APPENDIX F

1.10 DESIGN CRITERIA FOR STORM WATER DRAINAGE FACILITIES (1.10 of Appendix A)

SECTION 1 INTRODUCTION

1.01 Purpose

The purpose of this manual is to establish standard methods and principals for the design and construction of surface collection and drainage systems, storm water detention and retention systems and erosion control systems within the City of Ashland, Missouri. The design factors, formulae, graphs, and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of quantity, rate of flow, method of collection, storage, conveyance, disposal of storm water, and erosion control.

This manual is intended primarily for the use of developers and their engineers in the design of storm drainage management systems. These management systems consist of and include storm drains, small culverts, street and gutter flow, hydraulics, inlets, junction boxes, natural drainage swales, detention and retention facilities, and erosion control facilities.

1.02 Scope

This manual represents the application of accepted principals of surface drainage engineering and is a working supplement to basic information obtainable from standard drainage handbooks and other publications on drainage. It is presented in a format that gives logical development of solutions to the problems of storm drainage and urbanization.

SECTION 2 DETERMINATION OF STORM RUNOFF

2.01 GENERAL

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall. The hydraulic efficiency of a drainage area is generally improved by urbanization, which in effect reduces the storage capacity and is a direct result of the elimination of porous surfaces, small ponds, and holding areas. This comes about by the grading and paving of building sites, streets, drives, parking lots, and sidewalks and by construction of buildings and other facilities characteristic of urban development.

When analyzing an area for design purposes, urbanization of the full watershed shall be assumed. Zoning maps, land use plans, and master plans should be used as aids in establishing the anticipated surface character of the ultimate development. The selection of design runoff coefficients and/or percent impervious cover factors, which are explained in the following discussions of runoff calculation, must be based upon the assumed future urbanization of the complete watershed.

Numerous methods of runoff computation are available on which the design of storm drainage and flood control systems may be based. Storm drainage facilities for residential subdivisions and small commercial or industrial developments should generally be designed on the basis of discharges calculated by the Rational Formula if tributary areas are less than 200 acres. For tributary areas larger than 200 acres, it will be necessary to use other design techniques, such as the SCS method, or the USGS urban or rural regression equations.

2.02 RATIONAL METHOD

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q = kCiA$$

where,

Q = is defined as the peak rate of runoff in cubic feet per second (CFS)

k = 1.008; a constant converting acres and inches per hour of rainfall to CFS; for the purpose of this manual k shall be taken as unity.

C = The coefficient of runoff representing the ratio of direct runoff to rainfall.

i = The average intensity of rainfall in inches per hour for a period of time equal to the critical time of flow of the drainage area to the point under consideration (in/hr).

A = drainage area of the watershed (acres)

Basic assumptions associated with the rational method:

1. The maximum runoff rate occurs when the rainfall intensity lasts as long or longer than the time of concentration.
2. The frequency of the discharge is the same as that of the rainfall intensity.
3. The fraction of the rainfall that becomes runoff is independent of the rainfall intensity or volume.

The first assumption implies that a homogeneous rainfall event is applied uniformly to the entire drainage area, and may not be valid for larger watersheds where constant rainfalls of high intensity do not occur simultaneously over the entire watershed. This assumption also provides the basis for using the watershed's time of concentration as the duration of the design storm. The second assumption again limits the size of the drainage area because for larger basins, factors other than rainfall frequency can play a large role in determining the flood frequency. Finally, the third assumption is reasonable for highly impervious areas, but less reasonable for pervious areas where the antecedent moisture condition plays a large role in determining the amount of rainfall that becomes surface runoff. For these reasons, use of the Rational Method is limited to small watersheds.

A. RUNOFF COEFFICIENT (C)

Nature of Surface.

The proportion of the total rainfall that will reach the storm drains depends on the relative porosity or imperviousness of the surface, and the slope and ponding characteristics of the surface. Impervious surfaces such as asphalt pavements and the roofs of buildings, will be subject to nearly 100 percent runoff, regardless of the slope, after the surfaces have become thoroughly wet. On site inspections and aerial photographs may prove valuable in estimating the nature of the surface within the drainage area.

Soil.

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

Runoff Coefficient.

It should be noted that the runoff coefficient "C" is the variable of the Rational Method, which is least susceptible to precise determination. Proper use requires judgement and experience on the part of the Engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

Table C-1 present recommended ranges for "C" values.

