

CITY OF COMMERCE CITY
Department of Public Works

**STORM DRAINAGE DESIGN
AND
TECHNICAL CRITERIA
MANUAL**



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501	$Q = CIA$	Rational Formula
502	$t_c = t_i + t_t$	Time Of Concentration
503	$t_i = \frac{1.8 (1.1 - C_s) \sqrt{L}}{\sqrt[3]{S}}$	Initial Flow (Non-Urbanized)
504	$t_i = \frac{L' + 10}{180}$	Initial Flow (Urbanized)
601	$H_{FB} = 0.5 + \frac{V^2}{2g}$	Channel Freeboard
602	$H_{FB} = 2.0 + 0.025V(d)^{1/3}$	Concrete Channel Freeboard
701	$H_L = K \left[\frac{V^2}{2g} \right]$	General Equation For Pipe Form Losses
702	$H_L = K_e \left[\frac{V_1^2}{2g} \right] \left[1 - \frac{A_1}{A_2} \right]^2$	Expansion Loss
703	$H_L = K_c \left[\frac{V_2^2}{2g} \right] \left[1 - \left[\frac{A_2}{A_1} \right]^2 \right]^2$	Contraction Loss
704	$H_L = K_b \left[\frac{V^2}{2g} \right]$	Bend Loss
705	$H_L = \frac{V_2^2}{2g} - K_j \left[\frac{V_1^2}{2g} \right]$	Junction Loss
901	$Q = (0.56) (Z/n) S^{1/2} Y^{8/3}$	Modified Mannings Formula

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EQUATION NO.	EQUATION	TITLE
1001	$H_T = 0.11 (TV/D)^2 (\sin A)$	Trashrack Loss
1201	$V = KA$	Detention Volume Equation
1202	$K_{100} = (1.78I - 0.002I^2 - 3.56)/1000$	Detention Coefficient 100-Year Storm
1203	$K_5 = (0.77I - 2.26) / 1000$	Detention Coefficient 5-Year Storm
1204	$Q = CLH^{3/2}$	Weir Equation
1205	$Q = 2.5 \tan(\theta / 2) H^{5/2}$	Weir Equation
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CHAPTER 1: GENERAL PROVISIONS

CHAPTER 1 GENERAL PROVISIONS

1.1 SHORT TITLE

These regulations coupled with future amendments shall be known as the City of Commerce City "*Storm Drainage Design and Technical Criteria Manual*"; hereinafter called CRITERIA.

1.2 PURPOSE

1.2.1 This CRITERIA shall apply in the administration of Chapter 17 of this Code entitled "Planning and Development" as well as in the administration of the Commerce City Zoning Ordinance and all other ordinances and regulations which require a review of drainage conditions on any property within the City to which this CRITERIA applies except in a Flood Overlay District. Further, the purpose of this CRITERIA is to establish minimum standards for the public safety, health, comfort, convenience, welfare and economic well-being of residents and owners of property within the City.

1.2.2 Presented in this CRITERIA is the minimum design and technical standards for the analysis and design of storm drainage systems. Additionally, the construction of such storm drainage system as designed shall be inspected and approved by the Department of Public Works before consummation of the action to which the analysis and design applies.

1.2.3 Maintenance of the constructed drainage system, as required to cause the functioning of such system as designed and first built, shall be the responsibility of the property owner and subsequent owners of the property upon which the system is built.

1.3 SCOPE

1.3.1 All subdivisions, resubdivisions, planned unit developments, or any construction of exterior impervious surface (see below for exceptions) submitted to this City for approval shall include a hydrologic analysis and hydraulic design for storm drainage. Such analysis and design shall conform to this CRITERIA.

1.3.2 Exceptions. Single lot, infill single family residential construction, reconstruction, or expansion containing less than 10,000 square feet of impervious area, regardless of the zoning of the property. Exceptions from the requirement for submittal of complete drainage studies may be granted by the Administrator provided that engineering evidence is presented to the satisfaction of the Administrator that such a study is not required for the protection of the public health, safety and welfare. Exceptions from the requirement for construction of drainage improvements including detention and/or retention facilities may be

granted by the Administrator provided that engineering evidence is presented to the satisfaction of the Administrator that adequate control measures are or will be implemented for proper management of the drainage. Additionally, the requirement for submittal of engineering evidence and/or for construction of drainage improvements may be waived if in the judgement of the Administrator, no useful purpose would be served for such submittal or construction.

1.4 APPLICABILITY

This CRITERIA shall apply to all land within the current and subsequently annexed boundaries of this City, including all City-owned lands. The criteria shall be applied to all new development and redevelopment. In cases of development on lots without any existing structures or paving, the CRITERIA shall be applied to the entire lot. On lots with existing structures and/or impervious surface, the CRITERIA shall be applied only to the area on which new structures or paving are to be constructed and therefore a drainage system retrofit for the entire property shall not be required. In cases of redevelopment, the CRITERIA shall be applied only to that portion of the parcel under redevelopment and therefore no retrofit shall be required for the remainder of the parcel not under redevelopment.

1.5 ADOPTION

This *City of Commerce City Storm Drainage Design and Technical Criteria Manual*, 1989 Edition, has been adopted by the City Council by ordinance.

1.6 ADMINISTRATOR

As pertains to this CRITERIA, the City Manager, or his designee, (hereinafter called "Administrator"), shall be responsible for administration and enforcement of this CRITERIA, including:

1.6.1 Review of all drainage studies, plans and specifications for drainage improvements, except that such review pursuant to this CRITERIA shall not include flood plain district management as set forth in this Code at Chapter 17, Article IV, entitled "Flood Overlay Districts".

1.6.2 Interpretation and enforcement of the provisions of this CRITERIA.

1.6.3 Exercising sound engineering judgement in extending or reducing the requirements found in this CRITERIA.

1.7 AMENDMENT

1.7.1 The policies and criteria contained in this manual are basic guidelines which may be amended as new technology is developed and/or experience is gained in the use of this CRITERIA.

1.7.2 Amendments shall be published and posted as required by the City Charter for adoption of ordinances and shall become effective as provided by the City Charter. Final drainage reports or construction plans which are submitted for approval of the City within 30 days after the effective date of any amendment to this CRITERIA are exempt from the requirements of any newly adopted amendment.

1.8 CROSS REFERENCE

Policies and technical criteria not specifically addressed in this CRITERIA shall follow the provisions of the Urban Drainage and Flood Control District (hereinafter called DISTRICT) "*Urban Storm Drainage Criteria Manual*" (hereinafter called MANUAL), which is incorporated in this CRITERIA by reference. Drainage facilities in place or under construction shall be accepted without regard to the provisions of this CRITERIA; however, any modifications to these facilities shall be in accordance with this CRITERIA.

1.9 REVIEW AND APPROVAL

1.9.1 The City will review all submittals of drainage studies and plans for general compliance with this CRITERIA. An approval by the City does not relieve the owner, engineer, or designer from responsibility for insuring that the calculations, plans, specifications, construction, and record drawings are in compliance with the CRITERIA, as stated in the certification of the owner's engineer, as required in Section 3.2.

1.9.2 The DISTRICT may be requested by the City to review reports and construction plans required by this CRITERIA. Where major drainageway improvements or floodplain delineation are involved, DISTRICT approval will be required; administration of this subject area shall be in accordance with Chapter 17 Article IV of the Commerce City Code.

1.10 VARIANCES

Applications for variance from this CRITERIA and applications for interpretation of this CRITERIA shall be submitted to the Board of Adjustment, and considered on a case-by-case basis in accordance with the Zoning Board of Adjustment procedures as set forth in the ordinances of the City.

CHAPTER 2: DRAINAGE POLICY

CHAPTER 2 DRAINAGE POLICY

2.1 INTRODUCTION

The provisions for adequate drainage in urban areas is necessary to preserve and promote the general health, welfare, and economic well being of the region. Drainage is a regional feature that affects all governmental jurisdictions and all parcels of property. This characteristic of drainage makes it necessary to formulate a program that balances both public and private involvement. Overall coordination and master planning must be provided by the governmental units most directly involved, but drainage must be integrated on a regional level.

When planning drainage facilities, certain underlying principles provide direction for the effort. The principles are made operational through this set of policy statements. The application of the policy is in turn facilitated by technical criteria and data. A brief discussion of the requirements or basis for a policy is presented followed by the actual policy statement, which is printed in boldfaced type.

2.2 BASIC PRINCIPLES

2.2.1 *Urban Sub-System*

Drainage is a sub-system of all urbanization. The planning of drainage facilities must be included in the urbanization process. The first step is to include drainage planning with all regional and local development master plans. The report shall address multiple purpose use of land for drainage and open space.

Storm water management facilities, such as channels and storm sewers, serve both as a conveyance and storage function. When a channel is planned as a conveyance feature, it requires an outlet as well as downstream storage space. When the space requirements are considered, the provision for adequate drainage becomes a competing use for space along with other land uses. If adequate provision is not made in a land use plan for the drainage requirements, storm water runoff will conflict with other land uses and will result in water damages, and will impair or even disrupt the functioning of other urban systems.

The policy of the City shall be to consider storm drainage a sub-system of the over-all urban system and to require storm drainage planning for all developments to include the allocation of space for drainage facilities.

2.2.2 *Multi-Purpose Resource*

Storm water runoff is an urban resource. Whereas the runoff can be a liability to urbanization, storm runoff has potential for beneficial use. This use, however, must be compatible with adjacent land uses and Colorado Water Law.

When storm water runoff is treated as a resource, quality aspects of the water become important. This in turn relates to issues such as street cleaning practices, solid waste collection and removal services, and regulations on the development of raw land to control erosion and resulting silt loads. These practices influence succeeding water uses.

The storm water urban sub-system should be multi-purpose to satisfy the competing demands placed on water within the urban environment. Drainage facilities can fulfill other purposes aside from just drainage; and facilities not designed primarily for drainage frequently can be designed to provide drainage benefits.

The policy of the City shall be to consider storm runoff as a multi-purpose resource with the potential for practical use and to encourage the multi-purpose use.

2.2.3 *Water Rights*

When the drainage sub-system interferes with existing water rights, the value and use of the water are affected. The existing drainageways and storage locations frequently interrelate with the water rights, which must be addressed when planning the facility to preserve their integrity.

Ditches which have direct flow rights from a stream are controlled by headgates. Drainage improvements, which alter the quantity (or quality) of the water available to the headgate, affect the ability to divert water. Other ditches obtain all or portions of the rights by intercepting the shallow groundwater (see page rate). If the water right has not been abandoned or transferred to another location, the drainage design (including the sub-surface system) must be planned and constructed to preserve the water right. Similar situations can also occur when planning drainage facilities near reservoirs.

The policy of the City shall be to recognize the possible effects on the water rights and to include the interrelationship in the planning and design of the proposed drainage facility.

2.2.4 *Jurisdictional Boundaries*

Since drainage considerations and problems are regional in nature, and do not respect jurisdictional boundaries, a successful plan must emphasize regional cooperation in accomplishing the goals.

The policy of the City shall be to pursue a jurisdictionally unified approach to drainage to assure an integrated plan.

2.2.5 *Major Drainageway*

The definition of a major drainageway is necessary for the clarification and administration of these CRITERIA. For the purpose of these CRITERIA, a Major Drainageway shall be defined as: Any drainage flow path with a tributary area of 130 acres or more.

2.3 REGIONAL AND LOCAL PLANNING

2.3.1 *Basin Transfer*

Planning and design of storm water drainage systems should avoid the premise that problems can be transferred from one location to another. Property owners, developers, and the regulatory agencies must be aware of the potential liabilities of altering the historic drainage continuum, or of transferring a storm drainage burden from one platted or unplatted lot, tract, or land parcel to another. Until all reasonable alternatives are studied, and unless prudent reasons justify, such diversion shall be avoided.

The subdivision process can and will significantly alter the historic or natural drainage paths. When these alterations result in a subdivision outfall system that discharges back into the natural drainageway at or near the historic location, then the alterations (intra-basin transfer) are generally acceptable. However, when the subdivision outfall system does not return to the historic drainageway, then inter-basin transfer may result. This inter-basin transfer must be prevented since it violates a basic civil drainage law principle by discharging water onto a subservient property in a manner or quantity to do more harm than formerly. If the development significantly increases the drainage area tributary to the subdivision outfall, then inter-basin transfer into the property has occurred, which also must be prevented.

The policy of the City shall be to avoid inter-basin transfer of storm drainage runoff and to maintain the historic drainage path within the basin. 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.

2.3.2 Master Planning

As set forth in the policy statement 2.2.1, drainage planning is required for all new development plans. In recognition that drainage boundaries are non-jurisdictional, the City has participated in regional basin wide master plans to define the Major Drainageway Facilities. The City will also encourage and participate in future master plans.

The policy of the City shall be to encourage the development of detailed regional drainage master plans by the city which will set forth site requirements for new development and identify the required public improvements. Master plans will be approved and adopted by City Council. Plans by developers will be integral parts of the whole.

2.3.3 Public Improvements

When the drainage master plans identify that public improvements are justified, mechanisms for funding the improvements are required. The funding should equitably distribute the initial costs and maintenance cost in proportion to the benefits received.

Included with the public improvements defined by drainage master plans is the Local Drainage System and the Major Drainageway System. The Local Drainage System consists of curb and gutter, inlets and storm sewers, culverts, bridges, swales, ditches, channels, detention areas, and other drainage facilities within the development required to convey the minor and major storm runoff to the Major Drainageways. The Major Drainageway System consists of channels, storm sewers, bridges, detention areas, and other facilities serving more than the subdivision or property in question.

The policy of the City requires that all new development and redevelopment shall participate in the required drainage improvements as set forth below:

- 1. Design and construct the local drainage system as defined by the final drainage study and plan (sections 3.3 and 3.4 of the CRITERIA).**
- 2. Design and construct connection of the local drainage system to the major drainageway system. The City will require that the connection of the minor and major systems provide capacity to convey the flows (including offsite flows) leaving the specific development site. To minimize overall capital costs, the City encourages adjacent developments to join in designing and constructing connection systems. Also, the City may choose to participate with a developer in the design and construction of the connection system.**
- 3. Equitable participation in the design and construction of the major drainageway system within the development as defined by adopted master**

drainage plans (section 2.3.2 of the CRITERIA) or as required by the City and designated in the final drainage study.

2.3.4 Storm Runoff Detention and Water Quality Enhancement

The value of storm runoff detention as part of the urban system has been explored by many individuals, agencies, and professional societies. Detention is considered a viable method to reduce urban drainage costs. Temporarily detaining a few acre-feet of runoff can significantly reduce downstream flood hazards as well as pipe and channel requirements in urban areas. Storage also provides for sediment and debris collection which helps to keep streams and rivers cleaner. Thus, public health benefits may accrue from storage of storm runoff.

The policy of the City requires onsite detention for all new development, expansion, and redevelopment. The required minimum detention volume and maximum release rates at these volumes for the minor and major storm interval shall be determined in accordance with the procedure and data set forth in this CRITERIA.

Exceptions to this Paragraph (2.3.4) are found at Chapter 1, Paragraph 1.3.2, this CRITERIA.

Onsite detention requirements in all incorporated city areas will be waived where regional detention facilities are sized with the capacity to accommodate 100-year storm event flows from a fully developed basin and are publicly owned and maintained.

2.3.5 Operations and Maintenance

An important part of all storm drainage facilities is the continued maintenance of the facilities to ensure they will function as designed. Maintenance of detention facilities involves removal of debris and sediment. Such tasks are necessary to preclude the facility from becoming unhealthy and to retain the effectiveness of the detention basin. Sediment and debris must also be periodically removed from channels and storm sewers. Trashracks and street inlets must be regularly cleared of debris to maintain system capacity. Channel bank erosion, damage to drop structures, crushing of pipe inlets and outlets, and deterioration to the facilities must be repaired to avoid reduced conveyance capability, unsightliness, and ultimate failure.

Maintenance responsibility lies with the owner of the land, except as modified by specific agreement. Maintenance responsibility shall be delineated on Final Plats and Final Development Plans. Maintenance access for detention ponds need not be specified but must be adequate for maintenance and be shown on the Final Plats and Final Development Plans.

The policy of the City requires that maintenance access be provided to all storm drainage facilities to assure continuous operational capability of the system. The property owner shall be responsible for the maintenance of all drainage facilities including inlets, pipes, culverts, channels, ditches, hydraulic structures, and detention basins located on their land unless modified by the subdividers agreement.

REQUIRED OPERATIONS AND MAINTENANCE EASEMENT

<u>DRAINAGE FACILITY</u>	<u>EASEMENT WIDTH</u>
1. Storm Sewer	
(a) Less than 36" dia.	20'
(b) Equal to or greater than 36" dia.	25'
	with sewer placed at a 1/3 point in the easement
2. Open Channel/Swales	
(a) Q ₁₀₀ less than 20 cfs	20'
(b) Q ₁₀₀ less than 100 cfs	25'
(c) Q ₁₀₀ greater than 100 cfs	see Figures-603, 604, and -605
3. Detention Pond	As required to contain storage and associated facilities plus adequate maintenance access around perimeter.

Drainage easements shall be shown on the Final Plats and Final Development Plan and state that the City has the right of access on the easements which shall be kept clear of obstructions to the flow and/or maintenance access.

2.4 TECHNOLOGY PLANNING AND DESIGN

2.4.1 Drainage Criteria

The design criteria presented herein represent the state-of-the-art for stormwater management. The criteria are intended to establish guidelines, standards, and methods for effective planning and design. The criteria shall be revised and updated as necessary to reflect advances in the field of urban drainage engineering and urban water resources management.

The policy of the City requires that all storm drainage facilities shall be planned and designed in accordance with the criteria set forth in this

document. The criteria will be revised or amended as new technology is developed and/or experience is gained in the use of this document.

Due to the dynamic nature of urbanization, the needs of the public will change with time, requiring adjustment of design concepts. Therefore, a time limitation on the approved construction plans shall be made:

Construction of any drainage facility not initiated within a 180 day period from time of approval of the drainage study will be re-evaluated and be subject to re-approval by the City.

2.4.2 Minor and Major Drainage System

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned or designed. One is the Minor Drainage System and the other is the Major Drainage System, which are combined to form the Total Drainage System.

The Minor Drainage System is designed to transport the runoff from five year frequency events with a minimum disruption to the urban environment. Minor storm drainage can be conveyed in the curb and gutter area of the street or roadside ditch (subject to street classification and capacity, as defined herein), by storm sewer, channel, or other conveyance facility.

The policy of the City requires that all minor drainage systems be sized without reduction for volumes detained on site.

The Major Drainage System is designed to convey runoff from the 100-year recurrence interval flood to minimize health and life hazards, damage to structures, and interruption to traffic and services. Major storm flows can be carried in the urban street system (within acceptable depth criteria), channels, storm sewers, and other facilities.

The policy of the City requires that all subdivisions, resubdivision, planned unit development, or any other proposed construction include the planning, designing, and implementing the minor and major drainage systems in accordance with the following recurrence intervals:

LAND USE	RECURRENCE INTERVAL (YEARS) MINOR DRAINAGE SYSTEM	RECURRENCE INTERVAL (YEARS) MAJOR DRAINAGE SYSTEM
RESIDENTIAL	5	100
COMMERCIAL/BUSINESS	5	100
INDUSTRIAL	5	100
OPEN SPACE	5	100
OTHER/PUBLIC	5	100

2.4.3 Storm Runoff

The MANUAL allows storm runoff to be determined by the Colorado Urban Hydrograph Procedure (CUHP) method for basins greater than 90 acres and by the Rational Method for basins less than 160 acres.

The policy of the City allows storm runoff to be determined by either the rational method or the Colorado Urban Hydrograph Procedure (CUHP), within the limitations as set forth in this CRITERIA. For basins larger than 160 acres, the peak flows and volumes shall be determined by the Colorado Urban Hydrograph procedure.

2.4.4 Streets

Streets are an integral part of the urban drainage system and may be used for transporting storm runoff up to design limits. The engineer designer should recognize that the primary purpose of streets is for traffic, and therefore the full cross sectional use of streets for storm runoff must be restricted.

The policy of the City allows the use of streets for drainage within the following limitations:

ALLOWABLE USE OF STREETS FOR MINOR STORM RUNOFF

DRAINAGE CLASSIFICATION	MAXIMUM THEORETICAL STREET ENCROACHMENT
Type A (36 feet or less, flowline to flowline width)	No curb overtopping. Flow may spread to crown of street.
Type B (36+ feet to 60 feet, flowline to flowline width)	No curb overtopping. Flow spread must leave at least one equivalent lane free from water at crown of roadway.
Type C (Over 60 feet, flowline to flowline width)	No curb overtopping. Flow spread must leave at least two center lanes free from water, (one lane each direction).

The maximum allowable street flow shall be the product of the flow calculated at the "Maximum Theoretical Street Depth" and the required reduction factor as provided in this CRITERIA.

ALLOWABLE CROSS STREET FLOW *

DRAINAGE CLASSIFICATION (See above description)	MINOR DRAINAGE SYSTEM MAXIMUM DEPTH	MAJOR DRAINAGE SYSTEM MAXIMUM DEPTH
Type A	6-inches of depth in cross pan or gutter	12-inches of depth at gutter flowline
Type B	6-inches of depth at the gutter flowline	12-inches of depth at gutter flowline
Type C	None	None

* Cross street flow can occur in an urban drainage system under three conditions. One condition occurs when the runoff in a gutter spreads across the street crown to the opposite gutter. The second is when cross pans are used. The third condition occurs when the flow in a drainageway exceeds capacity of a road culvert and subsequently overtops the crown of the street.

ALLOWABLE CULVERT OVERTOPPING

DRAINAGE CLASSIFICATION (See above description)	MINOR DRAINAGE SYSTEM MAXIMUM DEPTH	MAJOR DRAINAGE SYSTEM MAXIMUM DEPTH
Type A	No culvert overtopping	12-inches of depth at the gutter flowline
Type B	No culvert overtopping	12-inches of depth at the gutter flowline
Type C	No culvert overtopping	None. Minimum clearance between the culvert crown and the energy grade line shall be 6-inches for basins less than 2 square miles, 1 foot for basins up to 10 square miles, and 2 feet for basins greater than 10 square miles.

The maximum headwater for the 100-year design flows shall be 1.5 times the culvert diameter or 1.5 times the rise dimension for pipe shapes other than round. This applies to Type A and Type B streets only.

2.4.5 Floodproofing

Floodproofing can be defined as those measures which reduce the potential for flood damages to existing properties within a floodplain. The floodproofing measures can range from elevating structures to intentional flooding of non-critical building spaces to minimize structural damages. Floodproofing measures are only a small part of good floodplain management which encourages wise floodplain development to minimize the adverse effects of floods.

The policy for existing structures is as follows:

The policy of the City shall be to encourage the floodproofing of existing structures not in conformance with the adopted floodplain regulations by utilizing the criteria presented in the *Colorado Floodproofing Manual* prepared by the Department of Natural Resources, Colorado Water Conservation Board, dated September 1983, or later editions.

2.5 IRRIGATION FACILITIES

2.5.1 *Drainageway Interaction*

There are irrigation ditches in the City area. The ditches have historically intercepted the storm runoff from the rural and agricultural type basins, generally without major problems. With urbanization of the basins, however, the storm runoff has increased in rate, quantity and frequency, as well as changes in water quality. The irrigation facilities can no longer be utilized indiscriminately as drainage facilities and therefore, policies have been established to achieve compatibility between urbanization and the irrigation facilities.

In evaluating the interaction of irrigation ditches with a major drainageway for the purpose of basin delineation, the ditch should not be utilized as a basin boundary due to the limiting flow capacity of the ditch. The ditches will generally be flowing full or near full during major storms and, therefore, the tributary basin runoff would flow across the ditch.

The policy of the City shall be to require drainage analysis to assure that an irrigation ditch does not intercept the storm runoff from the upper basin and that the upper basin is a tributary to the basin area downstream from the ditch.

Irrigation ditches are designed with flat slopes and limited carrying capacity, which decreases in the downstream direction. As a general rule, irrigation ditches cannot be used as an outfall point for the storm drainage system because of these physical limitations. In addition, certain ditches are abandoned after urbanization and therefore, could not be utilized successfully for storm drainage.

In certain instances, however, irrigation ditches have been utilized successfully as outfall points for the initial drainage system, but only after a thorough hydrological and hydraulic analysis. Since the owner's liability from ditch failure increases with the acceptance of storm runoff, the responsibility must be clearly defined before a combined system is approved.

The policy of the City shall be as follows:

- 1. To require development to direct the storm runoff into historic and natural drainageways and avoid discharging into the canal or ditch except as required by proven water rights.**
- 2. Whenever new development will alter patterns of the storm drainage into irrigation ditches by increasing flow rates, volumes, or changing points of concentration, the written consent from the ditch company shall be submitted with the development application. The discharge of runoff into the irrigation ditch shall be approved only if such discharge is consistent with an adopted master drainage plan and it is in the best interest of the city to accept such design.**
- 3. Whenever irrigation ditches cross major drainageways within the developing area, the developer shall be required to design and construct the appropriate structures to separate storm runoff from ditch flows subject to the condition noted in Item No. 2. above.**

CHAPTER 3: DRAINAGE PLANNING SUBMITTAL REQUIREMENTS

CHAPTER 3 SUBDIVISION PLANNING AND SUBMITTAL REQUIREMENTS

3.1 REVIEW PROCESS

All subdivisions, resubdivisions, planned unit developments, or other development(s) (excluding PUD amendments and administrative resubdivisions, and "in-fill" single family residential lots) within the jurisdiction of this CRITERIA shall submit a Preliminary Drainage Study, a Final Drainage Study, and as-built drawings in accordance with the requirements of this section. Three (3) copies of the drainage study shall be submitted to the Department of Community Development for review by the Department of Public Works, Engineering Division. One unaltered copy and one copy with comments will be retained by the City. The third copy, with identical comments, will be returned by mail or picked up by the consultant. The submittal shall include a declaration of the type of study submitted (i.e., Preliminary or Final). Standard Form SF-1 will be used to determine the adequacy of the submittal. Incomplete or absent information may result in the report being rejected for review.

A pre-application consultation is suggested of all applicants for all processing steps of the drainage study. Items to be reviewed may include the required procedures, drainage challenges, and specific submittal requirements.

3.2 PRELIMINARY DRAINAGE STUDY

The purpose of the Preliminary Drainage Study is to identify and define conceptual solutions to the problems which may occur on site and off site as a result of the development. In addition, those problems that exist on site prior to development must be addressed during the preliminary phase. All studies shall be typed on 8-1/2" x 11" paper and bound. The drawings, figures, plates, and tables shall be bound with the study or included in a folder/pocket attached inside the back cover of the study. The study shall include a cover letter which presents the preliminary design for review and shall be prepared by or under the supervision of a professional engineer who is licensed in Colorado. The study shall be certified as follows:

"I hereby certify that this preliminary study for the (Name of Development) was prepared by me (or under my direct supervision) in accordance with the provisions of the City of Commerce City *Storm Drainage Design and Technical Criteria Manual* for the owners thereof."

Name
Registered Professional Engineer
State of Colorado No. _____
(Affix Seal)

3.2.1 Study Format and Required Information

The Preliminary Drainage Study shall be in accordance with the following outline and contain the applicable information listed.

I. GENERAL LOCATION AND DESCRIPTION

A. Location.

1. City and County, and local streets within and adjacent to the site or the area to be served by the drainage improvements.
2. Township, range, section, 1/4 section.
3. Major drainageways and facilities.
4. Names of surrounding developments.

B. Description of Property.

1. Area in square feet (or acres).
2. Ground Cover (type of existing and proposed ground cover and vegetation).
3. Major drainageways.
4. General project description.
5. Proposed land use.

II. DRAINAGE BASINS AND SUB-BASINS.

A. Major Basin Description.

1. Reference to major drainageway planning studies such as flood hazard delineation reports, major drainageway planning reports, and flood insurance rate maps (on file with the Department of Community Development).
2. Major basin drainage characteristics.
3. Identification of all nearby irrigation facilities within 100-feet of the property boundary, which will influence or be influenced by the local drainage.

B. Sub-Basin Description.

1. Discussion of historic drainage patterns of the property and 100 feet adjacent thereto.
2. Discussion of offsite drainage flow patterns and impact of development.

III. DRAINAGE DESIGN CRITERIA

A. Regulations: Discussion of compliance with or deviation from the CRITERIA, if any, and its justification.

B. Development Criteria Reference and Constraints.

1. Discussion of previous drainage studies (i.e., project master plans) for the site in question that influence or are influenced by the drainage design and how it will affect drainage design for the site.
2. Discussion of the effects of adjacent drainage studies.
3. Discussion of the drainage impact of site constraints such as streets, utilities, existing structures, and development of site plan.

- C. Hydrological Criteria.
 - 1. Identify design rainfall.
 - 2. Identify runoff calculation method.
 - 3. Identify detention discharge and storage calculation method.
 - 4. Identify design storm recurrence intervals.
 - 5. Discussion and justification of other criteria or calculation methods used that are not presented in or referenced by the CRITERIA.
 - D. Hydraulic Criteria.
 - 1. Identify various capacity references.
 - 2. Discussion of other drainage facility design criteria used that are not presented in the CRITERIA.
- IV. DRAINAGE FACILITY DESIGN
- A. General Concept.
 - 1. Discussion of concept and typical drainage patterns.
 - 2. Discussion of offsite runoff considerations.
 - 3. Discussion of the content of tables, charts, figures, plates, or/and drawings presented in the study.
 - 4. Discussion of anticipated and proposed drainage patterns.
 - B. Specific Details.
 - 1. Discussion of drainage problems encountered and proposed solutions.
 - 2. Discussion of detention storage and outlet design.
 - 3. Discussion of maintenance access and aspects of the design.
 - 4. Discussion of easements and tracts for drainage purposes, including the conditions and limitations for use.
- V. CONCLUSIONS.
- A. Compliance with Standards.
 - 1. "CRITERIA".
 - 2. "Major Drainageway Planning Studies".
 - 3. "MANUAL".
 - B. Drainage Concept.
 - 1. Effectiveness of drainage design to control damage from storm runoff.
 - 2. Influence of proposed development on the Major Drainageway Planning Studies recommendation(s).
- VI. REFERENCES
- Reference all criteria and technical information used.
- VII. APPENDICES.
- A. Hydrologic Computations.
 - 1. Land use assumptions regarding adjacent properties (upland and downland).

2. Historic and fully developed runoff computations for proposed land use.
 3. Initial and major storm runoff for each basin.
 4. Runoff coefficients for each basin.
- B. Hydraulic Computations.
1. Culvert (if applicable) capacities.
 2. Storm sewer (if applicable) capacity.
 3. Detention area/volume capacity and outlet capacity calculations. Depths of detention basins.
 4. Downstream/outfall system capacity to the Major Drainageway System.

3.2.2 Drawing Contents

(a) General Location Map: All drawings shall be 18" x 24" or 24" x 36" in size. A map shall be provided in sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map shall be at a scale of 1" = 1000' or 1" = 2000' and show the path of all drainage from the upper end of any offsite basin(s) to the defined major drainageways.

(b) Floodplain Information: A copy of the appropriate Flood Insurance Rate Map showing the location of the subject property shall be included with the report. All major drainageways (see Section 2.2.5) shall have the flow path defined and be shown on the report drawings. The Department of Community Development can be contacted for detailed information on floodplains and Flood Insurance Rate Maps.

(c) Drainage Plan: Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on an 18" x 24" or 24" x 36" drawing shall be included. The drawing shall show the following:

1. Existing and proposed contours at 2-foot maximum intervals tied to USGS datum unless topography is too shallow (flat), then use 1 foot contours. The contours shall extend a minimum of 100 feet beyond the property lines.
2. Property lines and easements with purposes noted.
3. Streets, indicating ROW width, flowline width, curb type, sidewalk, and approximate slopes.
4. Existing drainage facilities and structures, including irrigation ditches, roadside ditches, drainageways, gutter flow directions, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be included.
5. Over-all drainage area boundary and drainage sub-area boundaries.

6. Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditch, gutter, slope and flow directions, and cross pans.
7. Proposed storm sewers and open drainageways, including inlets, manholes, culverts, and other appurtenances, including riprap protection.
8. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.
9. Routing and accumulation of flows at various critical points for the initial storm runoff listed on the drawing using the format shown in Table-301.
10. Routing and accumulation of flows at various critical points for the major storm runoff listed on the drawing using the format shown in Table-301.
11. Volumes and release rates for detention storage facilities and information on outlet works.
12. Location and elevations of all existing floodplains affecting the property.
13. Location and (if known) elevations of all existing and proposed utilities affected by or affecting the drainage design.
14. Routing of offsite drainage flow through the development.
15. Definition of flow path leaving the development through the downstream properties ending at a major drainageway.
16. Legend to define map symbols (see Table-301 for symbol criteria).
17. Title block in lower right hand corner.

3.3 FINAL DRAINAGE STUDY

The purpose of the Final Drainage Study is to update the concepts, and to present the design details for the drainage facilities discussed in the Preliminary Drainage Study. Also, any change to the preliminary concept must be presented.

The format of the study shall be as stated in paragraph 3.2 above.

3.4 CONSTRUCTION DRAWINGS AND SPECIFICATIONS

Where drainage improvements are to be constructed in accordance with the approved Final Drainage Study, the construction plans and specifications shall be submitted for review and approval at time of application for a building permit. A reproducible copy of the approved plans shall be submitted to the City for file. The plans and specifications for the drainage improvements shall include:

1. Storm sewers, inlets, outlets and manholes with pertinent elevations, dimensions, type, and horizontal control indicated.
2. Culverts, end sections, and inlet/outlet protection with dimensions, type, elevations, and horizontal control indicated.
3. Channels, ditches, and swales (including side/rear yard swales) with lengths, widths, cross-sections, and erosion control (i.e., riprap, concrete, grout) indicated.
4. Checks, channel drops, erosion control facilities.
5. Detention pond grading, trickle channels, outlets, and landscaping.
6. Other drainage related structures and facilities (including underdrains and sump pump lines).
7. Maintenance access considerations.
8. Overlot grading plan.

