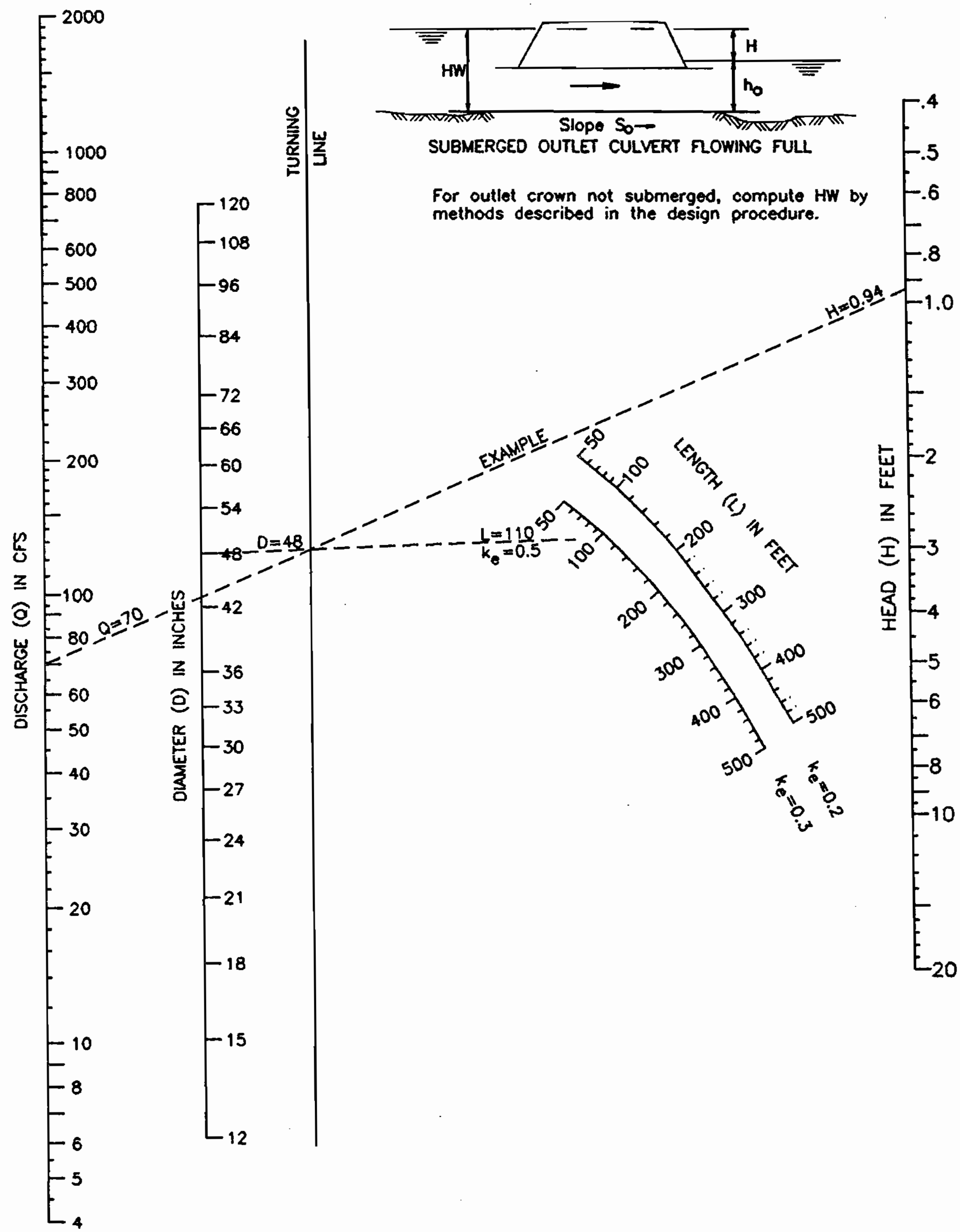
CITY OF AMARILLO

Head for Outlet Control Concrete Box Culverts Flowing Full n = 0.012

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 7-14



Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

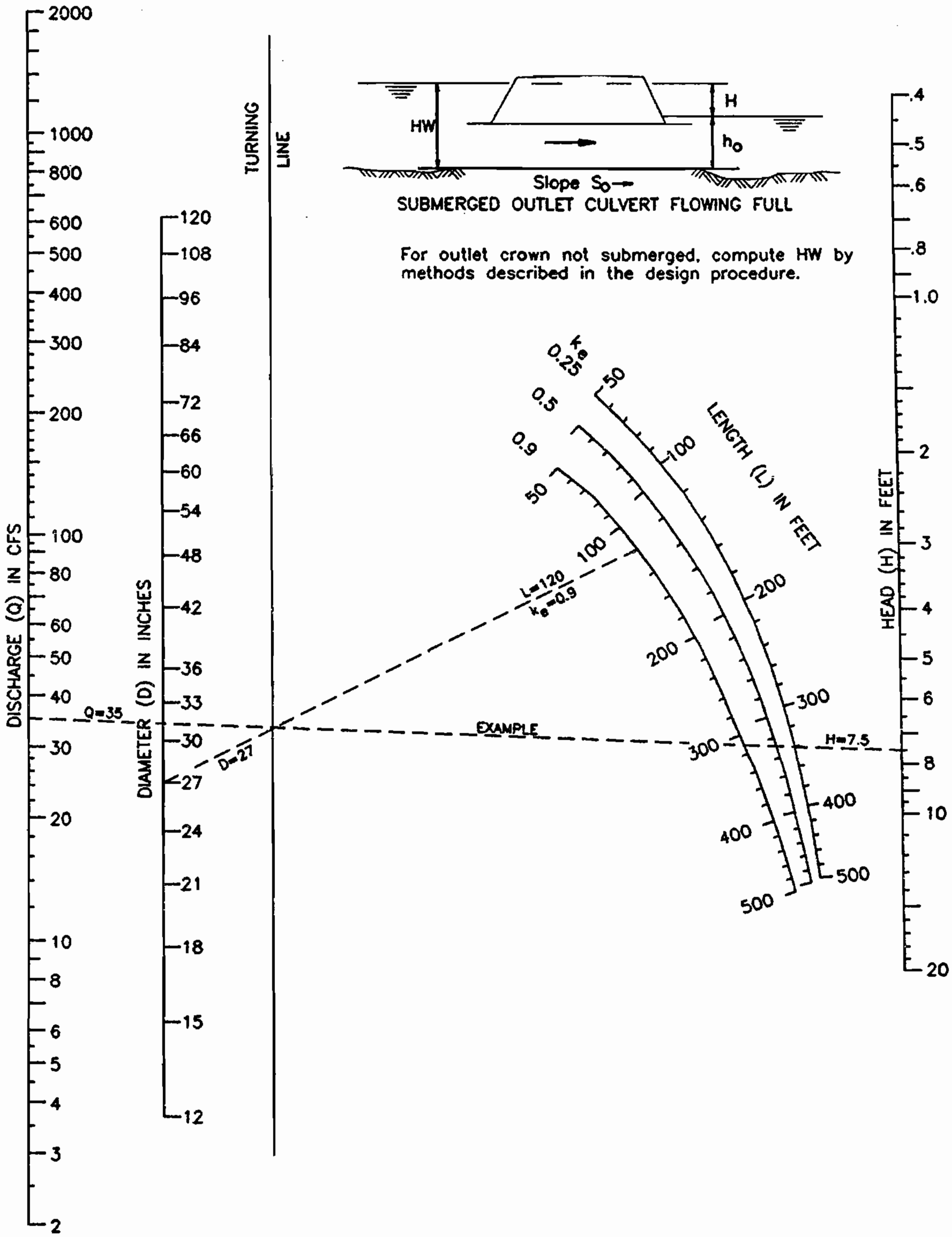
CITY OF AMARILLO

Head for Outlet Control Concrete Pipe Culverts Flowing Full n = 0.012

Storm Water Management Criteria Manual

Date NOV 1992

Figure 7-15



Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

CITY OF AMARILLO

Head for Outlet Control Standard Corrugated Metal Culverts Flowing Full n = 0.024

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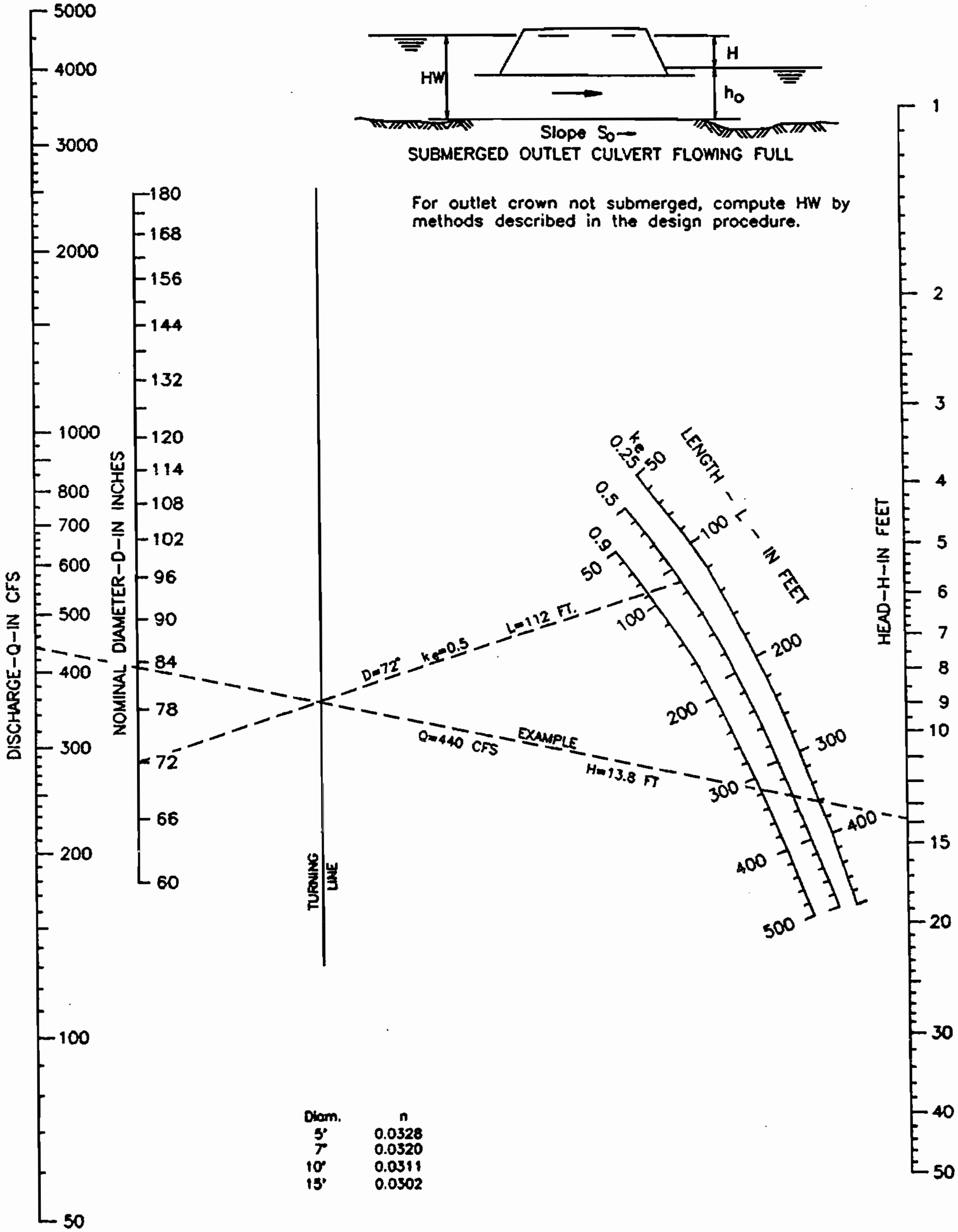
Date: NOV 1992

Figure: 7-16



Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Head for Outlet Control Structural Plate Corrugated Metal Pipe Culverts Flowing Full n = 0.0328 to 0.0302

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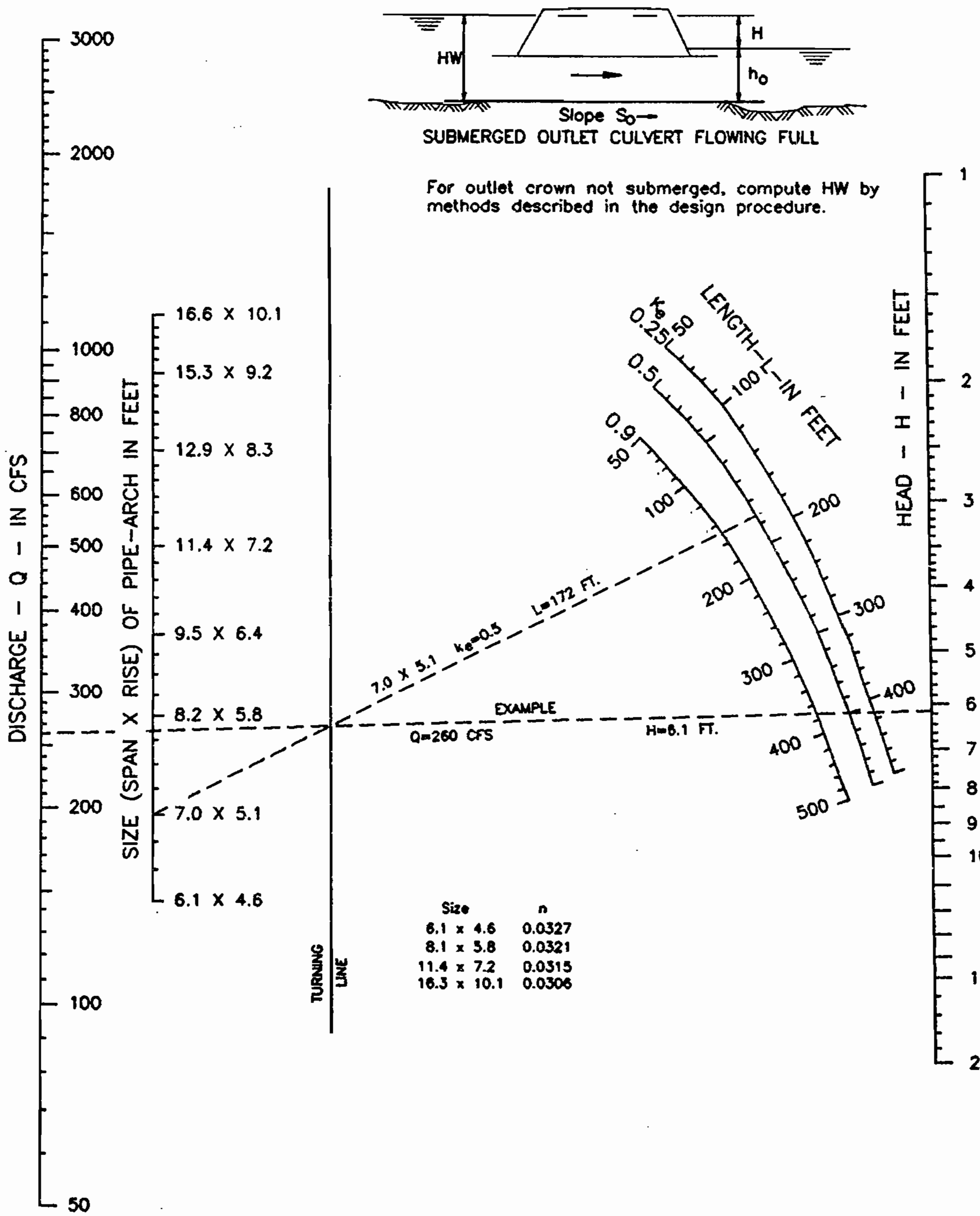
Date
NOV 1992

Figure
7-17



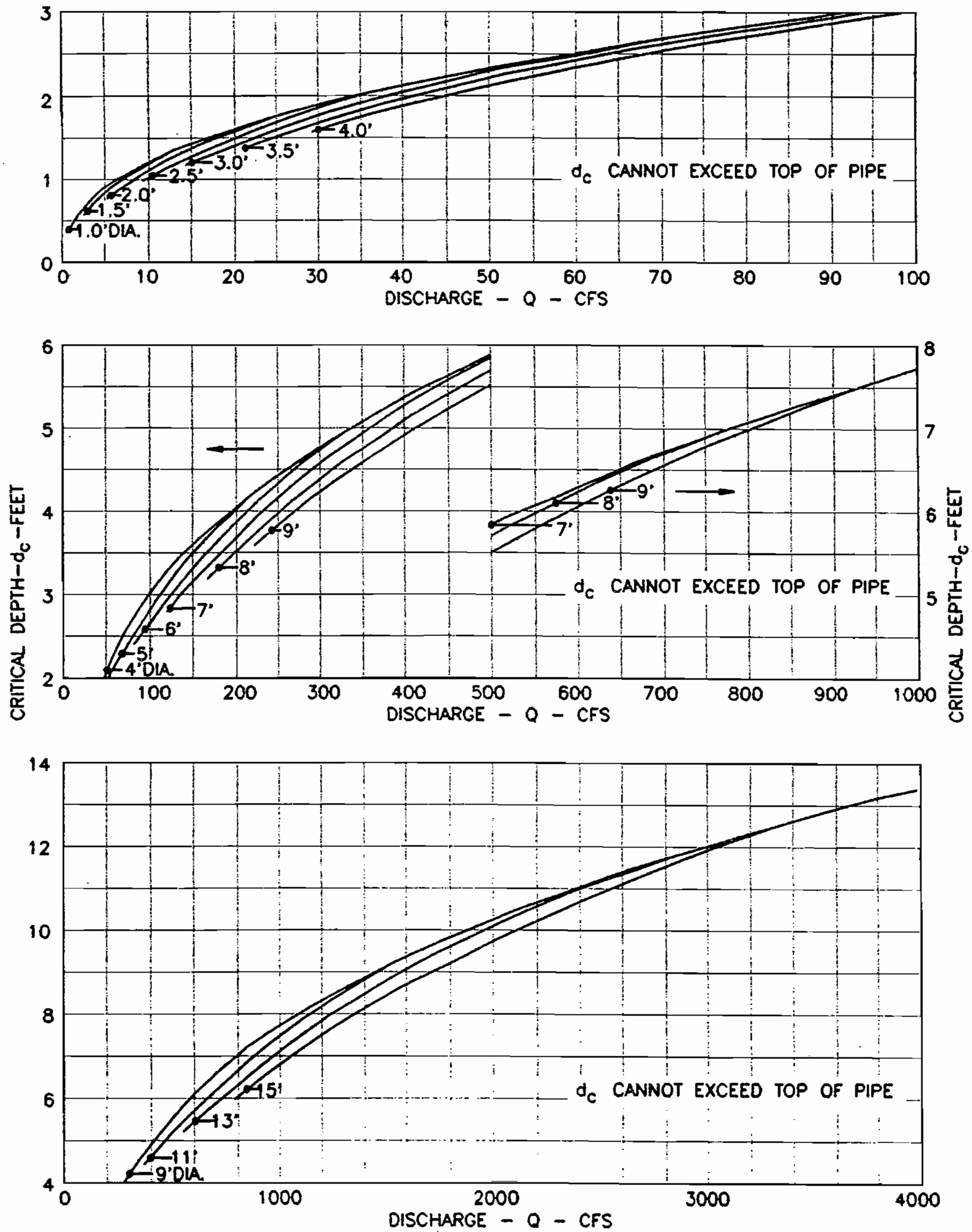
Size	n
6.1 x 4.6	0.0327
8.1 x 5.8	0.0321
11.4 x 7.2	0.0315
16.3 x 10.1	0.0306

Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Head for Outlet Control Structural Plate Corrugated Metal Pipe Arch Culverts, 18" Corner Radius Flowing Full n = 0.0327 to 0.0306

Date: NOV 1992

Figure: 7-18

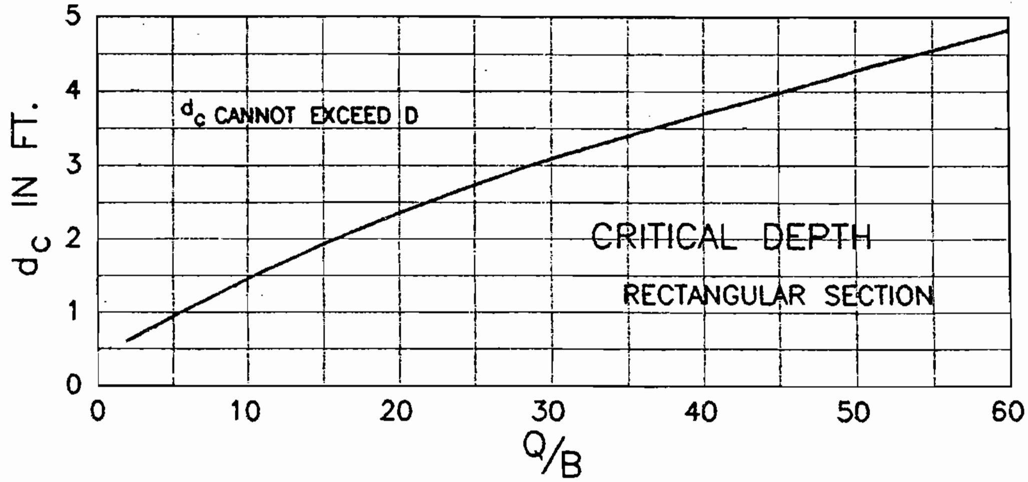


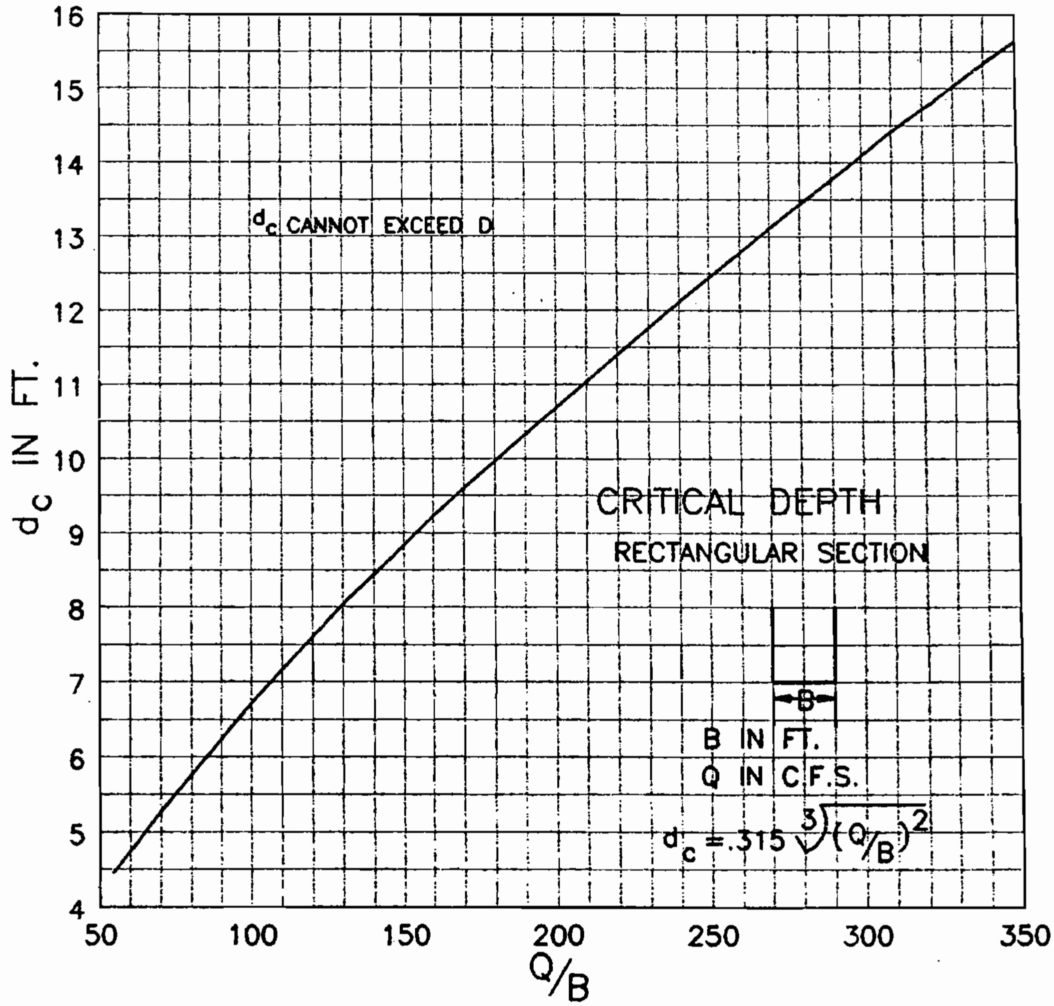
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Critical Depth, d_c Circular Pipe

Date
NOV 1992

Figure
7-19





Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

CITY OF AMARILLO

Critical Depth, d_c
Rectangular Section

Storm Water Management Criteria Manual

Date NOV 1992

Figure 7–20

CULVERT DESIGN FORM

PROJECT:

STATION:

SHEET ____ OF ____

DESIGNER/DATE: ____ / ____

REVIEWER/DATE: ____ / ____

HYDROLOGICAL DATA

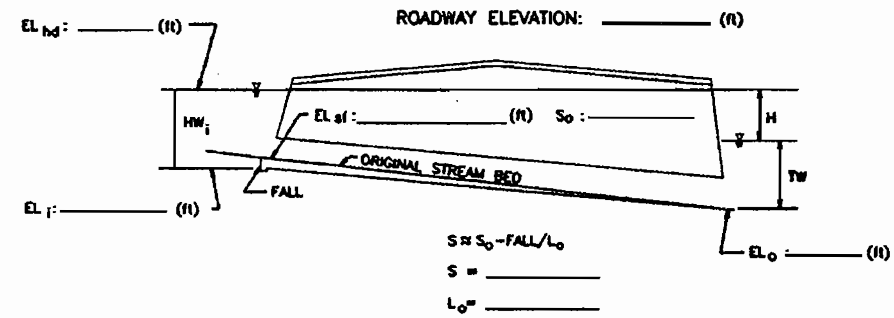
SEE ADD'L SHTS.