It is often desirable to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface types are given in Table C-2.

It should be noted that the runoff coefficient values given in Tables C-1 and C-2 have generally been derived for storms of 10 to 25 year frequency, and have been extended to the 100 year frequency.

B. TIME OF CONCENTRATION.

In order to determine the rainfall intensity used in the Rational Method, the time of concentration of the watershed must be estimated. The time of concentration of a watershed is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the watershed outlet. This is also the time required before the entire watershed begins to contribute flow to the watershed outlet. This characteristic response time of the watershed is used as the duration of the design storm and thus influences the value of rainfall intensity used in the Rational Method. Note that the location of the most hydraulically distant point in the watershed is a function of travel time and depends on both velocity and distance. The point in the watershed used to determine time of concentration may not necessarily be the point furthest from the watershed outlet. There may be as many as three distinct flow regimes in the watershed contributing to the time of concentration, including overland or sheet flow, ditch or channel flow, and storm sewer flow. For small rural watersheds, all flow regimes may be combined into a single equation used to calculate time of concentration.

The Kirpich equation is used for these watersheds:

$t\ c = KL^{0.77} S^{-0.385}$

C. RAINFALL INTENSITY**.** The design rainfall intensity is a function of the storm duration, the design frequency and the geographic location. The storm duration is taken as the time of concentration of the watershed or five minutes, whichever is greater. Knowing the storm duration and the design frequency, the rainfall intensity may be read from the appropriate Intensity-Duration-Frequency Figure C-1. For urban areas as defined in the CATSO area, use Figure C-8.

D. DRAINAGE AREA (A)

The drainage area (A) is the only parameter in the rational formula which is subject to accurate determination and represents the total area tributary to any point under consideration for which runoff is being determined. A current topographic map with a scale of not less than 1" = 200 ft., and a maximum contour interval of five feet should be obtained or prepared for use in drainage area calculations.

SECTION 3 FLOW IN STREETS

3.01 GENERAL

The location of inlets and permissible flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Interference to traffic is regulated by design limits of the spread of water into traffic lanes, especially in regard to arterials.

A. Interference Due to Flow in Streets

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

As the width of flow increases further it becomes impossible for vehicles to operate without moving through water. Splash from vehicles tends to obscure the vision of drivers. Eventually, if width and depth of flow become great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles be able to move along the crown of the street.

B. Interference Due to Ponding

Storm runoff that is ponded on the street surface because of grade changes, the crown slope of intersecting streets, or inlets has a substantial effect on the street carrying capacity. Because of the localized nature of ponding, vehicles moving at a relatively high speed may enter a ponded area. The manner in which ponded water affects traffic essentially the same as for curb flow, that is, the width of spread into the traffic lane is critical. Ponding in streets has the added hazard of surprise to drivers of vehicles, producing erratic and potentially dangerous response.

C. Interference Due to Water Flowing Across Traffic Lane

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. The cross-flow may be caused by super elevation of a curve, a street intersection, overflow from the higher gutter on a street with crossfall, or simply a poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature. Vehicles may be travelling at high speed when they reach the location. If vehicular movement is slow and the street is lightly travelled, as on residential streets, limited cross flows do not cause sufficient interference to be unacceptable.

The depth and velocity of cross flows shall be maintained within such limits that they will not have sufficient force to threaten moving traffic.

3.02 PERMISSIBLE SPREAD OF WATER

A. Arterial Streets

Inlets shall be spaced at such an interval as to provide one clear lane of traffic in each direction during the peak flows of a design storm having a 25-year return frequency. Two lanes of traffic being defined as 20 feet in width, being 10 feet on either side of the crown.

B. Collector Streets

Inlets shall be spaced at such an interval as to provide one clear lane of traffic having a minimum width of 12 feet during the peak flows of a design storm having a 25-year frequency.

C. Local Streets

Inlets shall be spaced at such an interval as to provide one clear lane of traffic having a minimum width of 10 feet during the peak flows of a design storm having a 10-year frequency.

3.03 DESIGN METHOD

A graph for calculating gutter flows for the City's standard residential street with a four-inch parabolic crown is provided in Exhibit C (Figure C-4). Figure C-4 may also be used for other streets with parabolic cross sections. For streets with non-parabolic cross sections, another graph (Figure C-6) is provided for two per cent cross slope to simplify the calculations for maximum gutter depth and gutter flows. Figures C-4 through C-7 are based upon the use of the City's standard barrier curb design. The use of roll-back curbs will require the designer to provide calculations verifying that the requirements of 3.02 are met.