The information required for the plans shall be in accordance with sound engineering principles, this CRITERIA, the City of Commerce City *Engineering Construction Standards and Specifications*, and the City requirements for subdivision designs. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the storm drainage facility. Construction plans shall be signed by a registered professional engineer as being in accordance with the City approved drainage report/drawings.

3.5 RECORD DRAWINGS (AS-BUILT DRAWINGS) AND FINAL ACCEPTANCE CERTIFICATE

Record drawings for all improvements are to be submitted to the City with the request for Probationary Acceptance. Two blue lines with certification shall be submitted. Certification of the record drawings is required as follows:

A professional engineer or land surveyor registered in the State of Colorado shall undertake such investigation as may be necessary to determine or confirm the as-built detention pond volumes and surface areas at the design depths, outlet structure sizes and elevations, storm sewer sizes and invert elevations at inlets, manholes and discharge location, and representative open channel cross-sections, and dimensions of all the drainage structures. If the improvements for a project are constructed in phases, as-built drawings may be required at the completion of each phase.

The professional engineer or land surveyor shall state on the as-built drawing: "to the best of my knowledge, belief, and opinion, the drainage facilities were constructed in accordance with the design intent of the approved drainage report and plan sheet(s)."

The City Engineer will compare the certified record drawing information with the construction drawings. A Certificate of Acceptance will be issued only if:

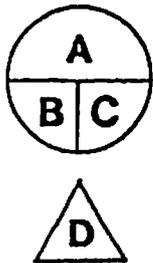
1. The record drawing information demonstrates (with accompanying calculations) that the construction is in compliance with the design intent.
2. The record drawings are certified by a professional engineer or land surveyor.

A summary of the required certifications and approvals follows:

<u>ITEM</u>	<u>CERTIFICATION REQUIRED</u>
Preliminary Drainage Study	Engineer
Final Drainage Study	Engineer
Plan Sheet Designs	Engineer
Record (As-Built) Drawings	Engineer or Land Surveyor

All studies are to be signed and stamped on cover sheet. All plan sheets are to be signed and stamped on each sheet.

**DRAWING SYMBOL CRITERIA AND HYDROLOGY
REVIEW TABLE**



A=BASIN DESIGNATION

B=AREA IN ACRES

C=COMPOSITE RUNOFF COEFFICIENTS

D=DESIGN POINT DESIGNATION

**SUMMARY RUNOFF TABLE
(to be placed on drainage plan)**

DESIGN POINT	CONTRIBUTING AREA (ACRES)	RUNOFF 5yr (CFS)	PEAK 100yr (CFS)
XX	XX·XX	XX·X	XX·X

CHAPTER 4: RAINFALL

CHAPTER 4 RAINFALL

4.1 INTRODUCTION

Presented in this section is the design rainfall data to be used with the Rational Method and the Colorado Urban Hydrographic Procedure (CUHP). All hydrological analysis within the jurisdiction of the CRITERIA shall utilize the rainfall data presented herein for calculating storm runoff.

Interpolation and analysis of the rainfall data published by the National Oceanic and Atmospheric Administration (NOAA) in the "Precipitation-Frequency Atlas of the Western United States, Volume III - Colorado" was used to develop one-hour and six-hour point rainfall values for the City of Commerce City. These point values were then used to develop two-hour and three-hour design rainfall distributions as well as time-intensity-frequency curves. The design storms were defined using the procedures developed by the DISTRICT.

4.2 CUHP DESIGN STORMS

4.2.1 *Basis of Design Storm Distribution*

Rainfall in the City of Commerce City is influenced by the orthographic effects of the Rocky Mountains, the topography of the high plains and the semi-arid climatology of the region. Rainstorms can often have an "upslope" character where easterly flow of moisture settles against the mountains. These types of rainstorms have durations that can exceed six-hours and produce large quantities of total precipitation.

Very intense rainfall in the City of Commerce City area results from convection storms or frontal stimulated convective storms. These types of storms are often less than two-hours in duration and can produce brief periods of high rainfall intensities. These short duration, but intense rainstorms cause most of the urban flooding problems.

Analysis of a 73-year record of rainfall at the Denver rain gage by the DISTRICT reveals that an overwhelming majority of the intense rainstorms produce their greatest intensities in the first hour of the storm. In fact, of the 73 most intense storms analyzed, 68 had the most intense period beginning and ending within the first hour of the storm and 52 had the most intense period beginning and ending within the first half hour of the storm. The data clearly shows that the leading intensity storms predominate among the "non-upslope" (northerly or westerly) type storms in the Denver Region.

The recommended design storm distribution takes into account the observed "leading intensity" nature of the convection storms. In addition, the temporal

distributions were designed to be used with the 1982 version of the CUHP, the published NOAA one-hour precipitation values, and the Horton's infiltration loss equation. They were developed to approximate the recurrence frequency of peak flows and volumes (i.e., 2- through 100-years) that were estimated for the watershed for which rainfall/runoff data was collected.

4.2.2 Basins Less Than Five Square Miles

For drainage basins less than five square miles, a two-hour storm distribution without area adjustment of the point rainfall values shall be used for CUHP. The incremental rainfall distribution is presented in Table-401.

4.2.3 Basins Between Five and Ten Square Miles

For drainage basins between five and ten square miles, a two-hour storm distribution is used but the incremental rainfall values are adjusted for the large basin area in accordance with the procedures in the NOAA Atlas for Colorado. The adjustment is to relate the average of all point values for a given duration and frequency within a basin to the average depth over the basin for the same duration and frequency. The incremental rainfall distribution is presented in Table-401.

4.3 TIME-INTENSITY-FREQUENCY CURVES

The one-hour design point rainfall values obtained from the NOAA Atlas for the Colorado Region and converted by the DISTRICT into maps of the Denver Region are presented below for the City of Commerce City:

<u>ONE-HOUR POINT RAINFALL (IN.)</u>				
<u>2 Year</u>	<u>5 Year</u>	<u>10 Year</u>	<u>50 Year</u>	<u>100 Year</u>
0.97	1.37	1.55	2.24	2.58

The Time-Intensity-Frequency curves were developed by distributing the one-hour point rainfall values using the factors obtained from the NOAA Atlas presented as follows:

<u>FACTORS FOR DURATIONS OF LESS THAN ONE HOUR</u>				
<u>Duration (minutes)</u>	<u>5</u>	<u>10</u>	<u>15</u>	<u>30</u>
<u>Ratio to 1-hour depth</u>	0.29	0.45	0.57	0.79

Source: NOAA Atlas 2, Volume III, Colorado, 1973

The point values were then converted to intensities and plotted on Figure-401. The data is also presented in Table-402.

Table 401

INCREMENTAL RAINFALL DEPTH/RETURN PERIOD															
TIME (MIN)	BASINS < 5 SQ MI					BASINS BETWEEN 5 AND 10 SQ MI									
	2-YR (IN)	5-YR (IN)	10-YR (IN)	50-YR (IN)	100-YR (IN)	2-YR (IN)	5-YR (IN)	10-YR (IN)	50-YR (IN)	100-YR (IN)	2-YR (IN)	5-YR (IN)	10-YR (IN)	50-YR (IN)	100-YR (IN)
5	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03
10	.04	.05	.06	.08	.08	.04	.05	.06	.08	.08	.04	.05	.06	.08	.08
15	.08	.12	.13	.11	.12	.08	.12	.13	.11	.12	.08	.12	.13	.11	.12
20	.15	.21	.23	.18	.21	.14	.20	.22	.18	.21	.14	.20	.22	.18	.21
25	.24	.34	.39	.34	.36	.23	.33	.37	.33	.35	.23	.33	.37	.33	.35
30	.13	.18	.19	.56	.65	.12	.17	.18	.54	.62	.12	.17	.18	.54	.62
35	.06	.08	.09	.27	.36	.06	.08	.09	.26	.35	.06	.08	.09	.26	.35
40	.05	.06	.07	.18	.21	.05	.06	.07	.18	.21	.05	.06	.07	.18	.21
45	.03	.05	.06	.11	.16	.03	.05	.06	.11	.16	.03	.05	.06	.11	.16
50	.03	.05	.05	.11	.13	.03	.05	.05	.11	.13	.03	.05	.05	.11	.13
55	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10
60	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10
65	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10	.03	.04	.05	.07	.10
70	.02	.04	.05	.05	.05	.02	.04	.05	.05	.05	.02	.04	.05	.05	.05
75	.02	.04	.05	.05	.05	.02	.04	.05	.05	.05	.02	.04	.05	.05	.05
80	.02	.03	.04	.04	.03	.02	.03	.04	.04	.03	.02	.03	.04	.04	.03
85	.02	.03	.03	.04	.03	.02	.03	.03	.04	.03	.02	.03	.03	.04	.03
90	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03
95	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03	.02	.03	.03	.03	.03
100	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03
105	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03
110	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03	.02	.02	.03	.03	.03
115	.01	.02	.03	.03	.03	.01	.02	.03	.03	.03	.01	.02	.03	.03	.03
120	.01	.02	.02	.03	.03	.01	.02	.02	.03	.03	.01	.02	.02	.03	.03

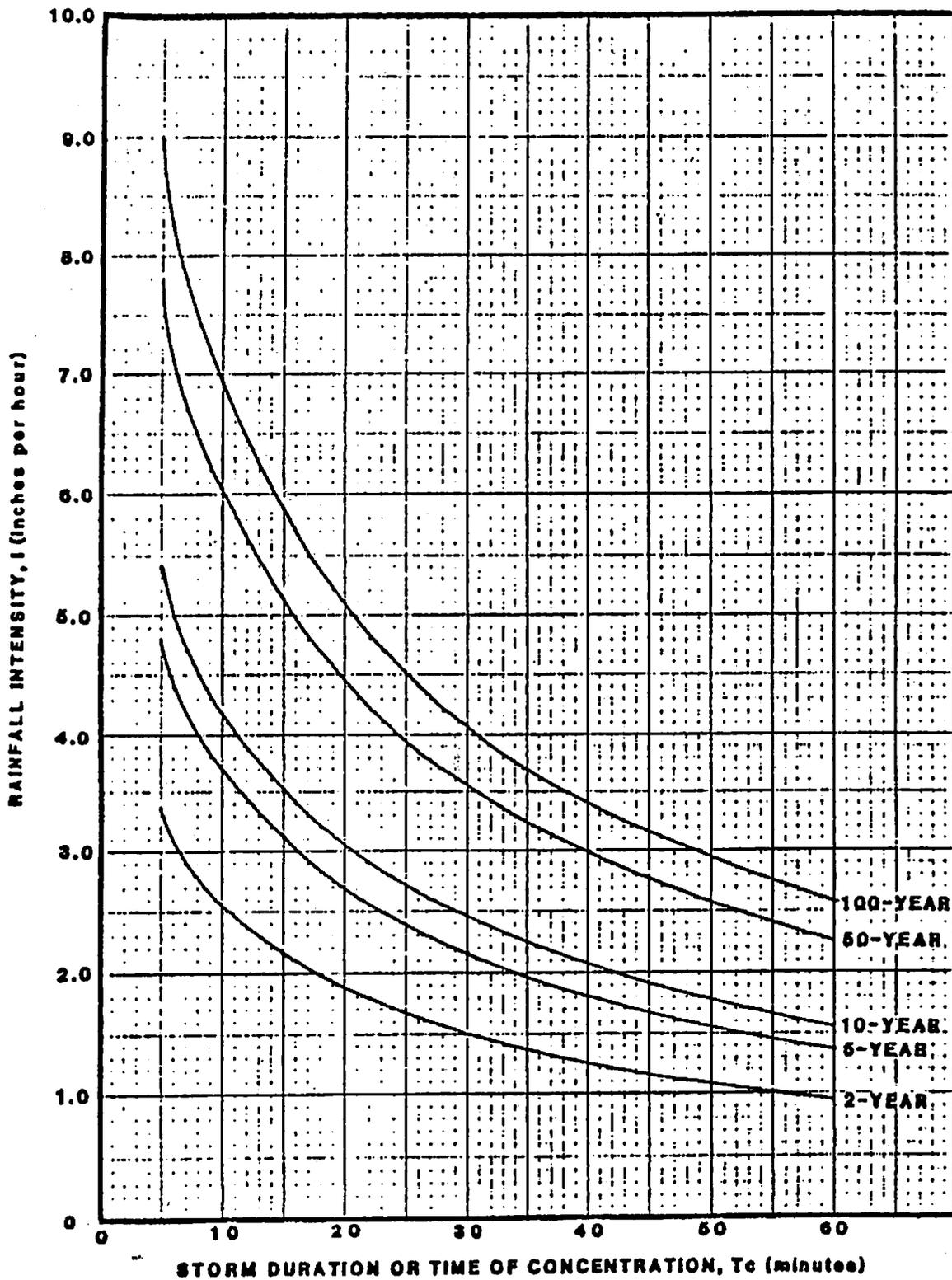
RAINFALL DEPTH/RETURN PERIOD

Table 402

DURATION FACTORS		5 MIN		10 MIN		15 MIN		30 MIN		60 MIN		
FREQ.		0.29		0.45		0.57		0.79		1.00		
	DEPTH (IN)	INTEN. (IN/HR)										
2-YEAR	0.28	3.36	0.43	2.58	0.54	2.16	0.75	1.50	0.95	0.95	0.95	
5-YEAR	0.40	4.80	0.62	3.72	0.78	3.12	1.08	2.16	1.37	1.37	1.37	
10-YEAR	0.45	5.40	0.70	4.20	0.88	3.52	1.22	2.44	1.55	1.55	1.55	
50-YEAR	0.65	7.80	1.01	6.06	1.28	5.12	1.77	3.54	2.24	2.24	2.24	
100-YEAR	0.75	9.00	1.16	6.96	1.47	5.88	2.04	4.08	2.58	2.58	2.58	

TIME-INTENSITY-FREQUENCY TABULATION

Note: Depth at each duration = (one hour rainfall depth) X (respective duration factor)



CHAPTER 5: RUNOFF

CHAPTER 5 RUNOFF

5.1 INTRODUCTION

This Chapter presents the criteria and methodology for determining the storm runoff design peaks and volumes to be used in the City for the preparation of storm drainage studies, plans, and facility design. The details of the rainfall/runoff models are presented in the MANUAL. The specific input data requirements and modifications to the procedures are presented in this Chapter.

5.2 RATIONAL METHOD

The Rational Method, in widespread use in the Denver Region, will continue to be utilized for the sizing of storm sewers and for determining runoff magnitude from unsewered areas. The new limit of application of the Rational Method is approximately 160 acres. It has been concluded that, for tributary basins in excess of 160 acres, the cost of the drainage works justifies significantly more study, thought, and judgment on the part of the engineer than is permitted by the Rational Method. When the urban drainage basin exceeds 160 acres, the CUHP method represents better practice and shall be used.

5.2.1 Rational Formula

The Rational Method is based upon the following formula:

$$Q = CIA \quad \text{(Equation 501)}$$

where

Q = Peak Discharge (cubic feet/second)
 C = Runoff Coefficient
 I = Rainfall Intensity (inches/hour)
 A = Drainage Area (Acres)

5.2.2 Time of Concentration (T_C)

For urban areas, the time of concentration consists of an inlet time or overland flow time (t_i) plus the time of travel (t_t) in a storm sewer, paved gutter, roadside drainage ditch, drainage channel, or other drainage facilities. 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 latter portion (t_t) of the time of concentration is 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 (t_c) shall be calculated using the following equation for both urban and non-urban areas:

$$t_c = t_i + t_t \quad \text{(Equation 502)}$$

where

t_c = time of concentration (minutes)

t_i = initial, inlet, or overland flow times (minutes)

t_t = travel time in the ditch, channel, gutter, storm, etc. (minutes)

5.2.2.1 Non-Urbanized Basins

The initial or overland flow time (t_i) is calculated using the following equation:

$$t_i = \frac{1.8 (1.1 - C_5) \sqrt{L}}{\sqrt[3]{S}} \quad \text{(Equation 503)}$$

where

t_i = initial or overland flow time (minutes)

C_5 = runoff coefficient for 5-year frequency

L = Length of overland flow, (feet)

S = average basin slope (percent)

The equation shall be used for distances up to 300 feet. For longer basin lengths, the runoff will be considered in a combined form and the travel time (t_t) shall be calculated using the hydraulic properties of the swale, ditch, or channel, or estimated using Figure 501. The time of concentration is then the sum of the initial flow time (t_i) and the travel time (t_t) in accordance with Equation 502. The minimum t_c shall be 10 minutes under non-urbanized conditions.

The use of Equations 502 and 503 is illustrated by the following example:

Therefore $t_c = 12.8$ minutes because it is the lesser of the two values for the first design point. For each successive downstream design point, calculate t_c by accumulating the travel times between each of the successive design points.

5.2.3 Rainfall Intensity (*I*)

The intensity (*I*) is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. After the design storm frequency has been selected, the rainfall intensity shall be obtained from Figure 401 or Table 402.

5.2.4 Runoff Coefficient (*C*)

The runoff coefficient (*C*) represents the integrated effects of infiltration, evaporation, retention, flow routing and interception, all of which effect the time distribution and peak rate of runoff. Table 501 presents the recommended values of *C* for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses. The coefficients for the various surface areas can be used to develop a composite value for a different land use, as described in the MANUAL.

5.2.5 Application of Rational Method

A valuable use of the Rational Method is in the evaluation of a proposed subdivision drainage system. The method can be used to define the design flows at various points in the subdivision to check the capacity of the streets, storm inlets, storm sewers, or other drainage facilities. The amount of detail needed to evaluate each subdivision depends upon the complexity of the drainage system. However, the use of the method is limited to watersheds of 160 acres or less.

5.3 COLORADO URBAN HYDROGRAPH PROCEDURE

The Colorado Urban Hydrograph Procedure (CUHP) was originally developed for the Denver area at the time the MANUAL was prepared. The method is generally applicable to basins greater than 90 acres. However, the CUHP is required for watershed areas larger than 160 acres. The procedures for the CUHP, as explained in the MANUAL, Volume-1 "Runoff", shall be followed in the preparation of drainage reports and storm drainage facility designs in the City. The design storms to be used with the CUHP method are presented in Table 501.

5.4. STORM FLOW ANALYSIS

5.4.1 Onsite Flow Analysis

When analyzing the flood peaks and volumes, the design engineer shall use the proposed fully developed land use plan to determine runoff coefficients. In addition, the engineer shall take into consideration the changes in flow patterns (from the undeveloped site conditions) caused by the proposed street alignments. When evaluating surface flow times, the proposed lot grading shall be used to calculate the time of concentration or the CUHP parameters.

5.4.2 Offsite Flow Analysis

The analysis of offsite runoff is dependent on the development status and whether the tributary offsite area lies within a major drainageway basin as defined in Section 2.2.5. In all cases, the minor system is designed for the fully developed five-year runoff (Section 2.4.2) without the benefits of onsite detention. In some cases credit is given for detention for the design of the major system.

5.4.2.1 Tributary Area Within a Major Drainageway Basin

- (a) Where the offsite area is undeveloped, the runoff shall be calculated assuming a fully developed basin under the current zoning as defined by the Planning Department. If this information is not available, then the runoff shall be calculated using the coefficients shown in Table 2-1 of Section 2.6, "Runoff", of the MANUAL. No credit will be given for onsite detention in the offsite area for any design frequency.
- (b) Where the offsite area is fully or partially developed, the storm runoff shall be based upon the existing platted land uses and topographic features. No credit will be given for onsite detention in the offsite area for any design frequency.

5.4.2.2 Tributary Area Not Within a Major Drainageway Basin

- a) Where the offsite area is undeveloped, the minor system (i.e., five-year) runoff shall be calculated assuming a fully developed basin under the current zoning as defined by the Planning Department.

If this information is not available, then the runoff will be calculated as stated in Section 5.4.2.1(a), without credit for onsite detention in the offsite area. The major system runoff (i.e., 100-year) may be calculated assuming the historic runoff rates computed in accordance with procedures described in Chapter 12 of these CRITERIA.

- (a) Where the offsite area is fully or partially developed, the storm runoff shall be based on the existing platted land uses and topographic

features, unless onsite detention in the offsite area has been constructed and accepted by the City. However, no credit will be given for onsite detention in the offsite area for the minor system design.

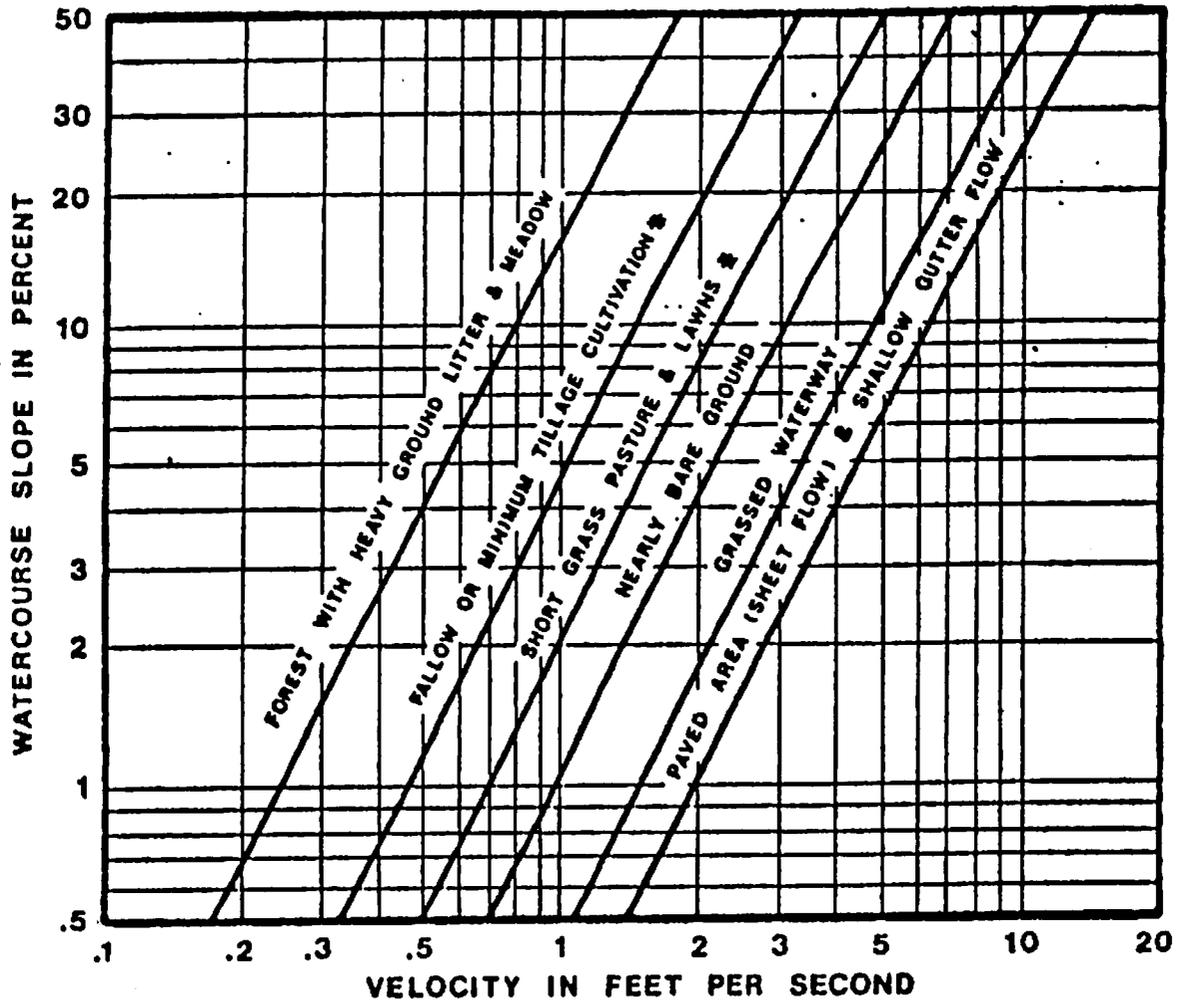
Table 501

RECOMMENDED RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	FREQUENCY			
		2	5	10	100
BUSINESS:					
Commercial Areas	95	.87	.87	.88	.89
Neighborhood Areas	70	.60	.65	.70	.80
RESIDENTIAL:					
Single-Family	45	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2 Acre Lot or Larger	40	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
INDUSTRIAL:					
Light Areas	80	.71	.72	.76	.82
Heavy Areas	90	.80	.80	.85	.90
PARKS, CEMETARIES:					
	7	.10	.10	.35	.60
PLAYGROUNDS:					
	13	.15	.25	.35	.65
SCHOOLS:					
	50	.45	.50	.60	.70
RAILROAD YARD AREAS:					
	40	.40	.45	.50	.60
UNDEVELOPED AREAS:					
Historic Flow Analysis	2	(See "Lawns")			
Greenbelts, Agricultural					
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.65
STREETS:					
Paved	100	.87	.88	.90	.93
Gravel	13	.15	.25	.35	.65
DRIVE AND WALKS:					
	96	.87	.87	.88	.89
ROOFS:					
	90	.80	.85	.90	.90
LAWNS, SANDY SOIL					
	0	.00	.01	.05	.20
LAWNS, CLAYEY SOIL					
	0	.05	.10	.20	.40

NOTE: These Rational Formula coefficients may not be valid for large basins.

RUNOFF COEFFICIENTS / PERCENT IMPERVIOUS



ESTIMATE OF AVERAGE FLOW VELOCITY FOR USE WITH THE RATIONAL FORMULA.

• MOST FREQUENTLY OCCURRING "UNDEVELOPED" LAND SURFACES IN THE DENVER REGION.

REFERENCE: "Urban Hydrology For Small Watersheds" Technical Release No. 55, USDA, SCS Jan. 1975.

CHAPTER 6 OPEN CHANNELS

CHAPTER 6 OPEN CHANNELS

6.1 INTRODUCTION

This chapter addresses the technical criteria for the hydraulic evaluation and hydraulic design of open channels in the City. In many instances, special design or evaluation techniques will be required. Except as modified herein, all open channel criteria shall be in accordance with the MANUAL.

6.2 CHANNEL TYPES

The channels in the City area are defined as natural or artificial. Natural channels include all water courses that have occurred naturally by the erosion process such as the South Platte River, Sand Creek River and First Creek. Artificial channels are those constructed or developed by human effort such as the large designated floodways, irrigation canals and flumes, roadside ditches, and grassed channels.

6.2.1 *Natural Channels*

The hydraulic properties of natural channels vary along the channel reach and can be either controlled to the extent desired or altered to meet given requirements. The initial decision to be made regarding natural channels is whether or not the channel is to be protected from erosion due to high velocity flows or protected from excessive silt deposition due to low velocities.

Many natural channels in urbanized and to-be-urbanized areas 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 urbanized area, the altered nature of the runoff peaks and volumes from urban development will cause erosion. Detailed hydraulic analysis will be required for natural channels in order to identify the erosion tendencies. Some onsite modifications 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 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 future conditions of flow. Supercritical flow does not normally occur in natural channels, but calculations must be made to assure that the results do not reflect supercritical flow.

6.2.2 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 engineer must give full consideration to sediment deposition and to scour as well as hydraulics. Unless existing development within the City restricts the availability of right-of-way (ROW), only channels lined with grass will be considered acceptable for major drainageways.

For the purpose of this CRITERIA, sandy soils are defined as non-cohesive sands classified as SW, SP, or SM in accordance with the Unified Soil Classification System.

6.2.3 Concrete Lined Channels

Concrete lined channels for major drainageways will be permitted only where ROW restrictions within existing development prohibit grass lined channels. The lining 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. This is especially true for supercritical flow conditions.

6.2.4 Rock Lined Channels

Riprap lined channels shall be permitted only in areas of existing development where ROW for major drainageways is limited and such limitation prohibits the use of grass lined channels. The advantage of rock lining a channel is that a steeper channel grade can be used due to the higher friction of the rock. Also, steeper side slopes are permitted. Rock linings (i.e., revetments) are permitted as a means of controlling erosion for natural channels.

6.2.5 Other Lining Types

The use of synthetic fabrics in drainage construction and geotechnical engineering has increased tremendously in the last decade. The placement of a slope revetment mat is a method of erosion control and the subject of discussion in this section.

The mattresses generally consist of double layers of woven fabric forms placed on the slope to be protected, and filled with concrete or grout. This type of forming system is a simple, fast, and economical technique for the placement of concrete for slope protection both above and below the water without the need for

dewatering. The performance characteristics and cost advantages make the process adaptable for stabilizing and protecting shorelines, levees, dikes, canals, holding basins, and similar erosion control projects.

The systems make use of the pressure injection of fluid fine-aggregate concrete into flexible fabric forms. Controlled bleeding of mixing water through the porous fabric produces all the desirable features of low water/cement ratio mortar-rapid stiffening, high strength, and exceptional durability.

For normal installations, the fabric forms, prefabricated to job specifications and dimensions, are simply spread over the terrain, which has received minimal grading. The fabric form is then pumped full of mortar. This same concept can be used where slide problems are caused by eroding of the toe of the slopes, and where access is difficult for placement of riprap. See Figure 601 for typical sections of mattresses.

There are several manufacturers of synthetic fabrics for erosion protection. Included in this category of channel lining are the products which consist of discrete blocks on a continuous fabric backing.

The use of synthetic fabrics for lining of channels for major drainageways within the City is restricted to areas of existing development where the ROW constraints prohibit the use of a grass lined section. The linings shall be restricted to channels with a Froude number of 0.8 or less.

6.3 FLOW COMPUTATION

Uniform flow and critical flow computations shall be in accordance with the MANUAL, Sections 2.2.3 and 2.2.4, "Major Drainage."

6.4 DESIGN STANDARDS FOR MAJOR DRAINAGEWAYS

6.4.1 *Natural Channels*

The design criteria and evaluation techniques for natural channels are:

1. The channel and overbank areas shall have adequate capacity for the 100-year storm runoff.
2. Natural channel segments which have a calculated Froude number greater than 0.95 for the 100-year flood peak shall be protected from erosion.
3. The water surface profiles shall be defined so that the floodplain can be zoned and protected.

4. Filling of the Floodplain District reduces valuable channel storage capacity and tends to increase downstream runoff peaks.
5. Roughness factors (n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles.
6. Roughness factors (n), which are representative of maintained channel conditions, shall be used to determine velocity limitations.
7. Erosion control structures, such as check drops or check dams, may be required to control flow velocities, including the minor storm runoff.
8. Plan and profile drawings of the floodplain shall be prepared and included in the Final Drainage Report. Appropriate allowances for known future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis. The applicant shall contact the Public Works Department for information on future bridges and culverts.

With most natural waterways, erosion control structures should be constructed at regular intervals to decrease the thalweg slope and to control erosion. However, these channels should be left in as near natural conditions 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 rules which are applicable to artificial channels, do not apply for natural channels. All structures constructed along the channel shall be elevated a minimum of one foot above the 100-year water surface. Significant advantages may occur if the designer incorporates into his planning the overtopping of the channel and localized flooding area into adjacent areas which have been developed for the purpose of inundation during the major runoff peak.

6.4.2 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.

1. Flow Velocity

The maximum normal depth velocity for the 100-year flood peak shall not exceed 7.0 feet per second for grass lined channels, except in sandy soil where the maximum velocity shall 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, wherever possible, shall be greater than 2.0 feet per second for the minor storm runoff

2. Longitudinal Channel Slopes

Grass lined channel slopes are dictated by velocity and Froude number requirements. Where the natural topography is steeper than desirable, drop structures shall be utilized to maintain design velocities and Froude numbers.

3. Freeboard

Except where localized overflow in certain areas is desirable for additional ponding benefits or other reasons, the freeboard for the 100-year flow shall be as determined by Equation 601. The minimum freeboard shall be 1.0 foot.

$$H_{FB} = 0.5 + \frac{V^2}{2g} \quad (\text{Equation 601})$$

where H_{FB} = Freeboard Height (feet)

V = Average Channel Velocity (feet per second)

g = Acceleration of Gravity = 32.2 ft/sec²

4. Curvature (Horizontal)

The center line curvature shall have a radius twice the top width of the design flow, but not less than 100 feet.

5. Roughness Coefficient

The variation of Manning's "n" (roughness coefficient) with the retardance and the product of mean velocity and hydraulic radius, as presented in Figure 602, shall be used in the capacity computation.

Retardance curve C (Figure 602) shall be used to determine the channel capacity, since a mature channel (i.e., substantial vegetation with minimal previous maintenance) will have a higher Manning's "n" value. However, a recently constructed channel will have minimal vegetation and the retardance will be less than the mature channel. Therefore, retardance

curve D (Figure 602) shall be used to determine the limiting velocity in a channel.

6. 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. Figure 603 and 604 present two cross sections for use with non-sandy soils. Figure 605 is the required cross section for sandy soils. The limitations within which the design must fall for the major storm design flow include:

a. Trickle Channel

The base flow shall be carried in a trickle channel except for sandy soils. The minimum capacity shall be 1.0 percent to 3.0 percent of the 100-year flow but not less than 1 cfs. Trickle channel shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance, and to aesthetically blend with the adjacent vegetation and soils. Typical trickle channel details are shown in Figure 606.

b. Main Channel

A main channel is required for sandy soils as shown in Figure 605. The side slopes can be from 2:1 to 2.5:1 if constructed from riprap. The depth of the main channel is not included in the normal depth limitation. A main channel can also be used for non-sandy soils, subject to the conditions shown in Figure 604.

c. Bottom Width

The minimum bottom width shall be consistent with the maximum depth and velocity criteria. The minimum width shall be four feet to accommodate the trickle channel, where required.

d. Right-of-Way (ROW) Width

The minimum ROW width shall include freeboard and a 12-foot wide maintenance access.

e. Flow Depth

The maximum design depth of flow (outside the trickle channel area and main channel area for sandy soils) for the 100-year flood peak shall be limited to 5.0 feet in grass lined channels.

f. Maintenance/Access Road

Continuous maintenance access shall be provided for all major drainageways with a minimum width of 12 feet. The City may require six inches of Class 6 road base or a concrete slab. (Refer to Section 2.3.5 of this CRITERIA.)

g. Side Slopes

Side slopes shall be 3 (horizontal) to 1 (vertical) or flatter.