- ☐ METHOD:
- ☐ DRAINAGE AREA: ☐ STREAM SLOPE:
- ☐ CHANNEL SHAPE:
- ☐ ROUTING: ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW(cfs)	TW(ft)

ROADWAY ELEVATION: ____ (ft)



$S \approx S_o - FALL/L_o$

$S =$ ____

$L_o =$ ____

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c+D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1+k_e+(29n^2L)/R^{1.33}\right]V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE:

SHAPE:

MATERIAL: ____ n ____

ENTRANCE:

Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Culvert Design Form

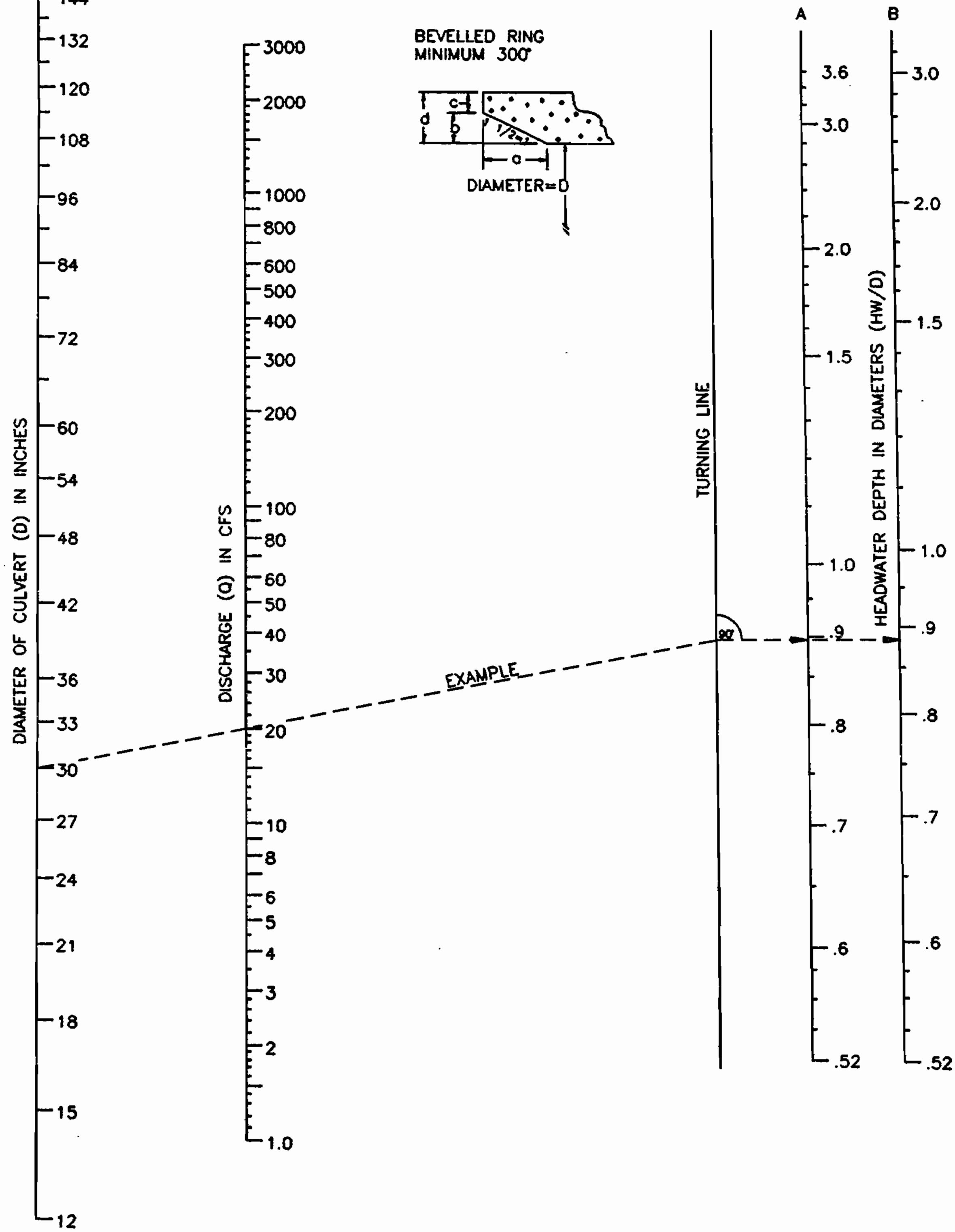
CITY OF AMARILLO

Storm Water Management Criteria Manual

Date: NOV 1992

Figure 7-21

b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

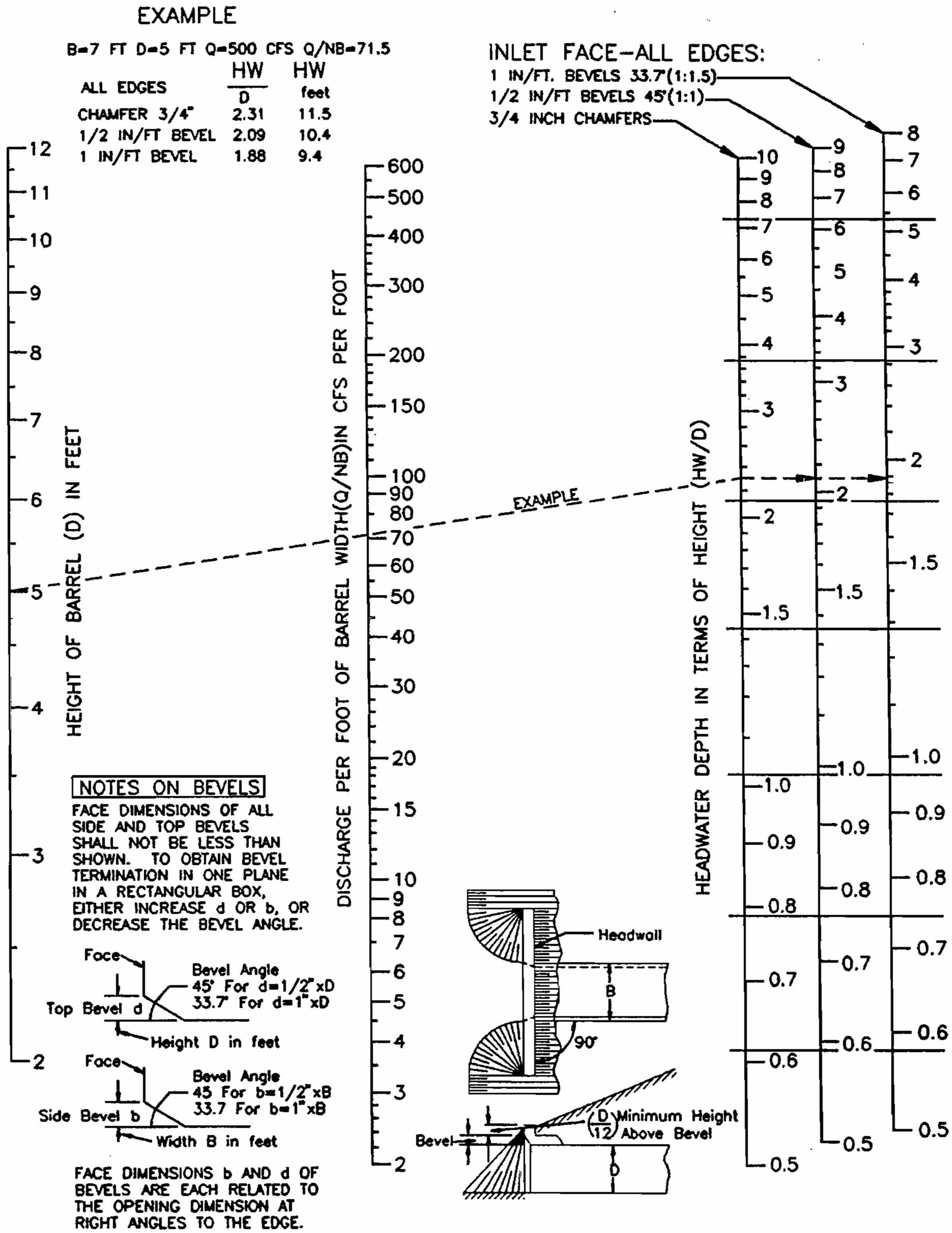
CITY OF AMARILLO

Headwater Depth for Circular Pipe Culverts with Beveled Ring with Inlet Control

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
7-22



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Headwater Depth for Inlet Control
Rectangular Box Culverts
90° Headwall
Chamfered or Beveled Inlet Edges

Storm Water Management Criteria Manual

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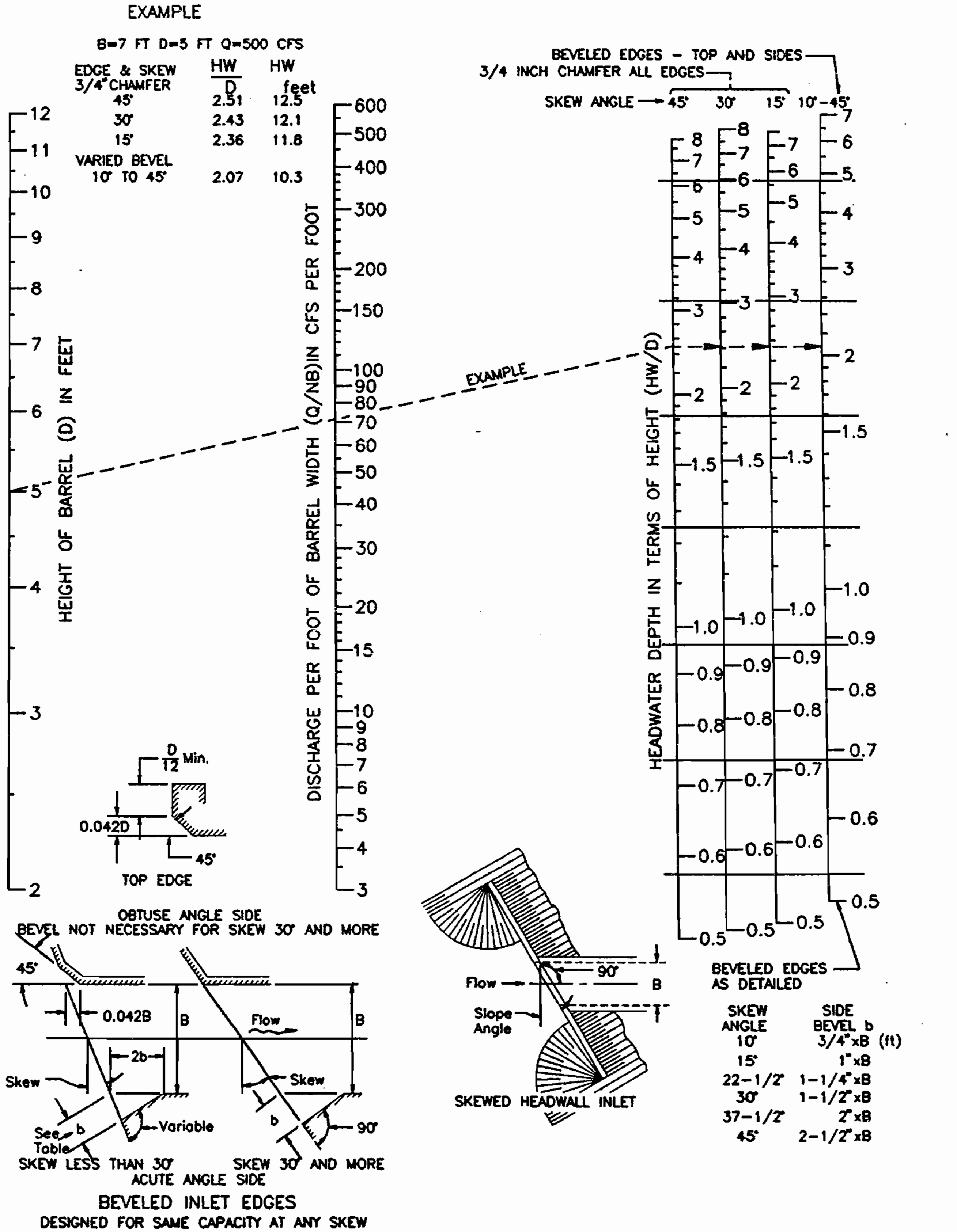
Figure 7-23



SKEW ANGLE	SIDE BEVEL b 3/4"xB (ft)
10°	3/4"xB
15°	1"xB
22-1/2°	1-1/4"xB
30°	1-1/2"xB
37-1/2°	2"xB
45°	2-1/2"xB

Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

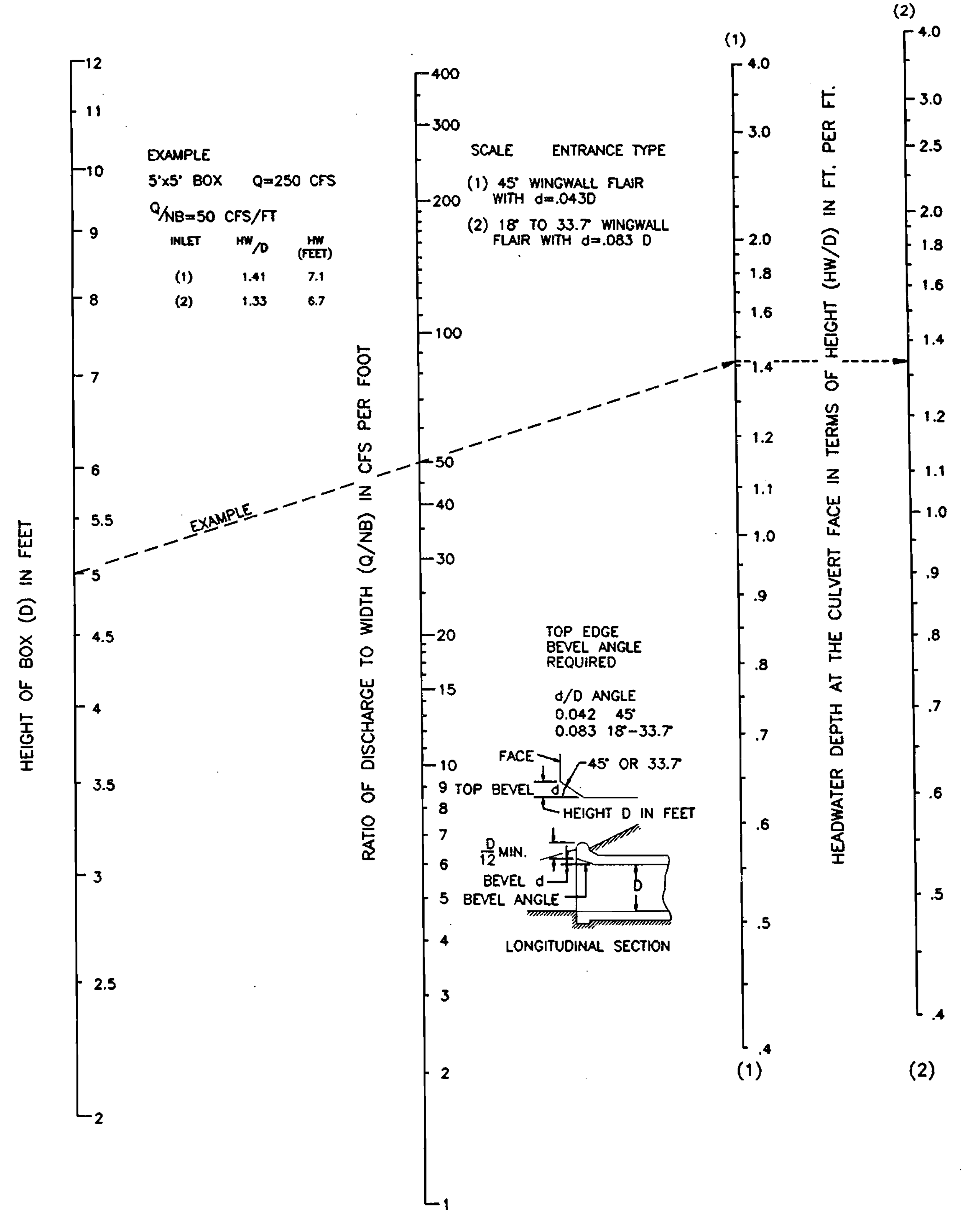
Headwater Depth for Inlet Control Single Barrel Box Culvert Skewed Headwalls Chamfered or Beveled Inlet Edges

Storm Water Management Criteria Manual

CITY OF AMARILLO

Date: NOV 1992

Figure 7-24



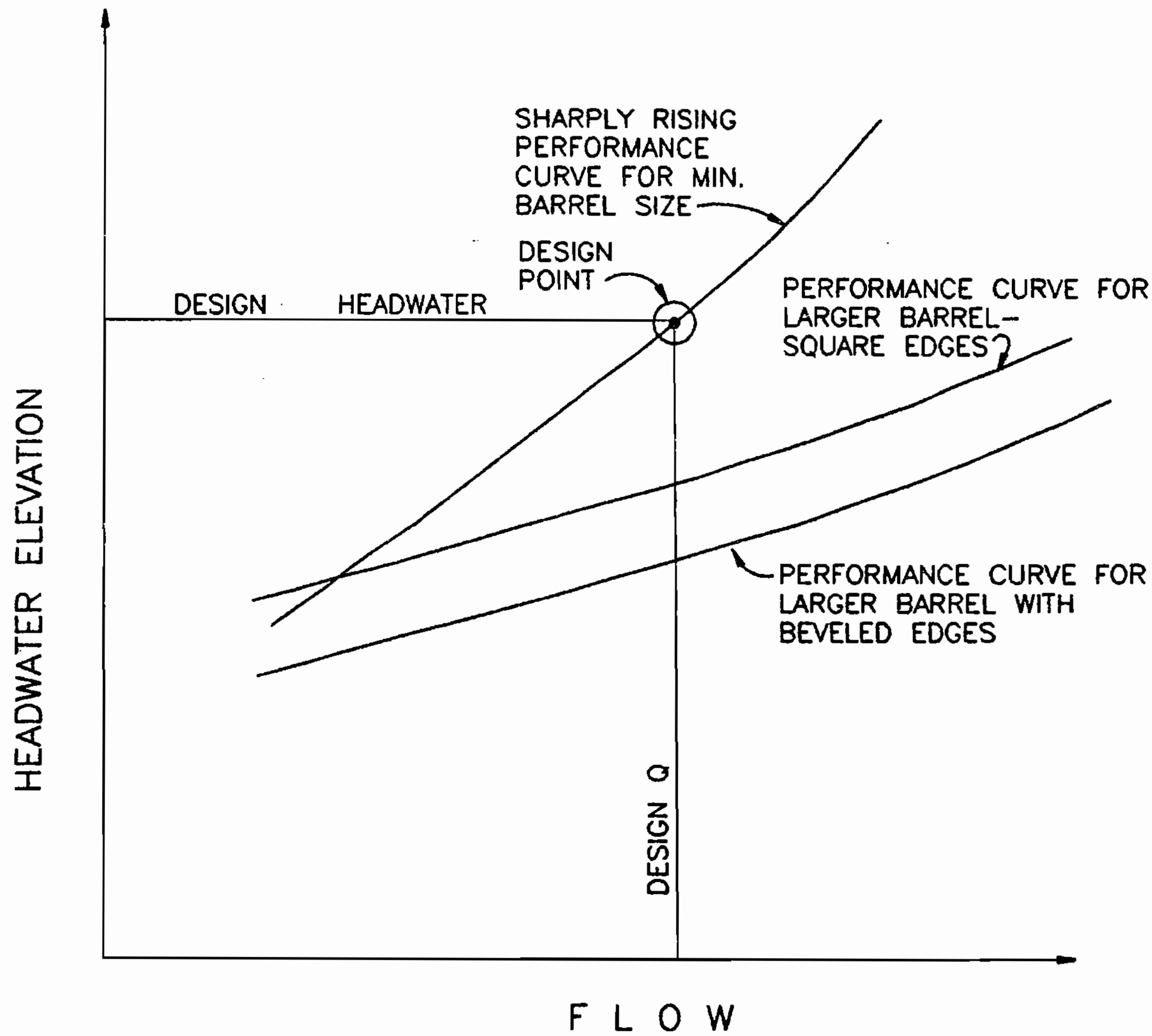
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Headwater Depth for Inlet Control Rectangular Box Culvert Flared Wingwalls 18° to 33.7° & 45° with Beveled Edge at Top of Inlet

Storm Water Management Criteria Manual

Date	NOV 1992
Figure	7-25



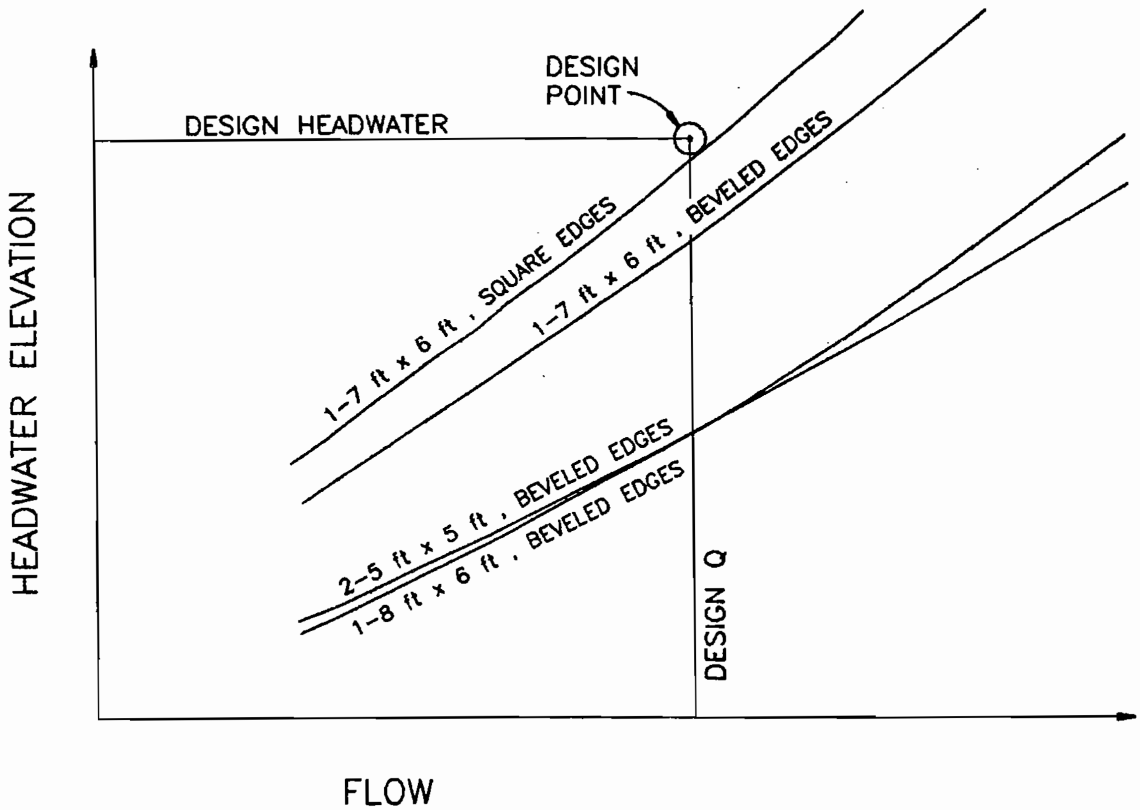
Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Outlet Control Performance Curves

Date
NOV 1992

Figure
7-26



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

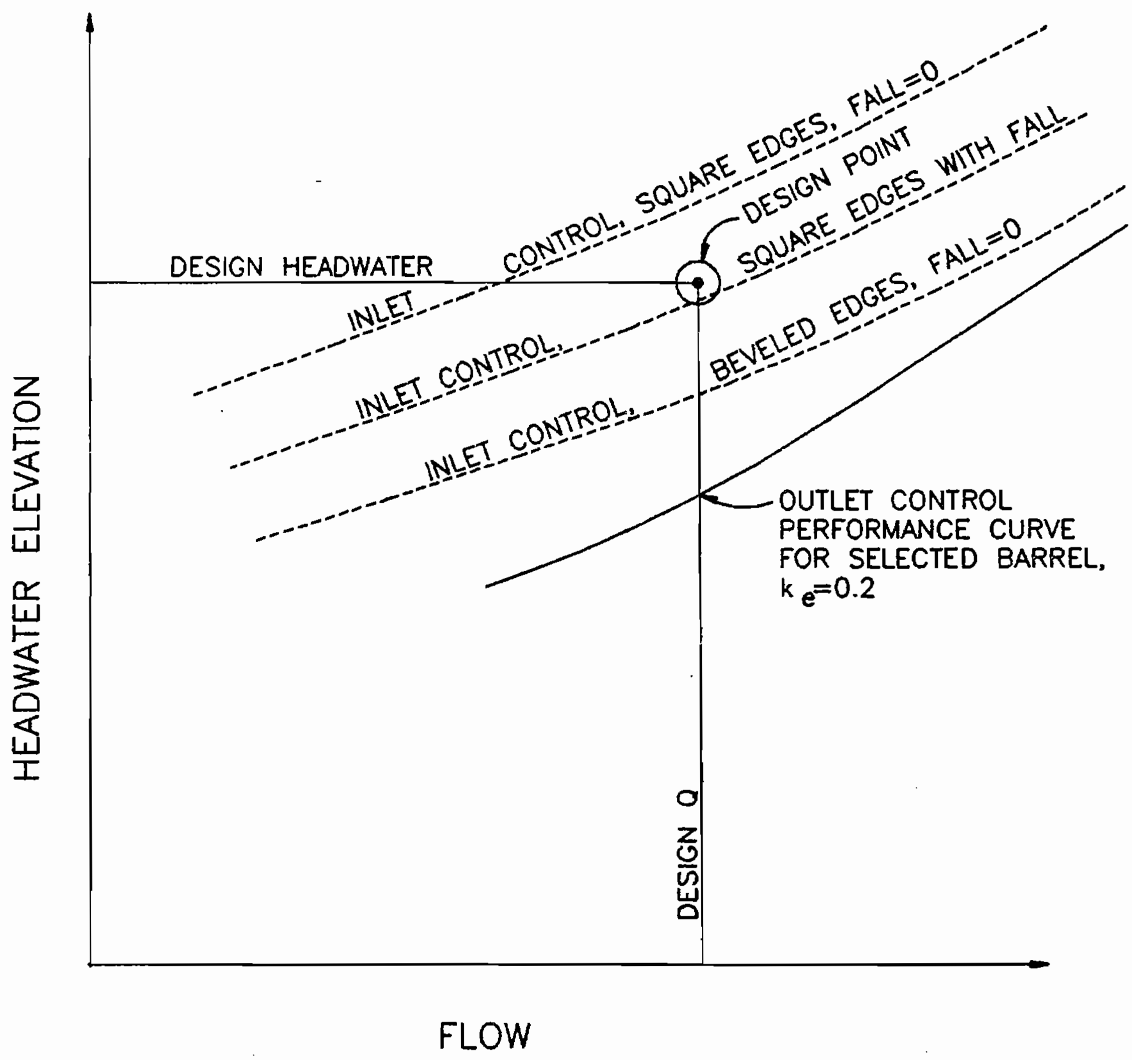
Box Culvert Outlet Control Performance Curves