SECTION 4 STORM DRAIN INLETS

4.01 GENERAL

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

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

A. Inlet design and location must be compatible with the criteria established in Section 3.
B. Design and location of inlets shall take bicycle and pedestrian traffic into consideration.
C. Additional recession or modification of the depression shall be considered when a traffic lane abuts the curb line.
D. When sidewalks abut the inlet they shall be tied with rebar, and shall be designed to maintain the full walk width.

4.02 INLET DESIGN

Spacing and location of inlets shall be such that the maximum allowable depth of gutter flow is not exceeded. Inlet capacity is a function of inlet configuration, street cross-slope, street longitudinal slope, and depth of gutter flow. Inlet capacity ordinarily should not be less than the quantity flow tributary to the inlet. Inlets at low points should have extra capacity as a safeguard against flooding because of possibility of flows in excess of the design flow or clogging by debris. Inlets should be placed at other than low points in addition to low points when curb capacities are exceeded.

City standard inlets are shown on Figure C-9. Appropriate uses for each type of inlet are summarized in Table 1.

TABLE 1

SUMMARY OF CITY STANDARD STORM INLETS

Type of Inlet	Street or Gutter Longitudinal Slope	Capacity Curve	Capacity Reduction Factor
Curb Inlet:			
Type "M"	Zero (sump)	Fig. C-10	0.80
	Up to 4%	Fig. C-11	0.80
Type "M" w/gutter deflector	Greater than 4%	Fig. C-11	0.80

Inlet capacities may be determined by the use of the theoretical inlet capacity curves in Exhibit C. Theoretical inlet capacities obtained from the capacity curves must be multiplied by the appropriate capacity reduction factor listed in the above table. The capacity reduction factor compensates for partial clogging of inlets by debris. When inlets are placed in a sump, spread should be checked at each of the throat transitions as well as directly in front of the inlet.

SECTION 5 FLOW IN STORM DRAINS AND THEIR APPURTENANCES.

5.01 GENERAL

A general description of storm drainage systems and quantities of storm runoff is in Section 2. It is the purpose of this section to consider the significance of the hydraulic elements of storm drains and their appurtenances to a storm drainage system.

Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exists. Storm drains accordingly are designed for open-channel flow to satisfy to the extent possible the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

5.02 VELOCITIES AND GRADES

A. Minimum Grades.

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material, otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps, which is considered to be the lower limit of scouring velocity. The minimum slope for standard construction procedures shall be 0.40%.

B. Maximum Velocities.

Maximum velocities in conduits are important mainly because of the possibilities of excessive erosion of the storm drain inverts. The maximum allowable velocity for storm drainage conduits shall be 15 fps.

C. Minimum Diameter.

Pipe which are to become an integral part of the public storm sewer system shall have a minimum diameter of 15 inches, and 18 inches under pavement.

5.03 MATERIALS

A. See Section 260 in Appendix A

5.04 FULL OR PART FULL FLOW IN STORM DRAINS

A. General

All storm drains shall be designed by the application of the continuity equation and Mannings Equation, either through the appropriate charts and nomographs or by direct solutions of the equations as follows:

$Q = AV$, and

$$Q = \frac{1.49\ AR^{2/3}Sf^{1/2}}{n}$$

Q = Pipe Flow (cfs)

A = Cross-sectional area of pipe (ft.^{2})

V = Velocity of flow (fps)

n = Coefficient of roughness of pipe

R = Hydraulic radius - A/W_p (ft.)

S_1 = Friction slope in pipe (ft./ft.)

W_p = Wetted perimeter (ft.)

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

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

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

3. At changes in pipe size match the soffits of the two pipes at the same level rather than matching the flow lines.

4. Conduits are to be checked at the time of their design with reference to critical slope.

   If the slope on the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

B. Pipe Flow Charts

Figures C-2, C-3 and C-16 are nomographs for determining flow properties in circular pipe. The nomographs are based upon a value of "n" of 0.015 for concrete and 0.025 for corrugated metal pipe.

5.05 HYDRAULIC GRADIENT AND PROFILE OF STORM DRAIN

In storm drain systems flowing full, all losses of energy through resistance of flow in pipes, by changes of momentum or by interference with flow patterns at junctions, must be accounted for by the accumulative head losses along the system from its initial upstream inlet to its outlet. The purpose of accurate determinations of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system. In this way, it is possible to determine the water surface elevation, which will exist at each structure.