7. Grass Lining

The grass lining for channels shall be in accordance with the MANUAL, Section 2.3.2, "Major Drainage".

8. Erosion Control

The requirements for erosion control for grass lined channels shall be as defined in the MANUAL, Section 2.3.6, "Major Drainage".

9. Water Surface Profiles

Computation of the water surface profile shall be presented in the final report for all open channels utilizing standard backwater methods, taking into consideration losses due to changes in velocity of channel cross section, drops, waterway openings, or obstructions. The energy gradient shall be shown on all final drawings.

6.4.3 *Concrete Lined Channels*

The criteria for the design and construction of concrete lined channels is presented below:

1. Hydraulics

a. Freeboard

Adequate channel freeboard above the designed water surface shall be provided and shall not be less than that determined by the following:

$$H_{FB} = 2.0 + 0.025 V (d)^{1/3} \quad (\text{Equation 602})$$

where H_{FB} = Freeboard Height (feet)

V = Velocity (feet per second)

d = Depth of Flow (feet)

Concrete side slopes shall be extended to provide freeboard.

Freeboard shall be in addition to superelevation, standing waves, and/or other water surface disturbances. These special situations should be addressed in the Final Drainage Report.

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 shall not exceed 18 fps during the 100-year flood.

2. Concrete Materials

- a. Cement type: sulphate resistant and air entrained.
- b. Minimum cement content: 550 lbs/C.Y.
- c. Maximum water-cement ratio: 0.50 (six gals. per sack).
- d. Maximum aggregate size: 1-1/2 inches.
- e. Air content range: 4 to 7 percent.
- f. Slump: 2 to 4 inches.
- g. Minimum compressive strength (f'_c): 3250 psi at 28 days.

3. Concrete Lining Section

- a. All concrete linings shall have a minimum thickness of 7 inches.
- b. The side slopes shall be a maximum of 2 (vertical) to 1 (horizontal), or a structurally reinforced wall if steeper.

4. Concrete Joints

- a. Channels shall be continuously reinforced without transverse joints. Expansion joints shall be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.
- b. Longitudinal joints, where required, shall be constructed on the side walls at least one foot vertically above channel invert.
- c. All joints shall be designed to prevent differential movement.
- d. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint.

5. Concrete Finish

Concrete surface shall be in accordance with the *Standard Specifications for Road and Bridge Construction*, 1986, and subsequent editions, Colorado Department of Highways, Section 608, Sidewalks, Subsection 608.03 (d).

6. Concrete Curing

All concrete shall be cured by the application of a liquid membrane-forming curing compound (white pigmented) upon completion of the concrete finish. "Protex" or equal shall be used.

7. Reinforcement Steel

- a. Steel reinforcement shall be minimum grade-40 deformed bars. Wire mesh shall not be used.
 - b. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than .005.
 - c. Ratio of transverse steel area to concrete cross sectional area shall be greater than .0025.
 - d. Reinforcing steel shall be placed at the center of the section with a minimum clear cover of two inches adjacent to the earth.
 - e. Additional steel as needed if a retaining wall structure is used.
8. Earthwork
The following areas shall be compacted to at least 95 percent of maximum density as determined by ASTM D698 (Standard Proctor):
- a. The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).
 - b. Top 12 inches of a drainageway maintenance road.
 - c. Top 12 inches of earth surface within 10 feet of concrete channel lip.
 - d. All fill material.
9. Bedding
Provide six inches of granular bedding equivalent in gradation to 3/4" concrete aggregate (Standard Specifications for Road & Bridge Construction, CDOH, current printing) under channel bottom and side slopes.
10. Underdrain
Longitudinal underdrains shall be provided on 10-foot centers and shall "daylight" at the check drops. A check valve or flap gate shall be provided at the outlet to prevent backflow into the drain. Weep holes shall be provided in vertical wall sections of the channel.
11. Safety Requirements
- a. A six-foot high vinyl coated chain link or comparable fence shall be installed to prevent access wherever the 100-year channel flow depths exceed three feet. Gates, with top latch, shall be placed at 250' intervals and staggered where fence is required on both sides of the channel.

- b. Ladder-type steps shall be installed not more than 400 feet apart on alternating sides of the channel. Bottom rung shall be placed approximately 12 inches vertically above channel invert.

6.4.4 Riprap Channel Linings

The criteria for the design and construction of riprap channel linings shall be in accordance with the MANUAL, Volume 2, "Major Drainage". Riprap lined channels shall be designed for a turbulence factor (Froude number) less than 0.8 for the 100-year flood peaks. The riprap shall be designed and constructed in accordance with Section 11.2 of this CRITERIA. Freeboard requirements shall be in accordance with the standards for grass lined channels defined in Section 6.4.2.3 of this CRITERIA.

6.4.5 Other Lining Types

The applicant is responsible to provide any necessary documenting data, as determined by the City Engineer, for any proposed material other than grass, rock, or concrete. The following minimum criteria will apply.

1. Flow Velocity
The maximum normal depth velocity will be dependent on the construction material utilized; however, the Froude number shall be equal to or less than 0.8.
2. Freeboard
Defined by Equation 601.
3. Curvature
See Section 6.4.2.4 of this CRITERIA.
4. Roughness Coefficient
A Manning's "n" value range shall be established by the manufacturer's data, with the high value used to determine depth/capacity requirements and the low value used to determine Froude number and velocity restrictions.
5. Cross Sections
Same as for grass lined channels, Section 6.4.2.6.

6.5 DESIGN STANDARDS FOR SMALL DRAINAGEWAYS

These standards cover the design of channels that are not classified as a major drainageway in accordance with the policy of Section 2.2.5. Additional flexibility and less stringent standards are allowed for small drainageways.

6.5.1 Natural Channels

The design criteria and evaluation techniques for natural channels are:

1. The channel and overbank areas shall have adequate capacity for the 100-year storm runoff.
2. Natural channel segments which have a calculated Froude number equal to or greater than 0.95 for the 100-year flood peak shall be protected from erosion.
3. Roughness factors (n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles.
4. Roughness factors (n), which are representative of maintained channel conditions, shall be used to determine velocity limitations.
5. Erosion control structures, such as check drops or check dams, may be required to control flow velocities, including the minor storm runoff.
6. Plan and profile drawings shall be prepared showing the 100-year water surface profile, floodplain, and details of erosion protection, if required.

6.5.2 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.

1. Flow Velocity: See Section 6.4.2.1 of this CRITERIA.
2. Longitudinal Channel Slopes: See Section 6.4.2.2 of this CRITERIA.
3. Freeboard: A minimum freeboard of 1 foot shall be included in the design for the 100-year flow. For swales (i.e., small drainageways with a 100-year flow less than 20 cfs), the minimum freeboard requirements are 6 inches.
4. Curvature (Horizontal): The center line curvature shall have a minimum radius twice the top width of the design flow but not less than 50 feet. The minimum radius for channels with a 100 year runoff of 20 cfs or less shall be 25 feet.

5. Roughness Coefficient: The variation of Manning's "n" with the retardance (curve "C") and the product of mean velocity and hydraulic radius, as presented in Figure 602, shall be used in the computation of capacity and velocity.
6. Cross Sections: The channel shape may be almost any type suitable to the location and to the environmental conditions. Some suggested cross sections are shown on Figures 603, 604, and 605. The section may also be simple V-Section for swales (i.e., Q_{100} less than 20 cfs). The limitations on the cross section are as follows:
 - a. Trickle Channel
The base flow (except for swales) shall be carried in a trickle channel for non-sandy soils. The minimum capacity shall be from 1.0 percent to 3.0 percent of the 100-year flow but not less than 1 cfs. The trickle channel can be constructed of concrete, rock, cobbles, or other suitable materials. For sandy soils, a main channel is required in accordance with Section 6.4.2.6(b). For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainageway slope, soil type, and upstream impervious area.
 - b. Right-of-Way (ROW) Width
The minimum ROW width shall include freeboard and should include a 12-foot wide maintenance access.
 - c. Flow Depth See Section 6.4.2.6.(e) of this CRITERIA.
 - d. Side Slopes See Section 6.4.2.6.(g) of this CRITERIA.
7. Grass Lining
The grass lining for channels shall be in accordance with the MANUAL, Section 2.3.2, "Major Drainage".
8. Erosion Control
The requirements for erosion control for grass lined channels shall be as defined in the MANUAL, Section 2.3.6, "Major Drainage". The design of riprap devices shall be in accordance with Sections 11.1 and 11.2 of this CRITERIA.
9. Hydraulic Information
Calculations of the capacity, velocity, and Froude numbers shall be submitted with the construction drawings.

6.5.3 Concrete Lined Channels

Same as Section 6.4.3. of this CRITERIA.

6.5.4 Riprap Lined Channels

Same as Section 6.4.4. of this CRITERIA.

6.5.5 Other Lining Types

The applicant is responsible to provide any necessary documenting data, as is determined by the City Engineer, for any proposed material other than grass, rock, or concrete. The following minimum will apply.

1. Flow Velocity See Section 6.4.5.1. of this CRITERIA.
2. Freeboard See Section 6.5.2.3. of this CRITERIA.
3. Curvature See Section 6.5.2.4. of this CRITERIA.
4. Roughness Coefficient See Section 6.4.5.4. of this CRITERIA.
5. Cross Sections See Section 6.5.2.6. of this CRITERIA.

6.6 ROADSIDE DITCHES

The criteria for the design of roadside ditches is similar to the criteria for grass lined channels with modifications for the special purpose of minor storm drainage. The criteria is as follows (refer to Figure 607):

1. Capacity Roadside ditches shall have adequate capacity for the five-year storm runoff peaks. Capacity shall be as defined in Table 601. Where the storm runoff exceeds the capacity of the ditch, a storm sewer system shall be required.
2. Flow Velocity The maximum velocity for the five-year flood peak shall not exceed 5.0 feet per second for Type-I ditch and 7.0 feet per second for Type-II or Type-III ditches.
3. Longitudinal Slope The slope shall be limited by the average velocity of the five-year flood peaks. Check drops may be required where street slopes are in excess of 2.0 percent.
4. Freeboard Freeboard shall be equal to the velocity head, or a minimum of 6 inches.
5. Curvature The minimum radius of curvature shall be 25 feet.
6. Roughness Coefficient Manning's "n" values presented in Figure 602 shall be used in the capacity computation for roadside ditches.
7. Grass Lining The grass lining shall be in accordance with the MANUAL, Section 2.3.2, "Major Drainage".
8. Driveway Culverts Driveway culverts shall be sized to pass the five-year ditch flow capacity without overtopping the driveway. The minimum size culvert shall be a 12" round pipe or equivalent with flared end sections. More than one culvert may be required.

9. **Major Drainage Capacity** The capacity of roadside ditches for major drainage flow is restricted by the maximum flow depth allowed at the street crown (Section 2.4.4 of this CRITERIA). However, the flow spread should not extend outside the street ROW.

6.7 CHANNEL RUNDOWNS

A channel rundown is used to convey surface storm runoff from gutters or paved areas at the top of the channel bank to the invert of an open channel or drainageway. The purpose of the structure is to minimize channel bank erosion from concentrated overland flow. Inlets and storm sewers may be used to convey the minor storm runoff with the rundown conveying the difference between the major and minor storm runoff.

6.7.1 Cross-Sections

A typical cross-section for a channel rundown is presented in Figure 608.

6.7.2 Design Flow

The channel rundown shall be designed to carry a minimum of the major storm runoff or 1 cfs, whichever is greater. When the minor storm flow is piped separate from the rundown, the design flow shall be the difference between the major and minor storm runoff or 1 cfs., whichever is greater.

6.7.3 Flow Depth

The maximum depth at the design flow shall be 12 inches. Due to the typical profile of a channel rundown beginning with a flat slope and then dropping steeply into the channel, the design depth of flow shall be computed critical depth for the design flow. In addition, each entrance condition shall be analyzed separately to ensure the entrance will have sufficient capacity without overtopping the channel sides.

6.7.4 Freeboard

Provide a minimum 6-inches of freeboard above the critical depth of the flow.

6.7.5 Outlet Configuration

The channel rundown outlet shall enter a drainageway at the trickle channel flowline. Erosion protection of the opposite channel bank shall be provided by a 24-inch layer of grouted Type-L riprap. The width of this riprap erosion protection shall be at least three times the channel rundown width or pipe diameter. Riprap protection shall extend up the opposite bank to the minor storm flow depth in the drainageway or two feet, whichever is greater.

6.8 CHECKLIST FOR ALL OPEN CHANNELS

To aid the designer and reviewer, the following checklist has been prepared.

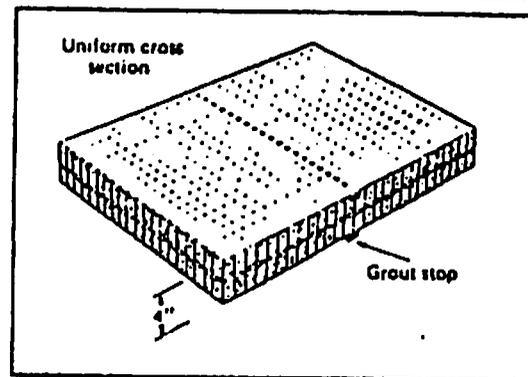
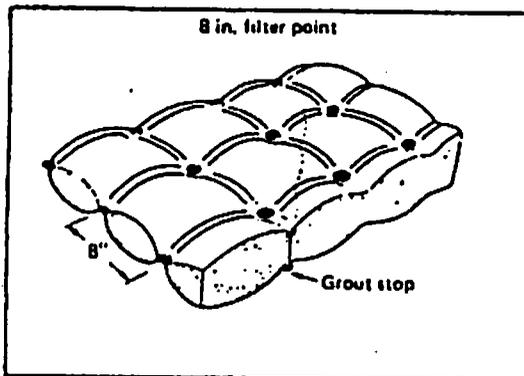
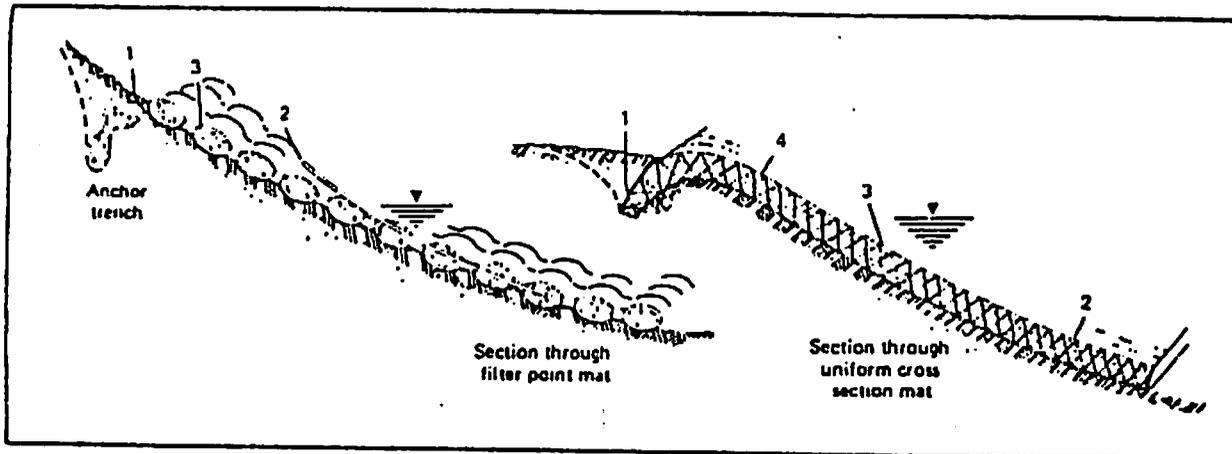
1. Check flow velocity with low retardance factor and capacity with high retardance factor.
2. Check Froude number and critical flow conditions.
3. Grass channel side slopes and roadside ditches must be 3:1 or flatter.
4. Show energy grade line and hydraulic grade line on design drawings.
5. Consider all backwater conditions (i.e., at culverts) when determining channel capacity.
6. Check flow velocity for flood conditions without backwater effects.
7. Provide adequate freeboard.
8. Provide adequate ROW for the channel and continuous maintenance access.

Table 601

SLOPE (%)	DITCH TYPE I		DITCH TYPE II		DITCH TYPE III	
	VEL. (FPS)	Q (CFS)	VEL. (FPS)	Q (CFS)	VEL. (FPS)	Q (CFS)
0.5	1.4	4	2.2	14	4.1	28
1.0	2.5	11	3.3	20	Not Permitted	Not Permitted
1.5	3.4	20	Not Permitted			
2.0	4.3	26	Not Permitted			
2.5	5.0	32	Not Permitted			
3.0 (4)	5.7	37	Not Permitted			

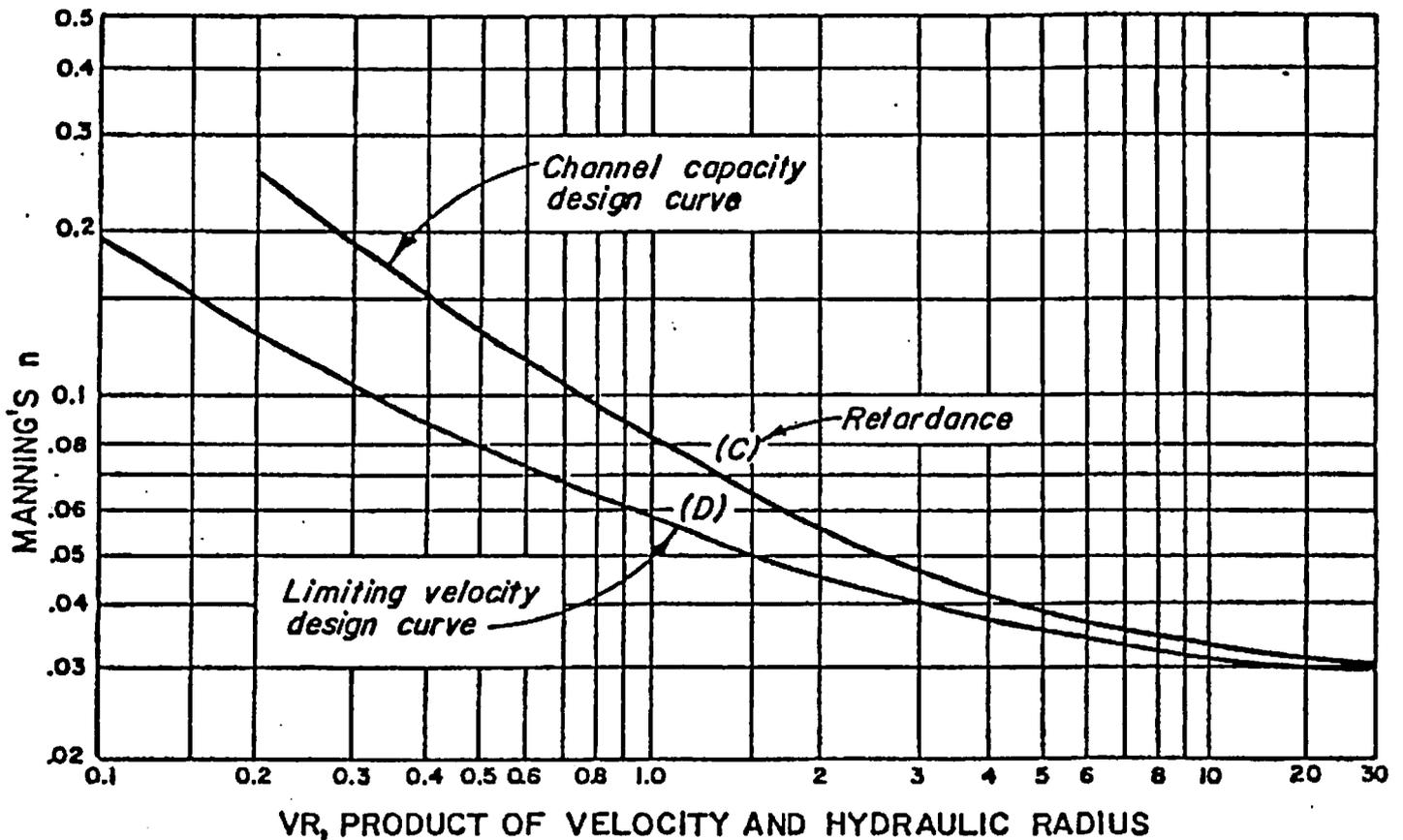
- Note: 1. See Figure 606 for geometry of roadside ditch.
2. Velocity is based upon the SCS Retardance Curve "D". See Figure 602.
3. Capacity is based upon the SCS Retardance Curve "C" See Figure 602.
4. Maximum permissible slope for roadside ditch is 3.0%. Slope limitation is based on a maximum Froude Number of 0.8 for Type I and II and 0.9 for Type III ditch.
5. Linearly interpolate for intermediate slopes.

ROADSIDE DITCH CAPACITIES



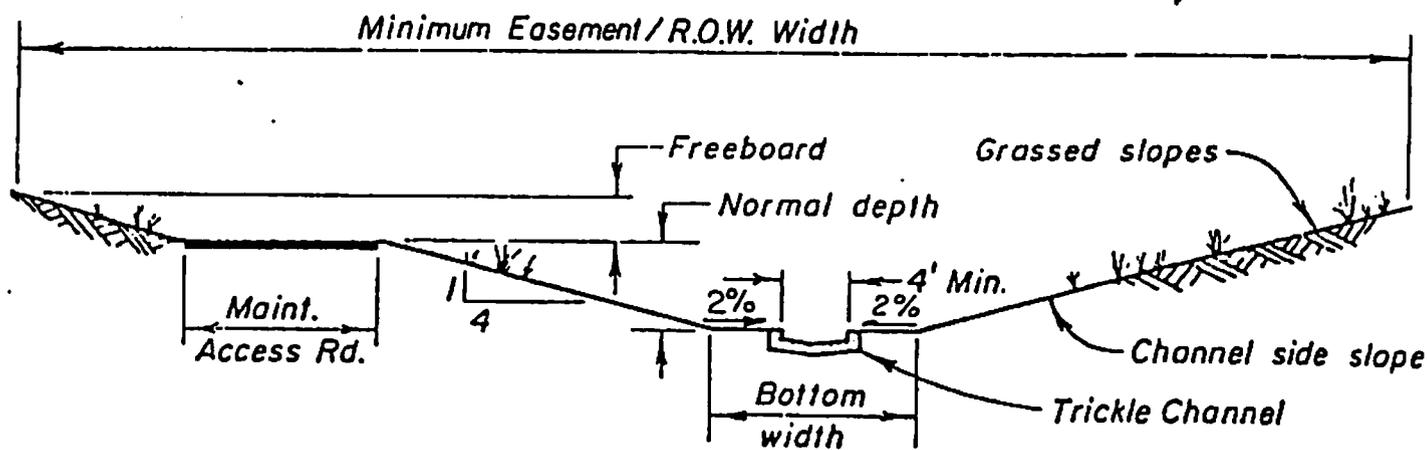
NOTE: Numbers refer to sequence of mortar injection.

Liners may be used provided the Froude Number of the channel section is less than 0.8.



From "Handbook of Channel Design For Soil and Water Conservation," U.S. Department of Agriculture, Soils Conservation Service, No. SCS-TP-61 March, 1947, Rev. June, 1954

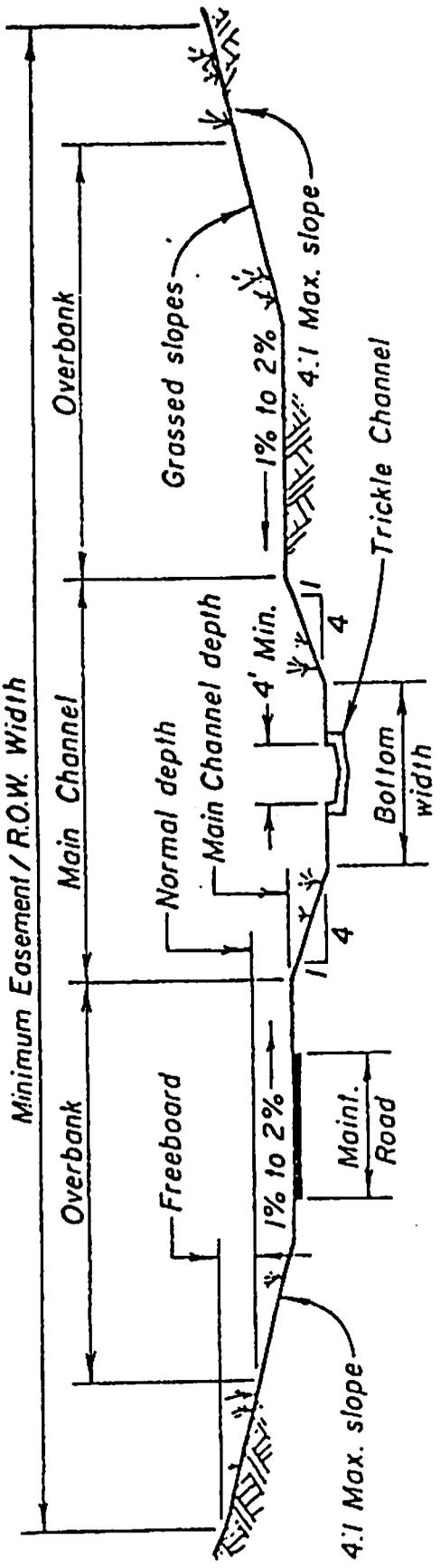
TYPE A



Notes:

1. Bottom Width: Consistent with maximum allowable depth and velocity requirements, shall not be less than trickle channel width.
2. Trickle Channel: Minimum capacity to be 1% to 3% of 100-year flow but not less than 1 cfs. Channel to be constructed of concrete, grouted riprap, or other approved materials. See Figure 605 for requirements on sandy soils.
3. Normal Depth: Normal depth at 100-year flow shall not exceed 5 feet. Maximum 100-year flow velocity at normal depth shall not exceed 7 fps.
4. Freeboard: Freeboard to be a minimum of 1 foot.
5. Maintenance Access Road: Minimum width to be 12 feet. City may require all or part of the road to be surfaced.
6. Easement/ROW Width: Minimum width to include freeboard and maintenance access road.
7. Channel Side slope: Maximum side slope for grassed channels to be 3:1.
8. Froude Number: Maximum value shall not exceed 0.8 for minor and major floods.
9. Refer to Section 6.4.1 of this CRITERIA.

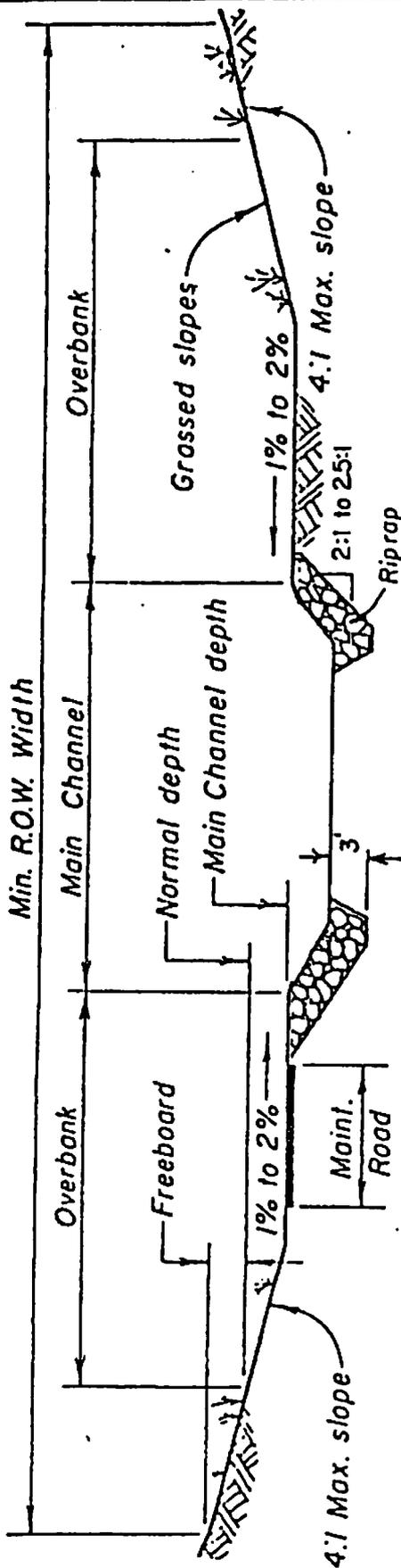
TYPE B



Notes:

1. Main Channel: Capacity to be no less than 20% of 100-year at Main Channel depth. Maximum 100-year flow velocity is 7 fps.
2. Trickle Channel: Minimum capacity to be 1% to 3% of 100-year flow but not less than 1 cfs. Channel to be constructed of concrete, grouted riprap, or other approved materials. See Figure 605 for requirements in sandy soils.
3. Normal Depth: Flow depth for 100-year flow shall not exceed 5 feet.
4. Freeboard: Freeboard to be a minimum of 1 foot.
5. Maintenance Access Road: Minimum width to be 12 feet. City may require all or part of the road to be surfaced.
6. Easement/ROW Width: Minimum width to include freeboard and maintenance access road.
7. Overbank: Flow in excess of main channel to be carried in this area. Area may be used for recreation purposes.
8. Refer to Section 6.4.2 of this CRITERIA.

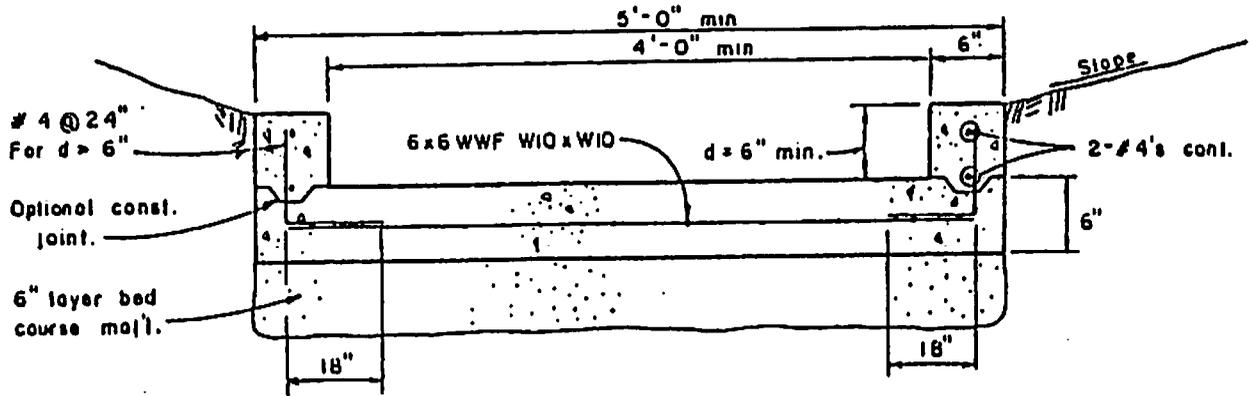
TYPE C



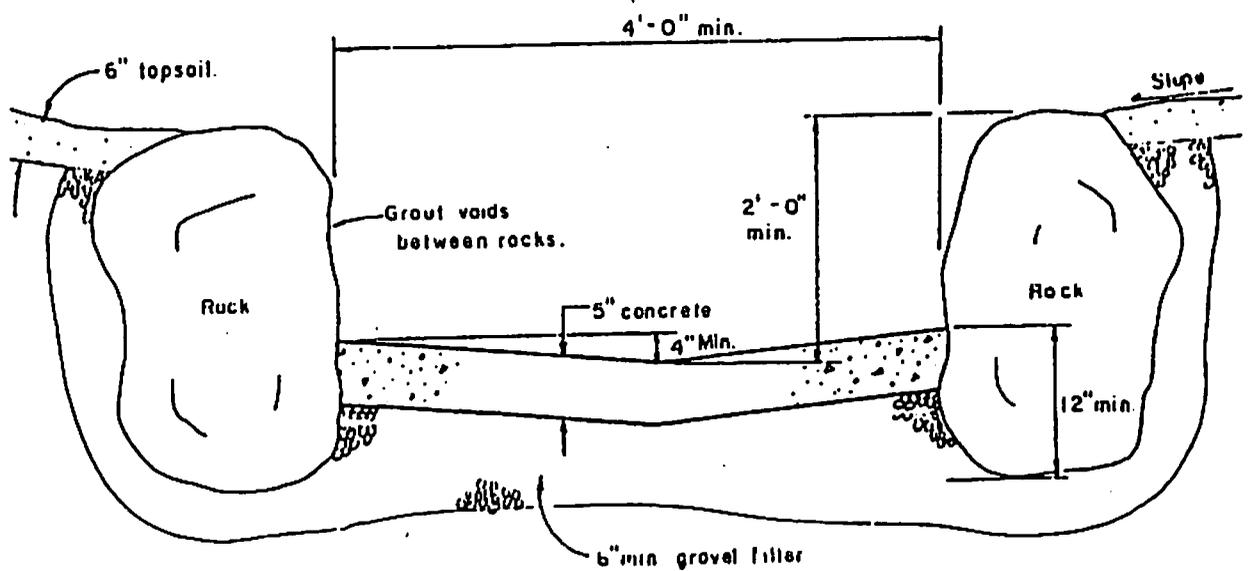
Notes:

1. This section is required for channels in sandy soils.
2. Main Channel: Capacity to be the equivalent of the 5-year. Maximum 100-year flow velocity is 5 fps. Protect slopes with riprap. Use a Mannings n-value of 0.03 for hydraulic calculations.
3. Normal Depth: Flow depth for 100-year flow shall not exceed 5 feet, not including the main channel depth.
4. Freeboard: Freeboard to be a minimum of 1 foot.
5. Maintenance Access Road: Minimum width to be 12 feet. City may require all or part of the road to be surfaced.
6. ROW Width: Minimum width to include freeboard and maintenance access road.
7. Overbank: Flow in excess of main channel to be carried in this area. Area may be used for recreation purposes.
8. Refer to Section 6.4.2 of this CRITERIA.