Date
NOV 1992

Figure
7-27

CITY OF AMARILLO

Storm Water Management Criteria Manual



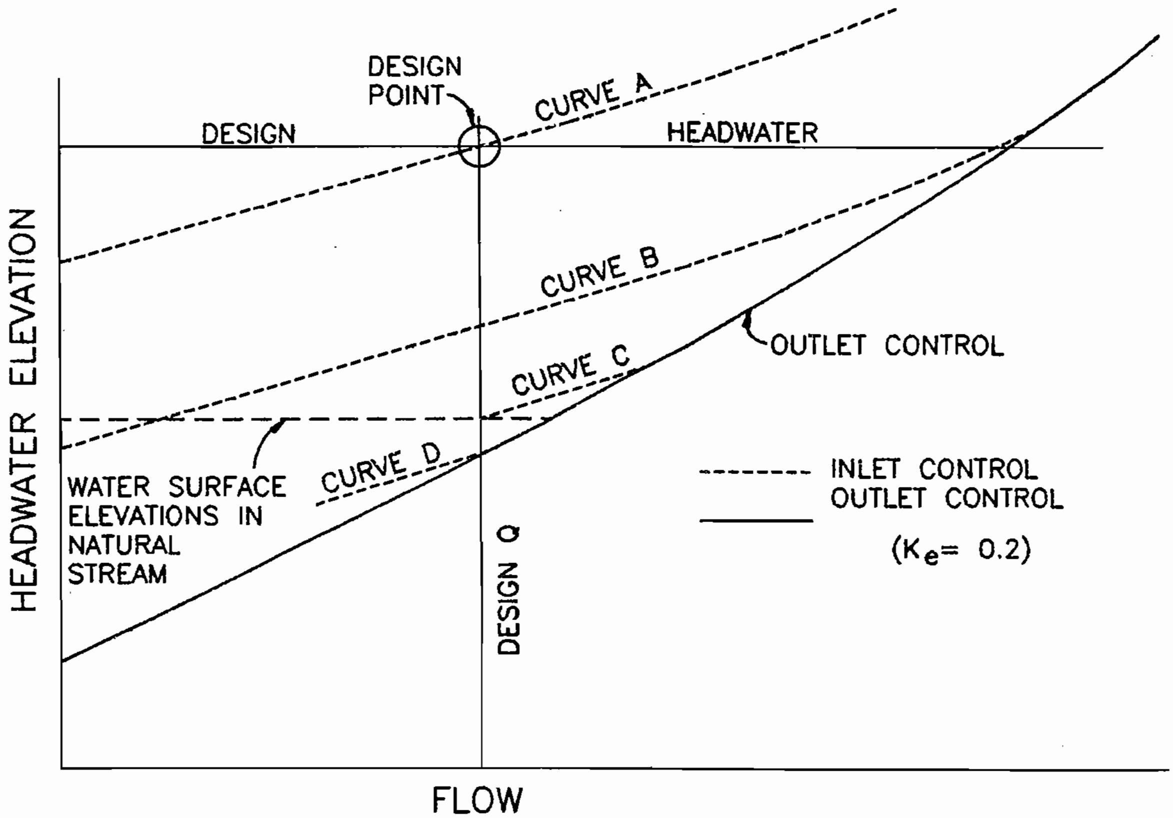
Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

Inlet Control Performance Curves for Various Inlet Configurations

Date: NOV 1992

Figure 7-28

CITY OF AMARILLO



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Optimization of Performance in Inlet Control

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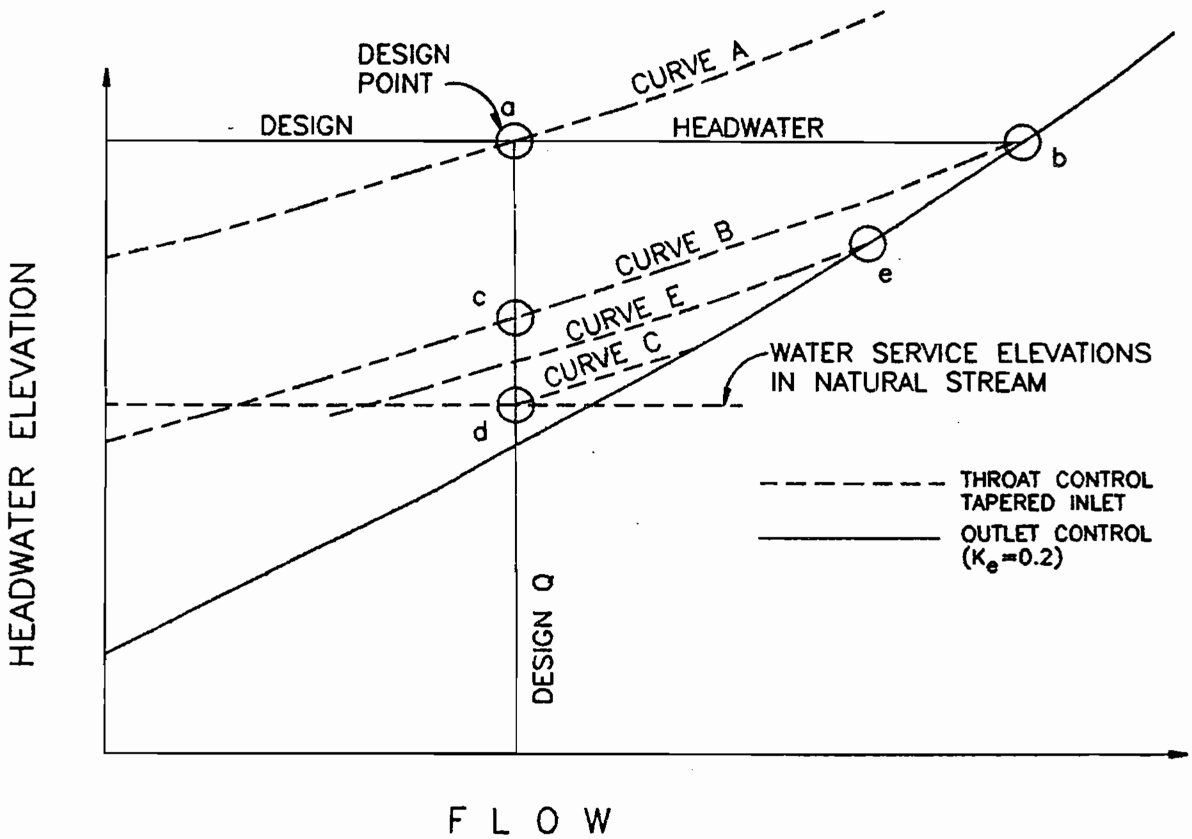
Figure
7-29



Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

CITY OF AMARILLO

Possible Face Design Selections Tapered Inlet

Storm Water Management Criteria Manual

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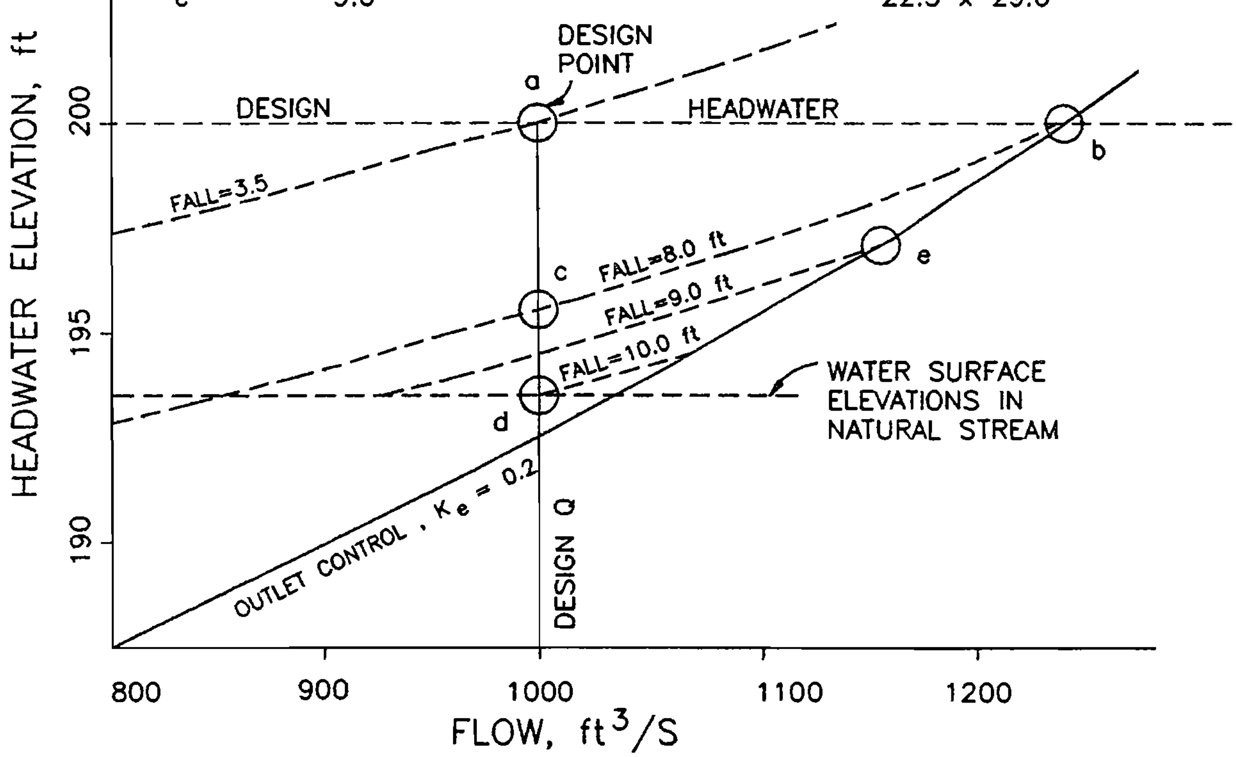
Figure 7-30

INLET STRUCTURE DIMENSIONS

POINT	FALL	SIDE TAPERED $B_f \times L_1$ (ft)	SLOPE TAPERED $B_f \times L_1$ (ft)
a	3.5	12.0 x 8.0	13.5 x 11.0
b	8.0	" "	17.0 x 20.0
c	8.0	" "	27.5 x 39.0
d	10.0	" "	52.5 x 89.0
e	9.0	" "	22.5 x 29.0



Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

Tapered Inlet Design Options for 8ft by 6ft Reinforced Box Culvert

Date NOV 1992

Figure 7-31

CITY OF AMARILLO

Storm Water Management Criteria Manual

PROJECT: ______________________	STATION: ______________ SHEET ______ OF ______	TAPERED INLET DESIGN FORM DESIGNER/DATE: ________ / ______ REVIEWER/DATE: ________ / ______

DESIGN DATA:

Q ____ = ______ cfs ; EL_{hi} __________ ft

EL. THROAT INVERT __________ ft

EL. STREAM BED AT FACE __________ ft

FALL ________ ft TAPER ________ :1 (4:1 TO 6:1)

STREAM SLOPE , S_o= __________ ft/ft

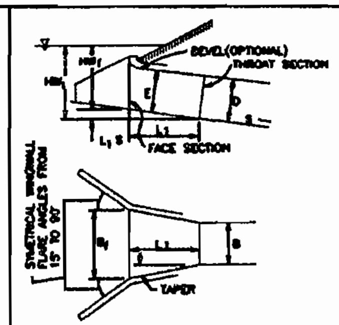
SLOPE OF BARREL , S= __________ ft/ft

S_f ________ :1 (2:1 TO 3:1)

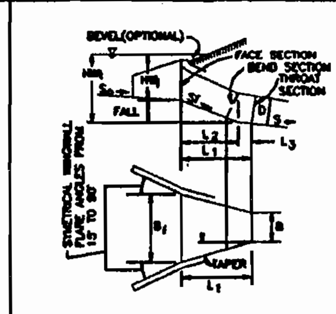
BARREL SHAPE AND MATERIAL: __________

N= __________ , B= __________ , D= __________

INLET EDGE DESCRIPTION __________



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

									SLOPE - TAPERED ONLY						SIDE-TAPERED W/FALL		
Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	$\frac{HW_f}{E}$ (3)	$\frac{Q}{B_f}$ (4)	MIN. B_f (5)	SELECTED B_f	MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	EL. CREST INV.	HW_c (12)	MIN. W (13)

(1) SIDE-TAPERED : EL. FACE INVERT = EL. THROAT + 1 ft (APPROX.)
SLOPE-TAPERED : EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi}$ - EL. FACE INVERT

(3) $1.1\ D \geq E \geq D$

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5$ NB

(7) L_2=(EL. FACE INVERT - EL. THROAT INVERT)S_f

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_3$

(9) IF (8)>(7). ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_2$

(10) IF(7)>(8). ADJ TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(11) SIDE-TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER
SLOPE-TAPERED: $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi}$ - EL. CREST INVERT

(13) MIN. $W = 0.35\ Q/HW_c^{1.5}$

SELECTED DESIGN

B_f __________

L_1 __________

L_2 __________

L_3 __________

BEVELS ANGLE ____°

b= ____ in:d= ____ in

TAPER ________ :1

S_f = ________ :1

Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

CITY OF AMARILLO

Tapered Inlet Design Form

Storm Water Management Criteria Manual

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Figure 7-32

MITERED INLET DESIGN FORM

PROJECT: ______________________

STATION: __________ SHEET ______ OF ______

DESIGNER/DATE: __________ / ______

REVIEWER/DATE: __________ / ______

DESIGN DATA: N______ ; B ______ ; D ______

Q ______ = ______ cfs : EL_{hi} ______ ft

EL. THROAT INVERT ______ ft EL. STREAM BED AT CREST ______ ft

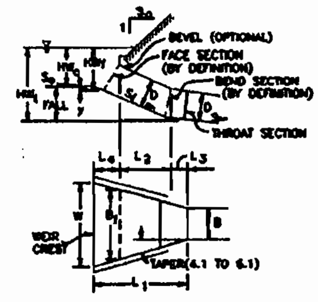
FALL ______ FT ; TAPER ______ : (4:1 TO 6:1)

STREAM SLOPE , S_o ______ ft/ft ; BARREL SLOPE , S= ______ ft/ft

SLOPE OF THE EMBANKMENT S_c= ______ :1 ; S_f ______ :1 (2:1 TO 3:1)

BARREL SHAPE AND MATERIAL: ______

INLET EDGE DESCRIPTION: ______



SLOPE-TAPERED INLET/MITERED FACE

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	y (1)	EL. FACE INVERT (2)	HW_f (3)	$\frac{HW_f}{E}$ (4)	$\frac{Q}{B_f}$ (5)	MIN. B_f (6)	SELECTED B_f	MIN. L_3 (7)	L_4 (8)	L_2 (9)	CHECK L_2 (10)	ADJ. L_3 (11)	ADJ. TAPER (12)	L_1 (13)	EL. CREST INV.	HW_c (14)	MIN. W (15)	W (16)

(1) $y = \left[\frac{(S_e - S_f) - 1}{(S_e + S_f)(S_f^2 + 1)^{0.5}}\right] \cdot D$

(2) EL. FACE INVERT − EL. STREAM BED AT CREST − y

(3) $HW_f = EL_{hi}$ − EL. FACE INVERT

(4) $1.1D \geq E \geq D$

(5) FROM DESIGN CHARTS

(6) MIN. $B_f = Q/(Q/B_f)$

(7) MIN. $L_3 = 0.5NB$

(8) $L_4 = S_f y + D/S_f$

(9) L_2 = (EL. CREST INVERT − EL. THROAT INVERT)$S_f - L_4$

*** IF L_2 IS NEGATIVE, DO NOT USE THIS INLET

(10) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right]$TAPER$-L_3$

(11) IF (10)>(9),ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right]$TAPER$-L_2$

(12) IF (9)>(10),ADJ. TAPER$=(L_2+L_3)/\left[\frac{B - NB}{2}\right]$

(13) $L_1 = L_2 + L_3 + L_4$

(14) $HW_c = EL_{hi}$ −EL. CREST INVERT

(15) MIN. $W = 0.35Q/(HW_c)^{1.5}$

(16) $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

IF W<MIN. W, ADJUST TAPER

SELECTED DESIGN

B_f ______

L_1 ______

L_2 ______

L_3 ______

L_4 ______

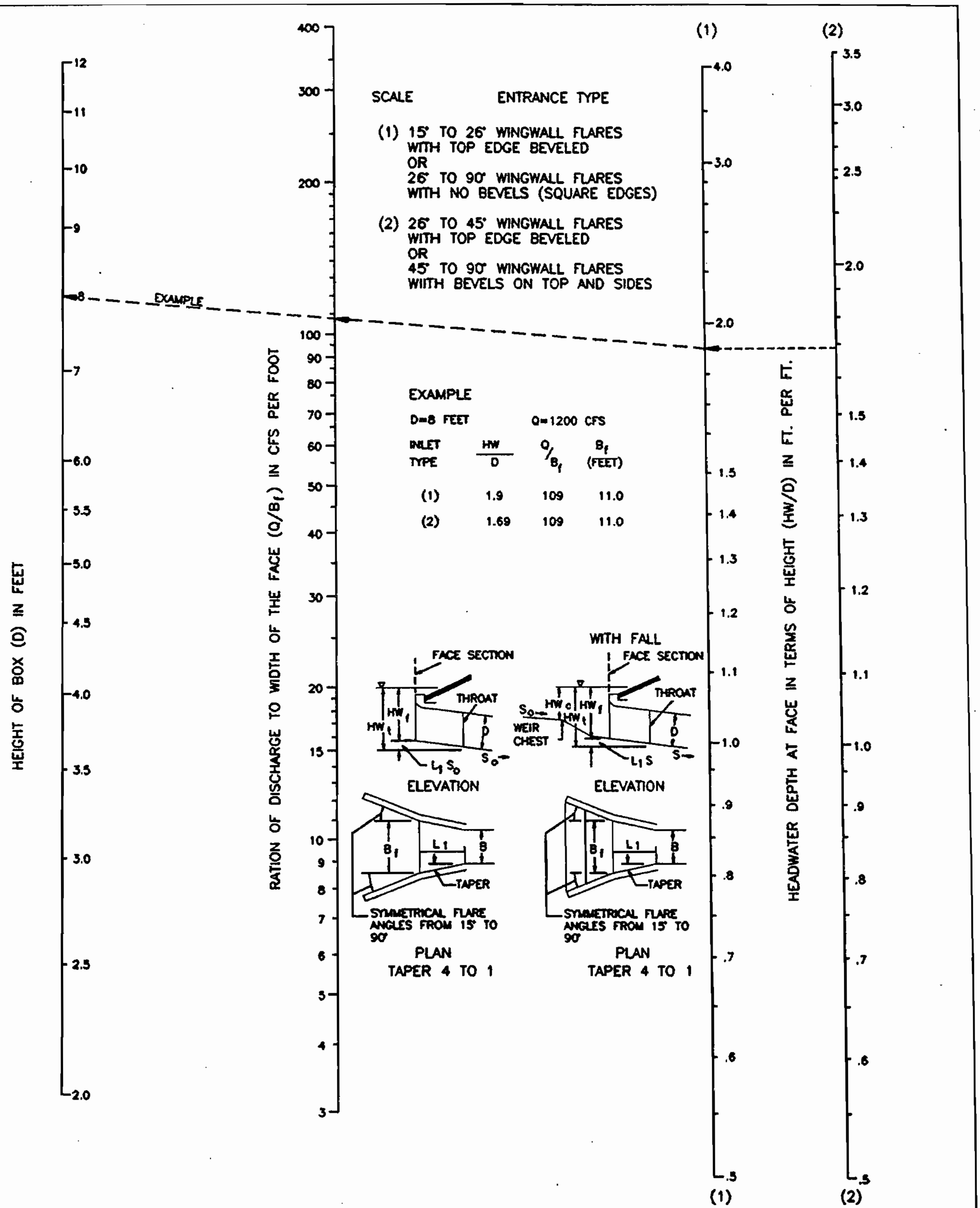
BEVELS ANGLE ____°

b= ____ in:d= ____ in

TAPER ______ :1

S_f = ______ :1

Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Headwater Depth for Face Control Box Culverts with Side-Tapered Inlets

CITY OF AMARILLO

Storm Water Management Criteria Manual

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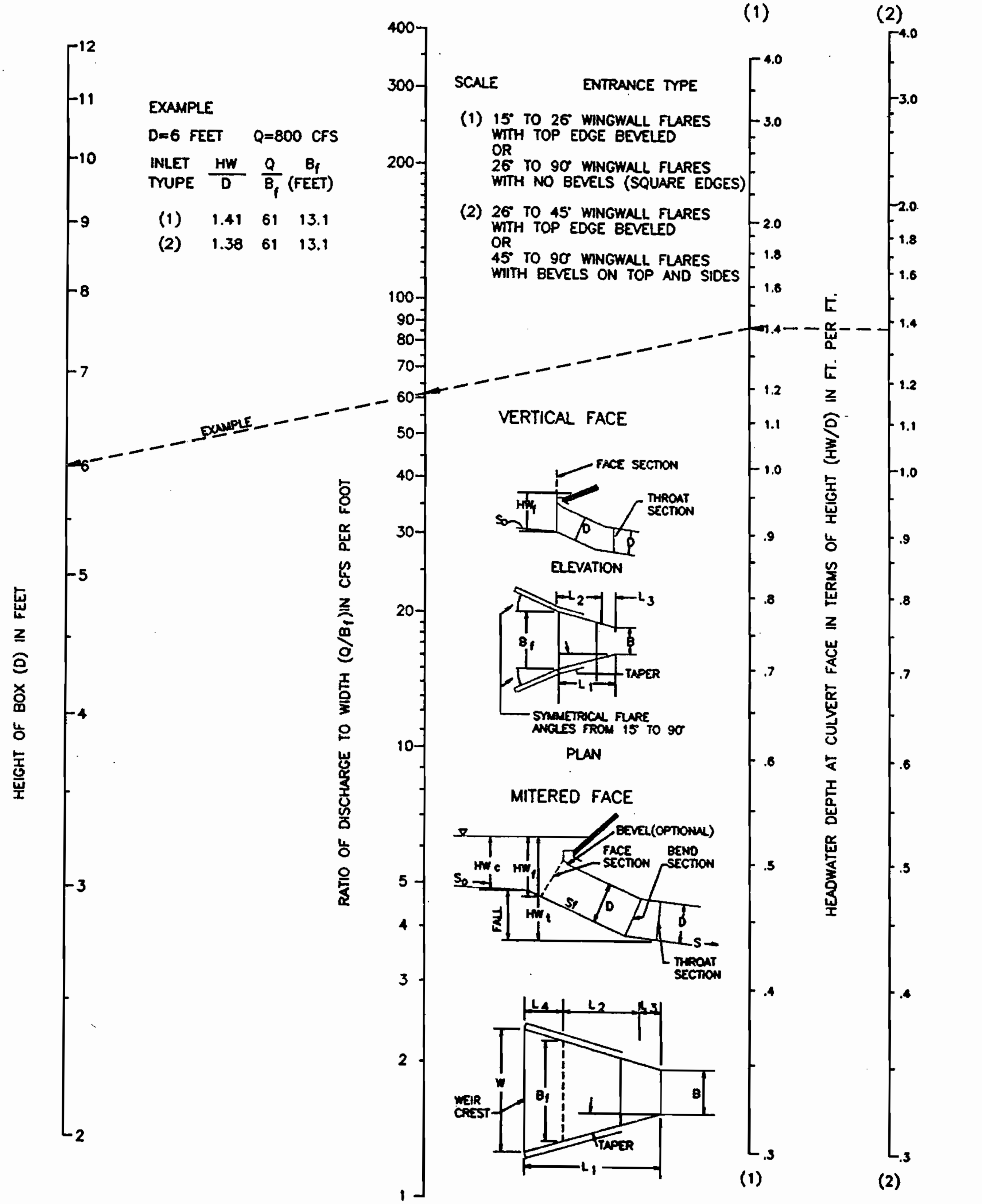
Figure 7–34



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Headwater Depth for Face Control Box Culvert with Slope-Tapered Inlets