While a check of the system by development of a hydraulic grade line requires minimum additional design time when utilizing an automated design process, a manual procedure can be very time consuming. Therefore, the designer must evaluate and justify the need for a hydraulic grade line check of a system on a case by case basis. Conditions that may warrant undertaking this additional design analysis are:

1. Systems with outlets that are subject to high tailwater conditions
2. Systems that transition from a steep to a flat gradient
3. Systems on flat gradient that have substantial junction and/or bend loses.

The maximum hydraulic grade line elevation shall be six inches (6”) below the lowest level of any inlet opening or twelve inches (12”) below the rim of a junction box or manhole.

5.06 MANHOLE LOCATION

Manholes shall be located at intervals not to exceed 400 feet for pipe 30 inches in diameter or smaller. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade and changes of alignment.

Manholes for pipe greater than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 600 feet.

5.07 PIPE CONNECTIONS

Prefabricated wye and tee connections may be utilized provided at least one of the pipes is greater than 30 inches in diameter.

5.08 MINOR HEAD LOSSES AT STRUCTURES

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends in the design of closed conduits. See figures C-12 and C-13 for details of each case. Minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved.

The basic equation for most cases, where there are both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient K_j shown in Tables C-5.

$$h_j = \frac{V_2^2 - K_j V_1^2}{2g}$$

h_j = Junction or structure head loss in feet.

v_1 = Velocity in upstream pipe in fps.

v_2 = Velocity in downstream pipe in fps.

K_j = Junction or structure coefficient of loss.

In the case where the initial velocity is negligible the equation for head loss becomes:

$$h_g = \frac{K_j V_2^2}{g}$$

Pipe shall be installed in a straight line and grade for all pipes 30 inches in diameter and smaller.

Short radius bends may be used on 33 inch and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

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

$$h_j = \frac{K_j V^2}{2g} \quad \text{where } v = \text{velocity in smaller pipe}$$

5.09 UTILITIES

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities such as water, gas and sanitary sewer lines.

When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to either the drainage system or the utility can then be determined.

SECTION 6 DESIGN OF ENCLOSED STORM DRAINAGE SYSTEMS

6.01 GENERAL

All storm drains shall be designed by the application of the Manning Equation either directly or through appropriate charts or nomographs. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway in question.

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

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

B. Runoff resulting from storms exceeding the design storm must be anticipated and disposed of with minimum damage to physical facilities and minimum interruption of normal traffic.

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

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

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

F. The storm drainage system must be adaptable to future expansion with minimum additional cost.

An example of the design of a storm drainage system is outlined in Paragraphs 6.03 and 6.04. The design theory has been presented in the preceding sections with corresponding tables and graphs of information.

6.02 PRELIMINARY DESIGN CONSIDERATIONS

Careful planning of storm drainage systems in the preliminary design phase offers the greatest potential for cost savings and for compliance with storm drainage objectives. The best time to prepare conceptual layouts of storm drainage systems is prior to finalization of street layout, easement location, and site grading. Options available to the drainage engineer are greatly reduced once surface characteristics of the drainage basin have been set.

In storm drainage system design, a significant part of the construction cost is represented by small diameter storm drains. The longer that overland flow can be kept from reaching the street network, the further the distance from the ridge line that the storm drain system need begin, and the fewer the number of inlets that will be required. Various layout concepts should be developed and analyzed prior to selection of a final concept for detailed design.

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

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

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

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

E. Show all existing underground utilities.

F. Establish design rainfall frequency.

G. Establish minimum inlet time of concentration.

H. Establish the typical cross section of each street.

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

J. Include A. through I. with plans submitted to the Engineering Department for review. The drainage map submitted shall be suitable for permanent filing in the Engineering Department and shall be a good quality reproducible.

6.03 INLET SYSTEM

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

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

B. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial sub-area. If the calculated runoff is less than the allowable street capacity, increase the size of the trial sub-area. Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacities versus runoff entering the street downstream.

C. Record the drainage area, time of concentration, runoff coefficient and calculated runoff for the sub-area. This information shall be recorded on the plans or in tabular form convenient for review.