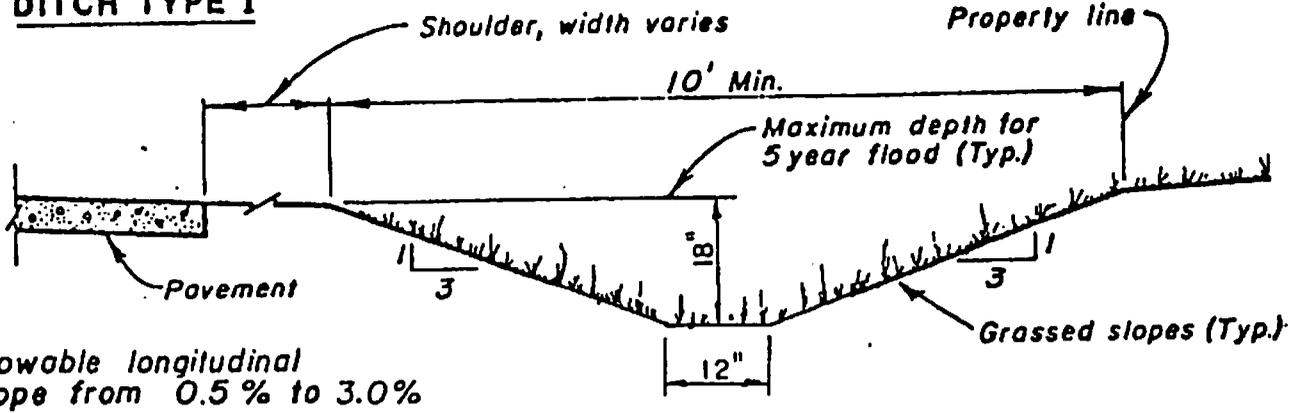
RECTANGULAR CHANNEL SECTION



COMBINATION CHANNEL SECTION

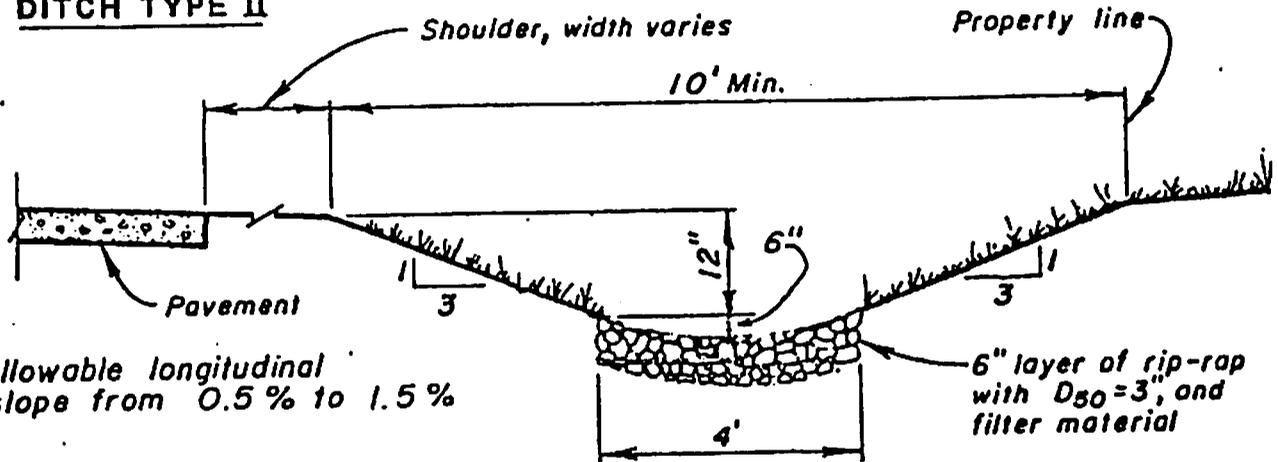


DITCH TYPE I



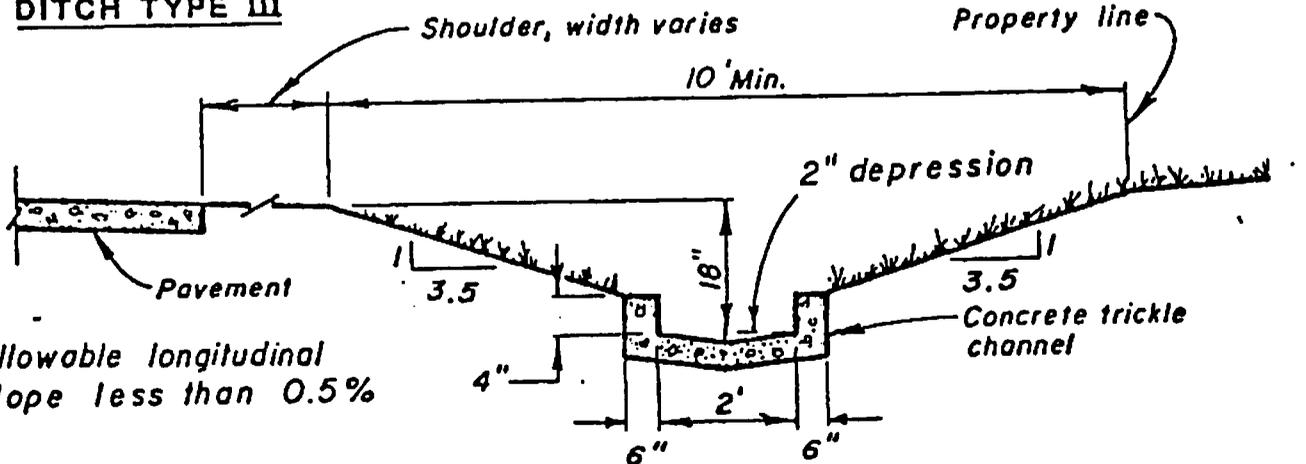
Allowable longitudinal slope from 0.5% to 3.0%

DITCH TYPE II



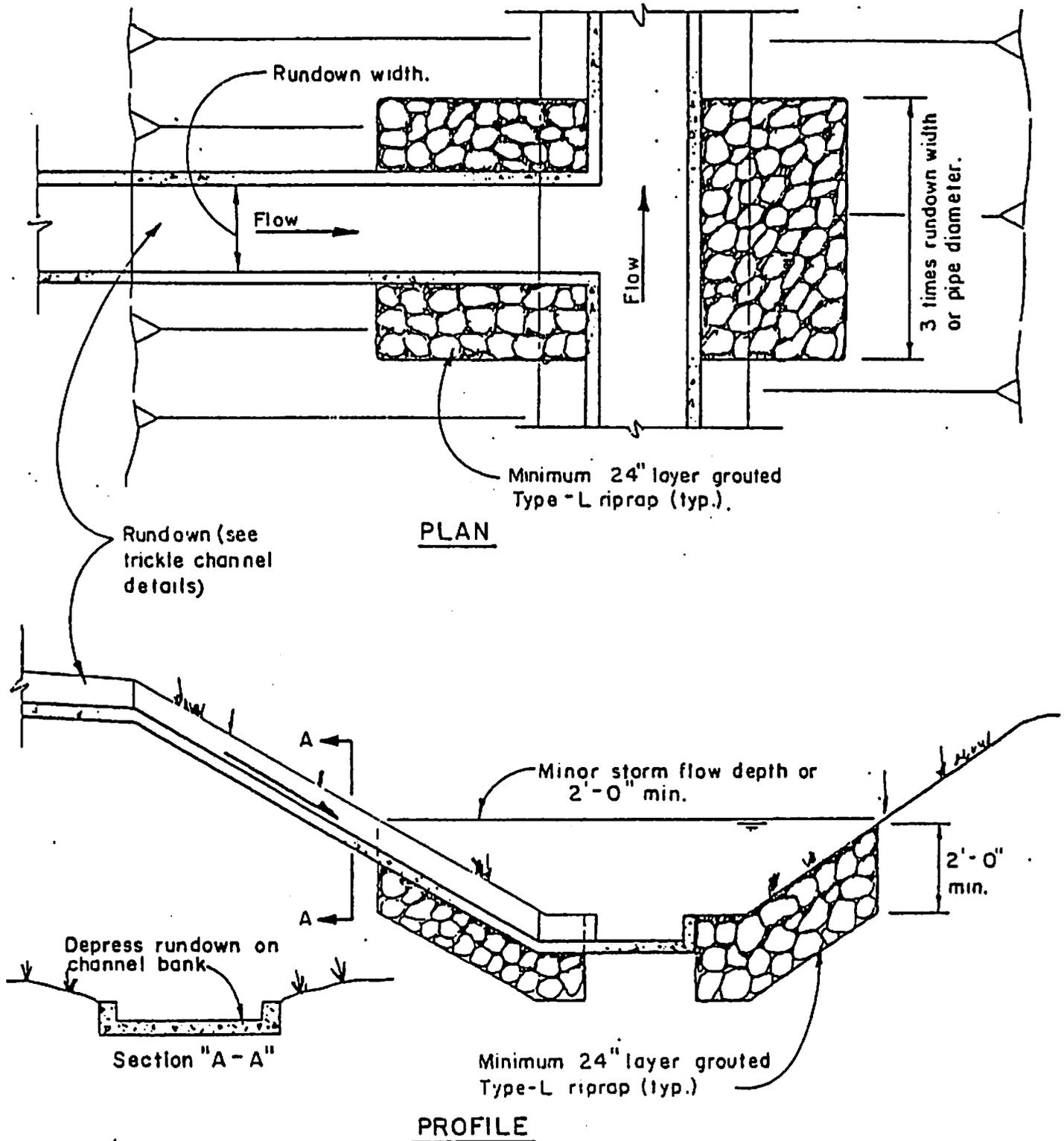
Allowable longitudinal slope from 0.5% to 1.5%

DITCH TYPE III



Allowable longitudinal slope less than 0.5%

- Notes:
1. See Table 601 for capacity of roadside ditch.
 2. For street slopes greater than maximum allowable, check drops (2' maximum height) will be required.



CHAPTER 7 STORM SEWERS

CHAPTER 7 STORM SEWERS

7.1 INTRODUCTION

Storm sewers are a part of the Minor Drainage System, and are required when the other parts of the minor system, primarily curb, gutter, and roadside ditches no longer have capacity for additional runoff.

Except as modified herein, the design of storm sewers shall be in accordance with the MANUAL Section on "Storm Sewers". Reference is made to follow specific sections in the MANUAL for clarity. The user is referred to the MANUAL and other references cited for additional discussion and basic design concepts.

A computer program for the design of a storm sewer system will be permitted provided the model is calibrated to three or more design points using the procedures presented in this CRITERIA.

7.2 CONSTRUCTION MATERIALS

Reinforced Concrete Pipe (RCP) in accordance with ASTM C-76, C-506, C-507 is acceptable for use in storm sewer construction. The minimum class of pipe shall be Class-II; however, the actual depth of cover, live load, and field conditions may require structurally stronger pipe. Corrugated Metal Pipe (CMP or steel pipe) is not permitted for storm sewer construction; corrugated aluminum pipe (CAP) and polyvinyl chloride (PVC) pipe is allowed.

7.3 HYDRAULIC DESIGN

Storm sewers shall be designed to convey the minor storm flood peaks without surcharging the sewer. To ensure that this objective is achieved, the hydraulic and energy grade line shall be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented in the following sections. The final energy grade line shall be at or below the proposed ground surface.

7.3.1 Pipe Friction Losses

The Manning's "n" values to be used in the calculation of storm sewer capacity and velocity are presented in Table 701.

7.3.2 Pipe Form Losses

Generally, between the inlet and outlet the flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of developed turbulence and can be expressed as follows:

$$H_L = K \left[\frac{V^2}{2g} \right] \quad (\text{Equation 701})$$

where

H_L = head loss (feet)

K = loss coefficient

$\frac{V^2}{2g}$ = velocity head (feet)

g = gravitational acceleration (32.2 ft/sec²)

The following is a discussion of a few of the common types of form losses encountered in sewer system design. The reader is referred to References 1 and 6 for additional discussion.

1. Expansion Losses

Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The loss of head can be expressed as:

$$H_L = K_e \left[\frac{V_1^2}{2g} \right] \left[\frac{1 - A_1}{A_2} \right]^2 \quad (\text{Equation 702})$$

in which A is the cross section area, V is the average flow velocity, and K_e is the loss coefficient. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. The value of K_e is approximately 1.0 for a sudden expansion, and about 0.2 for a well designed expansion transition. Table 702 presents the expansion loss coefficients for various flow conditions.

2. Contraction Losses

The form loss due to contraction is:

$$H_L = K_C \left[\frac{V_2^2}{2g} \right] \left[1 - \left[\frac{A_2}{A_1} \right]^2 \right]^2 \quad (\text{Equation 703})$$

where K_C is the contraction coefficient. K_C is equal to 0.5 for a sudden contraction and about 0.1 for a well designed transition. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. Table 702 presents the contraction loss coefficient for various flow conditions.

3. Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed by the relation:

$$H_L = K_b \left[\frac{V^2}{2g} \right] \quad (\text{Equation 704})$$

in which K_b is the bend coefficient. The bend coefficient has been found to be a function of, (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, (c) geometry of the cross section of flow, and (d) the Reynolds number and relative roughness. The recommended bend loss coefficients for standard bends, radius pipe, and bends through manholes are presented in Tables 703 and 704

4. Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main sewer, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances should be made for head loss due to the impact at junctions. The head loss for a straight through manhole or at an inlet entering the sewer is calculated from Equation 701. The head loss at a junction can be calculated as follows:

$$H_L = \frac{V_2^2}{2g} - K_j \left[\frac{V_1^2}{2g} \right] \quad (\text{Equation 705})$$

where V_2 is the outfall flow velocity and V_1 is the inlet velocity. The loss coefficient, K_j , for various junctions is presented in Table 705.

7.3.3 Storm Sewer Outlets

When the storm sewer system discharges into the Major Drainageway System (usually an open channel), additional losses occur at the outlet in the form of expansion losses (refer to Section 7.3.2.1). For a headwall and no wingwalls, the loss coefficient $K_e = 1.0$ (refer to Table 702), and for a flared-end section the loss coefficient is approximately 0.5 or less.

7.3.4 Partially Full Pipe Flow

When a storm sewer is not flowing full, the sewer acts like an open channel, and the hydraulic properties can be calculated using open channel techniques (refer to Chapter 6). For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Figures 701, 702, 703). The data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.

7.4 VERTICAL ALIGNMENT

The sewer grade shall be such that a minimum cover is maintained to withstand AASHTO HS-20 loading on the pipe. The minimum cover depends upon the pipe size, type and class, and soil bedding condition, but shall be not less than 1-foot at any point along the pipe.

The minimum clearance between storm sewer and water main, either above or below, shall be 12-inches. Concrete encasement of the water line will be required for clearances of 12-inches or less.

The minimum clearance between storm sewer and sanitary sewer, either above or below, shall also be 12-inches. In addition, when a sanitary sewer main lies above a storm sewer, or within 18-inches below, the sanitary sewer shall have an impervious encasement or be constructed of structural sewer pipe for a minimum of 10-feet on each side of where the storm sewer crosses.

7.5 HORIZONTAL ALIGNMENT

Storm sewer alignment between manholes shall be straight. Storm sewers may be constructed with curvilinear alignment when approved in writing by the City Engineer, by either the pulled-joint method, pipe bends, or by radius pipe in accordance with Table 701. The limitations on the radius for pulled-joint pipe is dependent on the pipe length and diameter, and amount of opening permitted in the joint. The maximum allowable joint pull shall be 3/4-inches. The minimum parameters for radius type pipe are shown in Table 701. The radius requirements for pipe bends are dependent upon the manufacturer's specifications.

7.6 PIPE SIZE

The minimum allowable pipe size for storm sewers except for detention outlets is dependent upon a practical diameter from the maintenance standpoint. The length of the sewer also affects the maintenance and, therefore, the minimum diameter. Table-701 presents the minimum pipe size for storm sewers.

7.7 MANHOLES

Manholes or maintenance access ports will be required whenever there is a change in size, direction, elevation, grade, or where there is a junction of two or more sewers. A manhole may be required at the beginning and/or at the end of the curved section of storm sewer. The maximum spacing between manholes for various pipe sizes shall be in accordance with Table-701. The required manhole size shall be as follows:

<u>SEWER DIAMETER</u>	<u>MANHOLE DIAMETER</u>
15" to 18"	4'
21" to 42"	5'
48" to 54"	6'
60" and larger	Std. Detail SD-6

Larger manhole diameters or a junction structure may be required when sewer alignments are not straight through or more than one sewer line goes through the manhole.

7.8 DESIGN EXAMPLE

The following calculation example, including the calculation Table 705, and Figures 704 and 705, were obtained from Modern Sewer Design, AISI, Wash., D.C., 1980 and edited for the calculation of manhole and junction losses in accordance with this Section.

EXAMPLE 3:

HYDRAULIC DESIGN OF STORM SEWERS

<u>Given:</u>	(a) Plan and Profile of storm sewer (Figures 704 and 705)
	(b) Station 0+00 (outfall) data as follows:
Design discharge	Q = 145 cfs [9]
Invert of pipe	= 94.50' [2]
Diameter	D = 66" RCP [3]
Starting water surface	W.S. = 100' [4]
Area of pipe	A = 23.76 sq. ft. [6]
Velocity = \underline{Q} ,	V = 6.1 f/s [8]

A

- Note:**
- (1) Number in brackets refers to the columns on Table 706.
 - (2) Sizes of the storm sewer were determined during the preliminary design phases.
- FIND:** Hydraulic Grade Line and Energy Grade Line for storm sewer.
- DISCUSSION:** The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80 percent), the following procedures can be used with minimal loss of accuracy. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.
- STEP 1:** The normal depth is greater than critical depth, $d_n > d_c$; therefore, calculations to begin at outfall, working upstream. Compute the following parameters:
- $$\zeta \text{ value [7]: } \zeta = \frac{2gn^2}{2.21} = \frac{(2)(32.2)(0.013)^2}{2.21}$$
- This equation is derived from the Manning's equation by solving for velocity and converting to velocity head.
- $$\zeta = 0.00492$$
- This value remains constant for this design since the n-value does not change.
- STEP 2:** Velocity head [10]: $H_V = V^2/2g = (6.1)^2/(2)(32.2)$
 $H_V = 0.58$
- STEP 3:** Energy Grade Point, E.G. [11]:
 E.G. = W.S. + $H_V = 100 + 0.58$
 E.G. = 100.58
 For the initial calculation, the Energy Grade Line is computed as described above. For subsequent calculations, the equation is reversed, and the water surface is calculated as follows (see Step 12):
 W.S. = E.G. - H_V
- This equation is used since the losses computed in Step 8 are energy losses which are added to the downstream energy grade elevation as the new starting point from which the velocity head is subtracted as shown above.

STEP 4: $S_f = C H_V/R^{4/3} = (0.00492)(0.58)/(1.375)^{4/3}$

NOTE: R = the hydraulic radius of the pipe.

$S_f = 0.0019$

STEP 5: Avg. S_f [13]:

Average skin friction: This is the average value between S_f of the station being calculated and the previous station. For the first station, Avg. $S_f = S_f$. Beginning with Column 13, the entries are placed in the next row since they represent the calculated losses between two stations.

STEP 6: Enter sewer length, L, in column 14

STEP 7: Friction loss H_f [15]:

$H_f = (\text{Avg. } S_f)(L)$

$H_f = (0.0019)(110)$

$H_f = 0.21$

STEP 8: Calculate the form losses for bends, junctions, manholes, and transition losses (expansion or contraction) using equations 701, 702, 703, 704, and 705. The calculation of these losses is presented below for the various sewer segments, since all the losses do not occur for one sewer segment.

(a) station 1 + 10 to 1 + 52.4 (bend).

$H_b = K_b H_V$, where the degree of bend is 60°

$K_b = 0.20$ (Table 703, Case I)

$H_b = (0.20)(0.58) = 0.12$, enter in column 16

(b) station 2 + 48 to 2 + 55.5 (transition: expansion).

$H_L = K_e (H_V - 1) (1 - A_1/A_2)^2$

$K_e = 1.06$ (Table 702) for $D_2/D_1 = 1.5$, and $= 45^\circ$

$H_L = (1.06)1.29 [1 - (15.9/23.76)]^2 = 0.15$, enter in column 19

(c) station 3 + 55.5 (manhole, straight through).

$H_m = K_m H_V$

$K_m = 0.05$ (Table 705, Case I)

- (d) $H_m = (0.05)(1.29) = 0.06$, enter in column 18
 station 4 + 55.5 to 4 + 65.5 (junction).
 $H_j = H_{v-2} - K_j (H_{v-1})$
 $K_j = 0.62$ (Table 705, Case III), $\theta = 30^\circ$
 $H_j = 1.29 - (0.62)(0.99) = 0.68$, enter in column 17
- (e) station 5 + 65.5 to 5 + 75.5 (junction) - since there are two laterals, the loss is estimated as twice the loss for one lateral.
 $K_j = 0.33$ (Table 705, Case III) $\theta = 70^\circ$
 $H_j = 0.99 - (0.33)(0.64) = 0.78$ for one lateral
- STEP 9: Sum all the form losses from columns 15 through 19 and enter in column 20. For the reach between Station 00+0 to 1+10, the total loss is 0.21.
- STEP 10: Add the total loss in column 20 to the energy grade at the downstream end (Sta. 0+0) to compute the energy grade at the upstream end (Sta. 1+10) for this example).
 $E.G. (U/S) = E.G. (D/S) + TOTAL LOSS$
 $= 100.58 + 0.21$
 $= 100.79$ (Column 11)
- STEP 11: Enter the new invert [2], pipe diameter D[3], pipe shape [5], pipe area A, [6], the compute constant C from Step 1 in column [7], the computed velocity V in column [8], the new Q [9], and the computed velocity head H_v [10].
- STEP 12: Compute the new water surface, W.S., for the upstream station (1+10 for this example).
 $W.S. = E.G. - H_v$
 $= 100.79 - 0.58$
 $= 100.21$ (column 4)
- STEP 13: Repeat Steps 1 through 12 until the design is complete. The hydraulic grade line and the energy grade line are plotted on the profile (Figure 705).

DISCUSSION OF RESULTS:

The HGL is at the crown of the pipe from Station 0+00 to 2+48. Upstream of the transition (Station 2+55.5) the 54" RCP has a greater capacity (approximately 175 cfs) at the slope than the design flow (145 cfs). The pipe is therefore not flowing

full but is substantially full (i.e., $145/175 = 0.84$ greater than 0.80). The computed HGL is below the crown of the pipe. However, at the outlet, the actual HGL is higher, since the outlet of the 54" RCP is submerged by the headwater for the 66" RCP. To compute the actual profile, a backwater calculation would be required; however this accuracy is not necessary for storm sewer design in most cases.

At the junction (Station 4+55.5), the HGL is above the top of the pipe due to the losses in the junction. In this case, however, the full flow capacity (100 cfs) is the same as the design capacity, and the HGL remains above and parallel to the top of the pipe. A similar situation occurs at the junction at Station 5+65.5.

If the pipe entering a manhole or junction is at an elevation significantly above the manhole invert, a discontinuity in the EGL may occur. If the EGL of the incoming pipe for the design flow condition is higher than the EGL in the manhole, then a discontinuity exists, and the higher EGL is used for the incoming pipe.

7.9 CHECKLIST FOR STORM SEWER DESIGN

To aid the designer and reviewer, the following checklist has been prepared:

1. Calculate energy grade line (EGL) and hydraulic grade line (HGL) for all sewers and show on the construction drawings or on a separate copy of the plans submitted with the construction drawings.
2. Account for all losses in the EGL calculation including outlet, form, bend, manhole, and junction losses.
3. Provide adequate protection at the outlet of all sewers into open channels.
4. Check for minimum pipe cover; strength of pipe with overburden, surcharge, and dynamic loads; and clearance from utilities.

Table 701

<u>Vertical Dimension of Pipe (Inches)</u>	<u>Maximum Allowable Distance Between Manholes and/or Cleanouts</u>	
15 to 36	400 feet	
42 and Larger	500 feet	

<u>Diameter of Pipe</u>	<u>Minimum Radius for Radius Pipe</u>	
48" to 54"	<u>Radius of Curvature</u> 28.50 ft.	
57" to 72"	32.00 ft.	
78" to 108"	38.00 ft.	

Short radius bends shall not be used on sewers 42 inches or less in diameter.

<u>Type</u>	<u>Minimum Pipe Diameter</u>	
	<u>Minimum Equivalent Pipe Diameter</u>	<u>Minimum Cross-sectional Area</u>
Main Trunk	18 in.	1.77 sq. ft.
*Lateral from inlet	15 in.	1.23 sq. ft.

*Minimum size of lateral shall also be based upon a water surface inside the inlet with a minimum distance of 1 foot below the grate or throat.

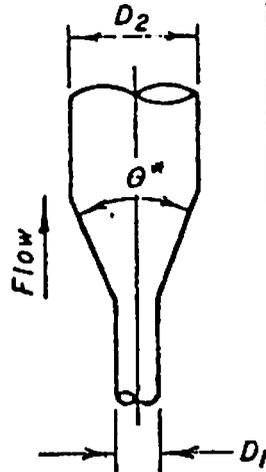
<u>Sewer Type</u>	<u>Manning's N-Value</u>	
	<u>Capacity Calculation</u>	<u>Velocity Calculation</u>
Concrete (newer pipe)	.013	.011
Concrete (older pipe)	.015	.012
Concrete (preliminary sizing)	.015	.012
Plastic	.011	.009

STORM SEWER ALIGNMENT AND SIZE CRITERIA

EXPANSION / CONTRACTION

(a) Expansion (K_e)

θ^*	$\frac{D_2}{D_1} = 3$	$\frac{D_2}{D_1} = 1.5$
	D_1	D_1
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00



* The angle θ is the angle in degrees between the sides of the tapering section.

(b) Pipe Entrance from Reservoir

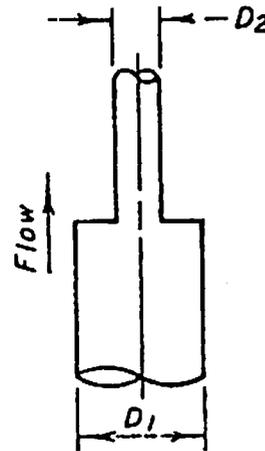
Bell-mouth $H_L = 0.04 \frac{V^2}{2g}$

Square-edge $H_L = 0.5 \frac{V^2}{2g}$

Groove end U/S
For Concrete
Pipe $H_L = 0.2 \frac{V^2}{2g}$

(c) Contractions (K_c)

$\frac{D_2}{D_1}$	K_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0



BENDS

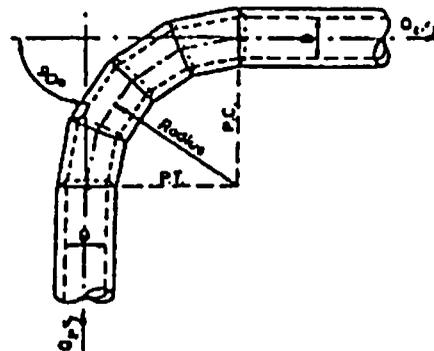
$$H_L = K_j (V^2 / 2g)$$

CASE I
CONDUIT ON 90° CURVES*

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

<u>RADIUS</u>	<u>K_b</u>
1 X D	0.50
(2 to 8) X D	0.25
(8 to 20) X D	0.04
>20 X D	0

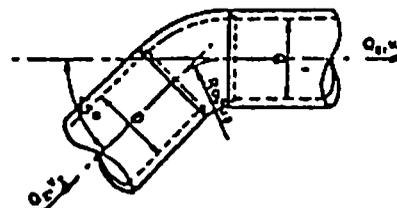
* When curves other than 90° are used, apply the following factors to 90° curves
 60° curve 85%
 45° curve 70%
 22-1/2° curve 40%



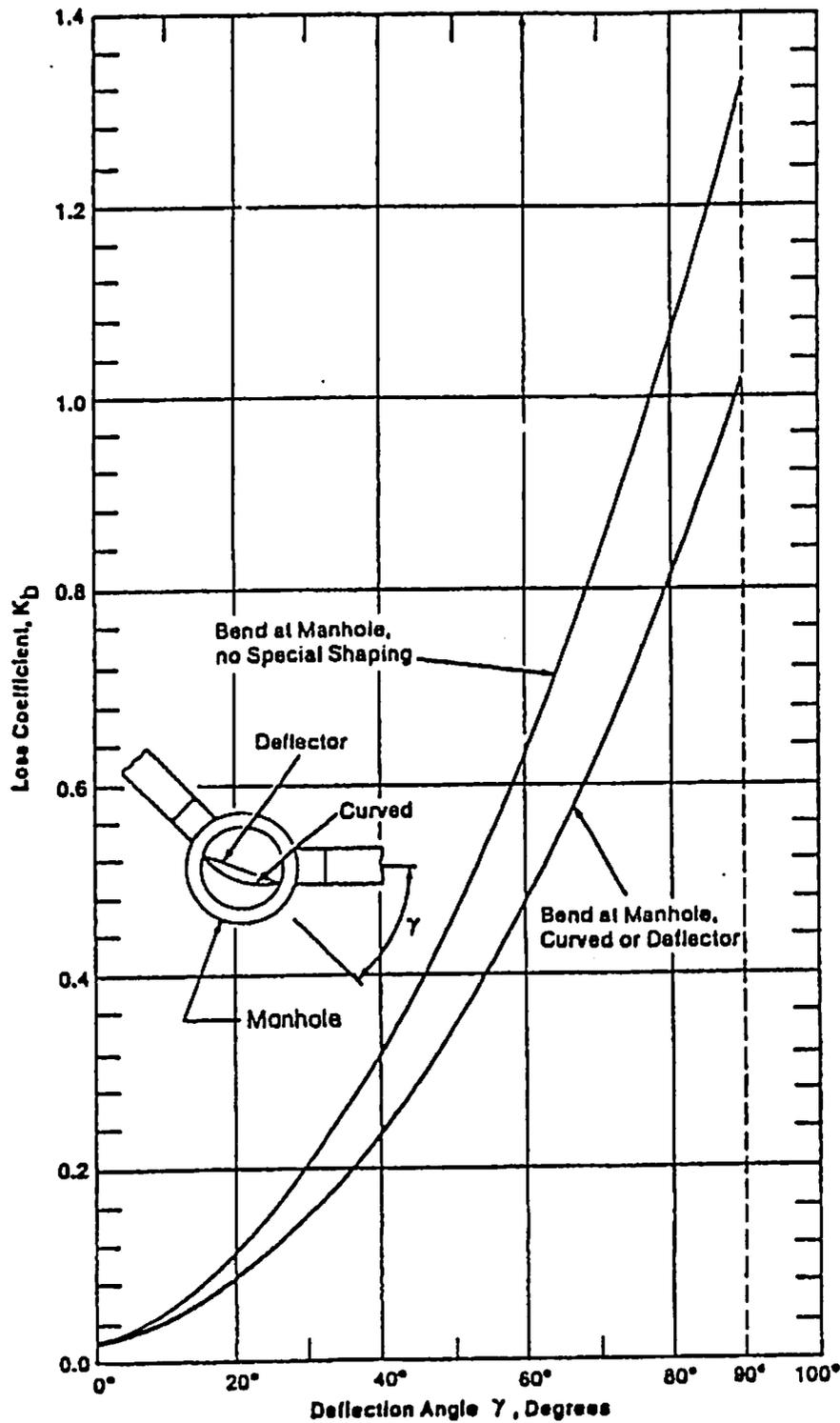
CASE II
BENDS WHERE RADIUS IS
EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at beginning of bend

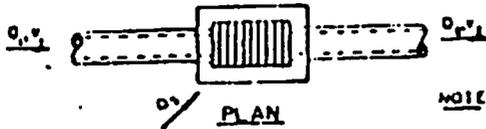
<u>θ° BEND</u>	<u>K_b</u>
90	0.50
60	0.43
45	0.35
22-1/2	0.20



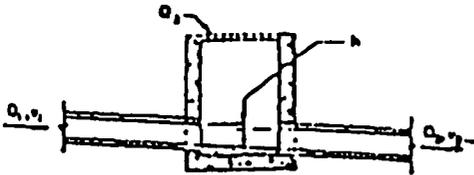
BENDS AT MANHOLES



NOTE: Head loss applied at outlet of manhole.



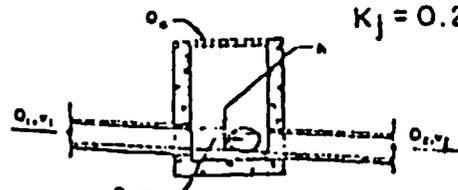
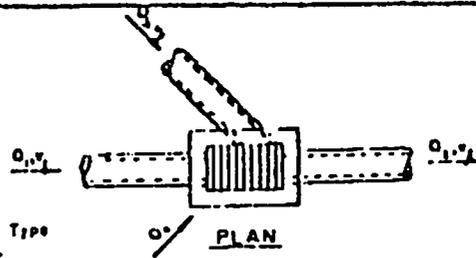
NOTE: For Any Type of Inlet.