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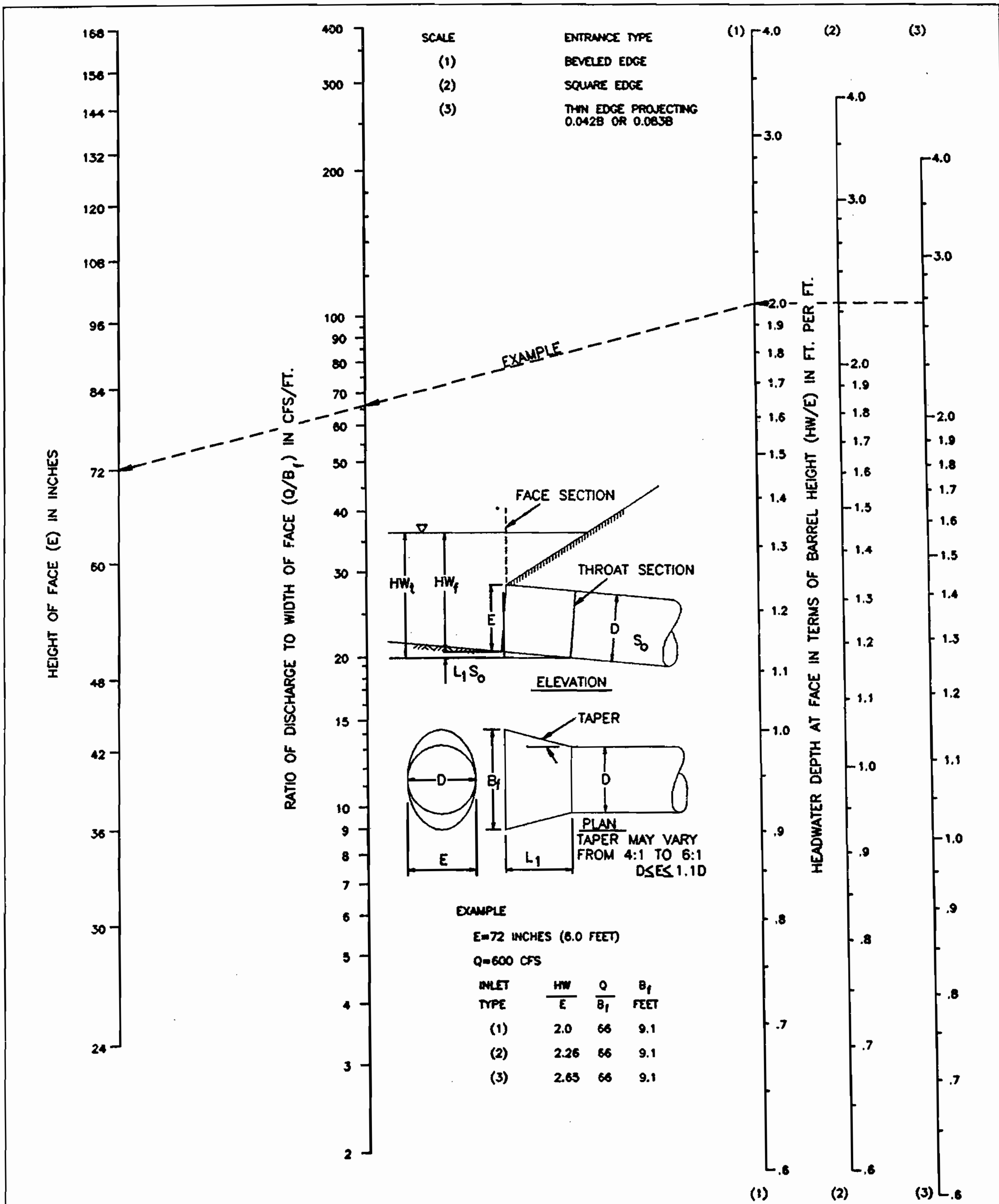
Storm Water Management Criteria Manual

Date: NOV 1992

Figure 7-35

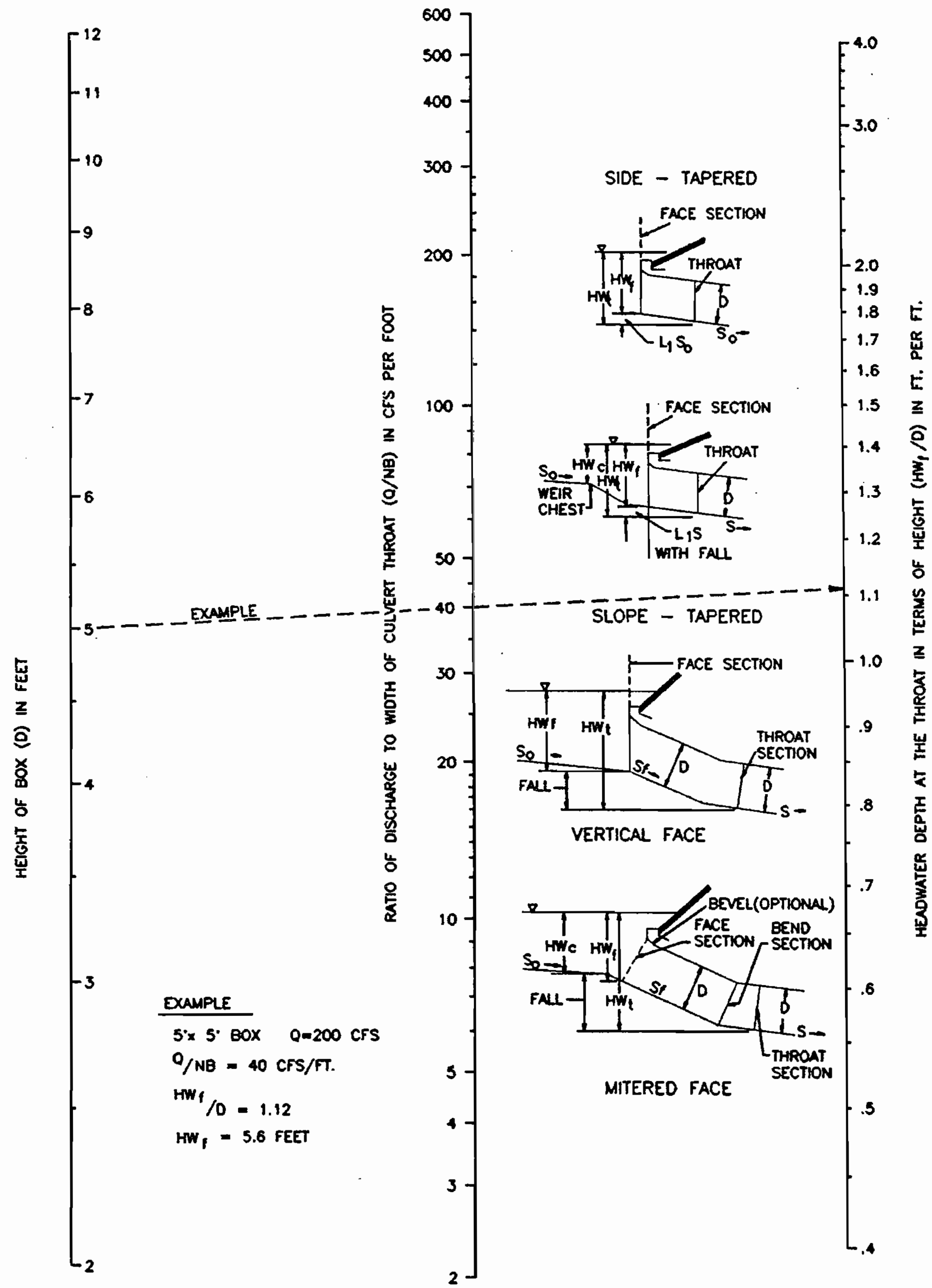


Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Headwater Depth for Face Control Pipe Culverts with Side-Tapered Inlets (Non-Rectangular Section Only)

Date: NOV 1992

Figure 7-36



Source: *Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.*

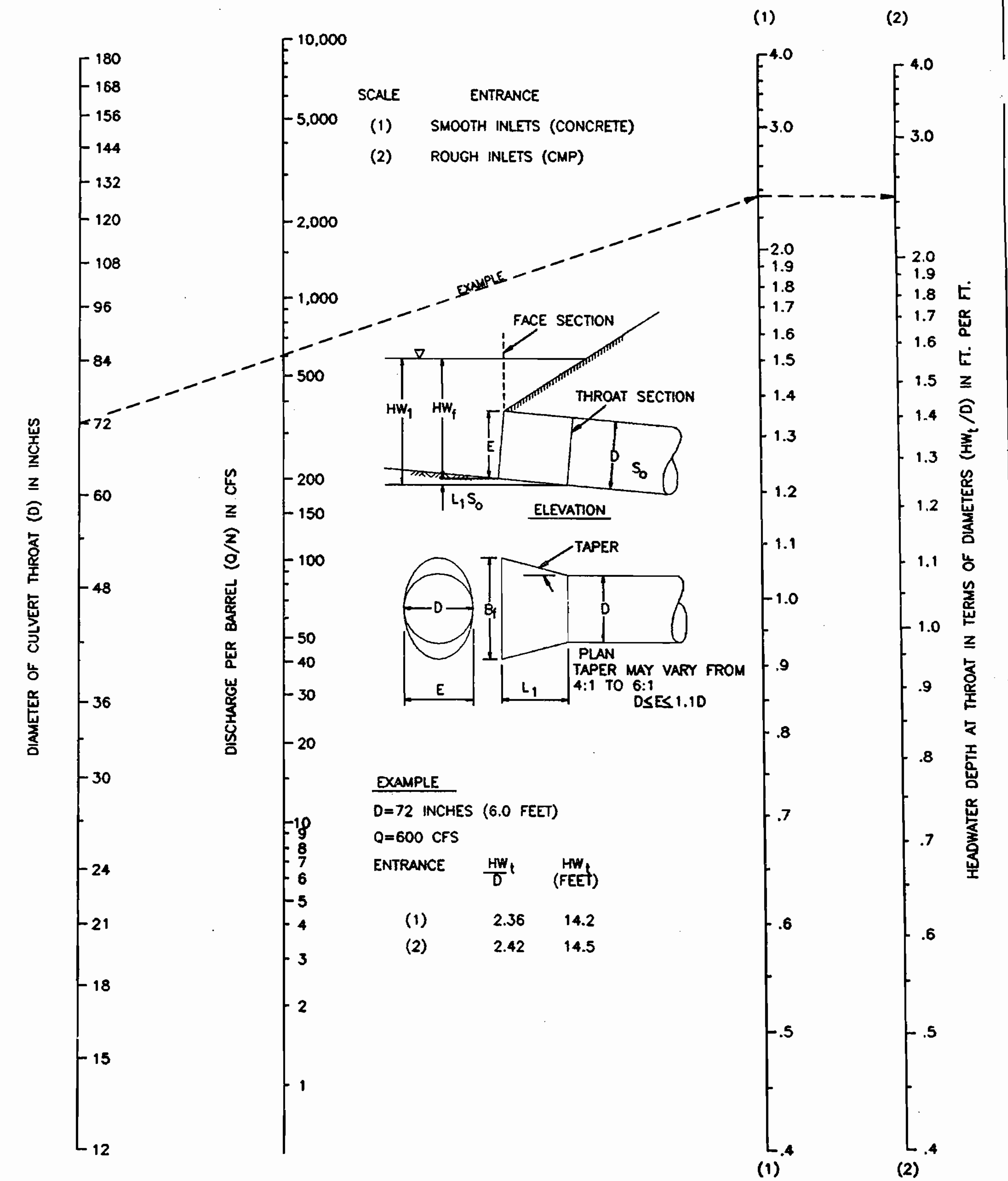
Headwater Depth for Throat Control Box Culverts with Tapered Inlets

Date NOV 1992

Figure 7-37

CITY OF AMARILLO

Storm Water Management Criteria Manual



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

CITY OF AMARILLO

Headwater Depth for Throat Control Pipe Culverts with Side-Tapered Inlets (Circular Section Only)

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Date: NOV 1992

Figure 7-38

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8.1 INTRODUCTION

On-site detention of runoff is an alternative to other methods of urban storm water management. Storage, which involves collecting excess runoff before it enters the main drainage system, can often be an effective and economical means of reducing peak flow rates and mitigating problems of flooding, pollution, soil erosion, and siltation.

Detention facilities can be used to lessen the impact of peak flows on downstream property, and for the improvement of water quality. Large regional facilities serving a number of developments are generally preferable to small on-site facilities serving only one subdivision or office complex.

The detention basin is the most widely used measure for controlling peak discharges from urbanizing areas. Basins can be designed to fit a variety of sites and can incorporate multiple-outlet spillways to meet requirements for multi-frequency control of flow. Measures other than a detention basin, such as infiltration trenches or porous pavement may be preferred in some locations. Any device selected, however, should be assessed as to its cost, function, maintenance requirements (frequency and type), and impact on downstream peak flows.

A drainage master plan has been developed for each watershed in the Amarillo area. The master plan identifies alternatives for playa modification and/or detention facilities to manage runoff subject to various development conditions. In order to provide the flexibility required for all storm water management objectives, this plan will occasionally require adjustment and modification. For specific developments, special requirements may necessitate the modification of playas and detention facilities or the construction of on-site facilities. Proposals not addressed in the master plan must be approved by the City Engineer.

Storage is a means to mitigate problems associated with increased runoff caused by development. It is preferable to avoid causing the problems in the first place by minimizing the increase in runoff volumes or rates. This section outlines strategies to achieve those goals.

8.2 PLAYAS

More than sixty naturally occurring playas are found in Amarillo and its extraterritorial jurisdiction. These playas generally function as terminal storage for storm water runoff which flows radially inward from the surrounding drainage basins. Most of the playas have no natural outlet; evaporation and seepage are the only natural mechanisms for emptying the playas. As a result, flooding is a recurrent hazard as development has encroached on the fringes of the playas. The City has responded to the problem of playa-related flooding by providing pumping facilities at several playas and by drafting a Flood Hazard Ordinance. The pumping facilities are intended to dewater playas during dry weather periods in order to increase the flood storage available in the playas during storm events. The Flood Hazard Ordinance imposes restrictions on development below the base flood elevation (as defined by the City) established for each playa. The effects of playas on upstream drainage improvements should be incorporated into the design of such improvements. Similarly, the effects of new development near playas or within playa boundaries should be evaluated to avoid increasing flood levels and related damages.

8.2.1 Expected Water Surface Elevations

The City has computed expected water surface elevations (based on ultimate development conditions) for the 2- and 100-year return periods as well as the maximum historical level for many of the playas in the vicinity of the City. These levels were determined using the Amarillo Simulation Analysis of Playa Performance (ASAPP) model. The elevations represent the level that a playa would attain or exceed for a given return period based on projected ultimate development of the playa drainage basin, current operating practices, and historical daily rainfall events. These expected water surface elevations may be obtained from the City Engineer.

8.2.2 Analysis of Tailwater Control Conditions Caused by Playas

When designing or evaluating hydraulic systems which are affected by tailwater resulting from playas, the design level for tailwater should be based on playa-specific flood levels. These design flood levels can be obtained from the City Engineer. The appropriate return interval for the tailwater level is the same as the contributing drainage system. For example, if a bridge was designed to pass the 100-year storm peak and located sufficiently close to a playa such that backwater effects from the playa would control tailwater conditions at the bridge, then the 100-year flood level for the playa would be used to begin backwater computations. While it is true that the joint probability of experiencing a 100-year storm simultaneously with a 100-year tailwater condition is less than the probability of either event alone, it will nonetheless result in a prudent design. Variations from this methodology will require approval by the City Engineer.

8.2.3 Development in and Adjacent to Playas

Development near playas is restricted to prevent increasing flood levels and associated damages above existing conditions. The restrictions involve requiring compensation for any development which is planned below the base flood level such that there is no increase in the post-development base flood level. Acceptable measures for compensation include excavation, pumping, gravity draining, or a combination of these measures.

Excavation generally requires that material, equal in volume to the proposed infill below the base flood elevation, be removed from the playa below the base flood elevation. Although it is believed that excavation from playas contributes to increased infiltration, no credit is to be provided due to the unquantifiable nature of the seepage increase. Excavation slopes, as well as infill slopes, should be no steeper than 4H:1V. If the proposed infill and excavation project results in a decrease in playa surface area, the City Engineer may increase the required excavation volume to compensate for reduced evaporation loss.

Pumping requires that permanent pumps be installed to draw down playa levels such that the resulting base flood level does not exceed the level before development. Furthermore, pump discharge may not adversely affect the flood levels downstream of the pumped discharge point.

Gravity drainage can be accomplished through open channel cut or tunneling. As with pumping measures, gravity drainage must be shown to completely compensate for any proposed infill without adversely affecting the receiving channel or downstream facilities. In general, only those playas which currently spill to neighboring playas are candidates for gravity drainage modifications.

Proposals for development and required compensatory measures can be evaluated using the Playa Simulation Model (ASAPP).

8.3 DETENTION BASIN DESIGN CRITERIA

The Rational Formula shall be used for detention basin design only for small areas (20 acres or less) as described in Section 8.5. Methods which include a runoff hydrograph, such as HEC-1, shall be used for watersheds larger than 20 acres. Runoff hydrographs must be developed as part of the evaluation of drainage system performance during minor and major storm events. Computations of runoff hydrographs which do not rely on a continuous accounting of antecedent moisture conditions shall assume antecedent moisture condition II.

Detention basins and playa modifications shall be designed to protect the safety of any children or adults coming in contact with the system during runoff events. Safety of storm water drainage system components is always a principal design criteria. The use of fencing around detention basins and playa modifications may be avoided by designing safe facilities. However, certain extreme cases may require the use of fences to protect the public. The shorelines of all detention basins and playa lakes at 100-year capacity shall be as level as practicable to prevent accidental falls into the basin and for stability and ease of maintenance.

The sideslopes of the banks of detention basins shall not be steeper than 4H:1V. All detention basins and playa lakes shall have a level safety ledge extending three (3) feet into the basin from the shoreline and two (2) feet below the normal water depth. Velocities throughout the drainage system shall be controlled to safe levels taking into consideration rates and depths of flow.

8.3.1 Design Storm

Detention basins shall be designed to limit the peak rate of discharge from the basin for the 2-year and 100-year events to the predevelopment rate or a rate which will not cause an increase in flooding or channel instability downstream when considered in aggregate with ultimate watershed development and downstream drainage capacities.

A minimum of one (1) foot of freeboard shall be added to the design water surface elevation. Backwater computations for runoff entering playas shall assume a starting elevation based on a compatible storm. It is the responsibility of the developer to determine if additional freeboard is necessary. The City Engineer reserves the right to require additional freeboard if the deemed necessary for safety or maintenance considerations. The design storm shall pass through the outlet without overtopping the structure.

8.3.2 Principal Outlet Works

Where a single pipe outlet is to be used to discharge, it shall have a minimum inside diameter of 18 inches. Maintenance of outlets smaller than 18 inches is likely to be a problem. If design release rates call for outlets smaller than this, release structures such as perforated risers or flow control orifices shall be incorporated.

Depending on the geometry of the outlet structure, discharge for various headwater depths can be controlled by the inlet crest (weir control), the riser or barrel opening (orifice control), or the riser or barrel pipe (pipe control). Each of these flow controls shall be evaluated when determining the rating curve of the principal outlet. The following weir, orifice and pipe flow equations can be used to evaluate a single opening outlet structure. Multiple openings required a more rigorous analysis and is beyond the scope of this Manual. Solicit hydraulics textbook for procedure.

Weir flow may be computed by the following equation:

$$Q = CLH^{3/2} \qquad (8\text{-}1)$$

where:

Q	=	discharge, in cubic feet per second
C	=	weir coefficient
L	=	length of the weir, in feet; for circular riser pipes, L is the pipe circumference
H_w	=	the depth of flow over the weir crest, in feet

Orifice flow may be computed by the following equation:

$$Q = CA(2gH_o)^{0.5} \qquad (8\text{-}2)$$

where:

Q	=	discharge, in cubic feet per second
C	=	orifice coefficient
A	=	cross-sectional area of the pipe, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
H_o	=	head above the centerline of the pipe, in feet

The weir and orifice coefficients are a function of various hydraulic properties and dimensional characteristics. The designer is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King[1] and use engineering judgement.

Pipe flow may be computed by the following equation:

$$Q = A\left[\frac{2gH}{1+k_b+k_e+k_f\,L}\right]^{0.5} \tag{8-3}$$

where:

Q	=	discharge, in cubic feet per second
A	=	cross-sectional area of the pipe, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
H	=	the difference between headwater and tailwater elevations, in feet
k_b	=	bend loss coefficient, use 0.6
k_e	=	entrance loss coefficient, use 0.5
k_f	=	friction loss coefficient
	=	$\frac{185n^2}{D^{4/3}}$
n	=	Manning's roughness coefficient
D	=	diameter of pipe, in feet
L	=	length of pipe, in feet

8.3.3 Emergency Spillways

The designer is responsible to determine if an emergency spillway or a spillway feature is needed for an embankment type detention facility. The City Engineer may require the designer to evaluate a more stringent design requirement including a breach analysis if there is a potential for loss of life. In addition, certain embankments are classified as dams and are required to meet rules established by the Texas Water Commission.

The position, profile, and length of the spillway are influenced by geologic and topographic features of the site. The cross section dimensions are governed by hydraulic elements and are determined by acceptable reservoir routing of the design storm. Most emergency spillways for detention ponds may be designed as grass-lined open channels. Table 8-1 presents acceptable grasses for vegetative spillways.

TABLE 8-1 Acceptable Grasses for Vegetated Spillways

Vegetative Type
Western Wheatgrass
Buffalograss
Bermudagrass
Tall Fescue
Blue grama

Discharge from the emergency spillways shall be directed to the main channel without causing erosion along the downstream toe of the dam. Emergency spillways proposed for the protection of earthen embankments shall be in full cut undisturbed soil, if possible, to avoid flows against constructed fill. The side slopes of the excavated channel in earth shall be no steeper than 4H:1V for ease of maintenance. Where the site limitations prevent a full channel cut, a wing dike shall be provided to direct spillway flows away from the downstream toe of the dam. Ready access to the emergency spillway system shall also be provided.

The configuration of the entrance channel from the reservoir to the control section of the emergency spillway shall be a smooth transition to avoid turbulent flow over the spillway crest. The outlet channel of the emergency spillway shall convey flow to the channel below the structure with a minimum of erosion. The slope of the exit channel usually follows the configuration of the abutment. Slopes, however, should not exceed 10 percent. In cases of highly erodible soils, it may be necessary to use other means of protection such as riprap, grouted rock or concrete paving to form the exit channel. As an alternative, detention storage can be increased to reduce the frequency or duration of use of the emergency spillway and thereby reduce erosion problems.

8.4 ON-SITE DETENTION

Potential advantages and disadvantages of on-site detention basins should be considered by the designer in the early stages of development. Discharge rates and outflow velocities are regulated to conform to the capacities and physical characteristics of downstream drainage systems. Energy dissipation and flow attenuation resulting from on-site storage can reduce soil erosion and pollutant loading. By controlling release flows, the impacts of the pollutant loading of stored runoff on receiving water quality can be minimized.

8.4.1 Parking Lots and Streets

There are two general types of storm water detention on parking lot surfaces. One type involves the storage of runoff in depressions constructed at drain locations. The stored water is drained into the storm sewer system slowly, using restrictions such as orifice plates in the drain. Proper design of such paved areas will restrict ponding to areas which will cause the least amount of inconvenience to the users of the parking areas.

For example, the parking lot of a shopping center will have the ponding areas located in the least-used portions of the lot, allowing customers to walk to their vehicles in areas of no ponding, except when the entire lot is filled with vehicles. Drainage of ponded water would be fairly rapid to prevent customer inconvenience. In most cases, the water should pond to a depth not to exceed 7 inches and the ponding area should be drained within 30 minutes or less after the rainfall. Computation of the amount of storage needed would be similar to the analysis used in designing detention basins on ground surfaces.

Another type of storm water detention on parking lots consists of using the paved areas of the lot to channel the runoff to grassed areas or gravel-filled seepage pits (Figure 8-1). Water from pavement should run through at least 30 feet of grass before entering an infiltration swale, trench or basin. The flow then infiltrates into the ground. Soil conditions and the effects of siltation in reducing infiltration must be considered.

Minimum slopes of one (1) percent are recommended in parking lot detention areas. Maximum slopes should not exceed four (4) percent to avoid gasoline spillage from tanks and to minimize vehicle traction problems on icy pavement.

8.4.2 Recreational Areas

Generally, recreational areas such as outdoor athletic fields have a substantial area of grass cover which often has a high infiltration rate. Generally, storm runoff from such fields is minimal. Grassed recreational fields can be utilized for the temporary detention of storm runoff without adversely affecting their primary function.

Amarillo contains many parks, both the neighborhood type and the large, central type. Parks, like recreational areas, create little runoff of their own; however, parks provide excellent detention storage potential for runoff from adjacent areas.

8.4.3 Property Line Swales

Subdivision planning and layout requires adequate surface drainage away from buildings. This is obtained by sloping the finished grade away from the buildings. When possible, the layout should call for a swale to be located along the back and/or side property line which then drains through the block (Figure 8-2). Such drainage should be guided away from storm sewers and towards natural channels. If storm sewers are the only point of discharge, the route should be as long as possible to allow infiltration. The final grading plan for the lot layout can be finalized to allow up to six (6) inches of temporary ponding along the property line.

Temporary ponding facilities along lot lines may include small controlled discharges or, if the subsoil conditions are favorable, such water may be percolated into the ground.

8.4.4 Road Embankments

The use of road embankments for temporary storage is an efficient method of attenuating the peak flows from a drainage basin.

The design criteria to be used for the temporary detention of water behind road embankments shall include consideration of the major storm runoff. The use of roadway embankments to help reduce downstream peak flows is encouraged. Planning for the usage of embankments must be done with thorough consideration to avoid damage to the embankment, the structure, and adjacent property.

8.4.5 On-Site Ponds

The construction of on-site ponds provides significant detention benefits when properly planned and designed. The use of such ponds is particularly encouraged in planned unit developments where large areas of grass and open space are common.

Controlled outlets for the surcharge storage can be used, and it is suggested that such outlets be designed to release at a rate that does not exceed the rate estimated for natural conditions or downstream channel capacity.

8.4.6 Combinations

In many instances, one on-site detention method cannot conveniently or economically satisfy the required amount of storm water storage. Limitations in storage capacities, site development conditions, soils limitations, and other related constraints may require that more than one method be utilized. For example, parking lot and surface pond storage might all be required to compensate for increases in runoff due to development of a particular site. Whichever combinations are suitable should be incorporated into the site development plan.

8.5 HYDRAULIC DESIGN METHODS

8.5.1 Modified Rational Method Analysis

The term Modified Rational Method Analysis is a procedure for manipulating the basic Rational Method to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. This greater volume of runoff produced by longer storm durations must be analyzed to determine the correct sizing of detention facilities.

The approach becomes more valid on progressively smaller basins, eventually reaching a size so small that watershed modeling is approached. The procedure should, therefore, be limited to relatively small areas such as rooftops, parking lots, or other upstream areas with tributary basins less than 20 acres. This would minimize major damage which could result from overtopping or failure of the proposed detention facility.