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

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

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

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

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

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

6.04 STORM SEWER SYSTEM

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

A. Storm Sewer Pipe

The ground line profile is now used in conjunction with the previous runoff calculations. The elevation of the hydraulic gradient is arbitrarily established approximately two (2) feet below the ground surface. When this tentative gradient is set and the design discharge is determined, a Manning flow chart may be used to determine the pipe size and velocity.

It is probable that the tentative gradient will have to be adjusted at this point since the intersection of the discharge and the slope on the chart will likely occur between standard pipe sizes. The smaller pipe should be used if the design discharge and corresponding slope does not result in an encroachment on the six (6) inch criteria below the inlet opening. If there is encroachment, use the larger pipe, which will establish a capacity somewhat in excess of the design discharge. Velocities can be read directly from a Manning Flow Chart based on a given discharge, pipe size and gradient slope (Figures C-2 and C-3).

B. Junctions, Inlets and Manholes

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

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

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

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

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

6. Determine head loss coefficient "k" at junction, inlet, or manhole at design point from Table C-5 or Figure C-12, C-13.

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

   $$h_j = \frac{V_2^2 - K_j V_1^2}{2g}$$

8. Compute hydraulic gradient at upstream end of junction as if junction were not there.

9. Add head loss to hydraulic gradient elevation determined to obtain hydraulic gradient elevation at upstream end of junction.

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

SECTION 7 FLOW IN OPEN CHANNELS

7.01 GENERAL

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

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are that:

A. Velocities are usually low, resulting in long concentration times and lower downstream peak flows.

B. Channel storage tends to decrease peak flows.

C. Maintenance needs are usually low because the channel is somewhat stabilized.

D. The channel provides a desirable green belt and recreational area adding significant social benefits.

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

In many areas facing urbanization, the runoff has been so minimal that natural channels do not exist. However, small trickle paths nearly always exist which provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal trickle channels to reduce development costs and minimize drainage problems. In some cases the prudent utilization of natural water routes in the development of a major drainage system will reduce the requirements for an underground storm sewer system.

Channel stability is a well-recognized problem in urban hydrology because of the significant increase in low flows and peak storm runoff flows. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures can be taken at reasonable cost, which will preserve the natural appearance without sacrificing hydraulic efficiency.

7.02 CHANNEL DISCHARGE

A. Manning's Equation

Careful attention must be given to the design of drainage channels to assure adequate capacity and minimum maintenance to overcome the results of erosion and silting. The hydraulic characteristics of channels shall be determined by Manning's equation.

$$Q = \underline{1.49} \ A \ R^{2/3} S^{1/2}$$

n

Q = Total discharge in cfs

n = Coefficient of roughness

A = Cross-sectional area of channel in sq. ft.

R = Hydraulic radius of channel in feet, cross sectional area of outflow divided by the wetted perimeter A/P.

S = Slope of the frictional gradient in feet per foot.

B. Uniform Flow

For a given channel condition of roughness, discharge, and slope, there is only one possible depth for maintaining a uniform flow. This depth is the normal depth. When roughness, depth, and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through the section. This discharge is the normal discharge.

If the channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in the field or to obtain in the laboratory. Channels are sometimes designed on the assumption that they will carry uniform flow at the normal depths, but because of conditions difficult if not impossible to evaluate and hence not taken into account, the flow will actually have depths considerably different from uniform depth. The engineer must be aware of the fact that uniform flow computation provides only an approximation of what will occur; however, such computations are useful and necessary for planning.

C. Normal Depth

The normal depth is computed so frequently that it is convenient to use nomographs for various types of cross sections to eliminate the need for trial and error solutions, which are time consuming. A nomograph for uniform flow is given in Figure C-14.

D. Critical Depth

For a channel cross section with a specified discharge, Q, uniform flow may occur at critical depth, at less than critical depth, or at more than critical depth, depending on the channel slope. Flow at or near critical depth, dc, is highly unstable and channel sections giving the depth of flow near the critical depth should be avoided. Subcritical velocity will prevail at normal depths greater than the critical depth and will occur on mild slopes. Supercritical velocity will prevail at normal depths less than the critical depth, and will occur on steep slopes.

Critical flow is characterized by a Froude number, F, equal to unity. If F is less than 1.0, the flow is subcritical and if F is greater than 1.0, the flow is supercritical. The Froude number, F, is defined as:

$$F = \frac{V}{g\, d_m}$$

in which:

V = velocity, in feet per second

g = gravitational constant, 32.2 feet per second squared

d_m = hydraulic depth A/b_w

where:

b_w = width of water surface

A = cross-sectional area of flow.