SECTION

USE EQUATION 701
 $K_j = 0.05$

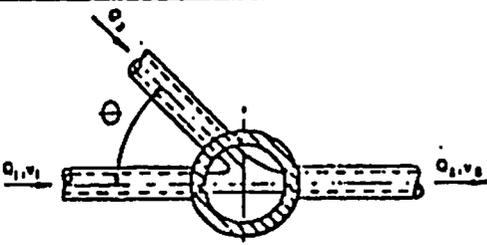
CASE I
INLET OR STRAIGHT THROUGH
MANHOLE ON MAIN LINE



SECTION

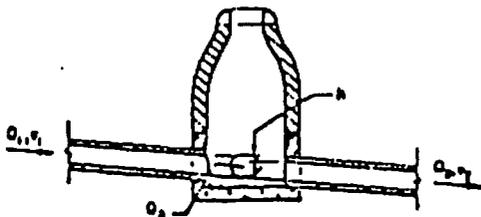
USE EQUATION 705
 $K_j = 0.25$

CASE II
INLET ON MAIN LINE
WITH BRANCH LATERAL



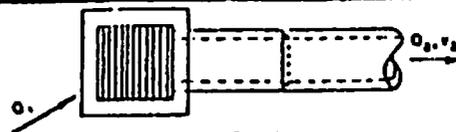
PLAN

USE EQUATION 705



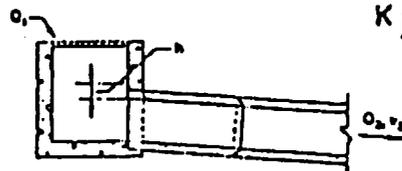
SECTION
CASE III

MANHOLE ON MAIN LINE
WITH 90° BRANCH LATERAL



PLAN

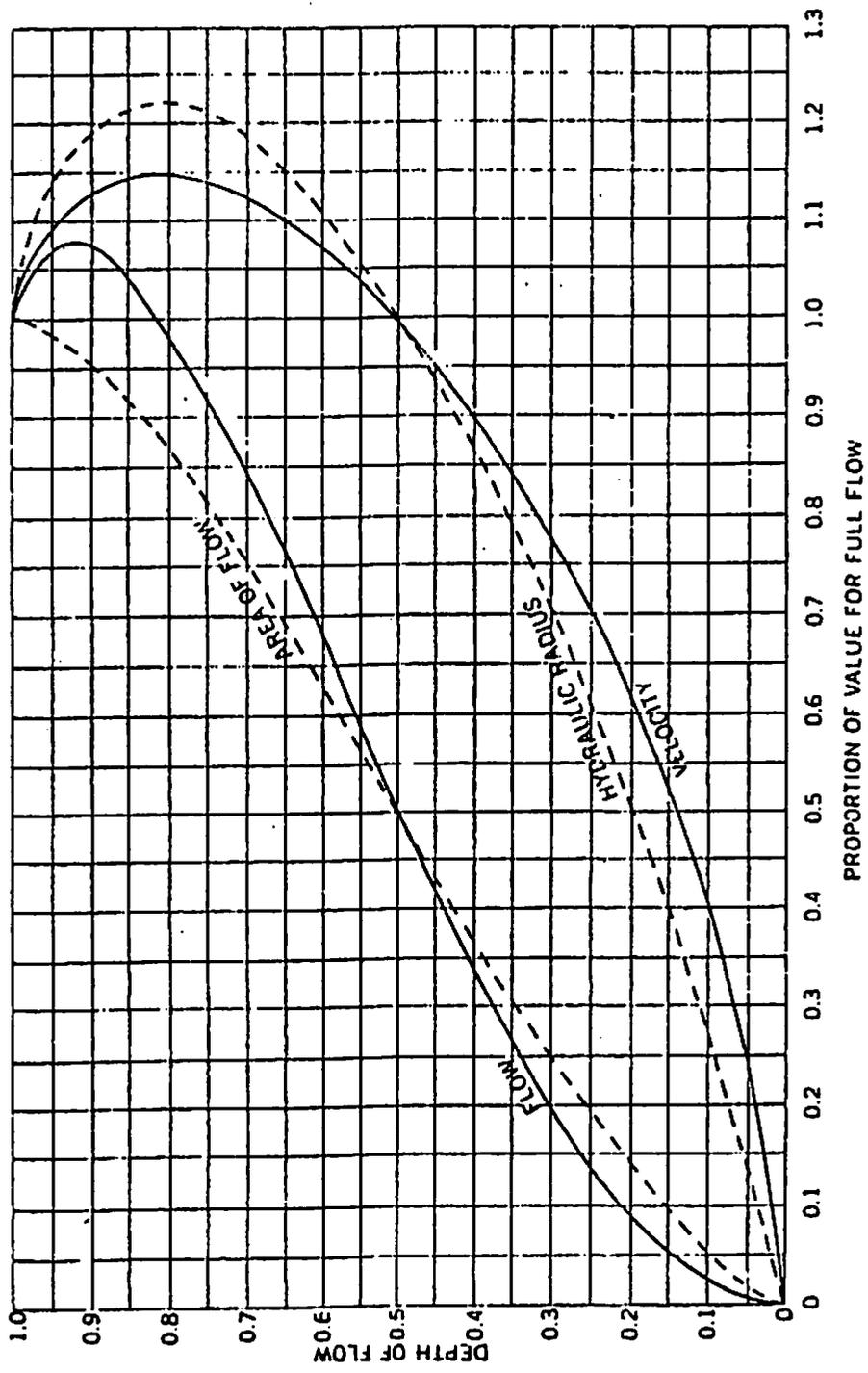
USE EQUATION 701
 $K_j = 1.25$

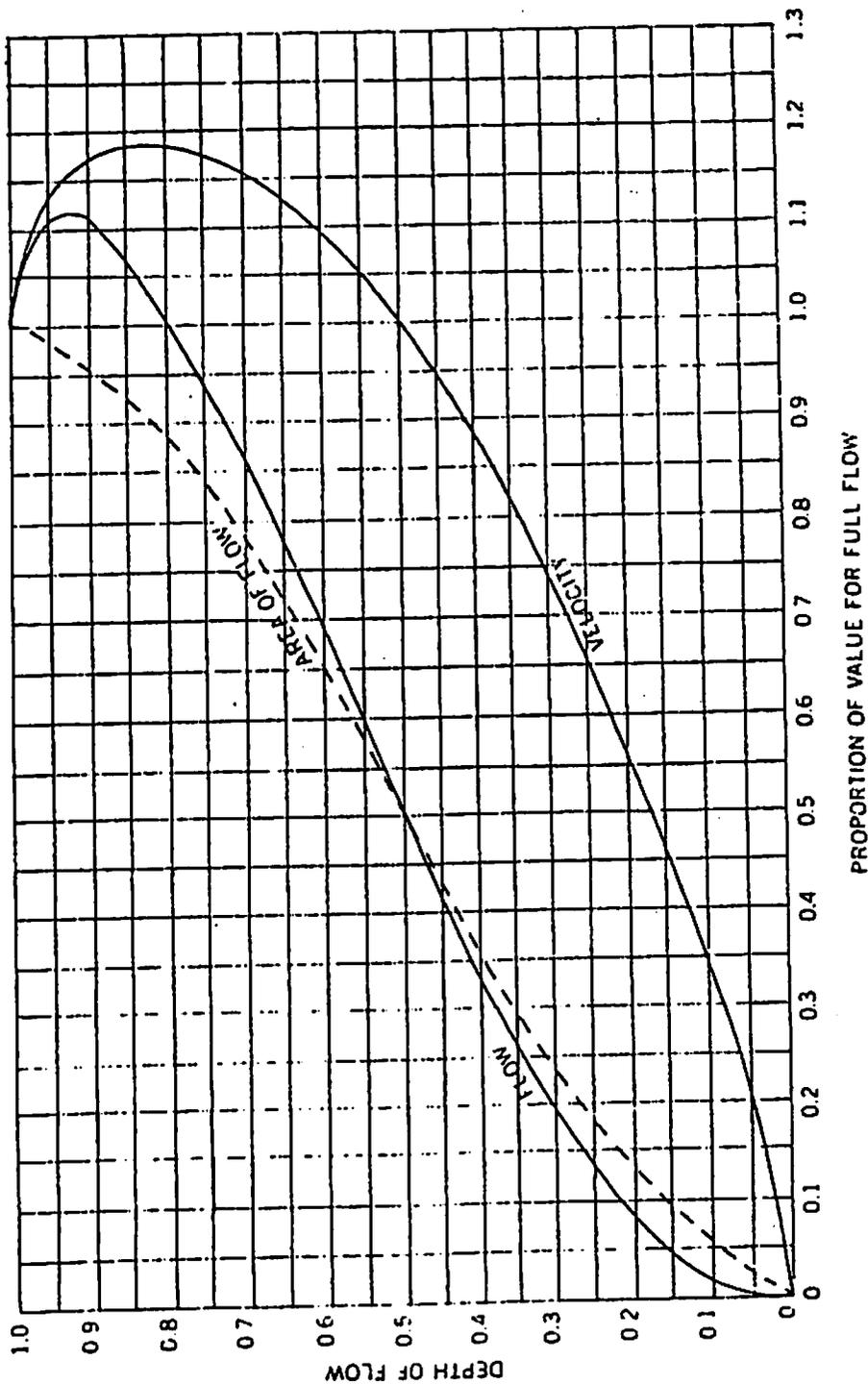


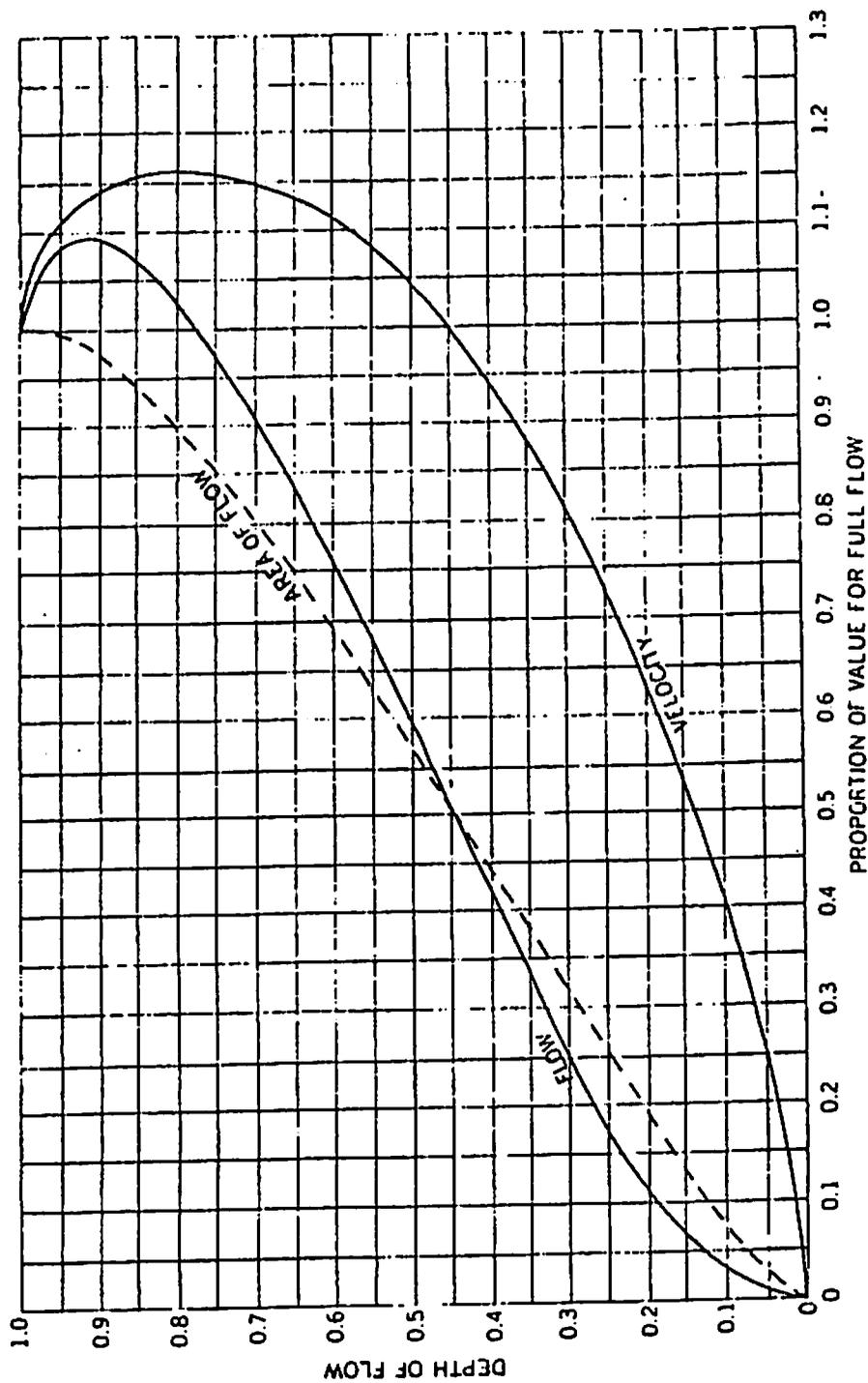
SECTION

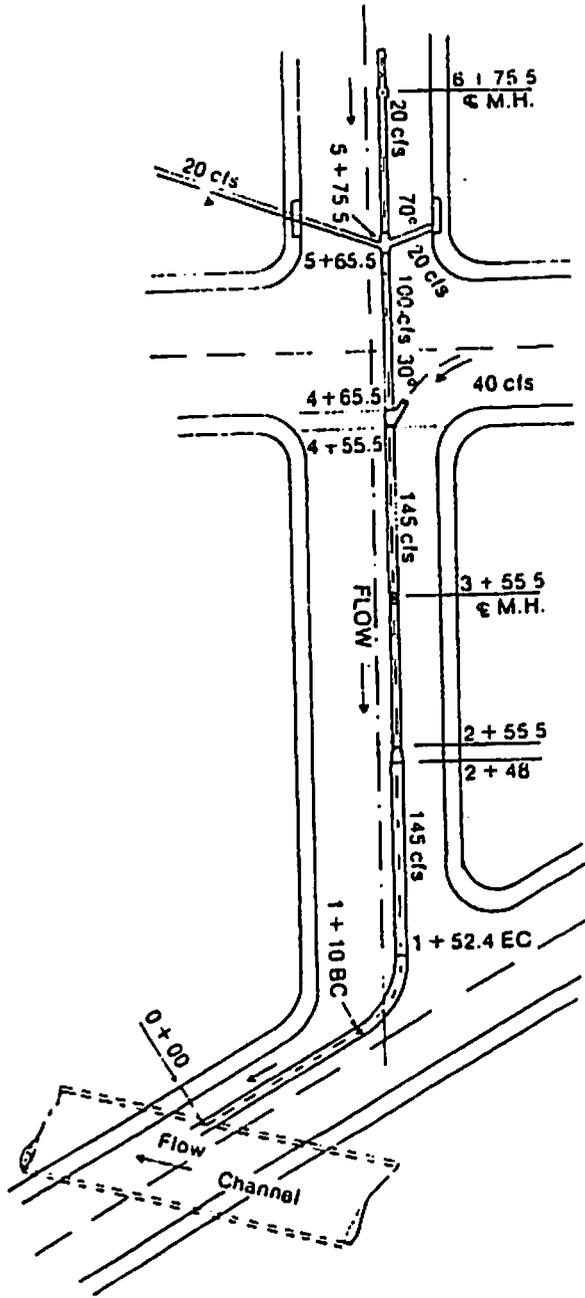
CASE IV
INLET OR MANHOLE AT
BEGINNING OF LINE

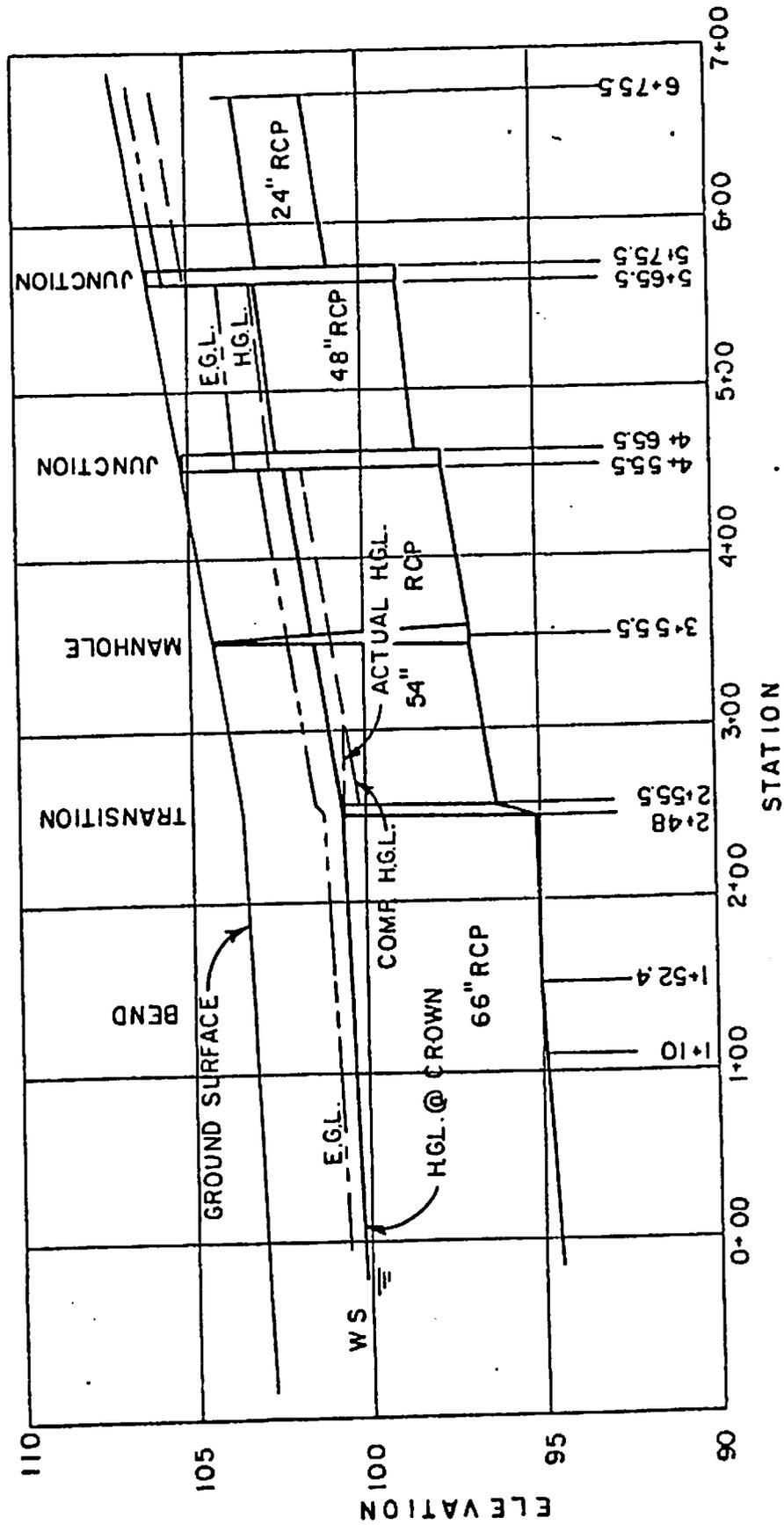
<u>CASE III</u>	
90°	K_j
22-1/2	0.75
45	0.50
60	0.35
90	0.25
No Lateral See Case I	











CHAPTER 8 STORM SEWER INLETS

CHAPTER 8 STORM SEWER INLETS

8.1 INTRODUCTION

There are three types of inlets: curb opening, grated, and combination inlets. Inlets are further classified as being on a "continuous grade" or in a sump. The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet and, therefore, ponding does not occur at the inlet. The sump condition exists whenever water is restricted or ponds because the inlet is located at a low point. A sump condition can occur at a change in grade of the street from negative to positive, or at an intersection due to the crown slope of a cross street.

Presented in this Chapter is the criteria and methodology for design and evaluation of storm sewer inlets in the City. Except as modified herein, all storm sewer inlet criteria shall be in accordance with the MANUAL.

8.2 STANDARD INLETS

The standard inlets permitted for use in the City are:

INLET TYPE	STANDARD DETAIL	PERMITTED USE
Curb Opening Inlet - Type R	SD-1	All street types
Grated Inlet - Type C	SD-2	All streets with a roadside or median ditch
Grated Inlet - Type 13	SD-3	Alleys or drives with a valley gutter (Private areas only)
Combination Inlet Type - 13	SD-4	All street types with vertical curb

8.3 INLET HYDRAULICS

The procedures and basic data used to define the capacities of the standard inlets under various flow conditions were obtained from the MANUAL, Volume 1, Section on "Storm Inlets", and from Reference-11 for curb opening inlets. The procedure consists of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. To account for effects which decrease the capacity of the various types of inlets, such as debris plugging, pavement overlaying, and variations in design assumptions, the theoretical capacity calculated for the inlets is reduced to the allowed capacity by the factors presented below for the standard inlets.

ALLOWABLE INLET CAPACITY

Percentage of Theoretical

Condition	Inlet Type	Capacity Allowed
Sump or Continuous Grade	CDOH Type R (SD-1)	
	5' length	88
	10' length	92
Sump or Continuous Grade	15' length	95
	Grated Type 13 (SD-3)	50
	Continuous Grade	66
Sump	Combination Type 13 (SD-4)	
	Grated Type C (SD-2)	50
Sump	Combination Type 13 (SD-4)	65

Allowable inlet capacities for the standard inlets have been developed and are resented in Figures 801 and 802 for "continuous grade" and Figure 803 for sump conditions. These figures include the reduction factors in the above table. The allowable inlet capacity is compatible with the allowable street capacity (refer to Chapter 9). The values shown were calculated on the basis of the maximum flow allowed in the street gutter (or roadside ditch for Type C). For the gutter flow amounts less than the maximum, the allowable inlet capacity must be proportionately reduced.

8.3.1 Continuous Grade Condition

For the "continuous grade" condition, the capacity of the inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

The use of Figures 802 and 804 is illustrated by the following example:

<u>Example 4:</u>	<u>Design of Type R Curb Opening Inlets</u>
<u>Given:</u>	Street Type = Major Arterial - Type C, S = 1.0 percent
	Maximum flow depth = 0.5 feet (refer to Chapter 9)
	Maximum allowable gutter capacity = 10.8 cfs (refer to Chapter 9)
	Starting gutter flow (Q_L) = 8.0 cfs
<u>Find:</u>	Interception and carryover amounts for the inlets and flow conditions illustrated on Figure 804.
<u>Solution:</u>	From Figure 804, we can see that inlets 1 and 2 are in a continuous grade condition and inlet 3 is in a sump condition. The first step is to calculate the interception ratio R, for the continuous grade inlets. This ratio is then

applied to the actual gutter flow (local runoff plus carryover flow) to determine amount intercepted by the inlet and the carryover flow. The final step is to calculate the size of the inlet required for the sump condition, which is discussed in Example #5 in the following section.

Step 1: From Figure 802 for an allowable depth of 0.50 feet, read the value 8.6 cfs. Note that even though the gutter flow is less than maximum allowable, the maximum depth is used for Figure 802. The effect of the lower depth on the inlet capacity will be accounted for in the following steps.

Step 2: Compute the interception ratio R

$$R = \frac{\text{Allowable Inlet Capacity}}{\text{Allowable Street Capacity}} = \frac{8.6}{10.8}$$

$$R = 0.80$$

Step 3: Compute the interception amount Q_I

$$Q_I = R \times Q_{\text{Street}}$$

$$= 0.80 \times 8.0$$

$$Q_I = 6.4 \text{ cfs amount intercepted by inlet}$$

Step 4: Compute the carryover amount Q_{CO}

$$Q_{CO} = Q_{\text{street}} - Q_I$$

$$= 8.0 - 6.4$$

$$Q_{CO} = 1.6 \text{ cfs}$$

Step 5: Compute the total flow at the next inlet, which is the sum of the carryover (Q_{CO}) from inlet #1 plus the local runoff to inlet #2.

$$Q_T (\text{inlet \#2}) = Q_{CO} (\text{inlet \#1}) + Q_L (\text{inlet \#2})$$

$$= 1.6 \text{ cfs} + 4 \text{ cfs}$$

$$Q_T (\text{inlet \#2}) = 5.6 \text{ cfs}$$

Step 6: Compute the interception ratio, intercepted amount, and carryover flow for inlet #2 using the procedure described in steps 1 through 4.

$$\text{Allowable inlet capacity} = 7.2 \text{ cfs} \quad (\text{Figure 802})$$

$$R = (7.2 \text{ cfs}) / (10.8 \text{ cfs}) = 0.67$$

$$Q_I (\text{inlet \#2}) = (0.67) (5.6 \text{ cfs}) = 3.7 \text{ cfs}$$

$$Q_{CO} (\text{inlet \#2}) = 5.6 \text{ cfs} - 3.7 \text{ cfs} = 1.9 \text{ cfs}$$

Step 7: Compute the total flow at inlet #3 using the procedure described in Step 5.

$$Q_T (\text{inlet \#3}) = 8 \text{ cfs} + 1.9 \text{ cfs} = 9.9 \text{ cfs}$$

Step 8: Size the inlet in the sump condition using the procedures described in Example #5, Section 8.3.2, and Figure 804. For this example, with an allowable maximum depth of flow of 0.5 cfs, a 10-foot type R inlet will intercept more than the total gutter flow and is therefore acceptable.

8.3.2 Sump Condition

The capacity of the inlet in a sump condition is dependent on the depth of flow above the inlet. Typically the problem consists of estimating the number of inlets or depth of flow required to intercept a given flow amount. The use of Figure 804 is illustrated by the following example:

Example 5: Allowable Capacity for Combination Type 13 Inlet in a Sump

Given: Flow in gutter = 9.9 cfs (From Example #4)
 Maximum allowable street depth = 0.50 feet
 Type 13 Combination double inlet

Find: Depth of flow

Solution:

Step 1: From Figure 803, read the "Depth of Flow" for a double Type 13 Combination inlet as $D = 0.33'$ at the gutter flow of 9.9 cfs.

Step 2: Compare computed to allowable depth. Since the computed depth is less than the allowable depth, the inlet is acceptable, otherwise the amount of inlets or the type of inlet would be changed and the procedure repeated.

8.4 INLET SPACING

The optimum spacing of storm inlets is dependent upon several factors including traffic requirements, contributing land use, street slope, and distance to the nearest outfall system. The suggested sizing and spacing of the inlets is based upon the interception rate of 70% to 80%. This spacing has been found to be more efficient than a spacing using 100% interception rate. Using the suggested spacing only, the most downstream inlet in a development would be designed to intercept 100% of

the flow. Also, considerable improvements in the over-all inlet system efficiency can be achieved if the inlets are located in the sumps created by street intersections. The following example illustrates how inlet sizing and interception capacity may be analyzed:

Example 6: Inlet Spacing

Given: Maximum allowable street flow depth = 0.48 ft.
 Street slope = 1.0 percent
 Maximum allowable gutter flow = 9.4 cfs
 Gutter flow = 9.4 cfs

Find: Size and type of inlet for 75 percent interception

Solution:

Step 1: Compute desired capacity

$$Q = (0.75) (9.4 \text{ cfs}) = 7.1 \text{ cfs}$$

Step 2: Read the allowable inlet capacities from Figures 801 and 802 for various inlets. The following values were obtained:

<u>INLET TYPE</u>	<u>CAPACITY</u>	<u>% INTERCEPTION</u>
Triple Type 13 Comb	4.7 cfs	50
Double Type R	6.5 cfs	69
Triple Type R	7.7 cfs	82

Therefore, a curb opening inlet Type R, L = 15 feet is required and will intercept 7.7 cfs. The remaining 1.7 cfs will continue downstream and contribute to the next inlet. Spacing between such inlets will depend on the local runoff, and the amount of flow bypassed at the upstream inlet.

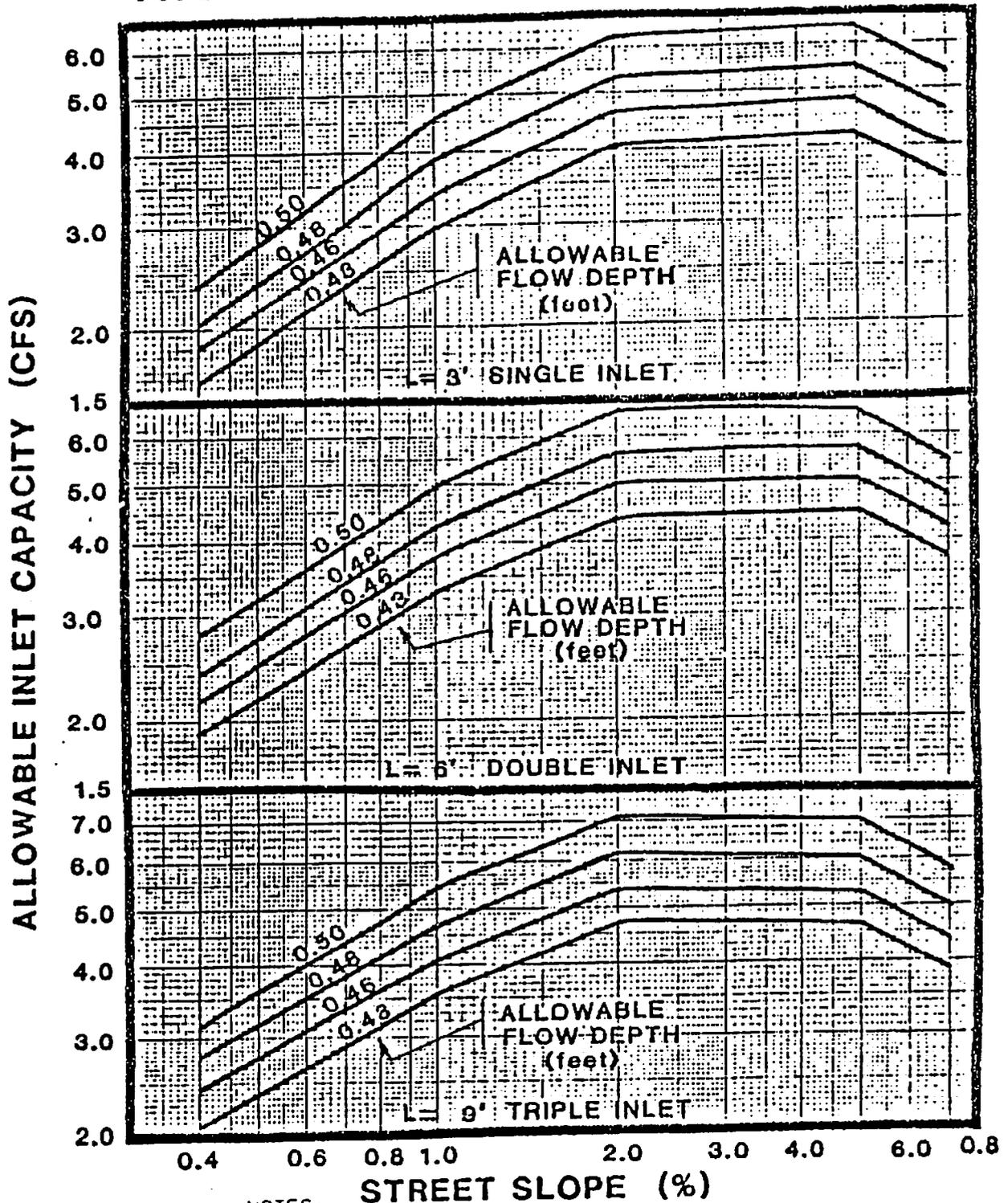
A comparison of the inlet capacity with the allowable street capacity (refer to Chapter 9) will show that the percent of street flow interception by the inlets varies from less than 50 percent to as much as 95 percent of the allowable street capacity. Therefore, the optimum inlet spacing cannot be achieved in all instances, and the spacing requirements should be analyzed by the design engineer.

8.5 CHECKLIST FOR INLET CAPACITY

To aid the designer and reviewer, the following checklist has been prepared:

1. Check the inlet capacity to determine the carryover flow, and account for this flow plus the local runoff in the sizing of the next downstream inlet.
2. Place inlets at the flattest grade or in sump conditions where possible to increase capacity.
3. Space inlets based upon the interception rate of 70 to 80% of the gutter flow to optimize inlet capacity.