Figure 8-3 presents a family of curves for a theoretical basin described in the following example. These hydrographs are developed by using the basic Rational Method assumptions of constant rainfall intensity, time of concentration (t_c) for the longest flow path, and the coefficient of runoff. The typical Rational Method hydrograph with the peak discharge coinciding with the time of concentration for the basin is first calculated using the formula, $Q = CC_f iA$. Following this, a family of hydrographs representing storms of greater duration are developed. The rising limb and falling limb of the hydrograph are, in each case, equal to t_c for the basin. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall multiplied by the duration of the rainfall.

Example 1 Modified Rational Method

Given: Area: A = 2.0 acres
Type of development: commercial parking lot, fully paved, C = 0.88
Time of concentration: t_c = 8 minutes
Design Frequency = 10 years
Use Intensity-Duration-Frequency Curves, Figure 2-1.

Find: Develop family of curves representing Modified Rational Method hydrographs for the 8-, 10-, 15-, 20-, 30- and 40-minute rainfall durations.

Solution:

$$Q_p = CC_f iA$$

Rainfall Duration (min)	Rainfall Intensity (in/hr)	Peak Runoff Rate (cfs)
8	6.37	11.2
10	5.82	10.2
15	4.92	8.7
20	4.46	7.9
30	3.54	6.2
40	3.14	5.5

The resulting storm hydrographs are depicted in Figure 8-3.

The next step in determining the necessary storage volume for the detention facility is to set a release rate and determine the volume of storage necessary to accomplish this release rate.

To determine the storage volume required, a reservoir routing procedure should be accomplished for each of the hydrographs, with the critical storm duration and required volume being determined. The importance of the particular project should govern the type of routing utilized. For small areas requiring repetitive calculations, such as parking lot bays, an assumed release curve is normally satisfactory. For larger areas, such as a pond in a small park with 20 acres or more of tributary area, a reservoir routing procedure would be more appropriate.

Figure 8-3 represents a method for small area detention analyses. The assumed release curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for the low release rate at the beginning of a storm and an increasing release rate as the storage volume increases.

In normal flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm of duration equal to the time of concentration for the basin. The release rate is held constant past this point. The storage volume is then found by determining the area between the inflow and release hydrographs. Example 2 continues the calculations initiated in Example 1 to determine the required storage volume.

The equation for the storm runoff volume, V_r, can be simplified as:

$$V_r = 60DQ_p \tag{8-4}$$

where:

V_r = storm runoff volume, in cubic feet

D = storm duration, in minutes

Q_p = peak runoff rate of the inflow hydrograph, in cubic feet per second

The equation for the required storage volume, V_s, can also be simplified as:

$$V_s = 60D(Q_p - Q_o) \tag{8-5}$$

where:

V_s = required storage volume, in cubic feet

D = storm duration, in minutes

Q_p = peak runoff rate of the inflow hydrograph, in cubic feet per second

Q_o = maximum release rate, in cubic feet per second

Example 2 Critical Storage Volume

Given: Drainage basin and other hydrologic information presented in Example 1.

Allowable release rate: $Q_o = 4.0$ cfs

Find: Determine the critical storage volume

Solution:

$$V_r = 60DQ_p$$

$$V_s = 60D(Q_p\text{-}Q_o)$$

Storm Duration (min)	Storm Runoff Volume (ft^3)	Required Storage Volume (ft^3)	
8	5,376	3,456	
10	6,120	3,720	
15	7,830	4,230	
20	9,480	4,680	Maximum
30	11,160	3,960	
40	13,200	3,600	

The critical storage volume is 4,680 cubic feet occurring for a 20-minute rainfall duration.

The limitations in the assumptions behind this method are evident. The approach becomes more valid on progressively smaller basins. The procedures should, therefore, be limited to relatively small areas where no major damage would result from overtopping or failure of the proposed detention facility. Care should be used when applying this method to areas in excess of 20 acres.

8.5.2 Hydrograph Procedure for Storage Analysis

The unit hydrograph procedure develops a hydrograph which provides a reliable solution for detention storage effects. The unit hydrograph procedure provides the engineer/designer greater flexibility for the representation of actual conditions to be modeled. The unit hydrograph procedure can be used for any size drainage area. For detention basin design, a minimum design storm duration of 24 hours should be used.

The development of the storm runoff hydrograph is presented in Section 2 of this Manual. A storm runoff hydrograph is presented in Figure 8-4 which represents inflow to a reservoir. The analysis for the reservoir storage must take into consideration the characteristics of the outlet pipe, the discharge of which is shown in Figure 8-4 as a solid line. The shape of the solid line reflects the carrying capacity of the outlet works with various headwater elevations. The higher the elevation of the water surface in the reservoir the greater the discharge through the outlet works. The area between the dashed line and the hydrograph of storm outflow can be planimetered to determine the volume of storage required to reduce channel flow from 200 cubic feet per second to 100 cubic feet per second.

For offstream storage the basic approach to the analysis is presented in Figure 8-4. In this case, the peak of the storm hydrograph is routed over a side channel spillway into a ponding area adjacent to the channel. The water removed from the channel, represented by the shaded area of the hydrograph, provides for a reduction in the peak channel flow from 200 cubic feet per second to about 100 cubic feet per second.

8.5.3 Modified Puls Routing Procedure

A flood routing procedure may be used to determine the required volume of the detention basin. Several flood routing procedures are available in published texts. One commonly used procedure is the Modified Puls. The data needed for this routing procedure are the inflow hydrograph, the physical dimensions of the storage basin, the maximum outflow allowed, and the hydraulic characteristics of the outlet structure or spillway.

To perform the Modified Puls procedure, the inflow hydrograph, depth-storage relationship, and depth-outflow relationship must be determined. They are then combined in a routing routine. The results of the routing are the ordinate of the outflow hydrograph, the depth of storage, and the volume of storage at each point in time of the flood duration.

The routing period, or time interval, Δt, is selected small enough so that there is a good definition of the hydrograph and the variation in the hydrograph during the period Δt is approximately linear. This can be accomplished by setting $\Delta t = 5$ minutes.

Several assumptions are made in this procedure and include the following:

A. The entire inflow hydrograph is known.

B. The storage volume is known at the beginning of the routing.

C. The outflow rate is known at the beginning of the routing.

D. The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the structure's hydraulic characteristics.

The derivation of the routing equation begins with the conservation of mass which states that the difference between the average inflow and average outflow during some time period Δt is equal to the change in storage during that time period. This can be written in equation form as:

$$I - O = \Delta S / \Delta t \tag{8-6}$$

where:

I	=	average inflow rate
O	=	average outflow rate
ΔS	=	change in storage volume
Δt	=	routing period

If inflow during the period is greater than outflow, then ΔS is positive and the pond gets deeper. If inflow is less than outflow during the period, then ΔS is negative and the pond gets shallower. Using the assumptions made previously, this equation can be rewritten as:

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \left[\frac{S_2 - S_1}{\Delta t}\right] \tag{8-7}$$

where:

I_1	=	inflow rate at time interval 1
I_2	=	inflow rate at time interval 2
O_1	=	outflow rate at time interval 1
O_2	=	outflow rate at time interval 2
S_1	=	storage volume at time interval 1
S_2	=	storage volume at time interval 2
Δt	=	routing period

Multiplying both sides by two and separating the right-hand side yields:

$$(I_1 + I_2) - (O_1 - O_2) = \left[2\frac{S_2}{\Delta t} - 2\frac{S_1}{\Delta t}\right] \tag{8-8}$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \left[2\frac{S_1}{\Delta t} - O_1\right] = \left[2\frac{S_2}{\Delta t} + O_2\right] \tag{8-9}$$

However, Equation 8-9 has two unknowns, S_2 and O_2. A second equation is needed which relates storage and outflow. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists between reservoir elevation, reservoir storage, and outflow. Therefore, for a particular elevation, there is an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t) + O$ is determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left-hand side. This yields a value for $(2S_2/\Delta t) + O_2$. This value is found on the log-log plot of $(2S/\Delta t) + O$ versus O.

The $(2S/\Delta t) + O$ versus O relationship is derived by combining the depth-storage relationship and the depth-outflow relationship, as previously discussed. This is shown in Table 8-2. Columns 1, 2, and 3 are tabulations of the depth-storage and depth-outflow relationships for a specific detention facility.

In column 4, the units of $2S/\Delta t$ and O must be the same. If O is in cubic feet per second, then $2S/\Delta t$ must be changed to cubic feet per second. For a routing time interval of 5 minutes:

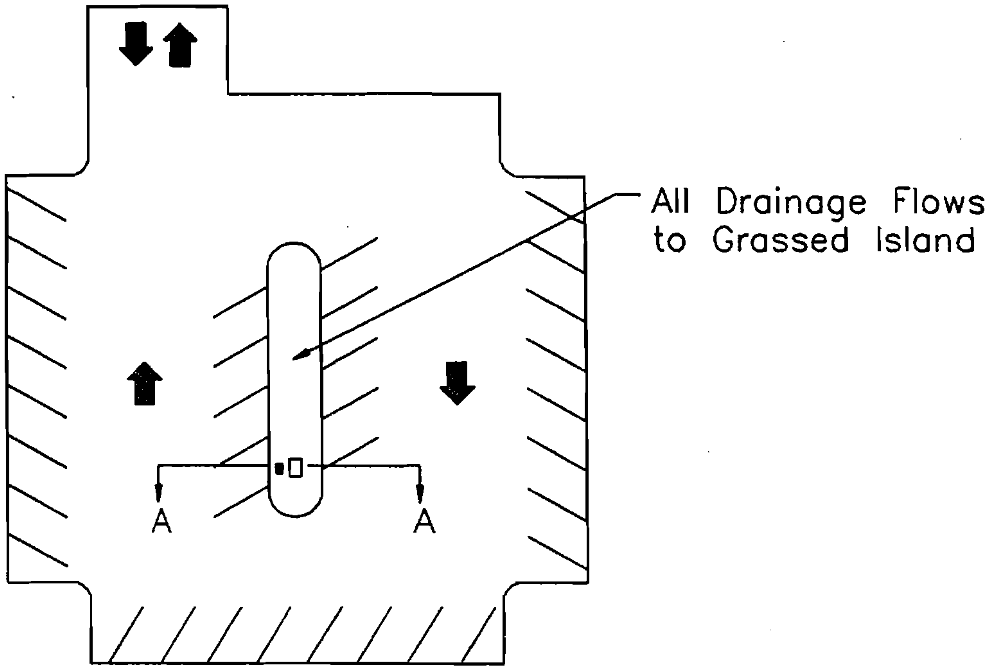
$$\frac{2\ S\ ac\text{-}ft}{5\ \text{min}} \ x\ \frac{1\ cfs\text{-}day}{1.98\ ac\text{-}ft}\ x\ \frac{1440\ \text{min}}{1\ day} = 291\ S$$

$$\text{Thus,}\quad \frac{S + O}{\Delta t} = 291\ S + O \quad \text{for } \Delta t = 5 \text{ min}$$

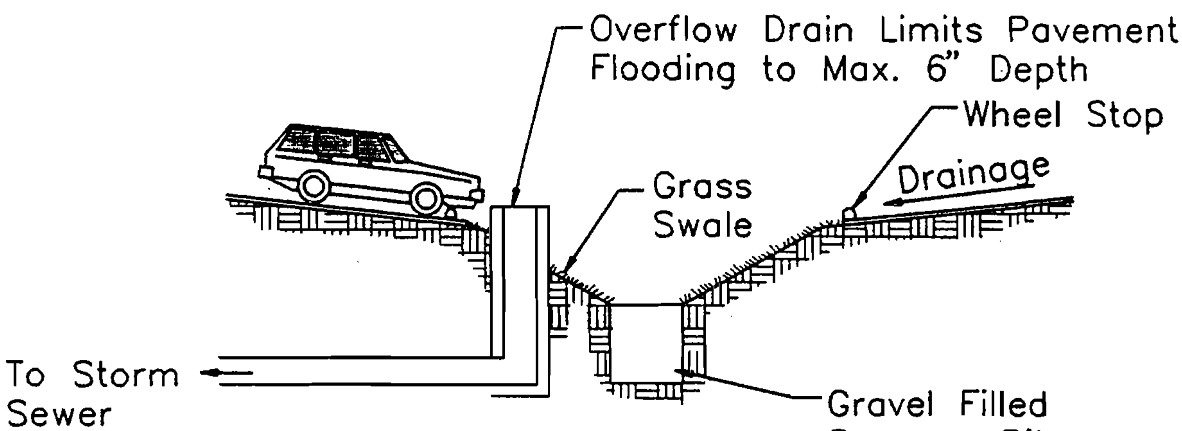
where: S has units of acre-feet, O has units of cfs, and $2S/\Delta t$ has units of cfs

TABLE 8-2 Development of a $(2S/\Delta t) + 0$ versus 0 Relationship

Depth (ft) (1)	Storage, S (acre-feet) (2)	Outflow, O (cfs) (3)	$(2S/\Delta t) + O$ (cfs) (4)
0	0.0	0	0
2	0.1	40	69
4	0.6	138	313
6	3.0	274	1,147
8	11.0	426	3,627
10	32.0	560	9,872
12	72.0	671	21,623
14	131.0	765	38,886



Plan
No Scale



Section A–A
No Scale

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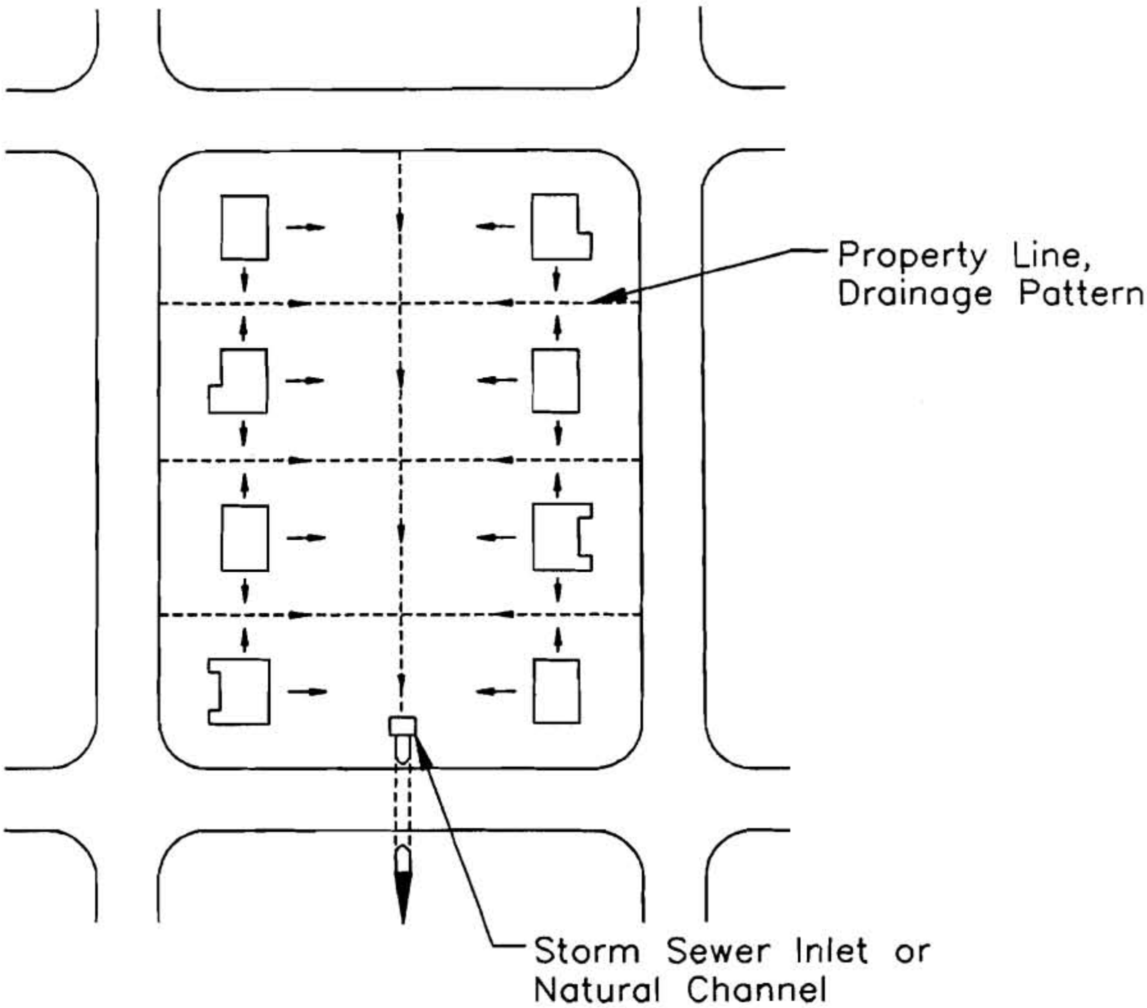
Parking Lot Detention

Storm Water Management Criteria Manual

Date
NOV 1992

Figure
8–1



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Property Line Swale

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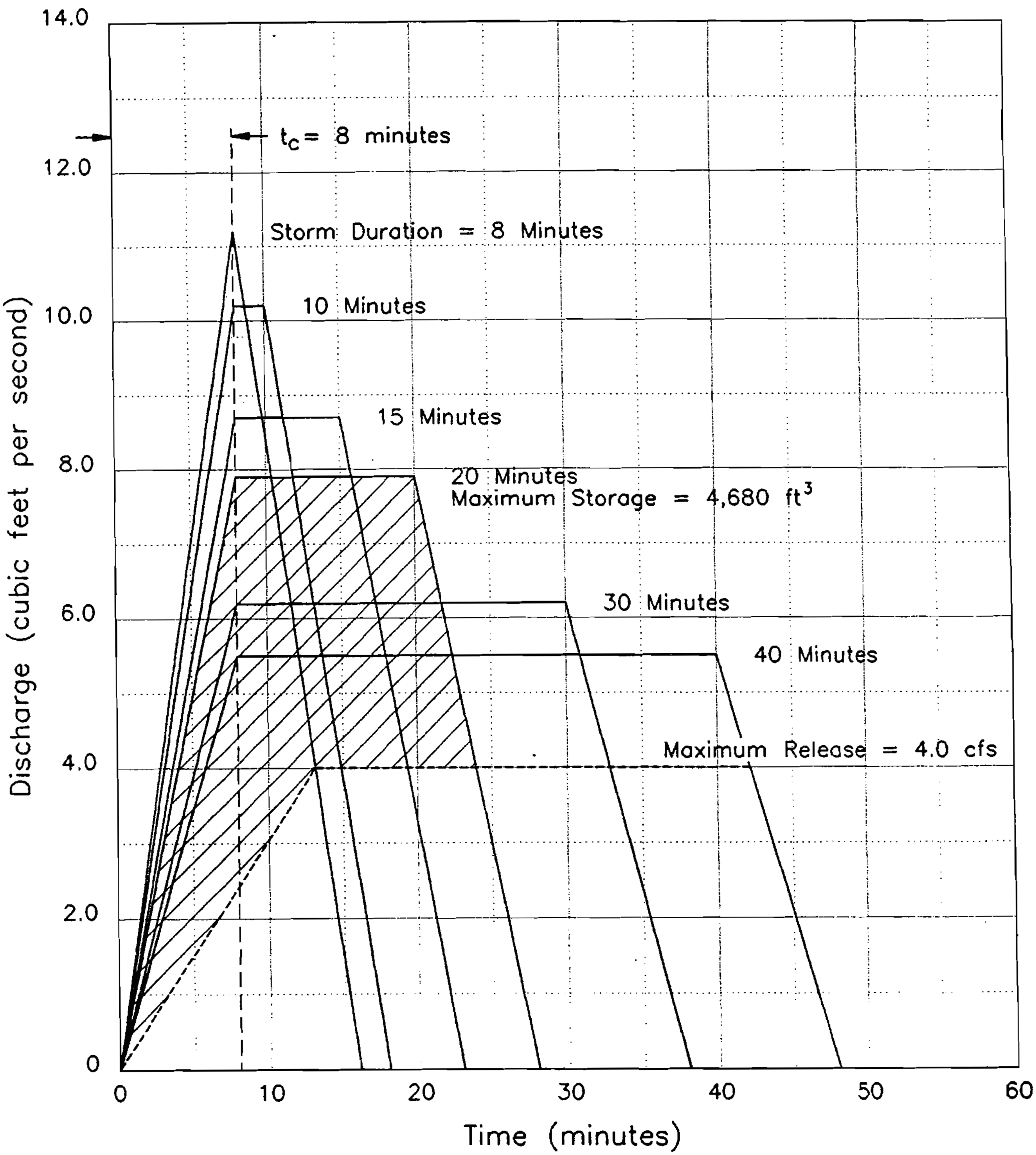
Figure
8-2



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Modified Rational Method Analysis
Example Problem 2

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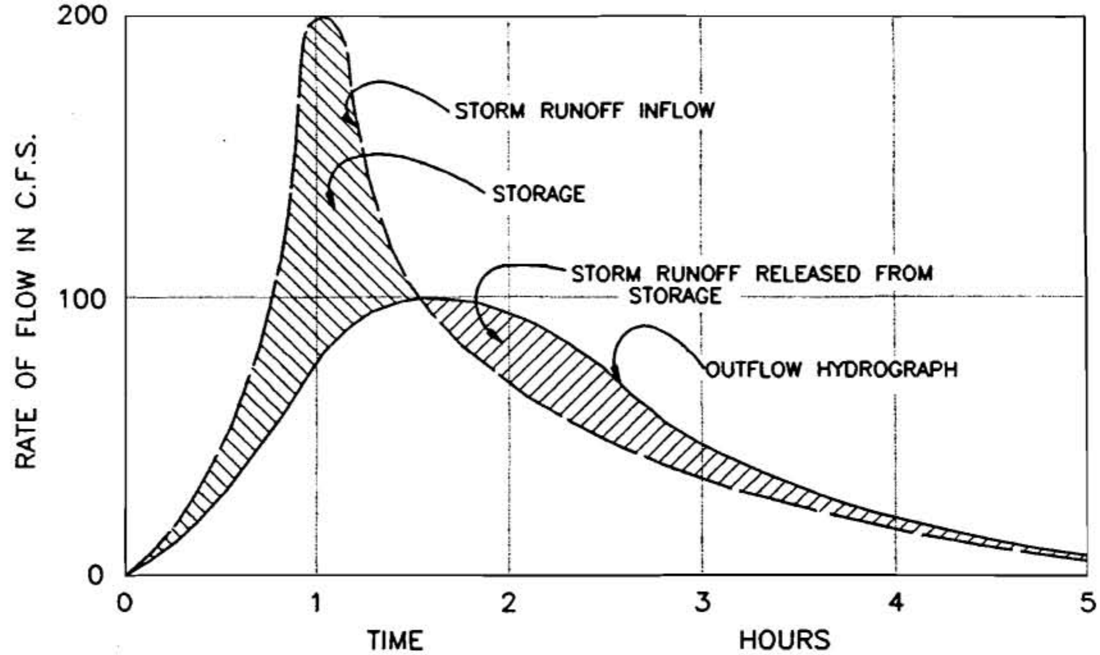
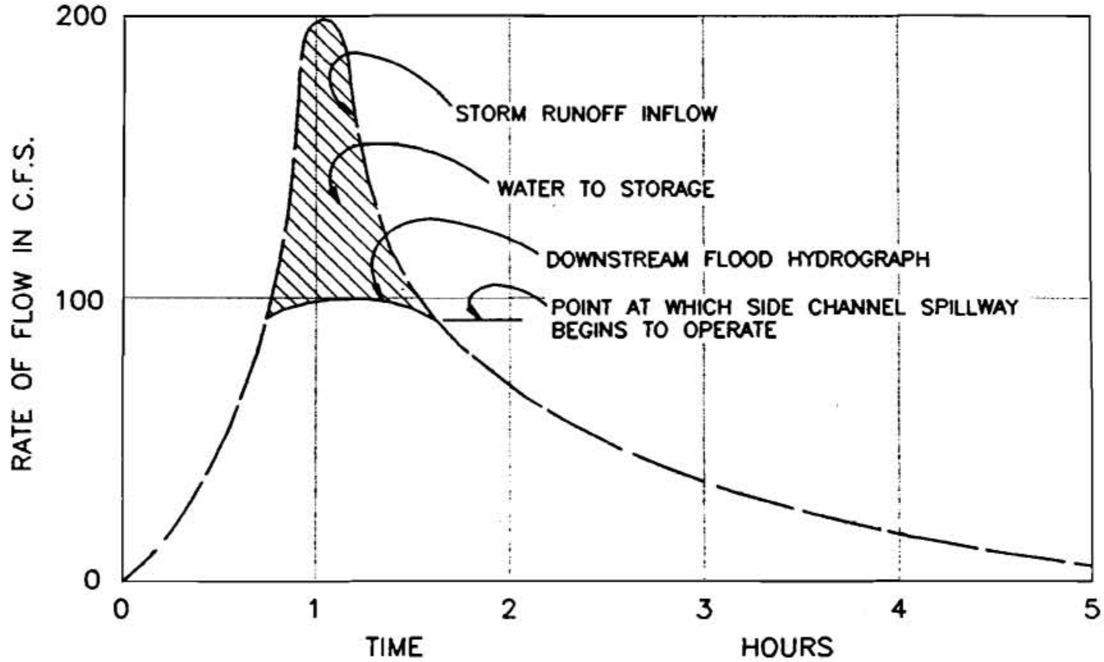
Figure	8-3



Effect of Onstream Reservoir on Storm Runoff Hydograph



Effect of Offstream Reservoir on Storm Runoff Hydograph

Source: Urban Drainage and Flood Control District.

CITY OF AMARILLO

Effects of Onstream and Offstream Reservoir on Storm Runoff Hydrograph

Storm Water Management Criteria Manual

Date NOV 1992

Figure 8-4

8.6 BIBLIOGRAPHY

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9.1 INTRODUCTION

Sedimentation involves three basic processes: erosion, transportation, and deposition. These are natural geologic phenomena which have been in continuous operation since the beginning of time. Man's land development activities; however, have initiated severe, highly undesirable, and damaging alterations in the natural sedimentation cycle by drastically accelerating the erosion-sedimentation process.