Flow that passes from supercritical to sub-critical may result in a hydraulic jump and should always be investigated for potential problems.

It is rare that uniform flow will occur in all reaches of a channel. There will normally be interconnected reaches of uniform and non-uniform flow. The determination of water surface profiles for a given discharge in the area of non-uniform flow may be necessary to ensure against extensive property damages. Computations should begin at a known point and extend upstream for sub-critical flow and downstream for supercritical flow.

7.03 WATER SURFACE PROFILES

Open channel flow in urban drainage systems is usually non-uniform because of bridge openings, curves and structures. This necessitates the use of backwater computations for all final channel design work.

A water surface profile must be computed for all channels and shown on all final drawings. Computation of the water surface profile should utilize standard backwater methods or acceptable computer routines, taking into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions. HEC-RAS would be an acceptable computer program for providing this information.

7.04 DESIGN CONSIDERATIONS

Channels should have trapezoidal section of adequate cross-sectional areas to take care of uncertainties in runoff estimates, changes in channel coefficients, channel obstructions and silt accumulations.

Accurate determination of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value for each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table C-7 gives a method of determining the composite roughness coefficient based on actual channel conditions.

Where practicable, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not be so great as to create erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the Director of Public Works, channel velocities in man-made channels shall not exceed 6 fps.

7.05 CHANNEL 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 to create additional sociological benefits.

A. Side Slope

Except in horizontal curves the flatter the side slope, the better. Normally slopes shall be no steeper than 3:1, which is also the practical limit for mowing equipment. Rock or concrete lined channels or those which for other reasons do not require slope maintenance may have slopes as steep as 1 ½ :1.

B. Depth

Deep channels are difficult to maintain and can be hazardous. Constructed channels should be as shallow as practical.

C. Bottom Width

Channels with narrow bottoms are difficult to maintain and are conducive to high velocities during high flows. It is desirable to design open channels such that the bottom width is at least twice the depth.

D. Trickle Channels

The low flows, and sometime base flows, from urban areas must be given specific attention. If erosion of the bottom of the channel appears to be a problem, low flows shall be carried if in a paved trickle channel which has a capacity of 5.0 percent of the design peak flow. Care must be taken to insure that low flows enter the trickle channel without the attendant problem of the flow paralleling the trickle channel.

E. Freeboard

For channels with flow at high velocities, the surface roughness, wave action, air bulking, and splash and spray are quite erosive along the top of the flow. Freeboard height should be chosen to provide a suitable safety margin. The height of freeboard shall be a minimum of one foot, or provide an additional capacity of approximately one-third of the design flow. For deep flows with high velocities one may use the formula:

Freeboard (in feet) = $1.0 + 0.025\ v^{3}\ d$, where

v = velocity of flow
d = depth of flow

For the freeboard of a channel on a sharp curve, extra height must be added to the outside bank or wall in the amount:

$$H = \frac{V^2\ T}{g\ R}$$

H = additional height on outside edge of channel (ft.)

V = velocity of flow in channel (fps)

T = width of flow at water surface (ft.)

R = centerline radius of turn (ft.)

g = acceleration of gravity (32.2 ft/sec.^{2})

For channels designed for supercritical flow, additional freeboard may be required depending upon the risk of damage which could occur if flow were to become sub-critical due to debris or other obstructions.

7.06 CHANNEL DROPS

The use of channel drops permit adjustment of channel gradients, which are too steep for the design conditions. In urban drainage work it is often desirable to use several low head drops in lieu of a few higher drops. Special attention must be given to protecting the channel from erosion in the area of channel drops.

The use of sloped drops will generally result in lower cost installations. Sloped drops can easily be designed to fit the channel topography.

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour, and shall not cause an upstream water surface drop, which will result in high velocities upstream. Side cutting just downstream from the drop is a common problem, which must be protected against.

The length L will depend upon the hydraulic characteristics of the channel and drop. For a q of 30 cfs/ft., L would be about 15 feet, that is, about 1/2 of the q value. The L should not be less than 10 feet, even for low q values. In addition, follow-up rip-rapping will often be necessary at most drops to more fully protect the banks and channel bottom. The criteria given is minimal, based on the philosophy that it is less costly to initially under protect the riprap, and to place additional protection later after erosional tendencies are determined in the field. Project approvals are to be based on provisions for such follow-up construction.