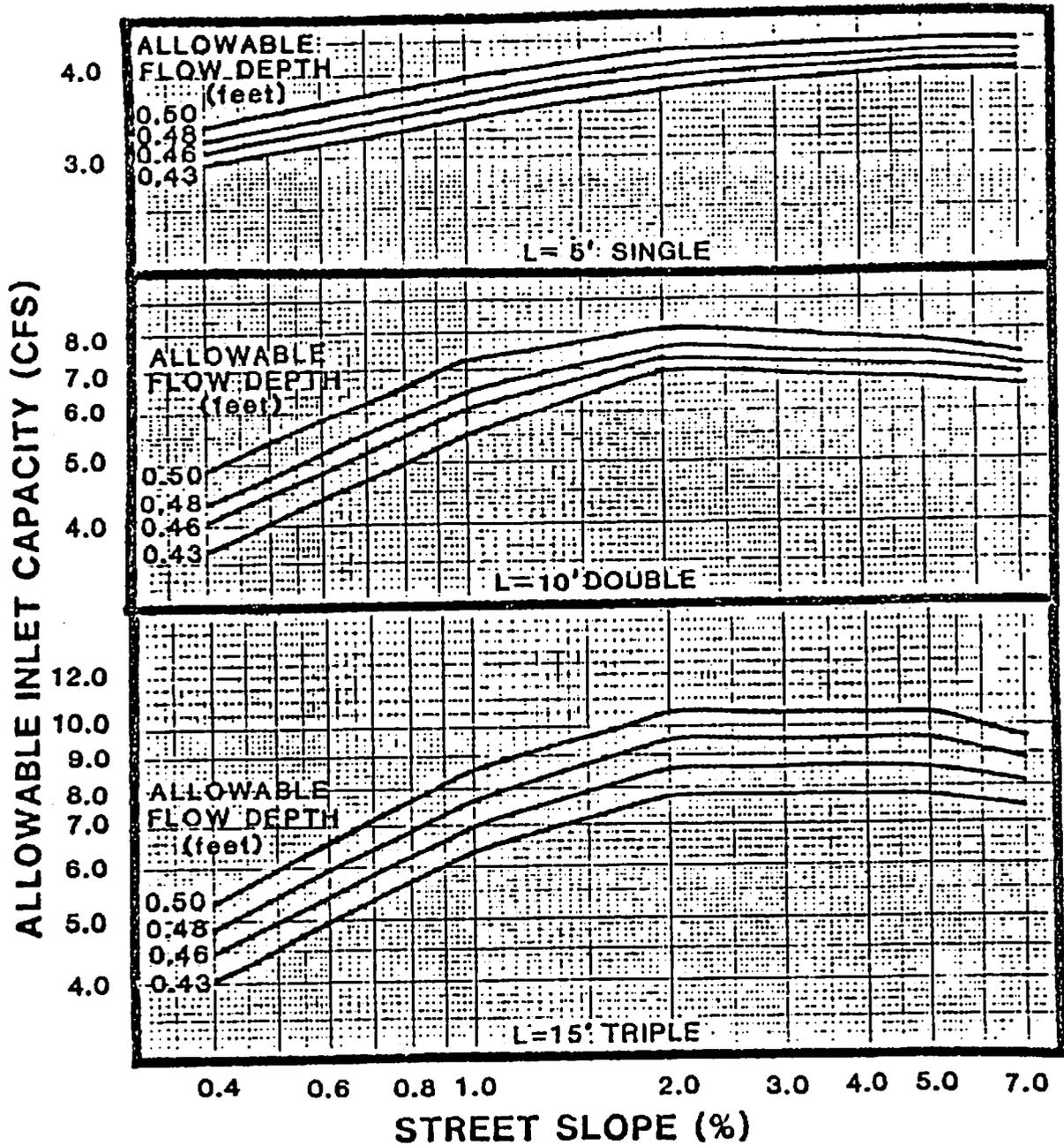
TYPE 13 COMBINATION ON A CONTINUOUS GRADE



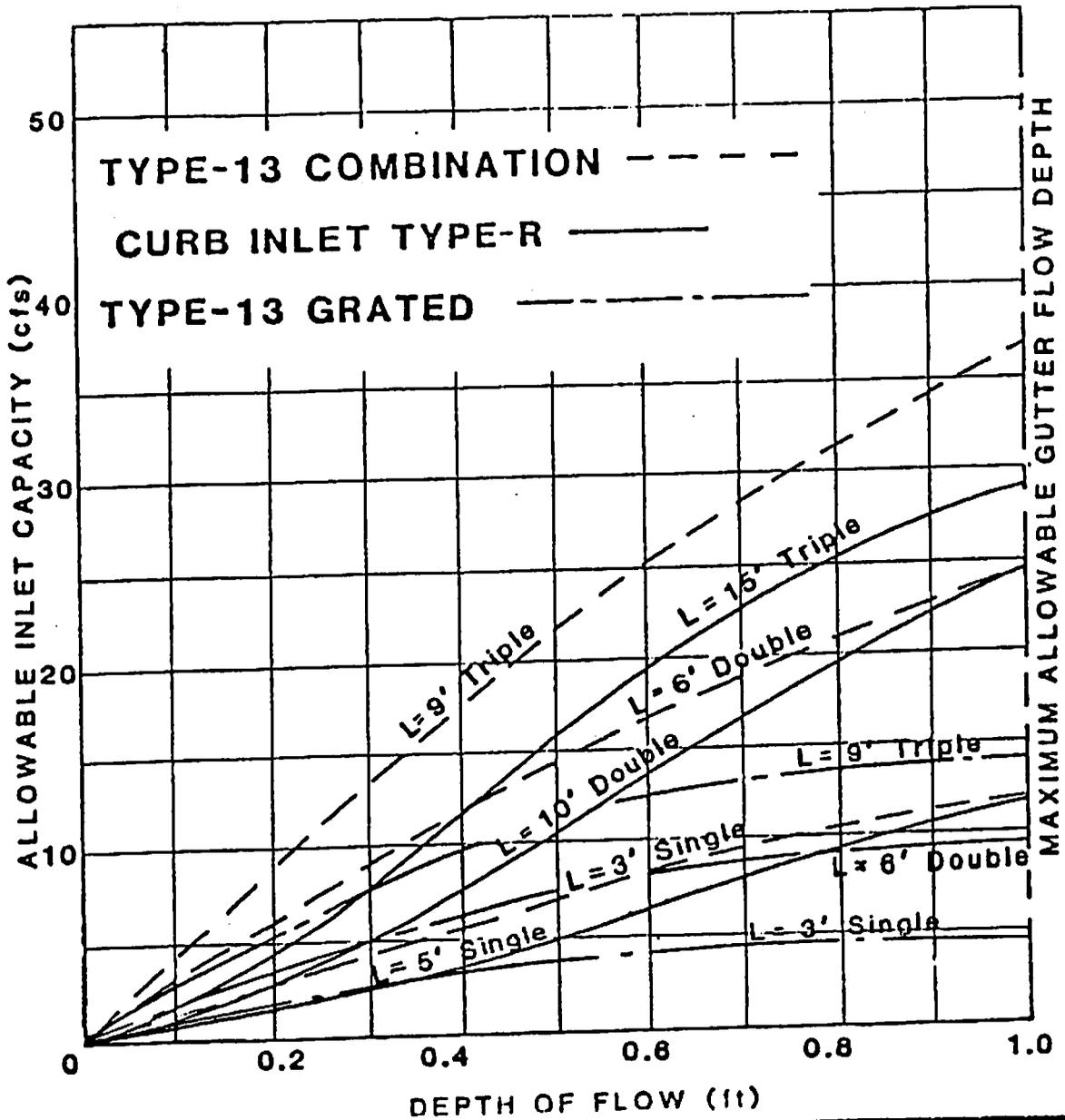
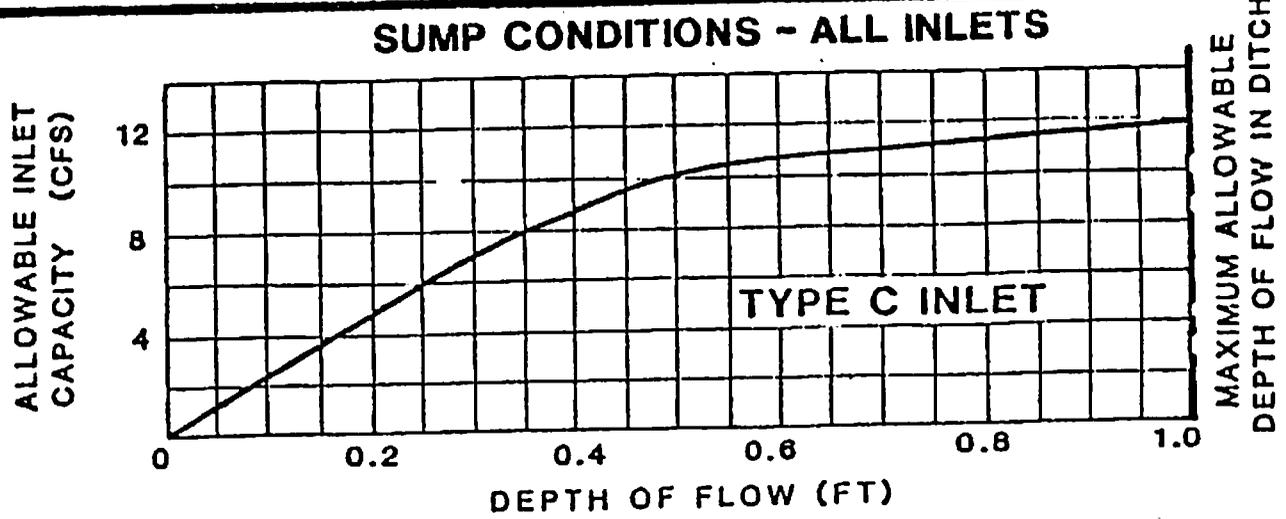
NOTES:

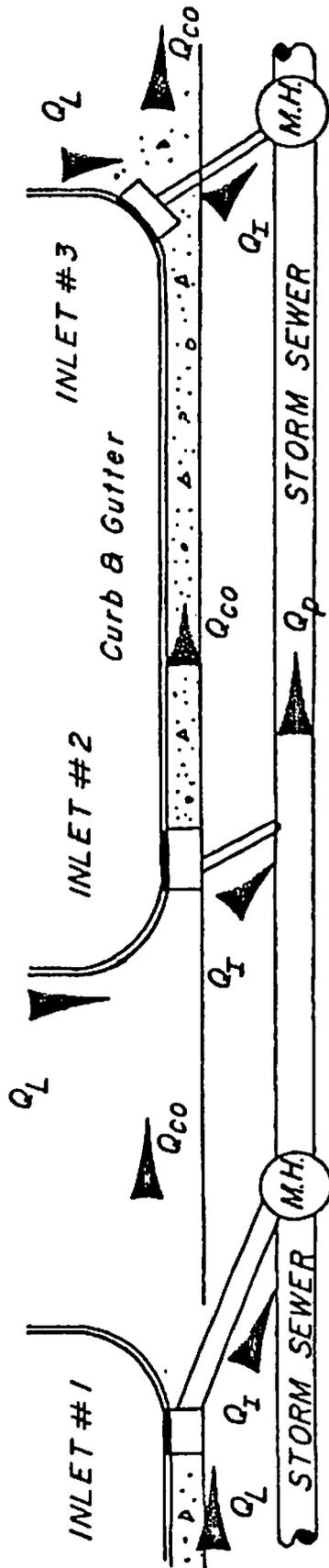
1. Allowable capacity = 66% theoretical capacity
2. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.

TYPE R CURB OPENING ON A CONTINUOUS GRADE



- NOTES: 1. Maximum inlet capacity at maximum allowable flow depth. Proportionally reduce for other depths.
2. Allowable Capacity =
 88% (L = 5')
 92% (L = 10')
 95% (L = 15') } of Theoretical Capacity
3. Interpolate for other inlet lengths.





Legend:

- Q_L = Local runoff for design storm tributary to designate inlet (cfs)
- Q_I = Runoff intercepted by inlet (cfs)
- Q_{CO} = Carry over runoff past inlet (cfs)
- Q_T = Total runoff at inlet = $Q_L + Q_{CO}$
- Q_P = Runoff in pipe

SUMMARY OF FLOWS FOR DESIGN EXAMPLE #4

INLET	Q^*	Q_L	Q_{CO}	Q_T	Q_I	Q_{CO}	Q	SEWER	COMMENTS
No. 1; 15' Type R	8.6	8	0	8	6.4	1.6	6.4	Inlet on grade	
No. 2; 10' Type R	7.2	4	1.6	5.6	3.7	1.9	10.1	Inlet on grade	
No. 3; 10' Type R	10.4	8	1.9	9.9	9.9	0	20.0	Inlet in sump condition.	

* Maximum allowable inlet capacity at maximum allowable gutter capacity, from Figure 802

CHAPTER 9 STREETS

CHAPTER 9 STREETS

9.1 INTRODUCTION

The criteria presented in this section shall be used in the evaluation of the allowable drainage encroachment within public streets. The review of all planning submittals (refer to Chapter 3) will be based on the criteria herein.

9.2 FUNCTION OF STREETS IN THE DRAINAGE SYSTEM

Urban and rural streets, specifically the curb and gutter or the roadside ditches, are part of the Minor Drainage System. When the drainage in the street exceeds allowable limits (refer to Section 2.4.4.), a storm sewer system (Chapter 7) or an open channel (Chapter 6) is required to convey the excess flows. The streets are also part of the Major Drainage System when they carry floods in excess of the minor storm (refer to Section 2.4.2), also subject to certain limitations (refer to Section 2.4.4). However, the primary function of urban streets is for traffic movement and, therefore, the drainage function is subservient and must not interfere with the traffic function of the street.

Design criteria for the collection and movement of runoff water on public streets is based on a reasonable frequency and magnitude of traffic interference. That is, depending on the character of the street, certain traffic lanes can be fully inundated once during the minor design storm return period. However, during lesser intense storms, runoff will also inundate traffic lanes but to a lesser degree. The primary function of the streets for the Minor Drainage System is therefore to convey the nuisance flows quickly and efficiently to the storm sewer or open channel drainage without interference with traffic movement. For the Major Drainage System, the function of the streets is to provide an emergency passageway for the flood flows with minimal damage to urban environment.

9.3 STREET CLASSIFICATION

The streets in the City are classified for drainage use as Type A, B, or C according to the average daily traffic (ADT) for which the street is designed. The larger the ADT, the more restrictive the allowable drainage encroachment into the driving lanes. The limits of storm runoff encroachment for each Drainage Classification and storm condition is set forth under the Policy Section of this manual (refer to Section 2.4.4).

Presented below is the Traffic Classification (i.e., Arterial, Collector, etc.), the corresponding Drainage Classification (i.e., Type A, B, or C), and the allowable theoretical flow depth before the reduction factor is applied for the minor storm. The limitations on the depth are based on the policy for encroachment (refer to Section 2.4.4).

Traffic Classification	Drainage Classification	Allowable Theoretical Minor Storm Flow Depth
Major Arterial(over 60 feet, flowline to flowline)	Type C	0.50'
Minor Arterial (36+ feet to 60 feet flowline to flowline)	Type B	0.50'
Collector (36+ feet to 60 feet flowline to flowline)	Type A	0.50'
Local (6" V.C.; 36 feet, flowline to flowline)	Type A	0.48'
Local (Hollywood Curb; 36 feet or less, flowline to flowline)	Type A	0.46'

9.4 HYDRAULIC EVALUATION

9.4.1 Allowable Capacity - Minor Storm

Based upon the policy of Section 2.4.4 and the Drainage Classification of each street in Section 9.3, the allowable minor storm capacity of each street section is calculated using the modified Manning's formula.

$$Q = (0.56) (Z/n) S^{1/2} Y^{8/3} \quad (\text{Equation 901})$$

Where

Q = discharge in cfs

Z = $1/S_x$, where S_x is the cross slope of the pavement (ft/ft)

Y = depth of water at face of curb (feet)

S = longitudinal grade of street (ft/ft)

n = Manning's roughness coefficient

The solution to the above equation can also be obtained through the use of the nomograph of Figure 901.

The allowable gutter capacity for each street cross section has been calculated and is presented in Figure 903. The calculations were performed for various street slopes and plotted in Figure 904. A Manning's n-value of 0.016 for the pavement area and 0.025 for the sidewalk/grass area were used to determine the capacity. The back slope from the curb was assumed to be 2 percent. The maximum allowable depth at the gutter is 12 inches (Section 2.4.4). the use of Figure 904 is illustrated by Example #7 in Section 9.5.

The back slope from the curb is assumed to be 2%. If this slope is not provided, the capacity graph cannot be used. In areas where the topography on each side of the street would not necessitate a 2% back slope, and if the over-curb capacity of

the street is needed, the drainage reports and construction plans must include cross sections of the street showing adequate back slope. Slope easements may be needed beyond the right-of-way line to provide for the back slope.

9.4.2 Allowable Capacity - Major Storm

The allowable street capacity for the major storm is calculated using the Manning's formula by first dividing the street cross section into the pavement area and sidewalk/grass area and then computing the individual flow contributions. The capacity is subject to the limitations set forth in the Policy Section 2.4.4 and the drainage classification of Section 9.3. The capacity calculations were performed for each street cross section and published in a Technical Memorandum on file with the City. The calculations were performed for various street slopes and plotted in Figure 904. A Manning's n-value of 0.016 for the pavement area and 0.025 for the sidewalk/grass area was used to determine the capacity. The back slope from the curb was assumed to be 2 percent. The maximum allowable depth at the gutter is 12 inches (Section 2.4.4). The use of Figure 904 is illustrated by Example #7 in Section 9.5.

9.4.3 Rural Streets

Rural streets are characterized by roadside ditches rather than curb and gutters for urban streets. The capacity is limited by the depth in the ditch and the maximum flow velocity. Refer to Section 6.4.5 for the design and capacity of roadside ditches.

9.5 DESIGN EXAMPLE

Example 7: Determination of Street Capacity

Given: Street with a traffic classification of Minor Arterial and a slope of 1.0 percent.

Find: Maximum allowable capacity for minor and major storm.

Solution:

Step 1: Determine maximum allowable depth:

From Section 9.3 for a "Minor Arterial", read the drainage classification as a Type B street. The maximum allowable theoretic depth for the initial storm is 0.50 feet.

Step 2: Determine the allowable minor storm gutter capacity:

From Figure 903, for a "Minor Arterial" with an allowable theoretic depth of 0.50 feet and a slope of 1.0 percent, read the allowable gutter capacity of 10.8 cfs. The flow

velocity can also be obtained from Figure 903 by interpolating between the velocity lines ($V = 3.0$ fps).

Step 3:

Determine the allowable major storm gutter capacity:

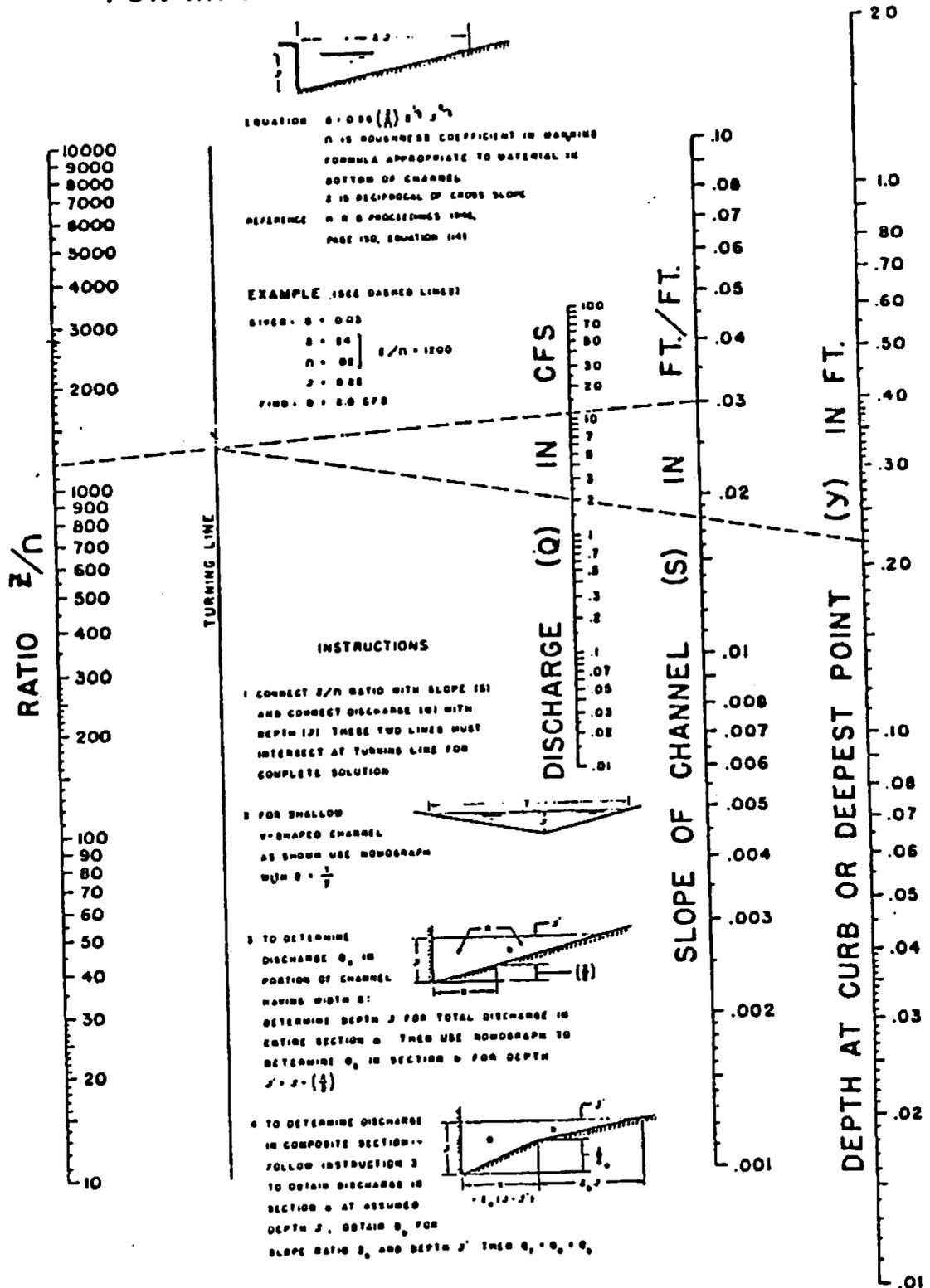
From Figure 904 for a "Minor Arterial" with a slope of 1.0 percent, read the allowable capacity of 110 cfs per gutter assuming the street is symmetrical. If the street cross-section is not symmetrical (i.e., one gutter flowline higher than the other), the capacity of the upper gutter would have to be reduced so that the lower gutter capacity is not exceeded by street cross flow from the upper gutter (see Figure 905).

9.6 CHECKLIST FOR STREET STORMWATER DESIGN

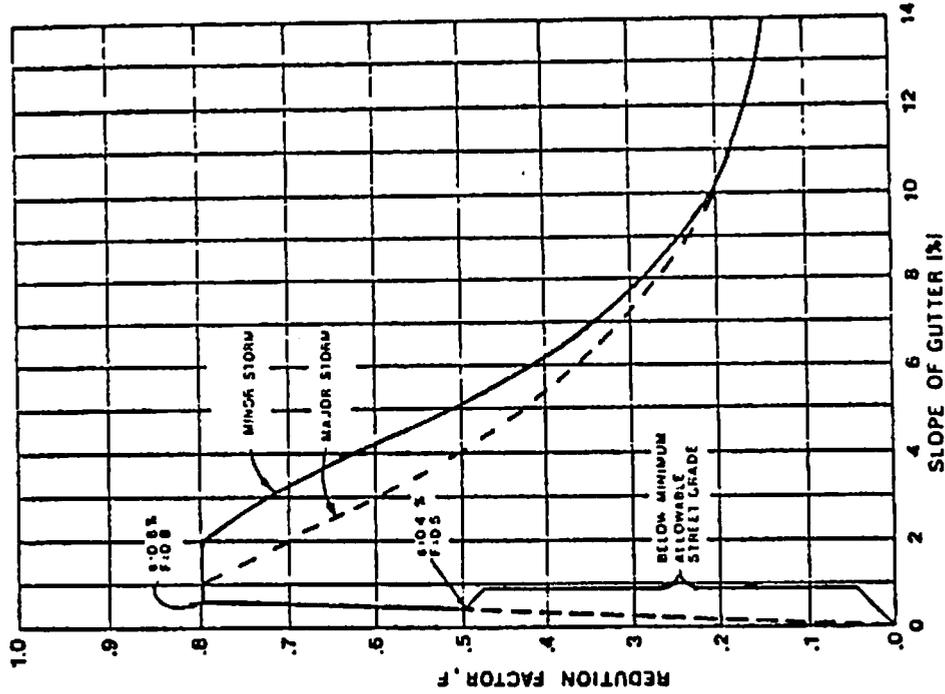
To aid the designer and reviewer, the following checklist has been prepared:

1. Use the flattest street slope to calculate the allowable gutter capacity.
2. Determine street classification first, then allowable depth and gutter capacity. Do not allow "overflow" onto adjacent private property.

FOR INFORMATION ONLY - NOT FOR DESIGN

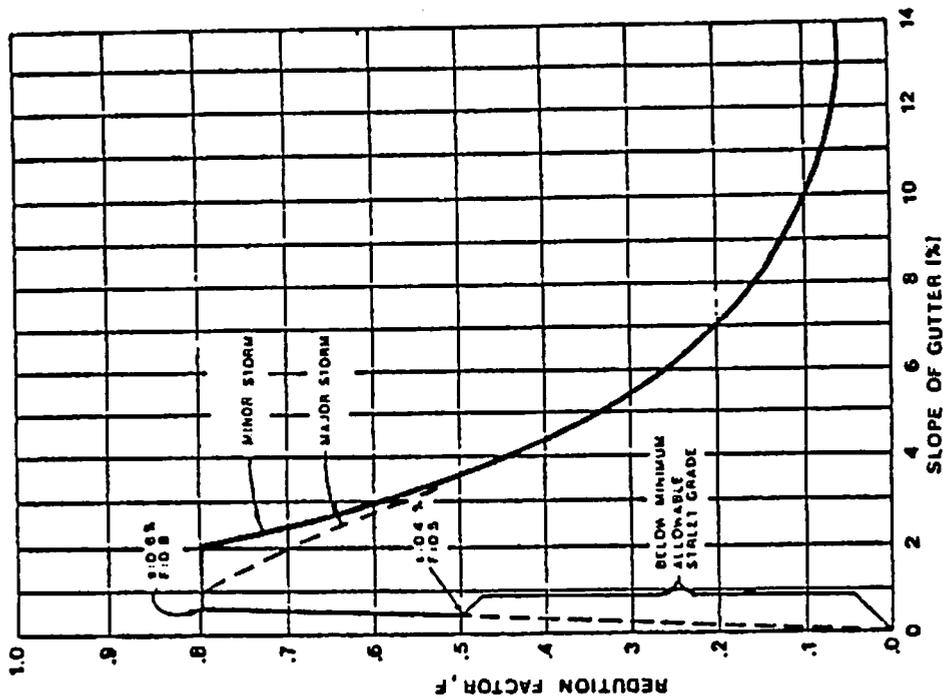


FOR INFORMATION ONLY - NOT FOR DESIGN



REDUCTION FACTOR FOR ALLOWABLE GUTTER CAPACITY LOCAL AND COLLECTOR STREETS

APPLY REDUCTION FACTOR FOR APPLICABLE SLOPE TO THE THEORETICAL GUTTER CAPACITY TO OBTAIN ALLOWABLE GUTTER CAPACITY APPROACHING ARTERIAL STREET



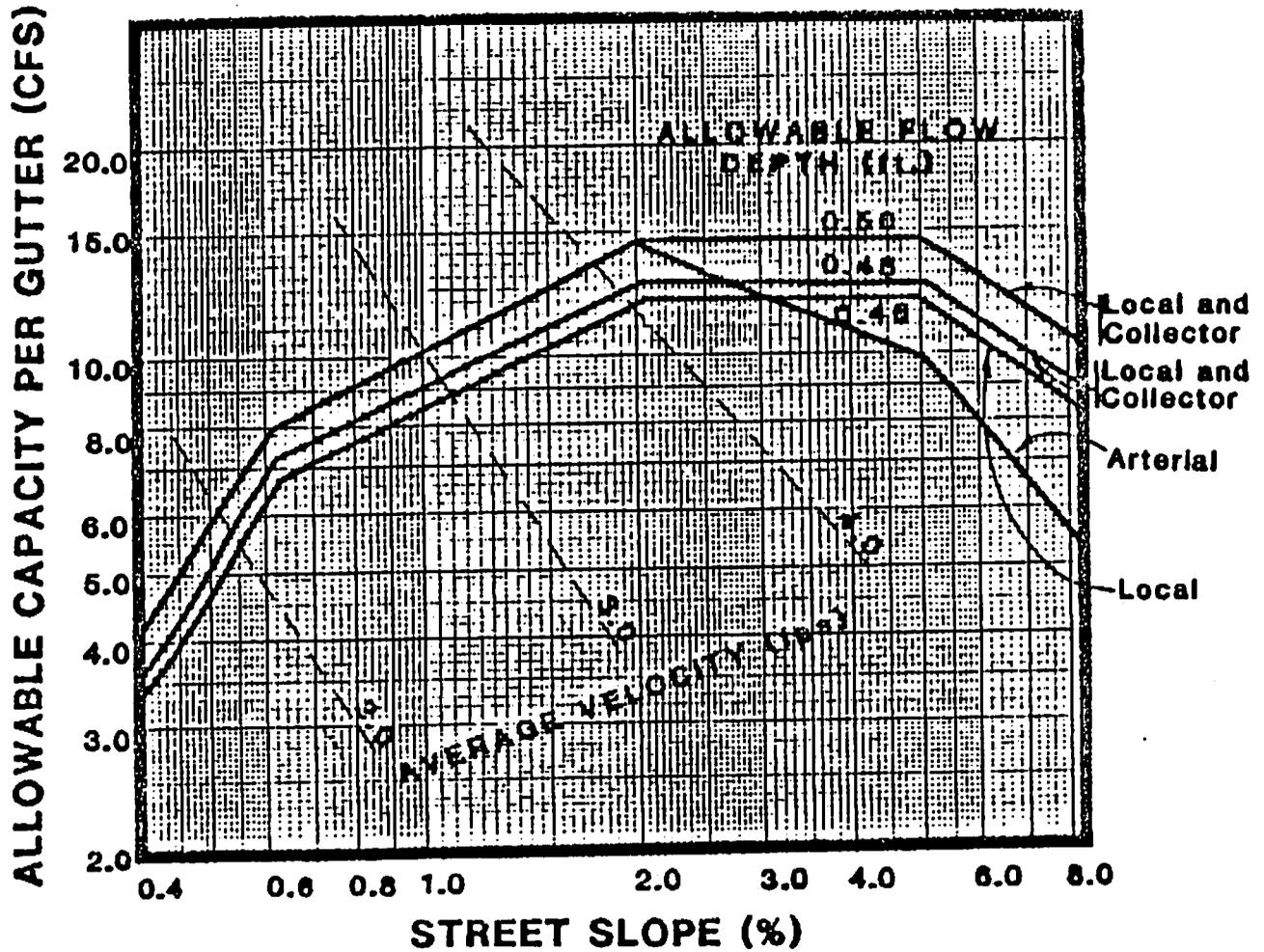
REDUCTION FACTOR FOR ALLOWABLE GUTTER CAPACITY WHEN APPROACHING AN ARTERIAL STREET

APPLY REDUCTION FACTOR FOR APPLICABLE SLOPE TO THE THEORETICAL GUTTER CAPACITY TO OBTAIN ALLOWABLE GUTTER CAPACITY APPROACHING ARTERIAL STREET

CCSDDTC
MARCH 1987

GUTTER CAPACITY REDUCTION CURVES

MINOR STORM



NOTES:

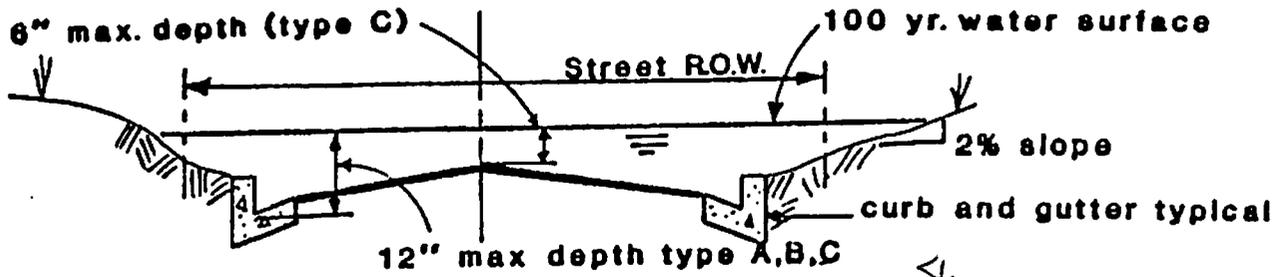
1. DESIGN CONDITIONS

$$Q = F(0.56(z/n)S^{1/2}d^{8/3})$$

F = (From Fig. 6-2, 8.2, Ref. 1)
 n = 0.016 for STREETS

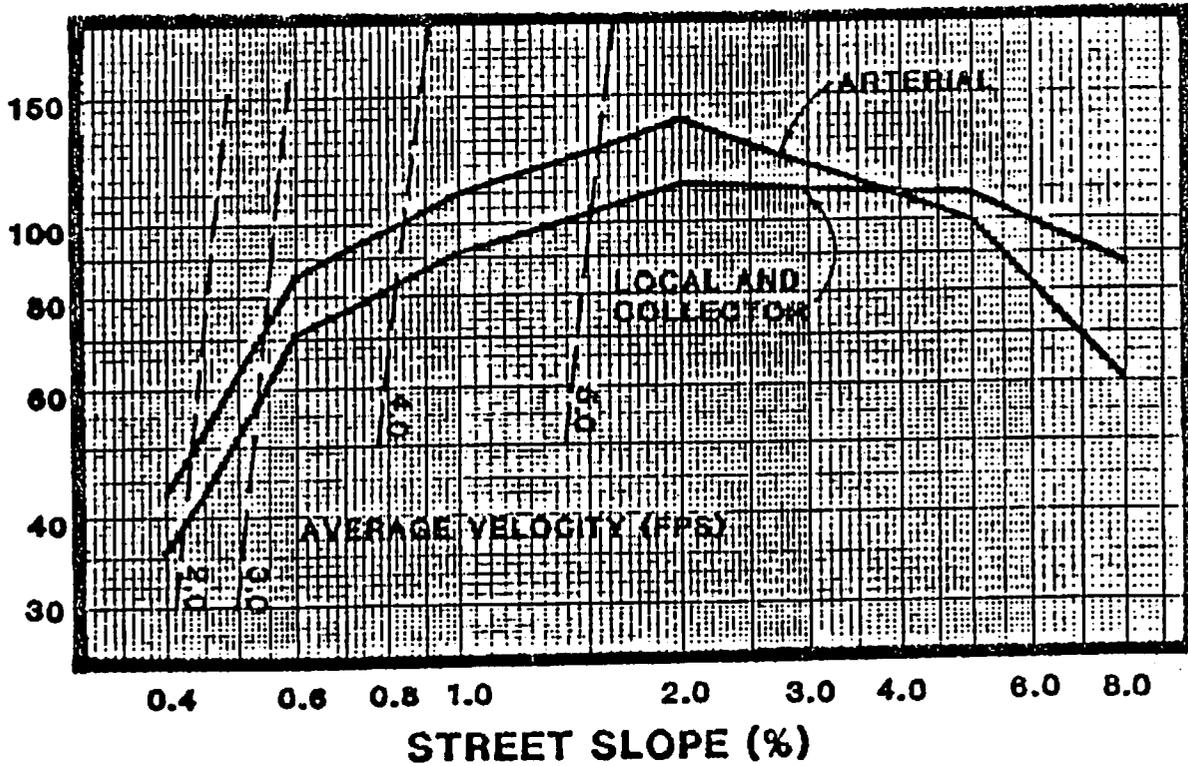
2. Figure includes reduction factor for allowable capacity

MAJOR STORM



NOTE: See section 8.4.4

ALLOWABLE CAPACITY PER GUTTER (CFS)



Notes:

1) DESIGN CONDITIONS

$$Q = \frac{F(0.56zS^{1/2}y^{8/3})}{n}$$

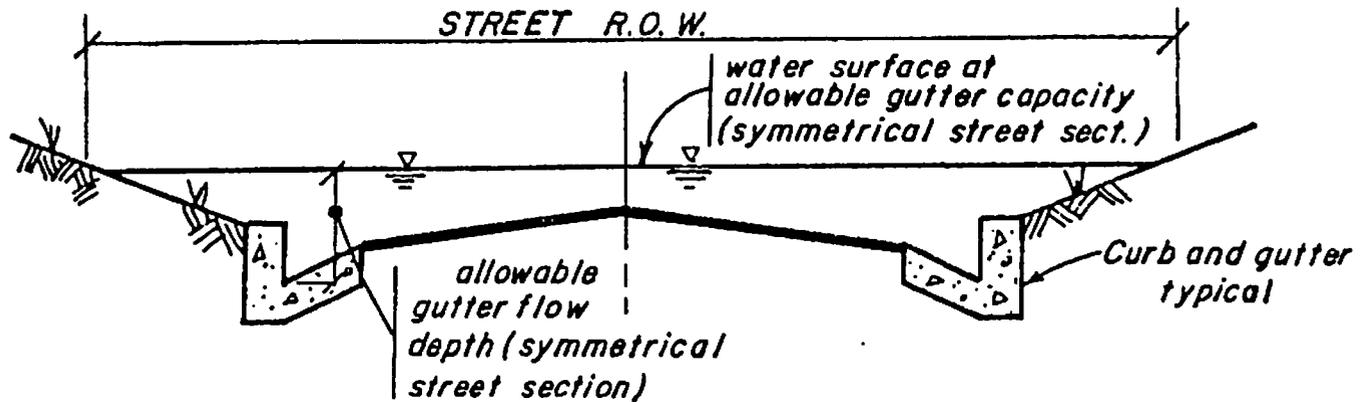
F = (from Fig. 6-2, 8.2, Ref 1)

n = 0.016 for STREETS

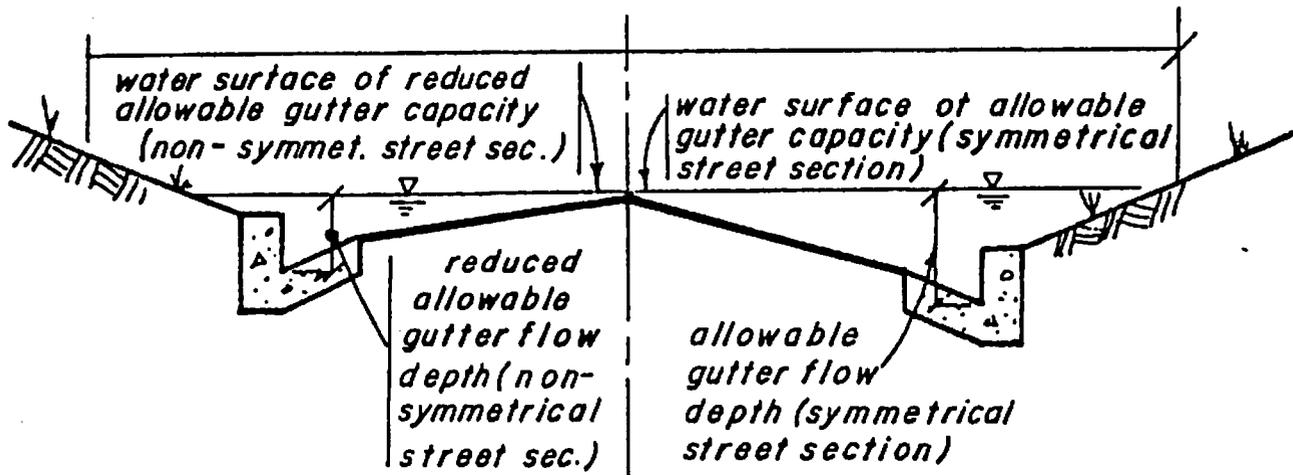
n = 0.025 for GRASS

2) Figure includes reduction factor for allowable capacity

MAJOR STORM



(a) SYMMETRICAL STREET SECTION



(b) NON-SYMMETRICAL STREET SECTION

NOTE: For non-symmetrical street section, adjust the total gutter capacity by reducing the allowable gutter capacity for the gutter with the higher flowline.