Erosion

This term includes all of the processes by which soil or rock material is loosened and removed, that is, weathering, solution, downcutting, and transportation. Soil erosion is usually caused by the force of water falling as raindrops and by the force of water flowing in rills and streams. The raindrops falling on bare or sparsely vegetated soil detach soil particles but have little capacity for transporting them. Water running in a sheet on the surface of the ground picks up these particles and carries them along as it flows downhill towards a stream system. As the runoff gains in velocity and concentration, it detaches more soil particles, cuts rills and gullies into the surface of the soil, and adds to its sediment load. Coalescing rivulets produce streams which have a larger volume and usually increased velocity; hence, a greater capacity to remove sediment and transport it downstream. The greater the distance the water runs uncontrolled, the greater its erosive force and the greater the resultant damage. Moreover, control becomes increasingly more difficult as the distance and volume increase.

Factors Influencing Erosion

The erosion potential of a site is principally determined by the erodibility of the soil, vegetative cover, topography, climate and season. Although the factors are interrelated as determinants of erosion potential, they are discussed separately for ease of understanding.

The vulnerability of a soil to erosion is known as erodibility. The soil structure, texture, and percentage of organic matter influence its erodibility. The most erodible soils generally contain high proportions of silt and very fine sand. The presence of clay or organic matter tends to decrease soil erodibility. Clays are sticky and tend to bind soil particles together. Organic matter helps maintain stable soil structure.

There are several ways in which vegetation protects soil from the erosive forces of raindrop impact and runoff scour. The top growth shields the soil surface from raindrop impact while the root mass holds soil particles in place. Grass buffer strips can be used to filter sediment from the surface runoff. Grasses slow the velocity of runoff which results in sedimentation, and also helps maintain the infiltration capacity of the soil. The establishment and maintenance of vegetation can be most effective in minimizing erosion during development.

Slope length and steepness are key influences on both the volume and velocity of surface runoff. Long slopes deliver more runoff to the base of slopes and steep slopes increase runoff velocity; both conditions enhance the potential for erosion to occur.
Erosion potential is also affected by the climate of the area. Rainfall characteristics, such as frequency, intensity, and duration directly influence the amount of runoff that is generated. As the frequency of rainfall increases, water has less change to drain through the soil between storms. The soil will remain saturated for, longer periods of time and storm water runoff volume may be potentially greater. Therefore, when rainfall events are frequent, intense, or lengthy, erosion risks are high.

Seasonal variation in wind, humidity, temperature and rainfall defines periods of high erosion potential during the year. A high erosion potential may exist in the spring when the surface soil first thaws and the ground underneath remains frozen. A low intensity rainfall may cause substantial erosion as infiltration is impossible because of the frozen subsoil. The erosion potential is also high during the summer months because of more frequent, high intensity rainfall.

9.2 STANDARDS FOR EROSION AND SEDIMENT CONTROL

The principles of reducing erosion and sedimentation from developing areas are:

A. Plan the development to fit the particular topography, soils, waterways, and natural vegetation at the site.

Initially, this is best achieved through adoption of a general land-use plan based upon a comprehensive inventory of soil, water, and related resources.

Slope length and gradient are key elements in determining the volume and velocity of the runoff and its associated erosion. As both slope length and steepness increase, the rate of runoff increases and the potential for erosion is magnified. Where possible, steep slopes should be left undisturbed. By limiting the length and steepness of the designed slopes, runoff volumes and velocities can be reduced and erosion hazards minimized.

Soils which contain a high proportion of silt and very fine sand are generally the most erodible. The erodibility of these soils is decreased as the percentage of clay organic matter content increases. Well-drained and well-graded gravel-sand mixtures with little silt are the least erodible soils. By reducing the length and steepness of a given slope, even a highly erodible soil may show little evidence of erosion. Long steep slopes should be broken by benching, or constructing diversion structures.

The natural vegetative cover is extremely important in controlling erosion since it: 1) shields the soil surface from the impact of falling rain; 2) increases infiltration of water into the soil; 3) reduces the velocity of the runoff water; and 4) holds soil particles in place while filtering surface runoff.

B. Keep disturbed areas small.

When earthwork is required and the natural vegetation is removed, keep the area and the duration of exposure to a minimum. Plan the phases or stages of development so that only the areas which are actively being developed are exposed. All other areas should have a good cover of temporary or permanent vegetation or mulch. Grading should be completed as soon as possible after it is begun. Minimizing grading of large or critical areas during the season of maximum erosion potential (May through October) reduces the risk of erosion.

C. Protect disturbed areas from storm water runoff.

This principle requires practices that control erosion on a site to prevent excessive sediment from being produced. Practices which keep soil covered as much as possible with temporary or permanent vegetation or with various mulch materials are best. Special grading methods such as roughening a slope on the contour or tracking with a cleated dozer may be used. Immediately after grading is complete, permanent vegetative cover should be established in the area. As cut slopes are made and as fill slopes are brought up to grade, these areas should be revegetated as the work progresses. Other practices include diversion structures to divert surface runoff from exposed soils and grade stabilization structures to control surface water.

Gross erosion in the form of gullies must be prevented by these control devices. Lesser types of erosion such as sheet and rill erosion should be prevented. When erosion is not adequately controlled, sediment control is more difficult and expensive.

D. Retain sediment within the site boundaries.

This principle relates to using practices that control sediment once it is produced and prevents it from leaving the site. Diversion ditches, sediment traps, vegetative filters, and sediment basins are examples of practices to control sediment. Vegetative and structural sediment control measures can be classified as either temporary or permanent depending on whether or not they will remain in use after development is complete. Generally, sediment can be retained by two methods: 1) filtering runoff as it flows through an area and 2) impounding the sediment-laden runoff for a period of time so that the soil particles are deposited. The best way to control sediment, however, is to prevent erosion.

E. Implement a thorough maintenance and follow-up program.

This principle is vital to success. A site cannot be effectively controlled without thorough, periodic checks of the control practices. An example of applying this principal would be to start a routine "end-of-day check" to ensure all control practices are working properly.

These five principles are integrated into a system of vegetative and structural measures, along with management techniques, to develop a plan to prevent erosion and provide sediment control. In most cases, a combination of limited grading, limited time of exposure, and a judicious selection of erosion control practices and sediment-trapping facilities will prove to be the most practical method of controlling erosion and the associated production and transport of sediment.

After the development process begins, effective erosion and sedimentation control depends upon careful, accurate installation in a timely fashion, and sufficient maintenance to ensure the intended results.

9.3 THE SEDIMENT CONTROL PLAN

The required Sediment Control Plan is a plan for controlling erosion and sediment during construction in compliance with the laws, ordinances, and these Standards. This plan shall be a part of the total site development plan and prescribes all the steps necessary, including scheduling, to assure erosion and sediment control during all phases of construction including final stabilization.

Planning for sediment control should begin with the conceptual plan and its preparation. Such features as soils and topography should be considered for the conceptual plan as well as any requirements for sediment control or storm water management.

Planning for sediment control should also begin with first-hand knowledge of the site by the designer. The plan shall be based on a sufficiently accurate topographic map that reflects the existing topography and site conditions. Adjacent areas affecting the site or affected by the site and its development shall be shown on the plans in sufficient detail to accomplish the need. Examples of this would be areas draining onto the site or areas where storm runoff leaves the site and travels to a stream or drainage system.

The Sediment Control Plan will consist of the best selection of erosion control practices and sediment-trapping facilities, in conjunction with an appropriate schedule, to accomplish an adequate level of control. Particular attention must be given to concentrated flows of water, either to prevent its occurrence or to provide conveyance devices according to the Standards to prevent "major" or "gross" erosion. Sediment-trapping devices will usually be required at all points of egress of sediment-laden water. The plan must include permanent structures for conveying storm runoff, final site stabilization, removal of temporary sediment control features such as sediment basins, and finally, stabilization of the sites where temporary features were removed. Plans showing improvements or construction to be done outside the property line for the site will generally not be approved unless the plan is accompanied by an appropriate legal easement for the area in which the work is to be done.

The standardization of sediment control plans makes them easier to study and review. The List of Standard Symbols (Figure 9-1) was developed to facilitate plan review. The symbols should be bold and easily identified on the plans.

Unless otherwise approved, one of the following scales shall be used for the detailed sediment control plans for urban development sites: 1" = 20', 1" = 30', 1" = 40', or 1" = 50'. The contour interval for these plans shall be 2 feet or smaller.

The Sediment Control Plan shall include the existing and proposed topography. Existing topography can be either from actual field survey obtained from approved photogrammetric methods or from information obtained from responsible agencies. No proposed slopes will exceed 2H:1V. All slopes steeper than 3H:1V will require low-maintenance stabilization.

The existing and proposed improvements shall be shown on the sediment control plan and will include all buildings, roads, storm drains, etc. Proposed removal or alterations of existing facilities shall be indicated on the plan.

9.3.1 Sediment Control Practices

All sediment control practices must be identified on the Sediment Control Plan. These practices will be shown in sufficient detail to facilitate implementation. All permanent sediment control structures will be labeled on the plan as PERMANENT. All temporary stabilization practices will be labeled on the plan as TEMPORARY. The location and methods of stabilization will be indicated on the Plan.

A schedule, or sequence, of operations will be included on the Sediment Control Plan. Special emphasis will be placed on the scheduled start of clearing and/or grading, sequence of installation of sediment control and storm water management facilities, duration of exposure, and the scheduled start and completion dates of stabilization measures (both temporary and permanent).

9.3.2 Drainage Plan

A Drainage Plan shall be provided as per Sections 2 and 3. Based on this Plan, indicate the velocity for: 1) pipe outfall, 2) outfall structure, and 3) natural or designed channel below outfall structures to point to entry into existing system or natural stream. On Sediment Control Plan show the proposed method of stabilizing the outfall, consistent with computed velocities.

9.4 STANDARDS FOR STRUCTURAL PRACTICES

This section describes several control measures which are available for use in controlling erosion and sedimentation. The design is encouraged to review the Soil Conservation Service publications, <u>Erosion and Sediment Control Guidelines in Developing Areas in Texas</u>[18], and <u>Texas Handbook Section 17, Erosion Control Practices</u>[19], for additional control measures.

9.4.1 Straw Bale Barrier

Definition

A temporary barrier of straw or similar material used to intercept sediment laden runoff from small drainage areas of disturbed soil. Figure 9-2 is a typical straw bale barrier.

Purpose

The purpose of a straw bale barrier is to reduce velocity and effect deposition of the transported sediment load. Straw bale barrier are to be used to intercept and detain small amounts of sediment from unprotected areas of less than 1/2 acre.

Application

The straw bale barrier is used where:

A. Contributing area is approximately 1/2 acre, or less.

B. There is no concentration of water in a channel or other drainage way above the barrier.

C. Erosion would occur in the form of sheet or rill erosion.

D. Length of slope above the straw bale dike shall not exceed 100 feet.

Straw bales must not be used on high sediment producing areas above "high risk" areas, where water concentrates, or where there would be a possibility of a washout.

Design Criteria

A design is not required. All bales shall be placed on the contour and shall be either wire bound or nylon-string tied. Bales shall be laid with the cut edge adhering to the ground and staked in place. At least two wooden or metal stakes shall be driven through each bale and into the ground at least one foot. The first stake shall be angled toward the previously placed bale and driven through both the first and second bale. Stakes shall be driven flush with the bale.

The possibility of piping failure shall be reduced by setting the straw bales in a trench excavated to a depth of at least four (4) inches and by firmly tamping the soil along the upstream face of the barrier.

9.4.2 Silt Fence

Definition

A silt fence is a temporary barrier made of geotextile fabric which is water-permeable but will trap water-borne sediment from small drainage areas of disturbed soil, as shown in Figure 9-3.

Purpose

The purpose of a silt fence is to reduce runoff velocity and effect deposition of transported sediment load. Limits imposed by ultraviolet stability of the fabric will dictate the maximum period the silt fence may be used.

Application

A silt fence may be used subject to the following conditions:

A. Maximum allowable slope lengths contributing runoff to a silt fence are listed in the Table 9-1.

TABLE 9-1 Silt Fence Slope Criteria

Constructed Slope	Maximum Slope Length (feet)
2H:1V	25
2.5H:1V	50
3H:1V	75
3.5H:1V	100
4H:1V	125
Flatter than 5H:1V	200

B. Maximum drainage area for overland flow to a silt fence shall not exceed 0.5 acre per 100 feet of fence.

C. Erosion would occur in the form of sheet erosion.

D. There is no concentration of water flowing to the barrier.

Design Criteria

Design computations are not required for a silt fence design. All silt fences shall be placed as close to the contour as possible. The filter fence shall be placed and constructed in such a manner that runoff from a disturbed upland area shall be intercepted, the sediment trapped, and the surface runoff allowed to percolate through the structure. The bottom of the fabric should be buried in a 6 inch by 6 inch trench. When a trench cannot be constructed, rock and soil shall be placed over the bottom of the fabric in such a manner as to prevent underflow.

A detail of the silt fence shall be shown on the plan, and contain the following minimum requirements:

A. The type, size, and spacing of fence posts;

B. the size of woven wire support fence;

C. the type of filter cloth used;

D. the method of anchoring the filter cloth; and

E. the method of fastening the filter cloth to the fencing support.

Where ends of filter cloth join they shall be overlapped, folded and stapled to prevent sediment bypass.

A. Silt Fence Fabric

The fabric shall meet the specifications in Table 9-2. Type W fabric is a Type 1 self-supported fence. Type NW is a nonwoven fabric which is used in a Type 2 net-reinforced fence or Type 3 triangular filter dike. Either fabric may be manufactured from polyester, polypropylene or polyamide and shall be resistant to ultraviolet degradation, mildew or rot. The edges of woven fabric shall be sealed or selvaged to prevent raveling.

TABLE 9-2 Silt Fence Fabric Criteria

Fabric Properties	Minimum Acceptable Value		Test Method
	Type W	Type NW	
Tensile Strength, lb	100	90	ASTM D4632
Elongation at Yield, %	10-40	100 Max	ASTM D4632
Trapezoidal Tear, lb	50	35	ASTM D4533
Permittivity, 1/sec	0.1	1.0	ASTM D4491
Apparent Opening Size	20-50	50-80	ASTM D4751
Ultraviolet Stability, %	80	80	ASTM D4355

B. Fence Reinforcement Materials

Silt fence reinforcement shall be one of the following systems.

1. Type 1: Self-Supported Fence

This system consists of fence posts, spaced no more than 8 1/2 feet apart, and Type W fabric without net reinforcement. Fence posts shall be a minimum of 42 inches long, embedded at least 1 foot, and constructed of either wood or steel. Soft wood posts shall be at least 3 inches in diameter or nominal 2 x 4 inches. and essentially straight. Hardwood posts shall be a minimum of 1.5 x 1.5 inches. Fabric attachment may be by staples or locking plastic ties at least every 6 inches, or by sewn vertical pockets. Steel posts shall be T or L shaped with a minimum weight of 1.3 pounds per foot. Attachment shall be by pockets or by plastic ties if the posts have suitable projections.

2. Type 2: Net-Reinforced Fence

   This system consists of fence posts, spaced no more than 8-1/2 feet apart, and Type NW fabric with an attached reinforcing net. Net reinforcement shall be galvanized welded wire mesh of at least 12.5-gauge wire with maximum opening size of 4 inches square. The fabric shall be attached to the top of the net by crimping or cord at least every 2 feet, or as otherwise specified.

3. Type 3: Triangular Filter Dike

   This system consists of a rigid wire mesh, at least 6-gauge, formed into an equilateral triangle cross-sectional shape with sides measuring 18 inches, wrapped with Type NW silt fence fabric. The fabric shall be continuously wrapped around the dike, with a skirt extending at least 12 inches from its upslope corner.

C. Prefabricated Units

Envirofence or approved equal may be used in lieu of the above method providing the unit is installed per manufacturer's instructions.

9.4.3 Stabilized Construction Entrance

Definition

A stabilized pad of aggregate located at any point where traffic will be entering or leaving a construction site to or from a public right-of-way, street, alley, sidewalk, or parking area.

Purpose

The purpose of a stabilized construction entrance is to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way or streets.

Application

A stabilized construction entrance applies to all points of construction ingress and egress.

Design Criteria

A design is not required for a stabilized construction entrance, however, the following criteria in Table 9-3 shall be used.

TABLE 9-3 Stabilized Construction Entrance Design Criteria

Aggregate:	Use 2 inch stone, or reclaimed or recycled concrete equivalent
Thickness:	Not less than six (6) inches
Width:	Not less than full width of all points of ingress and egress Twenty (20) foot minimum
Length:	As required, but not less than 50 feet

Maintenance

The entrance shall be maintained in a condition which will prevent tracking or flowing of sediment onto public rights-of-way or streets. This may require periodic top dressing with additional aggregate as conditions demand and repaired and/or cleaned of any measures used to trap sediment. All sediment spilled, dropped, washed, or tracked onto public rights-of-way must be removed immediately.

When necessary, wheels must be cleaned to remove sediment prior to entrance onto public right-of-way. When washing is required, it shall be done on an area stabilized with crushed stone which drains into an approved sediment trapping device. All sediment shall be prevented from entering any storm drain, ditch, or watercourse.

9.4.4 Sediment Basin

Definition

A sediment basin is constructed across a waterway or at other suitable locations to collect and store debris or sediment.

Purpose

The purpose of a sediment basin is to preserve the capacity of reservoirs, ditches, canals, diversions, waterways, and streams; to prevent undesirable deposition on bottom lands and developed areas; to trap sediment originating from construction sites; and to reduce or abate pollution by providing basins for deposition and storage of silt, sand, gravel, stone, agricultural wastes, and other detritus.

Application

This practice applies where physical conditions, land ownership or other restrictions preclude the treatment of a sediment source by the installation of erosion-control measures to keep soil and other material in place, or where a sediment basin offers the most practical solution to the problem.

Design Criteria

A. Compliance with Laws and Regulations

Design and construction shall comply with state and local laws, ordinances, rules, and regulations. The designer is cautioned that water impounding structures higher than six (6) feet may be considered dams and is encouraged to contact the Texas Water Commission regarding applicable rules.

B. Location

The sediment basin should be located to obtain the maximum storage benefit from the terrain and for ease of cleanout of the trapped sediment. It should be located to minimize interference with construction activities and construction of utilities.

C. Size of the Basin

The capacity of the sediment basin, as measured from the bottom of the basin to the elevation of the crest of the principal spillway, shall equal or exceed the trapped volumes of debris or sediment expected to be trapped at the site during the planned useful life of the structures or improvements it is designed to protect. The minimum capacity provided shall be in accordance with criteria in Texas Handbook, Erosion Control Practices, Section 17[19].

The Universal Soil Loss Equation (USLE) can be used to determine the size of the sediment basin. The USLE determines the gross sheet and rill erosion (tons/ac./yr). The actual sediment yield at the point of concern (sediment basin) is the gross erosion minus the sediment deposited enroute. The ratio of sediment yield to gross erosion can be estimated from relationships discussed in the SCS publication NEH-Chapter 6, Sedimentation.

The USLE equation is defined by six (6) factors. The designer should consult the Soil Conservation Service's Technical Release No. 51, Erosion Handbook Water and Wind[3], and USDA Handbook No. 537, for the proper tables and figures. The Universal Soil Loss Equation is defined by Equation 9-1.

$$A = RKLSCP \tag{9-1}$$

where:

A = sediment yield, in tons per acre per year

R = rainfall factor, R = 110 for Amarillo, Texas

K = soil erodibility factor, $0.05 \le K \le 0.41$

L = slope length factor

S = slope gradient factor

C = cropping management factor, $0.001 \leq C \leq 0.99$

P = erosion control practice factor, $0.10 \leq P \leq 1.0$

Sediment basins shall be cleaned out when the capacity as described above is reduced by sedimentation to 60% full, except in no case shall the sediment level by permitted to build up higher than one (1) foot below the principal spillway crest. At this elevation, cleanout shall be performed to restore the original design volume to the sediment basin. The elevation corresponding to the maximum allowable level shall be determined and shall be stated in the design data as a distance below the top of the riser and shall be clearly marked on the riser.

The basin dimensions necessary to obtain the required basin volume as stated above shall be clearly shown on the plans to facilitate plan review, construction, and inspection.

The Sediment Basin Plan shall indicate the method(s) of disposing of the sediment removed from the basin. The sediment shall be placed in such a manner that it will not erode from the site. The sediment shall not be deposited downstream from the basin or adjacent to a stream or floodplain.

The sediment basin plans shall also show the method of disposing of the sediment basin after the drainage area is stabilized, and shall include the stabilizing of the sediment basin site. Water lying over the trapped sediment shall be removed from the basin by pumping, cutting the top of the riser, or other appropriate methods prior to removing or breaching the embankment. Sediment shall not be allowed to flush into the stream or drainageway.

D. Entrance of Runoff into Basin

Points of entrance of surface runoff into excavated sediment basins shall be protected to prevent erosion. Diversions, grade stabilization structures or other water control devices shall be installed as necessary to ensure direction of runoff and protect points of entry into the basin.

E. Principal Spillways

A pipe spillway is recommended on all basins. The pipe spillway shall consist of a vertical pipe riser or box riser joined to a conduit which will extend through the embankment and outlet below the downstream toe of the fill.

The pipe spillway shall be proportioned to convey not less than 0.2 cfs per acre of drainage area without causing flow through the emergency spillway. The minimum size pipe shall be 4 inches in diameter. The vertical pipe riser or box riser shall have a cross-sectional area at least 1.5 times that of the pipe.

One anti-seep collar shall be installed around the pipe when any of the following condition exist:

1. The settled height of the dam exceeds 15 feet;

2. the conduit is of smooth pipe larger than 8 inches in diameter; or,

3. the conduit is of corrugated metal pipe larger than 12 inches in diameter.

The anti-seep collars and their connection to the pipe shall be watertight. Protection against scour at the discharge end of the spillway shall be provided.

Trash racks shall be installed where needed.

F. Earth Emergency Spillways

All debris basins shall have an earth emergency spillway unless the peak flow from the design storm is carried through a pipe spillway or other mechanical spillway. The earth spillway shall be excavated in undisturbed earth or compacted fill. The spillway shall be designed to be stable for the design flow.

Peak discharges for design of the emergency spillway shall be computed using an accepted method and shall be based on the soil and anticipated cover conditions in the drainage area during the expected life of the structure.

The crest of the emergency spillway shall be at least 0.5 foot above the crest of the principal spillway.

For debris basins with 20 acres or less watershed, the combined capacities of pipe and emergency spillways shall be sufficient to convey the peak discharge from the 10-year, 24-hour frequency storm. For debris basins with watersheds greater than 20 acres, the combined capacity of pipe and emergency spillway shall be adequate to convey the peak discharge from the 25-year, 24-hour storm.

The top of a dam for all debris basins shall be at least 0.5 foot higher than the stage reached by the designed storm.

The crest elevation of the emergency spillway will be determined by the head required on the principal spillway. The minimum top width shall be as per Table 9-4.

y o

TABLE 9-4 Minimum Top Width Embankment (Earth Fill)

Height of Dam	Top Width
10 feet or less	6 feet
10-14	8 feet
14-20	9 feet

Source: Soil Conservation Services Erosion and Sediment Control Guidelines for Developing Areas in Texas.

G. Safety

Sediment basins are attractive to children and can be very dangerous. Therefore they shall be fenced or otherwise secured unless this is deemed unnecessary due to the remoteness of the site or other circumstances. In any case, local ordinances and regulations regarding health and safety must be adhered to.

9.4.5 Diversion

Definition

A drainageway of parabolic or trapezoidal cross section with a supporting ridge on the lower side that is constructed across the slope.

Purpose

The purpose of a diversion is to intercept and convey runoff to stable outlets at non-erosive velocities. Temporary diversions are installed as an interior measure to facilitate some phase of construction and usually have a life expectancy of 1 year or less. A permanent diversion is an integral part of an overall water disposal system and remains for protection of property.

Application

Diversions are used where:

A. Runoff from higher areas is or has potential for damaging properties causing erosion or interfering or preventing the establishment of vegetation on lower areas.

B. Surface and shallow subsurface flow caused by seepage is damaging sloping upland.

C. The length of slopes need to be reduced so that soil loss will be kept to a minimum.

D. Required as a part of a pollution abatement system.

E. To control erosion and runoff on urban or developing areas and construction sites.

Design Criteria

The design procedures for trapezoidal channels are provided in Section 6.

A. Location

Diversion location shall be determined by considering outlet conditions, topography, land use, soil type, length of slope, and the layout of the proposed development. Avoid locations in or immediately below unstable or highly erosive soils, unless special treatment or stabilization measures are previously applied.

B. Capacity

Peak runoff values used in determining the capacity requirements shall be determined as outlined in Section 2. The minimum design 24-hour storm frequencies and freeboard shall comply with criteria in Table 9-5.

Diversions designed to protect urban area, buildings and roads, and those designed to function in connection with other structures, shall have enough capacity to carry the peak runoff expected from a storm frequency consistent with the hazard involved.

TABLE 9-5 Diversion Frequency and Freeboard

Diversion Type	Typical Areas of Protection	Design Frequency (Years)	Freeboard Required (Feet)
Temporary	Construction roads; land areas, etc.	2	0.0
	Building Sites	5	0.0
Permanent	Land areas; playfields, recreation areas, etc.	25	0.3
	Homes, schools, industrial bldg., etc.	50	0.5

Source: Soil Conservation Service, Erosion and Sediment Control Guidelines for Developing Areas in Texas.

C. Velocity and Grade

Channel grades may be uniform or variable. Maximum permissible velocities of flow for the stated conditions of stabilization are shown in Tables 9-6 and 9-7.

TABLE 9-6 Selection of Vegetal Retardance

Average Length of Vegetation (inches)	Retardance	
	Good Stand	Fair Stand
11-24	B	C
6-10	C	D
2-6	D	D

Source: Soil Conservation Service, Erosion and Sediment Control Guidelines for Developing Areas in Texas.