CHAPTER 10 CULVERTS

CHAPTER 10 CULVERTS

10.1 INTRODUCTION

A culvert is defined as a conduit for the passage of surface drainage water under a highway, railroad, canal, or other embankment (except detention outlets). Culverts may be constructed with many shapes and materials. Reinforced concrete pipe (RCP) is available in round, elliptical, or arch cross sections, in sizes ranging from 12 inches to 108 inches in diameter (Reference 9). The pipe may also be cast-in-place, although this construction method is generally used for storm sewers.

Corrugated Aluminum Pipe (CAP) culverts are available in round or arch cross sections (Reference-10). Sections of corrugated aluminum can also be bolted together to form several other cross sectional shapes, such as elliptical and pear shapes, forming structural plate pipe (SPP). Corrugations also come in various dimensions, which affect the hydraulics of the pipe flow.

Reinforced Concrete Box Culverts (RCBC) can be constructed with generally any rectangular cross section, the only limitations being the physical site constraints and the structural requirements. Precast box culverts are also available in several standard dimensions.

10.2 CULVERT HYDRAULICS

The procedures and basic data to be used for the hydraulic evaluation of culverts in the City shall be in accordance with the MANUAL Volume-2, "Inlets and Culverts", except as modified herein. The reader is also referred to the many texts covering the subject for additional information.

10.3 CULVERT DESIGN STANDARDS

10.3.1 *Construction Material and Pipe Size*

Within the City, culverts shall be constructed from aluminum, concrete, or polyvinyl chloride (PVC). Other materials for construction shall be subject to approval by the City Engineer.

The minimum pipe size for culverts within a public ROW shall be 12 inches diameter round culvert, or shall have a minimum cross sectional area of 0.75 ft² for arch shapes.

10.3.2 Inlet and Outlet Configuration

Within the City, all culverts are to be designed with headwalls and wingwalls, or with flared-end sections at the inlet and outlet. Flared-end sections are only allowed on pipes with diameters of 42-inches (or equivalent) or less.

Headwalls, wingwalls, and flared-end sections should be designed and constructed to use the existing land forms of the site and blend with the natural landscape. Naturally occurring stone or river rock used as a cover material is preferred.

Additional protection in the form of riprap will also be required at the inlet and outlet due to the potential scouring velocities. Refer to Section-11.2 and 11.3.

10.3.3 Hydraulic Data

When evaluating the capacity of a culvert, the following data shall be used:

- a. Roughness Coefficient - Table-1001.
- b. Entrance Loss Coefficients - Table-1002.
- c. Capacity Curves - There are many charts, tables, and curves in the literature for the computation of culvert hydraulic capacity. To assist in the review of the culvert design computations and to obtain uniformity of analysis, the following data shall be used:

All Culverts: *Urban Storm Drainage Criteria Manual*, DRCOG, Denver, Colorado 1969 (Reference-1) - Vol. 2, "Inlets and Culverts".

Concrete Pipe: *Concrete Pipe Design Manual*, ACPA, Arlington, Virginia February, 1970 (Reference-9).

Copies of the product manuals may be obtained through the local pipe suppliers.
- d. Design Forms - Standard Form SF-4 is to be used for determining culvert capacities. A sample computation is discussed in Section-10.4 and shown on Table-1003.

10.3.4 Velocity Considerations

A minimum velocity of flow is required to assure a self-cleaning condition of the culvert. At least three feet per second at the outlet is recommended.

The maximum velocity is dictated by the channel conditions at the outlet. If the outlet velocities are less than 7-fps for grassed channels, then the minimum amount of protection is required due to the eddy currents generated by the flow transition. Higher outlet velocities will require substantially more protection. A maximum outlet velocity of 12-fps is recommended with erosion protection. Refer to Section-11.2 and 11.3 for protection requirements at culvert outlet.

10.3.5 Headwater Considerations

The maximum headwater for the 100-year design flows will normally be 1.5 times the culvert diameter, or 1.5 times the culvert rise dimension for shapes other than round. Also, the headwater depth may be limited by the street overtopping policy in Section 2.4.4. For headwater depths greater than 1.5, the applicant shall submit detailed calculations determining the outlet velocity. If the outlet velocity is greater than 12 fps, an energy dissipater will be required. Refer to Section 11.3 of this CRITERIA.

10.3.6 Structural Design

As a minimum, all culverts shall be designed to withstand an HS-20 loading in accordance with the design procedures of AASHTO, "Standard Specifications for Highway Bridges", and with the pipe manufacturer's recommendation.

10.3.7 Trashracks

Trashracks may be required at the entrance and/or exit end of culverts and storm sewers for some installations as designated by the City Engineer. Installation of trashracks prevents debris from entering culverts. The culverts are protected from blockage since the debris accumulates at the trashracks. This centralized collection point allows routine cleaning of trashracks and hauling away of debris, which further protects culverts from blockage during flood events. In the event that someone is trapped in a channel during flood flows, a trashrack will enable the individual to climb to safety and not be swept into the culvert. The trashrack at the outlet end will prevent children from entering the pipe in park areas.

The following criteria shall be used for design of trashracks for culverts and storm sewers. These criteria are applicable for pipes of 24-inches in diameter (or equivalent) or greater. Design for pipes smaller than 24-inches in diameter will require much smaller structural members and a much larger rack-area to pipe-entrance-area ratio.

1. **Materials** All trashracks shall be constructed with smooth steel pipe with a minimum 1.25 inches outside diameter. The trashrack ends and bracing should be constructed with steel angle sections. All trashrack components shall have a corrosion protective finish.
2. **Trashrack Design** The trashracks shall be constructed without cross-braces (if possible) in order to minimize debris clogging. The trashrack shall be designed to withstand the full hydraulic load of a completely plugged trashrack based on the highest anticipated depth of ponding at the trashrack. The trashrack shall also be welded and non-removable.
3. **Bar Spacing:** The steel pipe bars shall be spaced with a maximum clear opening of six inches. In addition, the entire rack shall have a minimum clear opening area (normal to the rack) at the design flow depth of four times the culvert opening area.

4. **Trashrack Slope:** The trashrack at the pipe entrances shall have a longitudinal slope of no steeper than 2.5 horizontal to 1 vertical. The trashrack at the pipe exit shall fit the flared-end slope or headwall structure.
5. **Hydraulics:** Hydraulic losses through trash racks shall be computed using the following equation:

$$H_T = 0.11 (TV/D)^2 (\sin A) \quad (\text{Equation 1001})$$

where:

H_T = Head Loss through Trashrack (feet)

T = Thickness of Trashrack Bar (inches)

V = Velocity Normal to Trashrack (fps)

D = Center-to-center Spacing of Bars (inches)

A = Angle of Inclination of Rack with Horizontal

This equation applies to all racks constructed normal to the approach flow direction. The velocity normal to the trashrack shall be computed considering the rack to be 50 percent plugged.

10.4 DESIGN EXAMPLE

The procedure recommended to evaluate existing and proposed culverts is based on the procedures presented in HEC-5 Reference-12. The methodology consists of evaluating the culvert headwater requirements, assuming both inlet control and outlet control. The rating which results in the larger headwater requirements is the governing flow condition.

Example 8: Culvert Rating

A sample calculation for rating an existing culvert is presented in Table-1003. The required data are as follows:

Culvert size, length, and type (48" CMP, L = 150', n = .024).

Inlet, outlet elevation, and slope (5540.0, 5535.5, $s_0 = 0.030$).

Inlet treatment (flared end-section).

Low point elevation of embankment (EL = 5551.9).

Tailwater rating curve (see Table 1003, Column 5).

From the above data, the entrance loss coefficient, K_e , and the n-value are determined. The full flow Q and the velocity are calculated for comparison. The rating then proceeds in the following sequence:

STEP 1:

Headwater values are selected and entered in column 3. The headwater to pipe diameter ratio (H_w/D) is calculated and entered in column 2. If the culvert is other than circular, the height of the culvert is used.

STEP 2: For the H_W/D ratios, the culvert capacity is read from the rating curves (Section-10.3.3) and entered into column 1. This completes the inlet condition rating.

STEP 3: For outlet condition, the Q values in column 1 are used to determine the head values (H) in column 4 from the appropriate outlet rating curves (Section-10.3.3).

STEP 4: The tailwater depths (T_W) are entered into column 5 for the corresponding Q values in column 1 according to the tailwater rating curve (i.e., downstream channel rating computations). If the tailwater depth (T_W is less than the diameter of the culvert (D), columns 6 and 7 are to be calculated (go to Step 5). If T_W is more than D, the tailwater values in column 5 are entered into column 8 for the h_O values, and proceed to Step 6.

STEP 5: The critical depth (d_C) for the corresponding Q values in column 1 are entered into column 6. The average of the critical depth and the culvert diameter is calculated and entered into column 7 as the h_O values.

STEP 6: The headwater values (H_W) are calculated according to the equation:

$$H_W = H + h_O - LS_O$$

where H is from column 4, and h_O is from column 8 (for $T_W < D$). The values are entered into column 9.

STEP 7: The final step is to compare the headwater requirements (columns 9 and 3) and to record the higher of the two values in column 10. The type of control is recorded in column 11, depending upon which case gives the higher headwater requirements. The headwater elevation is calculated by adding the controlling H_W (column 10) to the upstream invert elevation. A culvert rating curve can then be plotted from the values in columns 12 and 1.

To size a culvert crossing, the same form can be used with some variations in the basic procedures. First, a design capacity is selected and the maximum allowable headwater is determined. An inlet type (i.e., headwall) is selected, and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control and then for outlet control. If the controlling headwater exceeds the maximum allowable headwater, a different culvert configuration is selected and the procedure repeated until the desired results are achieved.

10.5 CULVERT SIZING CRITERIA

The sizing of a culvert is dependent upon two factors, the drainage classification (i.e., Type-A, Type-B, or Type-C) and the allowable street overtopping. The allowable street overtopping for the various street classifications is set forth in Section-2.4.4. In addition to this policy, a criteria requiring that no street overtopping occur for a 5-year frequency storm has been established. Therefore, as a minimum design standard for street crossings, the following procedure shall be used:

1. Using the future developed conditions 100-year runoff, the allowable street overtopping shall be determined from overflow rating curves developed from the street profile crossing the waterway.
2. The culvert is then sized for the difference between the 100-year runoff and the allowable overtopping.
3. If the resulting culvert is smaller than that required to pass the 5-year flood peak without overtopping, the culvert shall be increased in size to pass the 5-year flow.

The criteria is considered a minimum design standard and must be modified where other factors are considered more important. For instance, if the procedure still results in certain structures remaining in the 100-year floodplain, the design frequency may be increased to lower the floodplain elevation. Also, if only a small increase in culvert size is required to prevent overtopping, then the larger culvert is recommended.

10.6 CHECKLIST FOR CULVERT DESIGN

To aid the designer and reviewer, the following checklist has been prepared:

1. Minimum culvert size is 12 inch diameter round or equivalent for other shapes.
2. Headwalls, wingwalls, or flared end sections required for all culverts.
3. Check outlet velocity and provide adequate protection.
4. Check maximum headwater for design condition.
5. Check structural requirements.

Table 1001

(A) Manning's n-values for Corrugated Steel Pipe

Corrugations	Annular 2 2/3" x 1/2"	Helical						
		1 1/2" x 1/4" 11,12		2 2/3" x 1/2"				
	All Diam.	8"	10"	12"	18"	24"	36"	48"
Unpaved	.024	.012	.014	.011	.014	.016	.019	.020
25% Paved	.021					.015	.017	.020
Fully Paved	.012					.012	.012	.012

Corrugations	Annular 3" x 1"	Helical --3" x 1"					
	All Diam	36"	48"	54"	60"	66"	72"
Unpaved	.027	.021	.023	.023	.024	.025	.026
25% Paved	.023	.019	.020	.020	.021	.022	.022
Fully Paved	.012	.012	.012	.012	.012	.012	.012

(B) Manning's n-values for Structural Plate Metal Pipe

Corrugations 6" x 2"	Diameters			
	5 ft.	7 ft.	10 ft.	15 ft.
Plain --unpaved	.033	.032	.030	.028
25% Paved	.028	.027	.026	.024

(C) Manning's n-values for Concrete Pipe/Culvert

<u>TYPE</u>	<u>n-VALUE</u>
Pre-cast	0.012
Cast-in-Place	---
With Steel Forms	0.013
With Wood Forms	0.015

CONTINUED NEXT PAGE

Table 1002

(D) CULVERT ENTRANCE LOSSES

<u>Type of Entrance</u>	<u>Entrance Coefficient, Ke</u>
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall & 45° Wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
<u>Box, Reinforced Concrete</u>	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Squared edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square edged at crown	0.70

Note: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.

CHAPTER 11 HYDRAULIC STRUCTURES

CHAPTER 11 HYDRAULIC STRUCTURES

11.1 EROSION CONTROL

Hydraulic structures are used in storm drainage work to control the flow of the runoff. The energy associated with flowing water has the potential to create damage to the drainage works, especially in the form of erosion. Hydraulic structures, which include riprap, energy dissipaters, check structures, bridges, and irrigation ditch crossings, all control the energy and minimize the damage potential of storm runoff.

The criteria to be used in the design of hydraulic structures shall be in accordance with the MANUAL Volume 2 in the "Major Drainage" and "Structures" sections. The specific criteria to be used with the modifications for the City are presented herein.

11.2 RIPRAP

The design of the riprap protection for culverts, channel bottom and banks, check drops, bridges, gabions or other areas subject to erosion, shall be in accordance with the MANUAL Volume 2, "Major Drainage Section 5 - Riprap", as revised.

11.3 ENERGY DISSIPATERS

Where riprap structures are insufficient or uneconomical to control the storm runoff, concrete energy dissipater structures (stilling basins) shall be provided in accordance with the MANUAL Volume 2 "Structures, Section 2.2 - Energy Dissipaters".

For culverts or storm sewers where the Froude number at the outlet is in excess of 2.5, the USBR Type VI impact stilling basin shall be used. An example of the USBR Type VI basin is shown on Figure 1101.

11.4 CHECK STRUCTURES

As discussed in Chapter 6, "Open Channels", there is a maximum permissible velocity for major design storm runoff in grass lined channels. One of the more common methods of controlling the flow velocity is to reduce the channel invert slope, which requires a check drop to make up for the elevation difference occurring when the channel slope is reduced.

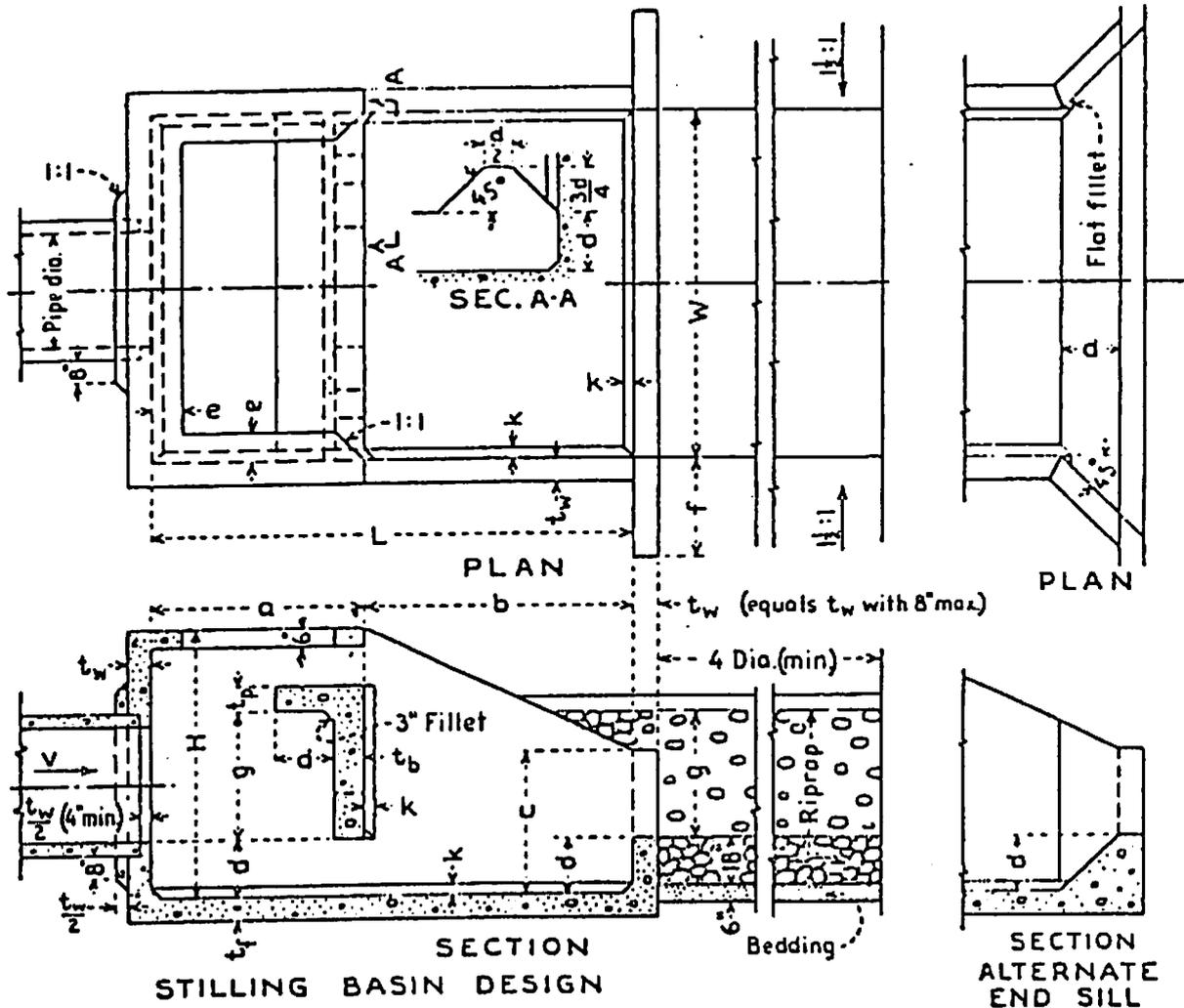
The design criteria for the check drops shall be in accordance with the MANUAL Volume 2 "Structures, Section 3 -Channel Drops". This section is being redrafted by the DISTRICT. Draft copies of the preliminary work may be obtained from the DISTRICT.

11.5 BRIDGES

The design of bridges within the City shall be in accordance with the MANUAL Volume 2, "Structures". The design capacity of the bridge shall be determined by the method presented in Section 10.5 of this CRITERIA.

11.6 IRRIGATION DITCH CROSSINGS

The City is traversed by various ditches and canals, including the O'Brian Canal and Burlington Ditch. Any proposed development in the vicinity of the ditches or canals that crosses or utilizes the canal for surface drainage shall have the plans approved by the ditch company prior to approval by the City.

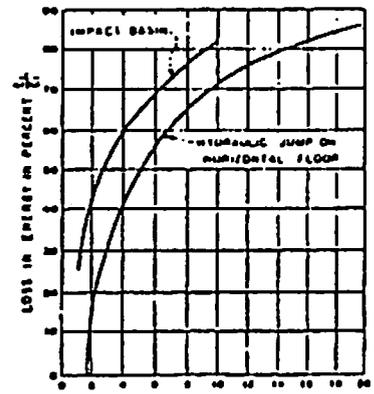
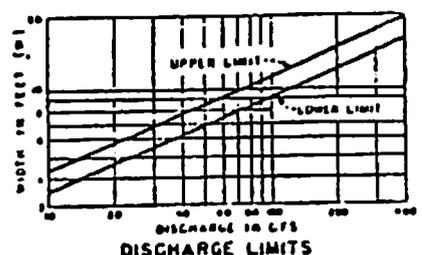
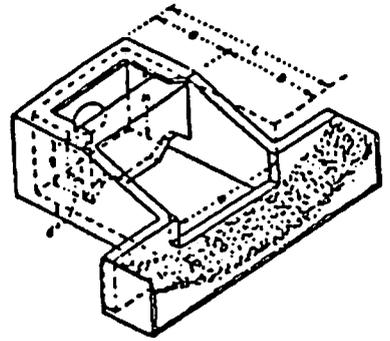


STILLING BASIN FOR PIPE ON OPEN CHANNEL OUTLET (BASIN VI)

For use on pipe or open channel outlets. Sizes and discharges from table VI should not exceed 30 feet per second. No tailwater required. Friction number usually 1.5 to 2 but not important.

PIPE DIA. (IN.)	DIA. (FT.)	FEET AND INCHES							
		a	b	c	d	e	f	g	H
10	0.83	21	0-0, 0-0	1-0	2-0	0-1	2-0	0-11	2-1
12	1.00	24	0-0	2-0	2-0	0-1	2-0	0-11	2-0
15	1.25	30	0-0	2-0	2-0	0-1	2-0	0-11	2-0
18	1.50	36	0-0	2-0	2-0	0-1	2-0	0-11	2-0
20	1.67	40	0-0	2-0	2-0	0-1	2-0	0-11	2-0
24	2.00	48	0-0	2-0	2-0	0-1	2-0	0-11	2-0
28	2.33	56	0-0	2-0	2-0	0-1	2-0	0-11	2-0
30	2.50	60	0-0	2-0	2-0	0-1	2-0	0-11	2-0
36	3.00	72	0-0	2-0	2-0	0-1	2-0	0-11	2-0
42	3.50	84	0-0	2-0	2-0	0-1	2-0	0-11	2-0
48	4.00	96	0-0	2-0	2-0	0-1	2-0	0-11	2-0
54	4.50	108	0-0	2-0	2-0	0-1	2-0	0-11	2-0
60	5.00	120	0-0	2-0	2-0	0-1	2-0	0-11	2-0

BASIC DIMENSIONS



COMPARISON OF ENERGY LOSSES

NOTE:
Refer to reference for structural details.

CHAPTER 12 DETENTION

CHAPTER 12 DETENTION

12.1 INTRODUCTION

The criteria presented in this section shall be used in the design and evaluation of all detention facilities. The review of all planning submittals (refer to Chapter 3) will be based on the criteria presented in this section.

The main purpose of a detention facility is to store the excess storm runoff associated with an increased basin imperviousness and discharge this excess at a rate similar to the rate experienced from the basin without development. The value of such detention facilities is discussed in Section 2.3.4. Any special design conditions which cannot be defined by this criteria shall be reviewed by the City Engineer before proceeding with design.

12.2 DETENTION METHODS

The various detention methods are defined on the basis of where the facility is constructed, such as open space detention, parking lot, underground or rooftop.

12.3 DESIGN CRITERIA

12.3.1 Volume and Release Rates

The minimum required volume shall be determined using the CUHP method or the following equations. These empirical equations were developed as part of the DISTRICT hydrology research program. The equations are based on a computer modeling study and represent average conditions. One of the most difficult aspects of storm drainage is obtaining consistent results between various methods for estimating detention requirements. These equations will provide consistent and more effective approaches to the sizing of onsite detention ponds. For larger water sheds where the Colorado Urban Hydrograph Procedure can be used (i.e., +_90 acres), hydrograph routing procedures will be permitted in the design of these ponds, provided the historic imperviousness of two percent or less is used.

Minimum Detention Volume:

$$V = KA \quad \text{(Equation 1201)}$$

For the 100-year,

$$K_{100} = (1.78I - 0.002I^2 - 3.56)/1000 \quad \text{(Equation 1202)}$$

For the 5-year,

$$K_5 = (0.77I - 2.26)/1000 \quad \text{(Equation 1203)}$$

Where

V = required volume for the 100- or 5-year storm (acre-feet)

I = Developed basin imperviousness (%) (See Table 501)

A = Tributary area (Acres)

The maximum release rates at the ponding depths corresponding to the 5- and 100-year volumes are as follows:

ALLOWABLE RELEASE RATES FOR DETENTION PONDS - CFS/ACRE

CONTROL FREQUENCY	SOIL GROUP*		
	A	B	C&D
5-year	0.07	0.13	0.17
100-year	0.50	0.85	1.00

* - According to the Soil Conservation Service soil group classification.

The predominate soil group for the total basin area tributary to the detention pond shall be used for determining the allowable release rate.

12.3.2 Design Frequency

All detention facilities are to be designed for two storm frequencies: the 5-year and the 100-year recurrence interval floods.

12.3.3 Hydraulic Design

Hydraulic design data for sizing of detention facilities outlet works is as follows:

1. Weir flow

The general form of the equation for horizontal crested weirs is:

$$Q = CLH^{3/2} \quad \text{(Equation 1204)}$$

Where

Q = discharge (cfs)

C = weir coefficient (Table 1201)

L = horizontal length (feet)

H = total energy head (feet)

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

$$Q = 2.5 \tan (\theta / 2) H^{5/2} \quad (\text{Equation 1205})$$

Where θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow, the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth. The example calculation for a weir design on Figure 1203 illustrates the submergence check.

2. Orifice Flow

The equation governing the orifice opening and plate is the orifice flow equation:

$$Q = C_d A (2gh)^{1/2} \quad (\text{Equation 1206})$$

Where

Q = Flow (cubic feet per second)

C_d = Orifice coefficient

A = Area (square feet)

g = Gravitational constant = 32.2 ft/sec²

h = Head on orifice measured from centerline (feet)

An orifice coefficient (C_d) value of 0.65 shall be used for sizing of square edged orifice openings and plates.

12.4 DESIGN STANDARDS FOR OPEN SPACE DETENTION

12.4.1 State Engineer's Office

Any dam constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statutes 37-87-105 as amended, shall require the approval of the plans by the State Engineer's Office. All detention storage areas shall be designed and constructed in accordance with these criteria. Those facilities subject to state statutes shall be designed and constructed in accordance with the criteria of the state.

12.4.2 Grading Requirements

Slopes on earthen embankments less than 5 feet in height shall not be steeper than 4 (horizontal) to 1 (vertical). For embankment heights between 5' and 10', the slopes shall not be steeper than 3 (horizontal) to 1 (vertical), but the horizontal distance of the slope shall not be less than 20'. For embankments greater than 10 feet in height, the slopes shall be such to maintain slope stability, but the horizontal distance of the slope shall not be less than 30 feet. All earthen slopes shall be covered with topsoil and revegetated with grass. Slopes on riprapped earthen embankments shall not be steeper than 3 (horizontal) to 1 (vertical). For grassed

detention facilities, the minimum bottom slope shall be 0.5 percent measured perpendicular to the trickle channel.

12.4.3 Freeboard Requirements

The minimum required freeboard for open space detention facilities is 1.0 feet above the computed 100-year water surface elevation.

12.4.4 Trickle Flow Control

All grassed bottom detention ponds shall include a concrete trickle channel or equivalent performing materials and design. Trickle flow criteria is presented in Section 6.4.2.6(a).

12.4.5 Outlet Configuration

Presented on Figure-1201 are two examples for detention pond outlet configuration. A Type 1 outlet consists of a grated drop inlet, outlet pipe, and an overflow weir in the pond embankment. The control for the 5-year discharge shall be at the throat of the outlet pipe under the head of water as defined on Figure-1201. The grate must be designed to pass the 5-year flow with a minimum of 50 percent blockage (i.e., twice the 5-year flow).

Since the minimum size of the outlet pipe is 12-inches, then a control orifice plate at the entrance of the pipe may be required to control the discharge of the design flow (see Section 12.4.2). An example orifice plate is shown on Figure-1202. Other outlet configurations will be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability.

The outlet shall be designed to minimize unauthorized modifications which affect proper function.

The difference between the 100-year discharge and the surcharged discharge on the 5-year outlet is released by the overflow weir or spillway. If sufficient pond depth is available, the drop inlet and the grate can be replaced by a depressed inlet with a headwall and trashrack. Depression of the inlet is required to reduce nuisance backup of flow into the pond during trickle flows. The maximum trashrack opening dimension shall be equal to the minimum opening in the orifice plate.

A Type 2 outlet consists of a drop inlet with an orifice controlled inlet for the 5-year discharge and a crest overflow and pipe inlet control for the 100-year discharge. The control for the 5-year discharge occurs at the orifice opening for the head as shown on the figure. The control for the 100-year discharge occurs at the throat of the outlet pipe as shown on the figure. However, the difference between the 100-year and 5-year discharge must pass over the weir and therefore the weir must be of adequate length. The effective weir length (L) occurs for three sides of the box.

To ensure the 100-year control occurs at the throat of the outlet pipe, a 50 percent increase in the required weir length is recommended. In addition, the outlet pipe must have an adequate slope to ensure throat control in the pipe.

12.4.6 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger than 100-year storm occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway should be set equal to or above the 100-year water surface elevation. Emergency spillway or overflow location should be separated from the pipe outlet alignment, if possible.

12.4.7 Vegetation Requirements

All open space detention ponds shall be revegetated by either irrigated sod or natural dry-land grasses in accordance with the manual "Guidelines for Development and Maintenance of Natural Vegetation" by Donald H. Godi & Associates, Inc., July 23, 1984, available through the DISTRICT.

12.5 DESIGN STANDARDS FOR PARKING LOT DETENTION

The requirements for parking lot detention are as follows:

12.5.1 Depth Limitation

The maximum allowable design depth of the ponding for the 100-year flood is 18 inches.

12.5.2 Outlet Configuration

The minimum pipe size for the outlet is 12" diameter where a drop inlet is used to discharge to a storm sewer or drainageway. Where a weir and a small diameter outlet through a curb are used, the size and shape are dependent on the discharge/storage requirements. A minimum pipe size of 3" diameter is recommended.

12.5.3 Performance

To assure that the detention facility performs as designed, maintenance access shall be provided in accordance with Section 2.3.7. The outlet shall be designed to minimize unauthorized modifications which effect function. Any repaving of the

parking lot shall be evaluated for impact on volume and release rates and are subject to approval by the Engineering Division.

12.5.4 Flood Hazard Warning

All parking lot detention areas shall be clearly identified as such with signs, each of which must have a minimum area of 1.5 square feet, and shall contain the following message:

WARNING

This area is a detention pond and is subject to periodic flooding to a depth of (provide design depth).

The signs shall be proportioned and installed in compliance with the Zoning Ordinance of the City of Commerce City.