TABLE 9-7 Permissible Velocities

	Permissible Velocity (fps)				
		Channel Vegetation			
Soil Texture	Bare Channel	Retardance	Poor	Fair	Good
Sand, silt		B		3.0	4.0
Sandy loam	1.5	C	1.5	2.5	3.5
Silty loam		D		2.0	3.0
Silty clay loam		B		4.0	5.0
Sandy clay loam	2.0	C	2.5	3.5	4.5
		D		3.0	4.0
Clay		B		5.0	6.0
	2.5	C	3.0	4.5	5.5
		D		4.0	5.0

Source: Soil Conservation Service, Erosion and Sediment Control Guidelines for Developing Areas in Texas.

D. Cross Section

The channel may be parabolic, V-shaped or trapezoidal in shape. The diversion is to be designed to have stable side slopes. The side slopes for permanent diversions should not be steeper than 3H:1V for maintenance purposes and preferably 4H:1V. The back slope of the ridge is not to be steeper than 3H:1V and preferably 4H:1V. The ridge after settlement is to be four (4) feet at the design water elevation. In determining the cross section on temporary diversions, consideration should be given to soil type, frequency and type of equipment that is anticipated to be crossing the diversion. In no case should side slopes be steeper than 1H:1V.

E. Outlets

Each diversion must have an adequate outlet. The outlet may be a grassed waterway, vegetated or paved area, grade stabilization structure, stable watercourse, or tile outlet. In all cases the outlet must convey runoff to a point where outflow will not cause damage. Vegetative outlets shall be installed prior to, and have vegetation established before diversion construction.

Underground outlets consist of an inlet and underground conduit, and the release rate when combined with storage is to be such that the design storms will not encroach on the design freeboard of the diversion ridge.

All areas where vegetation has been disturbed during construction and all other earth construction where vegetation is included in design, shall be seeded following completion of construction.

9.4.6 Grassed Waterway or Outlet

Definition

A natural or man-made drainageway or parabolic or trapezoidal cross section that is below adjacent ground level and is stabilized by suitable vegetation for the safe disposal of runoff or water.

Purpose

The purpose of a grassed waterway or outlet is to convey runoff from terraces, diversions, or from natural concentrations without causing damage from erosion or flooding.

Application

Grass waterways and outlets are used on sites where added capacity or vegetative protection, or both, are required to control erosion resulting from concentrated runoff. In short reaches of the grassed waterways or outlet where vegetation is not suitable for non-erosive disposal of runoff, other linings may be used to control erosion.

Grassed waterways are used where added channel capacity or stabilization is required to control erosion resulting from concentrated runoff and where such control can be achieved by this practice along or in combination with others.

Design Criteria

A. Compliance with Laws and Regulations

Planning and construction shall be in compliance with state and local laws and regulations. Such compliance is the responsibility of the landowner or developer.

B. Capacity

The minimum capacity is to be that required to convey the peak runoff expected from a 24-hour, 10-year frequency storm. Channel dimensions may be determined from Section 6.

C. Velocity

The design velocity is to be based upon soil, duration of flow, and type and quantity of vegetation. The maximum design velocity should be 4.0 feet per second for vegetation established by seeding and 6.0 feet per second for that established by sodding.

D. Cross Section

The cross section may be parabolic, trapezoidal, or triangular in shape. The bottom width of trapezoidal waterways or outlets shall not exceed 100 feet unless multiple or divided waterways are provided to control meandering of low flows.

The minimum depth of a waterway receiving water from diversions or tributary channels is to be that required to keep the design water surface in the waterway or outlet at or below the design water surface elevation in the diversion or other tributary channel at their junction. To provide for loss in channel capacity due to vegetal matter accumulation, sedimentation, and normal seedbed preparation, the channel depth and width should be increased proportionally to maintain the hydraulic properties of the waterway. In parabolic channels, this may be accomplished by adding 0.3 foot to the depth and 2 feet to the top width of the channel. This is not required on waterways located in natural watercourses.

Where a paved bottom is used in combination with vegetated side slopes, the paved section is to be designed to handle the base flow or runoff from a one-year frequency storm, whichever is greater. The flow depth of the paved section shall be a minimum of 0.5 foot.

E. Outlets

Each waterway shall have a stable outlet. The outlet may be another waterway, a stabilized open channel, or a grade stabilization structure.

In all cases, the outlet must discharge in such a manner as not to cause erosion. Outlets shall be constructed and stabilized prior to the operation of the waterway.

F. Drainage

In areas with high water table, seepage problems or prolonged low flows, subsurface drain, lined pilot channel, or other subsurface drainage methods should be provided. An open joint storm drain or lined pilot channel may be used to serve the same purpose and also handle frequently occurring storm runoff, base flow, or prolonged flow. The storm drain should be designed to handle base flow or the runoff from a one-year frequency storm, whichever is greater.

9.4.7 Lined Waterway or Outlet

Definition

A waterway or outlet with an erosion resistant lining of concrete, stone, or other permanent material. The lined section extends up the side slopes to designed depth. The earth above the permanent lining may be vegetated or otherwise protected.

Purpose

The purpose of a lined waterway or outlet is to provide for safe disposal of runoff from other conservation structures or from natural concentrations of flow, without damage by erosion or flooding, in situations where lined or grassed waterways would be inadequate. Properly designed linings may also control seepage, piping, and sloughing or slides.

Application

This practice applies where the following or similar conditions exist.

A. Concentrated runoff is such that lining is required to control erosion.

B. Steep grades, wetness prolonged base flow, seepage, or piping would cause erosion.

C. The location is such that damage from use by people or animals preclude use of vegetated waterways or outlets.

D. High value property or adjacent facilities warrant the extra cost to contain design runoff in a limited space.

E. Soils are highly erosive or other soil or climatic conditions preclude using vegetation.

F. For non-reinforced concrete flagstone linings installation shall be made only on low shrink-swell soils that are well drained or where subgrade drainage facilities are installed.

Design Criteria

A. Capacity

The minimum capacity shall be adequate to carry the peak rate of runoff. Capacity shall be computed using Manning's formula.

B. Velocity

Maximum design velocity shall be as stated in Section 6.0 for the appropriate channel type. Velocities exceeding critical will be restricted to straight reaches. Waterways or outlets with velocities exceeding critical shall discharge into an energy dissipator to reduce velocity to less than critical.

C. Cross Section

The cross section shall be triangular, parabolic, or trapezoidal. Monolithic concrete may be rectangular.

D. Freeboard

The minimum freeboard for lined waterways shall be as stated in Section 6 for the appropriate channel type.

E. Side Slopes

Steepest permissible side slopes shall be according to Table 9-8.

TABLE 9-8 Permissible Side Slopes for Lined Waterway

Non-Reinforced Concrete	Permissible Side Slope
Hand-placed, formed concrete:	
Height of lining 1.5 feet or less	Vertical
Hand-placed, screened concrete or in-place mortared flagstone:	
Height of lining less than 2 feet	
Height of lining more than 2 feet	1H:1V
	2H:1V
Slip form concrete:	
Height of lining less than 3 feet	1H:1V
Rock Riprap	2H:1V

F. Lining Thickness
Minimum lining thickness shall be as follows:
Concrete - 4 inches
Rock riprap - maximum stone size plus thickness of filter or bedding
Flagstone - 4 inches including mortar bed

G. Filters or Bedding

Filters or bedding are utilized to prevent piping. Drains shall be used to reduce uplift pressure, and to collect water as required. Filters, bedding, and drains shall be designed in accordance with Soil Conservation Service Standards. Weep holes and drains will be provided as needed.

H. Concrete

Concrete used for lining shall be so proportioned that it is plastic enough for thorough consolidation and stiff enough to stay in place on side slopes. A dense durable product will be required.

9.4.8 Riprap

Definition

A layer of loose rock or aggregate placed over an erodible soil surface.

Purpose

The purpose of riprap is to protect the soil surface from the erosive forces of water.

Application

This practice applies to soil-water interfaces where the soil conditions, water turbulence and velocity, expected vegetative cover, and groundwater conditions are such that the soil may erode under the design flow conditions. Riprap may be used, as appropriate, at such places as storm drain outlets, channel banks and/or bottoms, roadside ditches, drop structures, and shorelines. Broken concrete is not suitable as riprap.

Design Criteria

The minimum design discharge for channels and ditches shall be the peak discharge. See Section 6 for further design criteria.

9.5 STANDARDS FOR VEGETATIVE PRACTICES

9.5.1 Critical Area Stabilization

Definition

Critical area stabilization is planting short-term vegetation on critical areas.

Purpose

The purpose of critical area planting is to stabilize the soil, reduce damage from sediment and runoff to downstream areas, improve wildlife habitat, and enhance beauty of the area.

Application

Critical area stabilization is used on sediment-producing, highly erodible or severely eroded areas, such as dikes, levees, cuts, fills, and denuded or gullied areas where vegetation is difficult to establish with usual seeding or planting methods.

Design Criteria

A. Site Preparation

1. If necessary, divert outside water away from the critical area. This may require a permanent diversion, or in other instances, a temporary measure that will be effective during the period of establishment.

2. Where practical, grade to permit use of conventional equipment for seedbed preparation, seeding, mulch application and anchoring. (Cabling of equipment may be necessary on steep slopes.)

3. On construction sites where the exposed and underlying soil material will not maintain adequate vegetation, a topsoil dressing of six (6) inches will be applied as part of construction.

4. Where slopes must be steeper than 2H:1V use some means other than vegetation to stabilize the slope.

B. Seedbed Preparation

1. The seedbed, immediately before seeding, shall be firm but not so compact as to prohibit covering the seed. Tillage implements shall be used as necessary to provide approximately a three (3) inch depth of firm but friable soil that is free of large clods.

2. If fertilizer is to be applied, work this in during final seedbed preparation.

C. Fertilizing

1. Unless soil fertility is known to be adequate, refer to the City of Amarillo for appropriate fertilizer application rates.

D. Seeding

1. Method of Seeding

The proper amount of seed must be evenly distributed, placed at the proper depth (1" or less), and packed so that the seed is in contact with the soil. This may be done by one of the following methods.

a. Drilling

Drilling is the preferred method and should be used when possible. Drill must be equipped with seed hoppers that will properly meter out the kind of seed being planted. This may require a special drill for fluffy seeds. The drill should have double disk furrow openers with depth bands to obtain proper depth of placement. The drill should be equipped with packer wheels or the seeded area should be packed with a land roller immediately after drilling.

b. Broadcasting

This method is to be used only on areas that are inaccessible to a grass drill. The seeding rates shall be increased by one and one half (1-1/2) times when the seed is broadcasted. Seed must be evenly distributed. The seed must be covered and this can be done by light dicing, cultipacking, harrowing or raking by hand. If at all possible, the seeded area should then be packed.

c. Hydro-seeding

Where hydro-seeding equipment is used, seed, fertilizer, and wood-fiber mulch materials are mixed into a slurry with water. Care should be used to spread the mixture evenly and soon after the mixture is made. Keep the mixture well agitated when seeding.

E. Mulching

1. Where to Use

Mulch is essential on critical areas and slopes greater than 3H:1V. Mulch should be used on all treated critical areas where the goal is to attain a grass stand as soon as possible and where there is danger of damaging erosion occurring during the period of establishment.

2. Material

Mulch shall consist of clean cereal grain straw, grass hay, wood chips, long fibered wood cellulose or gravel.

3. Rate

Mulch shall be applied uniformly at a rate of 3,000 pounds minimum to 4,000 pounds maximum per acre of hay or straw. For long fibered wood cellulose the rate will be 1,500 pounds minimum to 2,500 maximum per acre.

4. Anchoring

a. Anchor mulch with a dull disk or other suitable machine. The operation should be across the slope. The mulch should be anchored a minimum of two inches in the soil and the disks spaced not more than 12 inches apart. Where it is impossible to use such a machine the mulch should be anchored by hand with a square point spade.

b. In some cases, properly anchored mulch netting may be used to hold the mulch in place.

9.6 WATER QUALITY

9.6.1 General

The land and its uses are a principal source of the nutrients, organic matter and silt that will change playa's condition. These sources are classified as either "point" or "nonpoint". Point sources are cultural in origin, usually pipes from sewage treatment plants, storm water discharges or industry. They are defined in federal regulations as "any discernable, confined and discrete conveyance, including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, containers... from which pollutants are or may be discharged." Agricultural storm water discharges and returns flows from irrigated agriculture are excluded from the definition of point source. Nonpoint sources, which are far more difficult to assess quantitatively and to control, include inputs from bank erosion, fertilizer and storm water runoff, groundwater, and biological sources such as livestock feedlots.

The quality of storm water runoff is only recently being addressed by local, state and federal regulatory agencies. Past efforts to address storm water management issues concentrated primarily upon management of storm water quantity, with relatively little attention to quality. However, state and federal regulatory programs (e.g. National Pollutant Discharge Elimination System, NPDES) aimed at the control of point discharges can result in dramatic improvements in surface water quality. With the improvement of water quality resulting from the control of point discharges has come the realization that nonpoint pollution sources represent a significant source of pollutants. Nonpoint source related problems include soil erosion and sedimentation, livestock wastes, agri-chemicals, hydrologic/habitat modification, urban storm water, resource extraction, and others.

Based on the previous discussion, storm water runoff and nonpoint source pollution in general, has the potential to significantly impact surface water quality. Because the quality of storm water runoff is closely tied to proper management of storm water quantity, it is important to address storm water quality during development of the storm water management plan. This section of the Drainage Criteria Manual discusses federal regulatory developments relating to storm water management, pollutants present in storm water management and Best Management Practices (BMPs) for storm water management.

9.6.2 Surface Water Impacts from Storm Water

Impacts associated with the discharge of storm water into surface waters depend upon the type of pollutants present. In general, organic compounds lead to a decrease in dissolved oxygen and potentially to loss of aquatic life; suspended solids lead to solids build-up in the river or lake and poor water quality. Nutrients such as phosphorous and nitrogen lead to the eutrophication of surface water, algal blooms, and eventually poor water clarity. Trace metals such as lead, cadmium, and zinc, and other toxic chemicals can be transported to aquatic organisms by the process of bioaccumulation. In fish, this process can result in concentrations of toxic substances that are large multiples of those of the water in which the fish dwell. This, in turn, can further result in food chain biomagnification causing increased concentrations of substances as each food chain level is consumed by the next. Trace metals and organic contaminants can also result in acute or chronic toxic effects if concentrations become extreme.

When considering surface water impacts, it is helpful to understand the phenomenon called "first flush." Studies have shown that the portion of storm water runoff containing high concentrations of pollutants is captured during the first fifteen minutes of a storm event, commonly referred to as the first flush. Thus the pollutant accumulation in, intense small runoff events can be more detrimental to water quality than larger flooding events. Because most pollutants are contained in the first flush, upstream management practices which control small volumes of initial runoff can be very effective, and enhance nonpoint source pollutant removal.

9.6.3 Environmental Protection Agency Rules

In 1972, Congress passed significant amendments to the Federal Water Pollution Control Act (referred to as the Clean Water Act or CWA). The amendments prohibited the discharge of any pollutant to waters of the United States from a point source unless the discharge was authorized by a National Pollutant Discharge Elimination System (NPDES) permit. NPDES permits specify monitoring, reporting and control requirements, including allowable levels of pollutants in discharges.

Many studies have shown that runoff from urban and industrial areas typically contain significant quantities of the same pollutants found in waste waters and industrial discharges. These pollutants include heavy metals, pesticides, herbicides, and organic compounds such as fuels, waste oils, solvents, lubricants, and grease. These pollutants may cause problems for both human health and aquatic organisms.

In 1987, the CWA was revised by adding Section 402(p) to address storm water. Section 402(p) requires the EPA to establish final regulations governing storm water discharge permit application requirements under the NPDES program. The permit application requirements pertain to storm water discharges associated with industrial and construction activity, and discharges from large (systems serving a population of 250,000 or more); or small (systems serving a population of 100,000 or more) municipal separate storm sewer systems. In response to this requirement, the EPA published in the November 16, 1990, Federal Register the regulations for NPDES permit application requirements for storm water discharges.

Permits for storm water discharges associated with industrial activity must comply with all of the applicable provisions of Sections 402 and 301, including technology and water quality-based standards. NPDES permits for storm water discharges from municipal storm sewers must require controls to reduce the discharge of pollutants to the "maximum extent practicable."

The regulations place the burden of storm water management upon the local community. The agency responsible for the construction, maintenance and operation of the storm water treatment system will also be responsible for the quality of storm water discharged. The reader should consult the City Engineer or EPA to identify specific NPDES permit requirements.

The NPDES program in Texas is administered by the US Environmental Protection Agency (EPA), Region VI. Additional information on the NPDES program can be obtained at the address listed below:

US Environmental Protection Agency
Region VI Office
1445 Ross Avenue
Dallas, Texas 75202
Attention: Rhonda Harris
Telephone: (214) 655-7175

9.6.4 Standards and Criteria

It is the responsibility of the City and property owner to meet storm water runoff quality standards. The methods of achieving this are by implementation of BMPs. However, there are many types of technical problems covering a variety of concerns, and it is beyond the scope of this manual to identify the level of protection that will be required for a specific site, or the BMP which will achieve the required result. At this time, the water quality program requires a best effort attempt at installing facilities which will address the commonly predictable problems of a development. The NPDES Program provides for periodic water sampling and testing. Areas of deficiencies will be identified and site specific requirements will be imposed to address these deficiencies. Therefore, it is essential that the site planner consider the future use of a site and provide for any predictable water quality problems. As a result of the complexity of the problem, the following section is somewhat general.

9.7 BEST MANAGEMENT PRACTICES

Best Management Practices (BMPs) is a concept from federal laws that govern the control of nonpoint pollution sources. The concept was to direct attention to management of inputs rather than collection, concentration and treatment of the effects of inputs. The techniques are designed to reduce soil loss or prevent surface runoff from carrying heavy sediment and nutrient loads into water bodies.

BMPs may generally be categorized as 1) storage; 2) infiltration; 3) source controls; or 4) treatment practices. Storage of storm water is intended to retain storm water on site either in a permanent pool or for later release at a predetermined rate. Infiltration techniques provide for a reduction in the quantity of storm water generated by enhancing the rate of movement of storm water into the earth's surface. Source controls consist of management or good housekeeping practices intended to reduce contact between storm water and pollutants. And, treatment of storm water is the use of chemicals (e.g. aluminum sulfate salts to remove phosphorus) to provide a reduction in the concentration of contaminants in storm water. While some methods for controlling storm water quantity may have the secondary benefit of improving water quality, others may not.

There are generally two reasons to implement BMPs from a water quality standpoint. The first is to protect the existing level of water quality from future degradation; the second is to correct existing water quality problems. In this case, BMPs are implemented as remedial measures, and a different process of BMP selection is appropriate.

9.7.1 Infiltration Basin

An infiltration basin is a water impoundment constructed over permeable soils. The purpose of the basin is to temporarily store surface runoff for a specific design frequency storm and allow it to infiltrate through the bottom and sides of the basin. This infiltration removes many pollutants, provides groundwater recharge, reduces the volume of runoff, and reduces peak discharges.

Infiltration basins are very effective for removing fine sediment and pollutants associated with it. This includes sediment, trace metals, nutrients, bacteria, and oxygen-demanding substances. Coarse sediment is effectively controlled, but should be removed from runoff before it enters an infiltration basin. Coarse sediments can clog the basin and take up storage volume. Dissolved pollutants are effectively controlled for storm events less than the design frequency, but these materials may not be removed from the runoff as it infiltrates.

Infiltration basins can be designed to provide total control of urban pollutants in surface runoff for the design runoff volume. Although infiltration basins are very effective for controlling pollutants in surface water, certain soluble substances can be expected to move to the groundwater. Chloride from road salt is an example of a soluble material that will not be removed during the infiltration process.

Infiltration basins are best suited for sites with drainage areas of 5 to 50 acres. A typical basin will have a depth of 3 to 12 feet. The maximum depth of a basin is limited by the infiltration rate of the soil and maximum detention time. Figures 9-4 and 9-5 show variations of infiltration/detention facilities which are effective in improvement of water quality.

The soils on a prospective site are an important consideration when determining the suitability for infiltration. The soils must have an infiltration rate of 0.27 inches/hour or greater to be considered for an infiltration basin. This generally corresponds to soils in the A or B hydrologic soil groups which include silt loam, loam, sandy loam, loamy sand and sandy soils. Soil surveys are necessary for preliminary screening of a site for soil infiltration rate.

Design Criteria

A. Ponding Time

The maximum ponding time recommended is 72 hours. This maximum ponding time combined with the infiltration rate of the soil will determine the maximum design depth of the basin. The maximum design depth can be related by:

$$d_{max} = f\,T_p \tag{9-2}$$

where:

d_{max} = maximum design depth, in inches

f = soil infiltration rate, in inches per hour

T_p = design ponding time, in hours

9.7.2 Filter Strips

Settleable solids, floating materials, and grease should be removed from runoff to the maximum extent possible before it enters the infiltration basin. If these materials enter the basin, they can clog the bottom of the basin and take up storage volume. Devices such as detention ponds, vegetative filters, oil/grit separators, or floatable skimmers can be used to remove these materials before they enter the infiltration basin. It may be feasible to allow these materials to enter the basin if their effects are considered during the design. One method of planning for this is to rely upon infiltration out of the sides of the basin rather than the bottom.

A. Design Criteria Embankment Design

Any structure using an embankment to impound water should have an emergency spillway to safely bypass flows from large rainfalls. See requirements of Section 8.

B. Principal Spillway for Combination Structure

If a combination detention pond/infiltration basin design is being used, the elevation of the principal spillway crest should be no higher than the three-day infiltration capacity of the basin. All other aspects of the basin design such as flood routing should meet the requirements of an extended detention pond and Section 8. An example of a combination basin is shown in Figure 9-6.

C. Hydrologic Design

The hydrologic design of infiltration basins should be in accordance with the recommended procedures included in Section 2. For combination basins where flood routing is required, HEC-1 is recommended for analysis.

D. Infiltration Capacity Protection

Initial excavation of the basin should be carried out to within one-foot of the final grade of the basin floor. Final excavation of the basin floor should be delayed until all disturbed areas in the watershed are stabilized. The final phase of excavation should be performed by equipment with tracks exerting relatively light ground pressures. This will prevent compacting of the basin floor which would reduce the infiltration capacity. After final grading, the basin floor should be tilled to a depth of at least 6 inches to provide a well aerated, porous surface texture.

The bottom of infiltration basins may be lined with a 6- to 12-inch layer of filter material such as coarse sand to help prevent the build-up of impervious deposits. The filter layer can be replaced or cleaned if it becomes clogged. The slopes of infiltration basins usually need little maintenance to maintain their infiltration capacity.

Establishing dense vegetation on basin floors and slopes is recommended. Vegetation will not only prevent erosion, but will also provide a natural means of maintaining infiltration rates. Vegetation should be selected and established with permanent vegetation. For the highly permeable areas of the basin, drought-tolerant species are recommended.

E. Maintenance

Proper maintenance of infiltration basins is critical. Basins should be inspected semiannually and after major storm events. Sediment removal should be performed when the sediment is dry enough so that it is cracked and readily separates from the basin floor.

Vegetation should be maintained as needed to control weed growth and maintain the health of the grass. This will include mowing and fertilization. The use of low maintenance and drought-resistant varieties will minimize maintenance needs. When fertilizer application is needed to maintain the vegetation, proper application methods should be used to minimize the potential for leaching. Practices such as split application and use of slow-release fertilizers will help to minimize the chance of leaching.

9.7.3 Vegetated Swales

Vegetated swales are broad shallow channels with a dense stand of vegetation established in them that are designed to promote infiltration and trap pollutants. The combination of low velocities and vegetative cover provides an opportunity for pollutants to settle out or be treated by infiltration. In addition to pollutant removal, this practice can result in reduced volumes of runoff and peak discharges.

Vegetated swales are most effective for removal of coarse sediment and pollutants associated with it. Fine sediment and soluble pollutants are not treated unless they are part of runoff that infiltrates through the swale.

Low-gradient grass swales are best suited to providing water quality benefits. Check dams can be used in higher gradient swales to impound water and slow velocities, but are impractical in steeper swales because of the close spacing required.

Vegetated swales are most applicable in residential or institutional areas where the percentage of impervious cover is relatively small. Swales are usually located in a drainage easement at the back or side of a residential lot. They can also be used along roads in place of curb and gutter.

In planning the drainage system for a development, the planner should consider the following characteristics of vegetated swales:

A. Vegetated swales are generally less expensive to install than curb and gutter.

B. Roadside swales keep flow away from the street surface during storms, thus reducing driving hazards.

C. Roadside swales become less feasible as the number of driveway entrances requiring culverts increases.

D. In areas with steep slopes, vegetated swales are best suited to locations where they are parallel to the contours.

Vegetated swales are most effective when the flow depth is shallow and the velocities are low. These characteristics limit the application of grass swales as a BMP to locations where flows are generally less than 5 to 10 cubic feet per second. Also, the soils should be suitable to establish a vigorous stand of vegetation. If dense vegetation cannot be maintained in the swale, its effectiveness as a BMP will be severely reduced. Sites on A or B hydrologic group soils will be more effective for infiltration, although swales on other soils will still provide some infiltration and treatment through sedimentation.

The seasonally high water table should be 1 to 2 feet below the bottom of the swale. This will allow treatment of most pollutants before they reach the groundwater.

Design Criteria

A. Channel Design

As mentioned above, the channel should be designed for low velocity flow. A velocity of 2 fps is the maximum design velocity recommended when vegetated swales are being designed as a BMP. Flow depths in the swale should be minimized to increase the amount of filtering and settling. A maximum design flow depth of 1 foot is recommended. This will generally result in wide shallow channel designs.

The grade of the channel should be as flat as possible, and should not exceed 2 percent. Check dams can be constructed in the waterway to temporarily store water, promote infiltration, and increase the effectiveness of the grass swale. The check dam should be constructed of durable material so that it will not erode. The area just downstream of the check dam should be protected from scour with properly designed rock riprap or channel lining.

B. Infiltration Enhancement

On soils with A or B hydrologic group rating, vegetated swales can be designed for infiltration as well as pollutant removal through sedimentation. To enhance the infiltration characteristics, check dams can be used to store water in the swale. These check dams should be designed so that the water ponded will infiltrate in 24 hours or less.