12.6 DESIGN STANDARDS FOR UNDERGROUND DETENTION

The requirements for underground detention are as follows:

12.6.1 Materials

Underground detention shall be constructed using corrugated aluminum pipe (CAP), polyvinyl chloride (PVC) or reinforced concrete pipe (RCP). The pipe thickness cover, bedding, and backfill shall be designed to withstand HS-20 loading.

12.6.2 Configuration

Pipe segments shall be sufficient in number, diameter, and length to provide the required minimum storage volume for the 100-year design. As an option, the 5-year design can be stored in the pipe segments and the difference for the 100-year stored above the pipe in an open space detention (Section 12.4) or in a parking lot detention (Section 12.5). The minimum diameter of the pipe segments shall be 36 inches.

The pipe segments shall be placed side by side and connected at both ends by elbow tee fittings and across the fitting at the outlet (see Figure-1205). The pipe segments shall be continuously sloped at a minimum of 0.25% to the outlet. Manholes for maintenance access (see Section 12.6.4) shall be placed in the tee fittings and in the straight segments of the pipe, when required.

Permanent buildings or structures shall not be placed above any part of the underground detention.

12.6.3 Inlet and Outlet Design

The outlet from the detention shall consist of a short (maximum 25 ft.) length(s) of CAP or RCP with a 12" minimum diameter. A two-pipe outlet may be required to control both design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pipes. The outlet pipe(s) shall discharge into a standard manhole (see Standard Detail SD-6) or into a drainageway with erosion protection provided per Sections 10.3.2, 11.2, and 11.3. If an orifice plate is required to control the release rates, the plate(s) shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe(s).

Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system.

12.6.4 Maintenance Access

Access easements to the detention site shall be provided in accordance with Section 2.3.7. To facilitate cleaning of the pipe segments, 3-foot diameter maintenance access ports shall be placed according to the following schedule:

MAINTENANCE ACCESS REQUIREMENTS

Detention Pipe Size	Maximum Spacing	Minimum Frequency
36" to 54"	150'	Every pipe segment
60" to 66"	200'	Every other pipe segment
66"	200'	One at each end of the battery of pipes

The manholes shall be constructed in accordance with the detail on Figure 1205.

12.7 DESIGN STANDARDS FOR ROOFTOP DETENTION

Rooftop detention is generally discouraged and will only be allowed when all other options have been proven to be insufficient. Under no circumstances will the City accept rooftop detention as a publicly owned facility. Only the area of the roof will be allowed in the computation of the drainage basin tributary to the rooftop pond. In addition, the roof pitch or slope must be included in the computation of the volume. However, if a developer must use this technique and agrees to maintain it, it will be allowed if designed according to this criteria.

12.7.1 Depth Limitation

The maximum allowable design depth of ponding at the outlet for the 100-year flood is two inches. The current local building code may limit this depth to less than two inches. The stricter of these criteria shall apply.

12.7.2 Outlet Configuration

Rooftop detention outlets shall be designed to meet the volume and release rates as specified in this CRITERIA.

12.7.3 Overflow Provisions

Overflow drains or scuppers shall be installed for all rooftop detention facilities. These overflow drains shall be designed in accordance with the current edition of the Uniform Building Code and shall have a minimum design capacity of the peak 100-year runoff considering the detention outlet to be plugged. These drains or scuppers shall have a minimum freeboard of one inch.

12.7.4 Structural Loading

Roofs to be used for detention shall be structurally designed to account for all loads including detention at the maximum possible depth in accordance with the applicable building code.

12.7.5 Maintenance Easement

A maintenance easement and right of access to the City are required for the entire roof area to allow for inspection and cleaning of the roof drains and overflows. This easement shall be provided in accordance with Section 2.3.5.

12.7.6 Approval

Construction, maintenance, and emergency access plans shall be included with the Final Drainage Report.

12.8 DESIGN STANDARDS FOR RETENTION

12.8.1 Facility Requirement

When the City Engineer determines from review of the study and plan that stormwater retention must be employed for a specific development, the facility shall be designed using the following criteria:

1. The minimum retention volume shall equal the runoff from a 100-year storm of 24 hour duration (i.e., storm depth of 4.8 inches). No credit shall be taken for infiltration in establishing the minimum volume.
2. An overflow section shall be provided for the detention facility that will protect embankments from overflow resulting from a 100-year storm when the pond is full and the tributary area is fully developed.
3. Side slopes shall not be steeper than three (3) horizontal to one (1) vertical.
4. One (1) foot minimum freeboard above the maximum retention volume water surface.
5. The applicant must evaluate or assess the impacts of the retention facility on local groundwater levels and the potential for damage to nearby properties.

6. A slow release will be permitted of .25 CFS or less if the small flows will be conveyed to a major drainage way and will not cause nuisance conditions such as icing on highways.
7. This policy does not relieve the land developer of making permanent detention improvements on his property as a condition of subdivision approval.
8. A drainage maintenance easement shall be granted to the City to assure that emergency maintenance and access can be legally provided to keep the facility operable. This easement may be vacated when the retention pond function is no longer needed.
9. Acceptable alternatives to these requirements may include:
 - a. Agreements among landowners wherein historic flow rates are exceeded by upstream landowners and will be accepted by downstream landowners. Such agreements are subject to review by the Department of Highways/Engineering.
 - b. The developer providing offsite drainage improvements to convey stormwater, at historic rates, to an acceptable outfall point.

12.9 DESIGN EXAMPLES

Example 9: Detention Design

<u>Given:</u>	A basin that has the following characteristics: Basin Area (A) = 23 acres Basin Imperviousness (I) = 55% Predominant Soil Group = D
<u>Required:</u>	100-year and 5-year storage volumes and release rates.

Solution:

Step 1: Determine K_{100} using Equation 1202

$$\begin{aligned} K_{100} &= (1.78I - 0.002I^2 - 3.56)/1000 \\ &= (1.78(55) - 0.002(55)^2 - 3.56)/1000 \\ &= 0.0883 \end{aligned}$$

Step 2: Determine K_5 using Equation 1203

$$\begin{aligned} K_5 &= (0.77I - 2.26)/1000 \\ &= 0.0401 \end{aligned}$$

Step 3: Determine minimum required 100-year storage volume using Equation 1201

$$\begin{aligned} V &= KA \\ &= 0.0883 \times 23 \\ &= 2.03 \text{ acre-feet (88,500 ft}^3\text{)} \end{aligned}$$

Step 4: Repeat Step 3 for 5-year storage

$$\begin{aligned} V &= KA \\ &= 0.0401 \times 23 \\ &= 0.92 \text{ acre-feet (40,200 ft}^3\text{)} \end{aligned}$$

Step 5: Determine maximum allowed 100-year release rate

$$\begin{aligned} Q_{100} &= 1.00 A \\ &= 1.00 \times 23 \\ &= 23.0 \text{ cfs} \end{aligned}$$

Step 6: Repeat Step 5 for 5-year release rate

$$\begin{aligned} Q_5 &= 0.17 A \\ &= 0.17 \times 23 \\ &= 3.9 \text{ cfs} \end{aligned}$$

Example 10: Detention Outlet Structure Design

Given: Detention pond with the following characteristics (see Example 9)

Maximum 100-yr release rate = 23.0 cfs

Maximum 5-year release rate = 3.9 cfs

Type 2 outlet (refer to Figure 1201)

100-year water surface elevation = 105.0

5-year water surface elevation = 103.0
 100-year outlet pipe invert elevation = 100.0
 5-year outlet orifice invert elevation = 100.0
 18-inch diameter outlet pipe

Required: 5-year and 100-year outlet sizing

Solution: (see Figure 1204)

Step 1: Determine 5-year orifice opening size, depth to centerline of orifice = 2.5

Note: Orifice equation uses ponding depth taken from the center of the orifice.

$$A = Q/(C_d) (2gh)^{1/2} \quad (\text{Rearranged Equation 1206})$$

$$= 3.9/(0.65) ((2)(32.2)(2.5))^{1/2}$$

$$= 0.47 \text{ ft}^2$$

Step 2: Determine 5-year orifice width (using a rectangle orifice with a height of 1 foot).

Area of orifice = 1 foot X width

$$\text{width} = 0.47/1$$

$$= 0.47 \text{ feet (approximately 6 inches)}$$

Therefore, a rectangular orifice of 6 inches wide by 12 inches high is required at the entrance to the outlet box.

Step 3: Determine discharge through 5-year outlet for 100-year headwater.

$$Q = C_d A (2gh)^{1/2} \quad (\text{Equation 1206})$$

$$= 0.65(.47) ((2)(32.2)(4.5))^{1/2}$$

$$= 5.2 \text{ cfs}$$

Step 4: Size weir plate for 100-year outlet (18" RCP and H=4.25 feet)

$$A = Q/(C_d) (2gh)^{1/2} \quad (\text{Equation 1206})$$

$$= 23.0/(0.65) ((2)(32.2)(4.25))^{1/2}$$

$$A = 2.14 \text{ ft}^2$$

Step 5: Determine 100-year orifice diameter

$$\text{Diameter} = (4A/\pi)^{1/2}$$

$$= ((4)(2.14)/\pi)^{1/2}$$

$$= 1.65 \text{ feet} = 20 \text{ inches}$$

If the reduced orifice diameter is approximately equal to the pure diameter $\pm 10\%$, then no orifice plate is required.

Step 6: Determine minimum box dimensions (i.e., weir length) to assure control of the pipe inlet.

$$L = Q_{\text{weir}} / (CH^{3/2}) \quad (\text{Equation 1204})$$

$$\begin{aligned} Q_{\text{weir}} &= Q_{100} - Q \text{ (from Step 3)} \\ &= 23.0 - 5.2 = 17.8 \end{aligned}$$

$$C = 3.4 \text{ from Table 1201}$$

$$L = 17.8 / (3.4) (2')^{3/2}$$

$$L = 1.85 \text{ feet} - \text{Required length} = 1.85 (1.5) = 2.8$$

Since required weir length is only 2.8 feet, selected box dimensions suit construction and maintenance access. A minimum size of 3' x 3' is recommended.

Step 7: Check minimum size for trash rack opening area

$$\begin{aligned} \text{Min. area} &= 2 \times \text{orifice area} \\ &= (2)(1.76) \end{aligned}$$

$$\text{Min. area} = 3.5 \text{ feet}^2$$

Since box opening is 3 x 3 = 9 sq. ft., then design requirements are satisfied.

Step 8: Check minimum size for 5-year trashrack total open area.

$$\begin{aligned} \text{Min. area} &= 9 \times 5\text{-year orifice area (Figure 1202)} \\ &= 9 \times 0.47 \end{aligned}$$

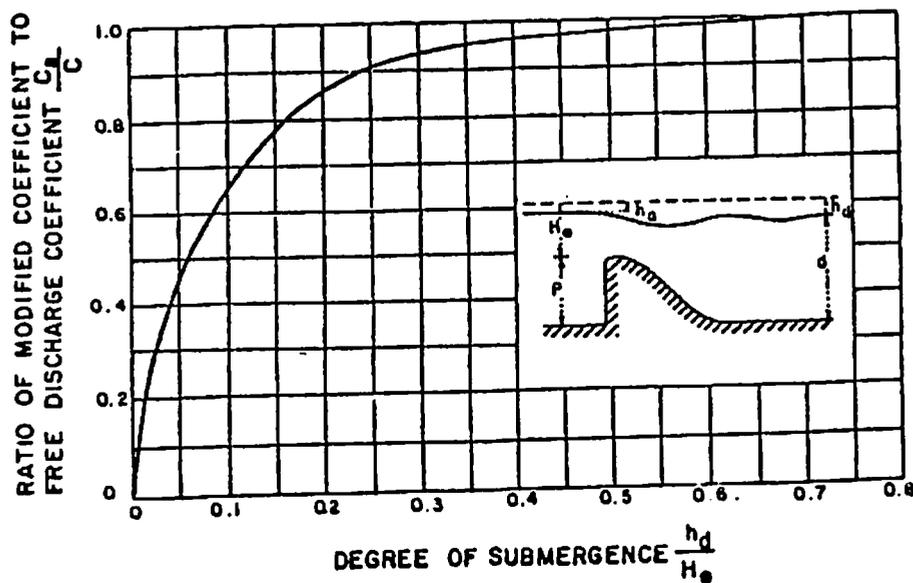
$$\text{Min. area} = 4.23 \text{ ft.}^2$$

12.10 CHECKLIST FOR DETENTION POND DESIGN

To aid the designer and reviewer, the following checklist has been prepared:

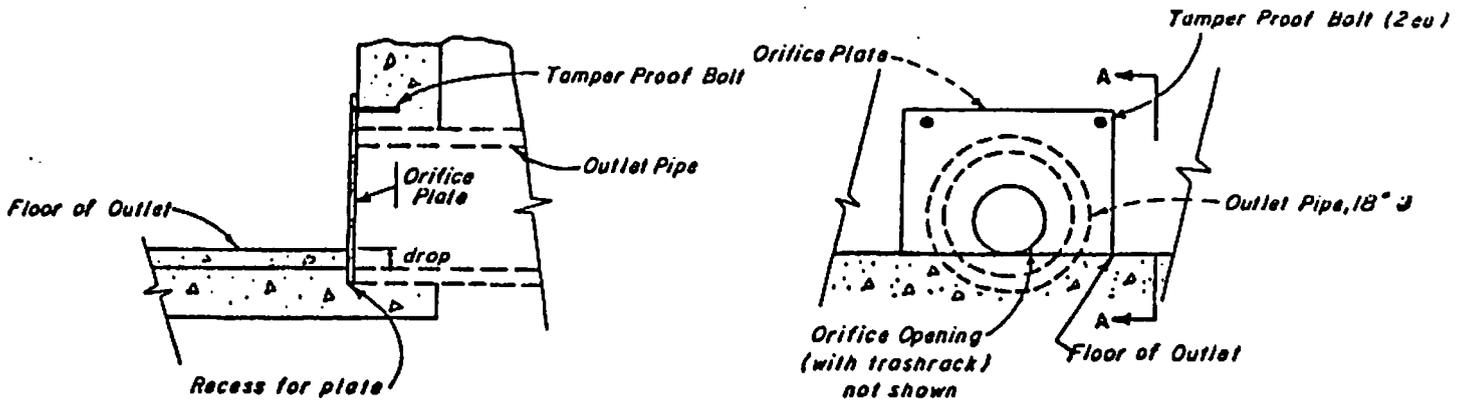
1. Earth slopes are to be 3:1 or flatter.
2. Minimum freeboard of 1 foot for the 100-year detention is required.
3. Open space detention areas to include trickle channels.
4. Protect embankment for overtopping condition by adding riprap.
5. Provide trash racks at all outlet structures.
6. Provide signs as required.
7. Provide maintenance access.

SHAPE	COEFFICIENT	COMMENTS (H in feet)	SCHEMATIC
Sharp Crested	-		
Projection Ratio (H/P = 0.4)	3.4	H < 1.0	
Projection Ratio (H/P = 2.0)	4.0	H > 1.0	
Broad Crested	-		
W/Sharp U/S Corner	2.6	Minimum Value	
W/Rounded U/S Corner	3.1	Critical Depth	
Triangular Section	-		
A) Vertical U/S Slope	-		
1:1 D/S Slope	3.8	H > 0.7	
4:1 D/S Slope	3.2	H > 0.7	
10:1 D/S Slope	2.9	H > 0.7	
B) 1:1 U/S Slope	-		
1:1 D/S Slope	3.8	H > 0.5	
3:1 D/S Slope	3.5	H > 0.5	
Trapezoidal Section			
1:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0	
2:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0	
Road Crossings			
Gravel	3.0	H > 1.0	
Paved	3.1	H > 1.0	



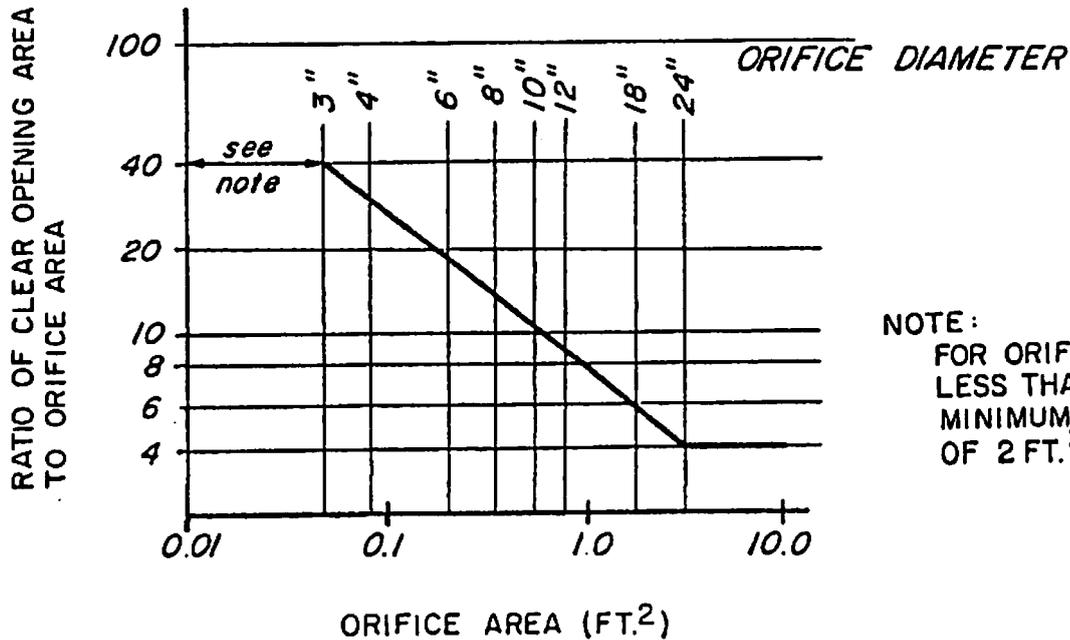
ADJUSTMENT FOR TAILWATER

(A) ORIFICE PLATE DETAILS

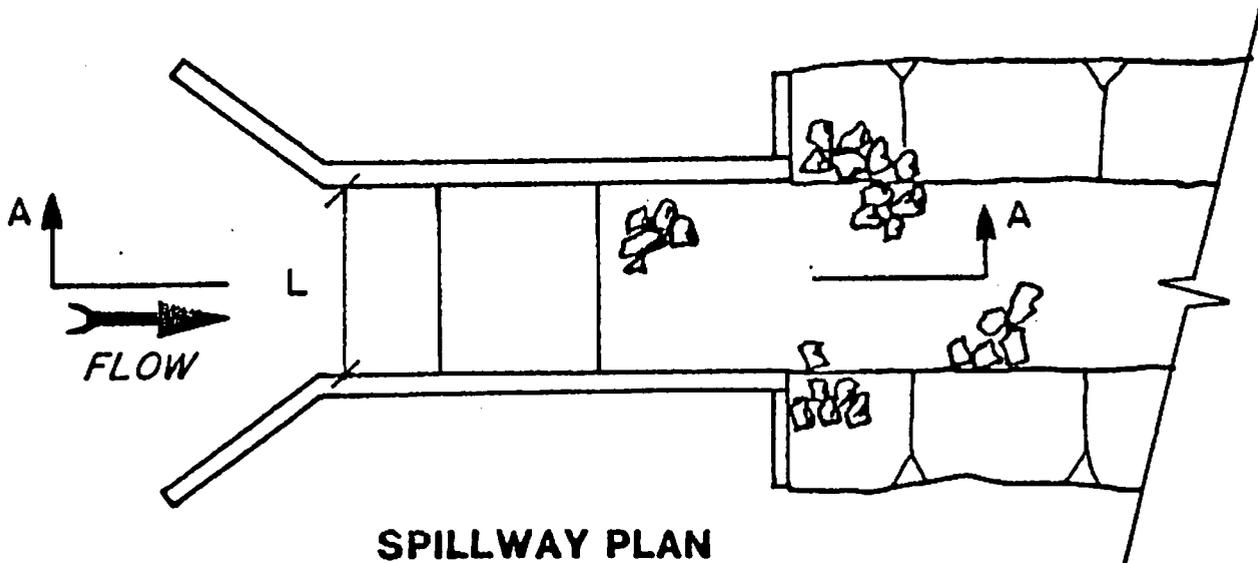


NOTE: Trashrack capacity to be 10 times orifice capacity.

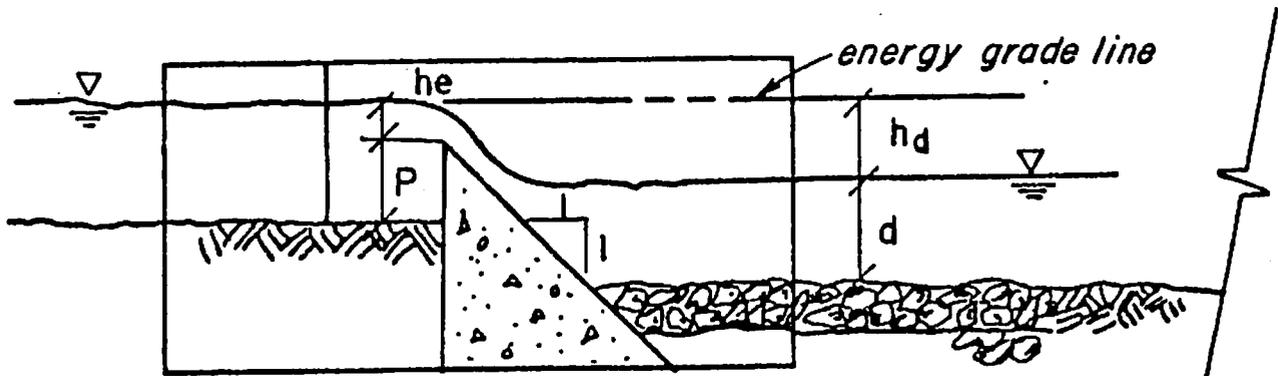
(B) TRASHRACK AREA REQUIREMENTS



NOTE:
FOR ORIFICE DIAMETER LESS THAN 3", USE A MINIMUM CLEAR OPENING OF 2 FT.²



SPILLWAY PLAN



SPILLWAY SECTION A-A

GIVEN: $Q = 100$ CFS, Triangular weir with vertical face, and 1:1 downstream slope, $P = 2'$, $h_e = 2'$, tailwater depth = $4.5'$, $h_d = 1.5$

FIND: L , and check submergence

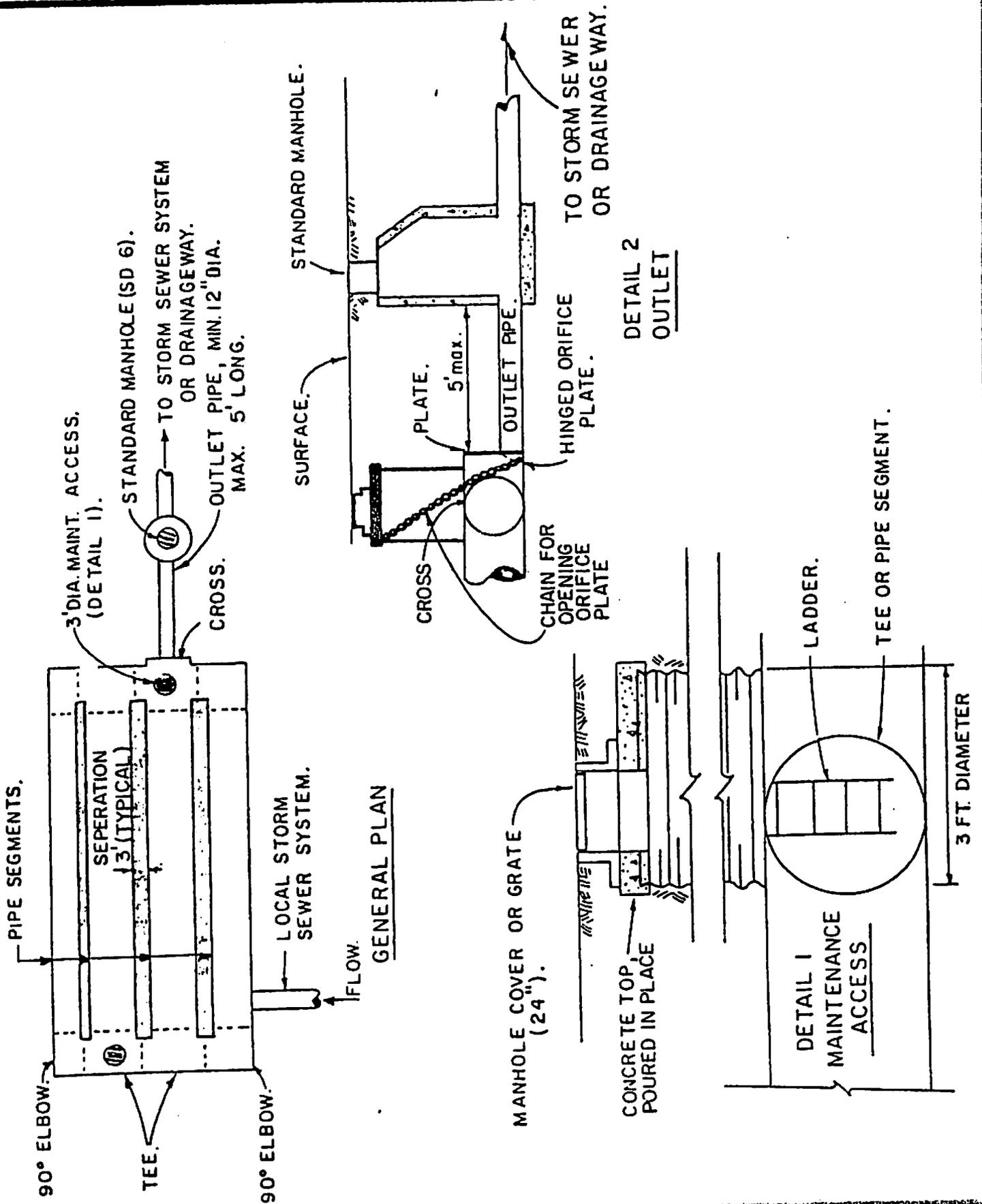
SOLUTION: $C_w = 3.8$ (Table-1201)

$$L = \frac{Q}{C_w h_e^{3/2}} = \frac{100}{(3.8)/(2)^{3/2}} = 9.3 \text{ FT}$$

Submergence check

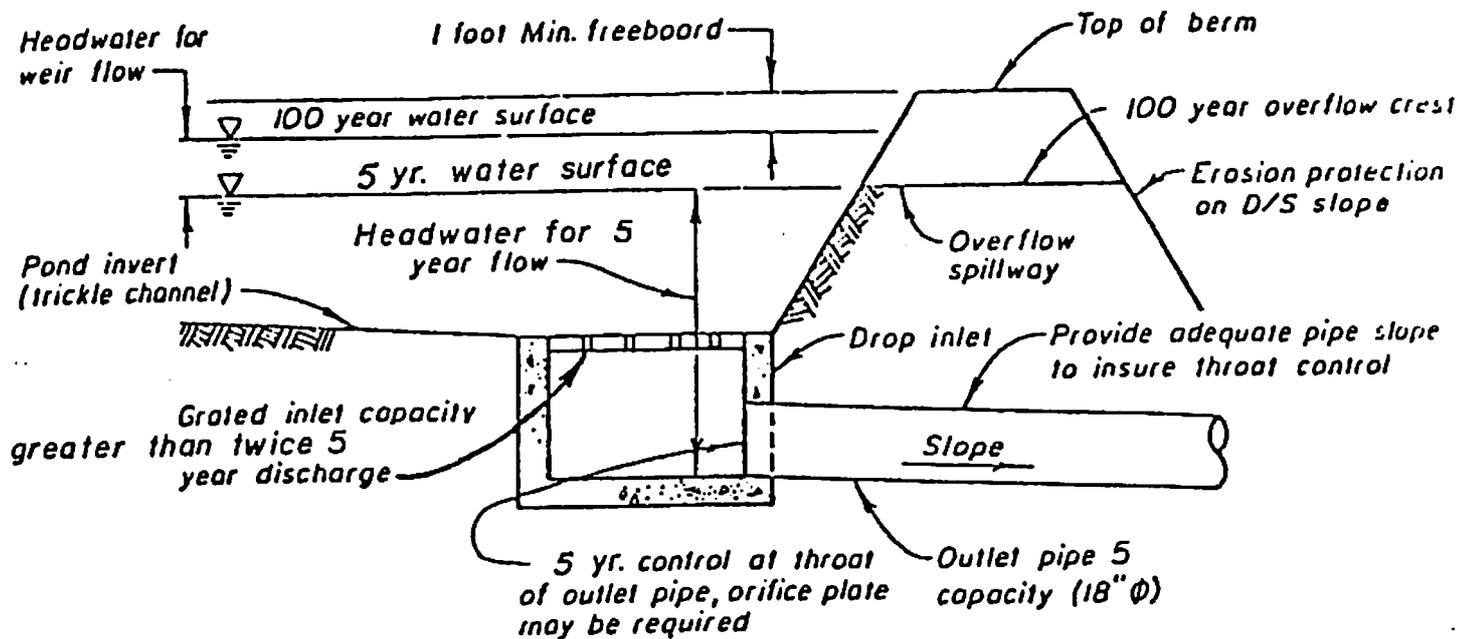
$$\frac{h_d}{h_e} = \frac{1.5}{2.0} = 0.75, \text{ then from Table-1201, } C_s/C = 1.0, \text{ therefore no submergence adjustment}$$

is required



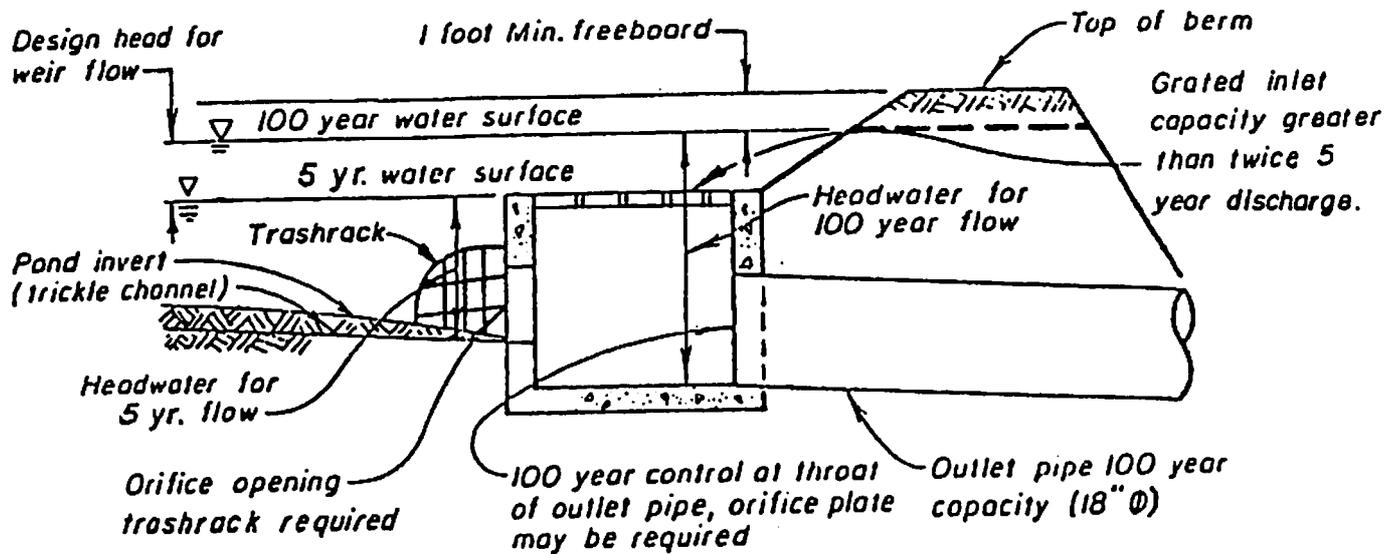
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TYPE 1 OUTLET

No Scale



TYPE 2 OUTLET

No Scale

STORM DRAINAGE AND TECHNICAL CRITERIA

SF-1

DRAINAGE STUDY SUBMITTAL CHECKLIST

PREPARED BY: DATE: _____ DATE: _____

The drainage study with plan drawings, as noted below has been received and found to lack the information noted. This information must be submitted before the study will be accepted for review. Please provide the required information and return this checklist with your submittal.

SUBDIVISION: _____

LOCATION: _____

DATE SUBMITTED: _____ TYPE OF STUDY: PRELIMINARY _____ FINAL _____

SUBMITTED BY: FIRM: _____

CONTACT: _____ PHONE: _____

SUBMITTED DATE: (1) _____ (2) _____ (3) _____ (4) _____

DATE APPROVED: _____

ITEM	DESCRIPTION	CHECKLIST	RECEIVED OR NOT APPLICABLE	TO BE SUBMITTED
1.	Typed, bound study		_____	_____
2.	Professional Engineers Certificate		_____	_____
3.	General Location and Description		_____	_____
	(a) Location Map		_____	_____
	(b) Existing Site Description		_____	_____
	(c) Description of Existing Drainage Patterns and Facilities		_____	_____
4.	Drainage Basins and Sub-Basins		_____	_____
	(a) Major Basin Description		_____	_____
	(b) Sub-basin Description		_____	_____
5.	Design Criteria		_____	_____
	(a) Development Master Plan Discussion		_____	_____
	(b) Hydrologic Criteria Discussion		_____	_____
	(c) Hydraulic Criteria Discussion		_____	_____
6.	Drainage Facility Design		_____	_____
	(a) Discussion of Proposed Facilities		_____	_____
	(b) Discussion of Drainage Patterns		_____	_____
	(c) Impact on Offsite Facilities		_____	_____
	(d) Impact on Master Plan		_____	_____
7.	Drainage Plan		_____	_____
	(a) Topographic Contours		_____	_____
	(b) R-O-W and Easements		_____	_____
	(c) Delineation of Basin and Sub-Basins		_____	_____
	(d) Existing Drainage Patterns and Facilities		_____	_____
	(e) Proposed Drainage Patterns and Facilities		_____	_____
	(f) Proposed Outfall Points		_____	_____
	(g) Routing of Offsite Drainage		_____	_____
	(h) Routing From Site to Major Drainageway		_____	_____