C. Vegetation

A dense stand of vegetation is needed for vegetated swales to be effective. If the water table is high, it may need to be controlled with subsurface drainage for the swale to support the desired species. Drainage would also improve the condition of the swale to prevent rutting during maintenance. If the swale is constructed on a very permeable soil, a drought-resistant species of grass should be selected.

D. Maintenance

Vegetated swales should be maintained to keep the grass dense. The grass should be mowed occasionally, but it should not be trimmed close to the ground. If the grass is trimmed extremely short, the filtering effect of the swale will be reduced. Major maintenance operations will involve weed control, mowing, and occasional fertilization. Fertilization should only be done when needed to maintain the health of the grass. Over-application of fertilizer can result in the swale becoming a source of nutrients.

9.7.4 Playas

Playa treatment involves passing runoff through a natural playa to remove or treat pollutants. Playas provide favorable conditions for removal of pollutants from urban runoff through sedimentation and also provide an intense pool of biological activity to use nutrients during the growing season. Although playas can be effective for removing pollutants, certain drawbacks limit their use as a BMP. The major problem with playa treatment are the environmental damage that may be done to natural playas.

Playa treatment can be very effective for removing sediment and pollutants associated with it (such as trace metals, nutrients, and hydrocarbons), oxygen-demanding substances, and bacteria from urban runoff. Playas can also be effective during the growing season for removal of dissolved nutrients as well as those adsorbed to sediment.

The effectiveness of playa treatment systems for the removal of urban pollutants will depend upon the physical characteristics of the system such as playa size to watershed size ratio, runoff residence time in the playa, and water budget. In general, as the playa to watershed ratio increases, the average runoff residence time increases, and the effectiveness of the playa for pollutant removal also increases.

9.7.5 Oil/Grit Separators

Oil/grit separators are chambers designed to remove sediment and hydrocarbons from urban runoff. They are normally used close to the source before pollutants are conveyed to storm drains or other BMPs such as infiltration trenches. Oil/grit separators are typically used in areas with heavy traffic or high potential for petroleum spills such as parking lots, gas stations, roads, and loading areas. See Figure 9-7.

As the name implies, oil/grit separators are intended to remove floating oils and coarse sediment from runoff. They are also effective for removing floating trash from runoff.

Runoff is only detained briefly in oil/grit separators, so only moderate removal of coarse sediments, oil, and grease can be expected. Even more limited removal is likely for fine grained sediment and pollutants attached to the sediment such as trace metals and nutrients. Soluble pollutants will most likely pass through oil/grit separators.

9.8 PERMIT QUALIFICATIONS

The Corps of Engineers and the EPA are the agencies of the federal government that regulate water quality and wetlands. Both agencies have authorities under Section 404 of the CWA, but the Corps alone has authority under Section 10 of the Rivers and Harbors Act. The principal law providing wetlands regulatory authority is the Clean Water Act. The principal distinction in the roles of the two agencies is that the Corps administers the wetlands permitting program and the EPA mostly exercises certain review and policy-setting functions. The Fish and Wildlife Service (FWS) of the Department of Interior has the significant role in the wetlands regulation process of commenting on the habitat importance of areas for which a permit is sought.

9.8.1 Clean Water Act (CWA)

Formerly known as the Federal Water Pollution Control Act (FWPCA), this statute was enacted principally to address the problem of water pollution, mainly by reducing discharges of pollutants, particularly from industrial sources, in U.S. lakes, rivers, and streams. The basic means to achieve the goals of the Act is through a system of water quality standards, discharge limitations, and permits. The Act authorizes EPA to require owners and operators of point source discharges to monitor, sample, and maintain effluent records. The 1972 amendments to the act established a permit program, known as the National Pollution Discharge Elimination System (NPDES), for point source discharges. If the water quality of a water body is potentially affected by a proposed action (i.e., construction of a wastewater treatment plan), a NPDES permit (Section 402) may be required. In most cases, the EPA has turned this responsibility over to the States as long as the individual State program is acceptable to the Agency.

Similarly, if a project may result in the placement of material into waters of the United States, a Corps of Engineers Dredge and Fill Permit (Section 404 of CWA) may be required. It should be noted that the Section 404 permit also pertains to activities in wetlands and riparian areas.

Prior to the issuance of either a NPDES or a Section 404 permit, the applicant must obtain a Section 401 certification. This declaration states that any discharge complies with all applicable effluent limitations and water quality standards. Certain Federal projects may be exempt from the requirements of Section 404 if the conditions set forth in Section 404(r) are met.

Section 401 - Clean Water Act

Under Section 401 of CWA, states have the authority to review any federal permit of license which may result in a discharge to waters of the United States, including Section 404 permit applications, to ensure that actions would be consistent with the State's water quality standards. A Section 401 review provides the State's water quality certification.

Section 404 - Clean Water Act

This Section provides a special authority for the Corps, away from the general EPA permit authority, to issue permits for the discharge of two types of pollutants: dredged material and fill material. Under Section 404, a permit is required from the Corps before dredge and fill materials can be discharged into any waters of the U.S., including wetlands. Definition of "discharge" is broad and includes filling any U.S. waters for any type of development. This Section also encompasses more than just traditionally navigable waters as regulated by Section 10 of the Rivers and Harbors Act. In general, the Corps will not issue a Section 404 permit unless the proposed project complies with state laws, including state water quality certification (Section 401) and state wetlands laws.

The Section 404 program is administered by both the Corps and EPA, while the U.S. Fish and Wildlife Service (FWS) and the National Marine Fisheries Service (NMFS) have advisory roles. The Corps has the primary responsibility for the permit program and is authorized to issue permits; EPA has a responsibility to review and comment on permit application being evaluated by the Corps. The primary role of the FWS is one of consultation and assistance under the Fish and Wildlife Coordination Act (FWCA). The NMFS has review responsibilities similar to the FWS for certain Section 404 permit actions when they relate to marine resources.

404(a) - authorizes the Corps to issue permits for filling navigable waters, which includes wetlands. The act gave the Corps authority to issue permits, but no guidelines to evaluate them; so, the Corps relies heavily on its public interest review to evaluate permits.

404(b) - requires, in essence, that the Corps issue permits in accordance with guidelines developed by EPA--the so-called "b-1 guidelines." These guidelines state that, among other things, "no discharge of dredged or fill material shall be permitted if there is a practicable alternative to the proposed discharge which would have less adverse impact on the aquatic ecosystem..." In addition, "no discharge of dredged or fill material shall be permitted which will cause or contribute to significant degradation of the waters of the U.S."

404(c) - authorizes EPA to veto a decision by the Corps to issue a permit to fill in a wetland.

404(e) - authorizes the Corps to issue general permits on a state, regional, or nationwide basis for certain categories of activities in wetlands that are "similar in nature, and will cause only minimal adverse effect to the environment."

404(f) - exempts certain activities from the permit requirements, such as "normal farming, silviculture, and ranching activities, minor drainage, harvesting for the production of food, fiber and forest products, or upland soil and water conservation practices."

404(g) - authorizes states to assume the permit program from the Corps (except in the case of coastal waters), provided their program is approved by EPA.

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Straw (or Hay) Bale Barrier	SBB
Silt Fence	SF
Stabilized Construction Entrance	
Outlet Structure	OS L=
Grass Outlet Structure	GOS L=
Diversion	D D
Grassed Waterway	GW
Lined Waterway	LW
Sediment Basin	
Riprap	

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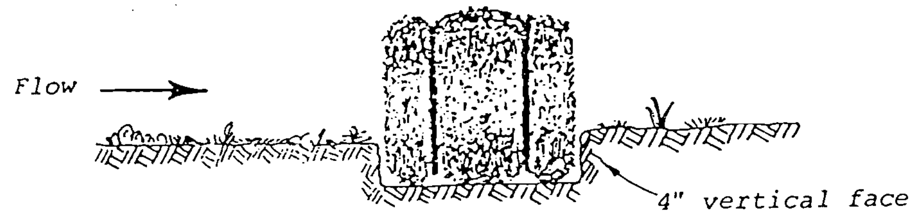
Standard Symbols for Soil Erosion and Sediment Control

Storm Water Management Criteria Manual

Date NOV 1992

Figure 9-1



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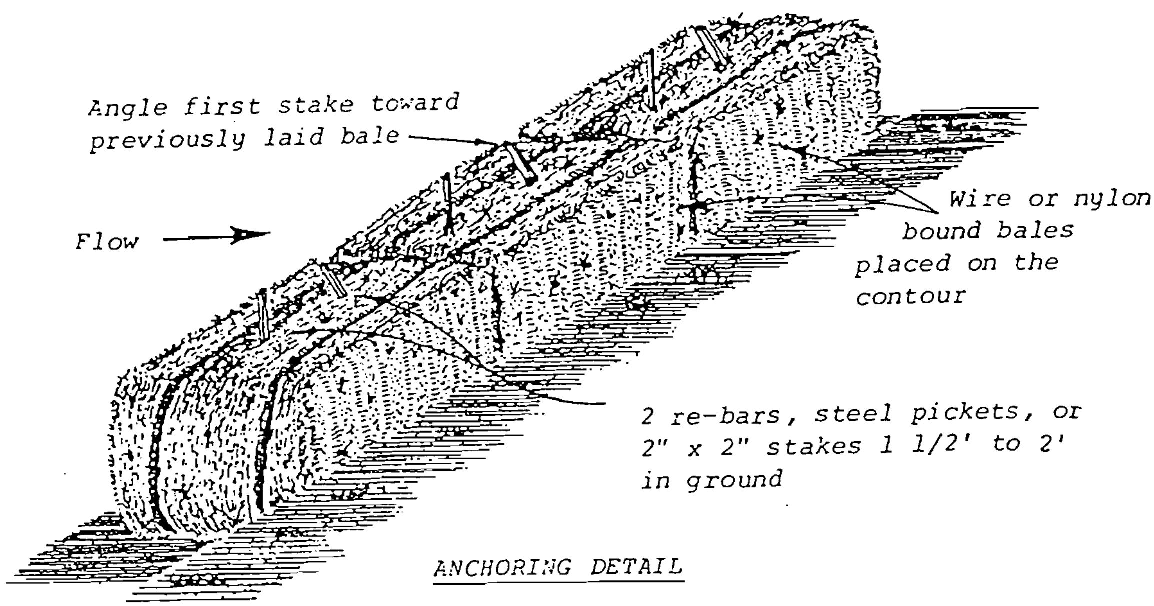

ANCHORING DETAIL

Standard Symbol

SBB

Source: Soil Conservation Service.

Straw (or Hay) Bale Barrier

Date: NOV 1992

Figure: 9–2

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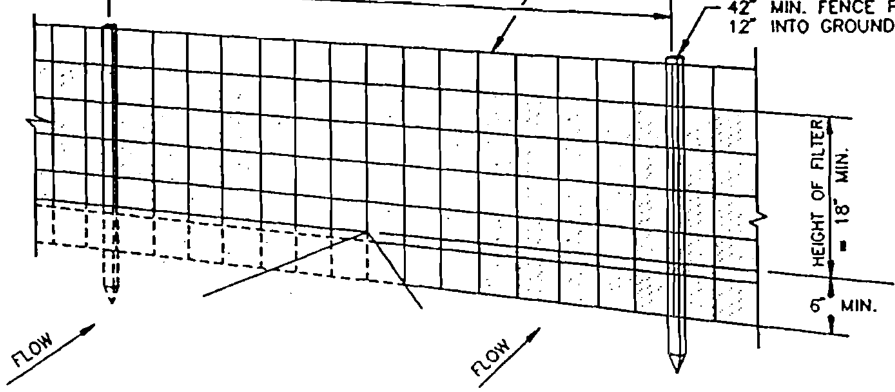
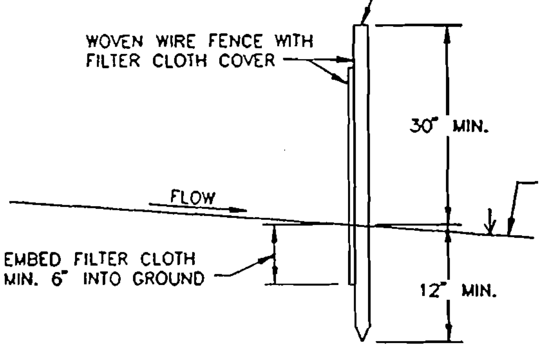


PERSPECTIVE VIEW



SECTION

Standard Symbol

SF

Source: Soil Conservation Service.

Silt Fence

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Figure 9–3

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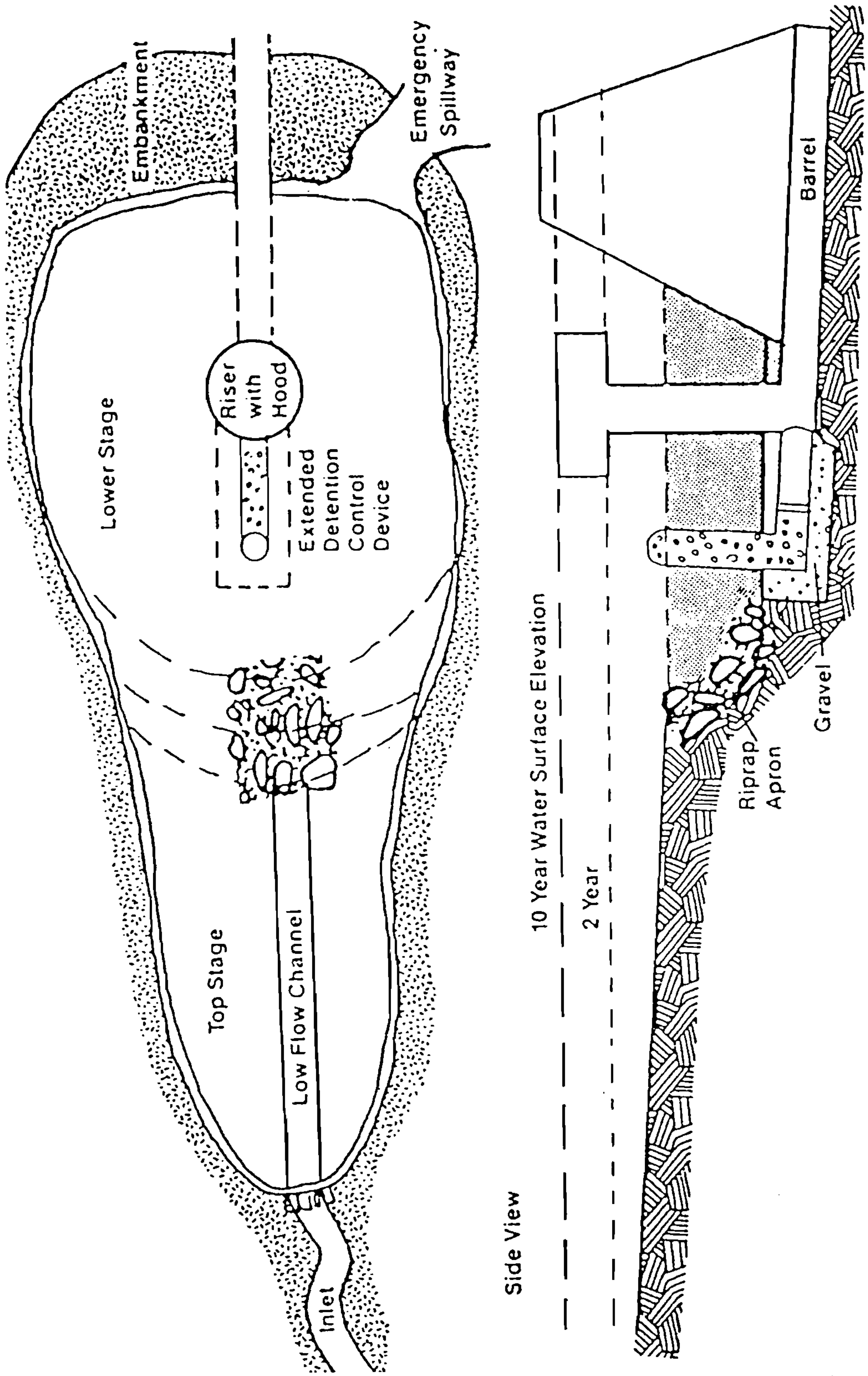

Source: Schueler, 1987.

Schematic of a Dry Extended Detention Pond

Date: NOV 1992

Figure: 9-4

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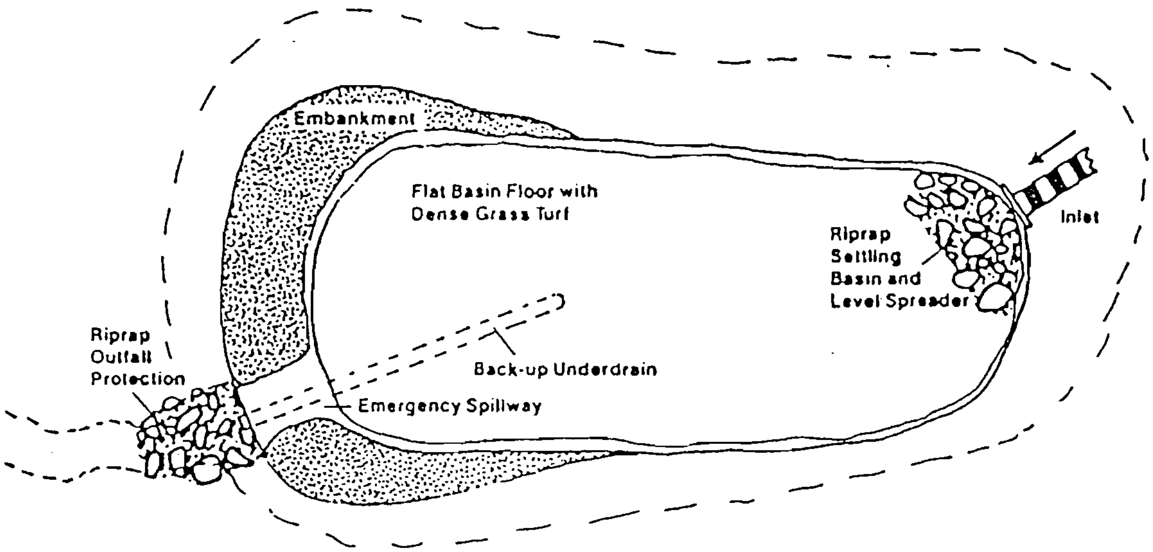
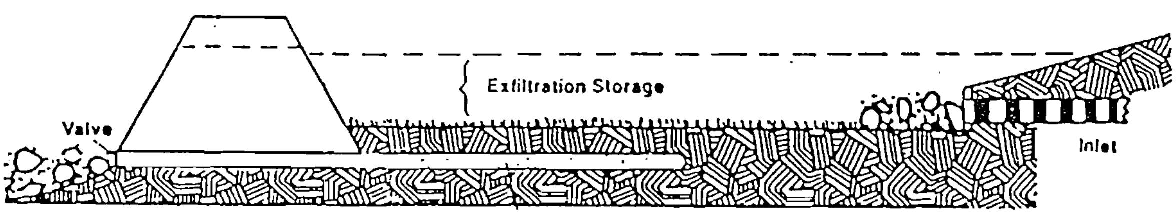


Side View



Back-up Underdrain Pipe in Case of Standing Water Problems

Source: Schueler, 1987.

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Typical Infiltration Basin Schematic

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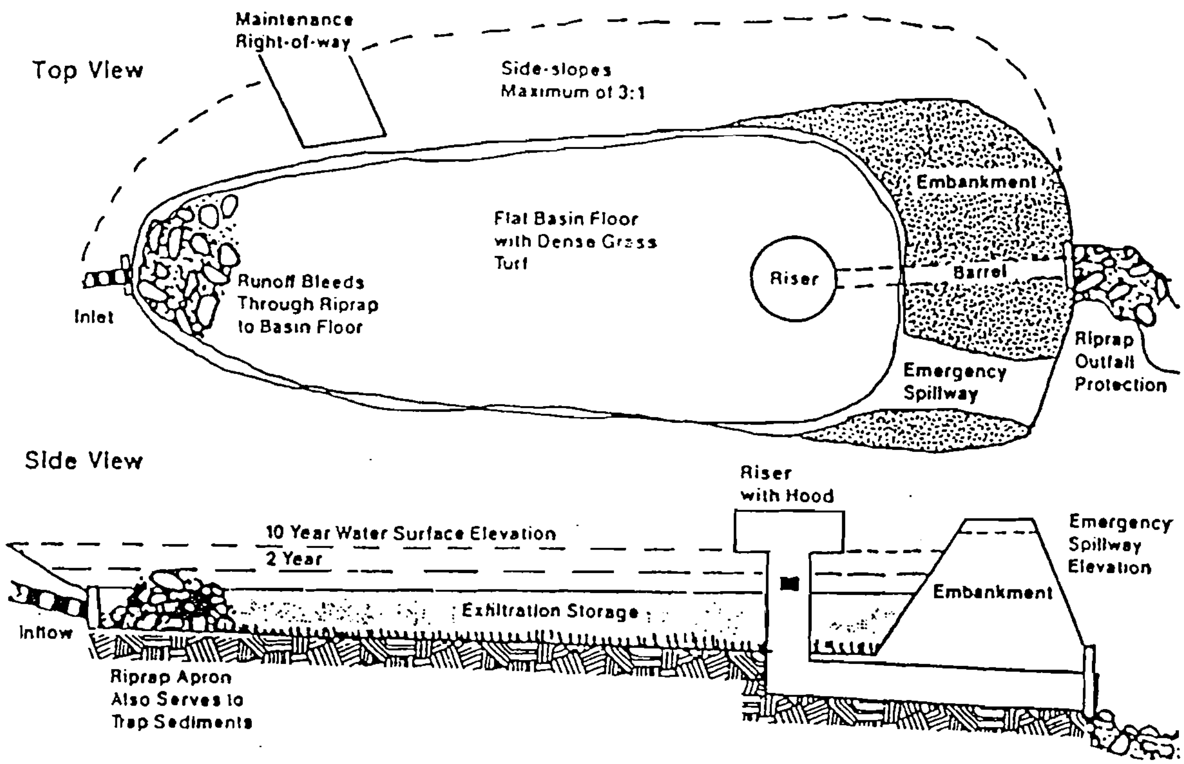
Figure
9–5



Source: Schueler, 1987.

Typical Combined Infiltration Detention Basin

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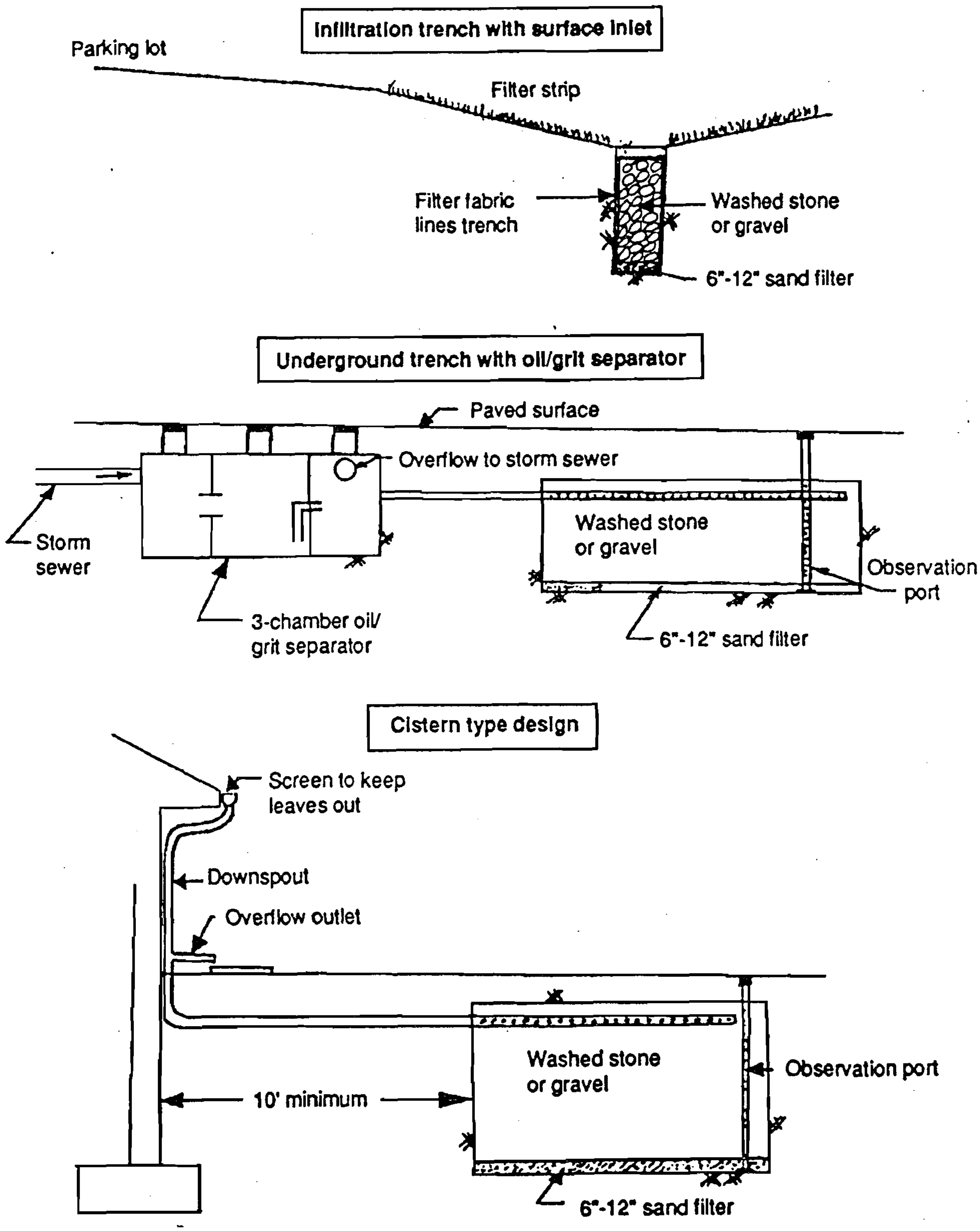
Figure 9-6



Source: Schueler, 1987.

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Typical Infiltration Trench Use and Oil/Grit Separator

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Date NOV 1992

Figure 9-7

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