7.07 BAFFLE CHUTES

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective. They are particularly useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes are used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are low, no stilling basin is needed. The chute, on a 2:1 slope or flatter, may be designed to discharge up to 60cfs per foot of width, and the drop may be as high as structurally feasible. The lower end of the chute is constructed to below stream- bed level and backfilled as necessary. Degradation of the stream-bed does not, therefore, adversely affect the performance of the structure. In urban drainage design, the lower end should be protected from the scouring action.

The baffled apron shall be designed for the full design discharge. Baffle chutes shall be designed using acceptable methods such as those presented by A.S. Peterka of the United States Bureau of Reclamation in Engineering monograph No. 25.

SECTION 8 DESIGN OF CULVERTS

8.01 GENERAL

The function of a drainage culvert is to pass the design flow under a streetway, railstreet, or yard area without causing excessive backwater and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure, which will meet these requirements.

8.02 QUANTITY OF FLOW

The design storm flow shall be computed by the rational method or other approved method as set forth in Section 2 of this manual. The system shall be designed to handle frequency storms as outlined in Table C-4 in the Exhibit.

8.03 HEADWALLS, ENDWALLS, AND END SECTIONS

A. General

The normal function of properly designed headwalls, endwalls, and end sections are to anchor the culvert to prevent movement due to lateral pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. End sections shall be the same material as the pipe except that corrugated metal end sections may be galvanized metal. Concrete end sections shall have a toewall, either pre-cast or cast in place. All headwalls and endwalls shall be reinforced concrete, and may be either straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by site conditions.

B. Conditions at Entrance

It is important to recognize that the operational characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts must involve consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

$$h_e = \frac{V_2^2 - K_e V_1^2}{2g}$$

where:

h_e = Entrance head loss in feet

V_2 = Velocity of flow in culvert in fps.

V_1 = Velocity of approach flow in fps.

K_e = Entrance loss coefficient shown in Table C-6

In order to compensate for the retarding effect on the velocity of approach in channels produced by the creation of the headwater pools at culvert entrances, the velocity of the approach in the channel (Va) shall be reduced by the factors below:

Reduction Factors for Approach of Velocity

Velocity of Approach "Va" (fps)	Description of Conditions	V1 to be used in formula for he
0-6	All Culverts	V1 = Va
Above 6	Good alignment of approach channel headwater pool within drainage easement	V1 = 0.5 Va
Above 6	Good alignment of the approach channel; channel slopes have been line; limited backwater pool permissible	V1 = 0

C. Type of Headwall, Endwall, or End Section

In general the following guidelines should be used in the selection of the type of headwall, endwall, or end section.

Parallel (to streetway) Headwall and Endwall

1. Approach velocities are low (below 6 fps).
2. Backwater pools are permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall, Endwall, or End Section

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3. Medium amount of debris exist.

The wings of flared walls should be located with respect to the direction of the approaching flow instead of the culvert axis.

Warped Headwall and Endwall

1. Channel is well defined and concrete lined.

2. Approach velocities are between 8 and 20 fps.

3. Medium amount of debris exist.

These headwalls are effective with drop down aprons to accelerate flow through culvert, and are effective endwalls for transitioning flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

8.04 CULVERT DISCHARGE VELOCITIES

The velocity of discharge from culverts should be limited as shown below. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and streetway embankment.

Culvert Discharge - Velocity Limitations

Downstream Condition	Maximum Allowable Discharge Velocity (fps)
Erosion Control Blanket	8 fps
Rip-rap Apron	15 fps (**See Appendix B-1, Drawing 530.03)**

8.05 SELECTION OF CULVERT SIZE AND TYPE

A. Culvert Types

Culverts shall be selected based on hydraulic principals, economy of size and shape, and with a resulting headwater depth, which will not cause damage to adjacent property. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. Five types of operating conditions are issued below with a discussion of each of the following. See Appendix A for sample calculation procedure and Appendix for sample calculation forms.

Type I Flowing part full, with outlet control and tailwater depth below the critical depth (Figure 8-1).

Type IIFlowing part full with outlet control and tailwater depth above the critical depth (Figure 8-2).

Type III Flowing part full with inlet control (Figure 8-3).

Type IVA Flowing full with submerged outlet (Figure 8-4).

Type IVB Flowing full with partially submerged outlet (Figure 8-5).

Type 1

Culvert Flowing Part Full

With Outlet Control and Tailwater Depth

Below Critical Depth

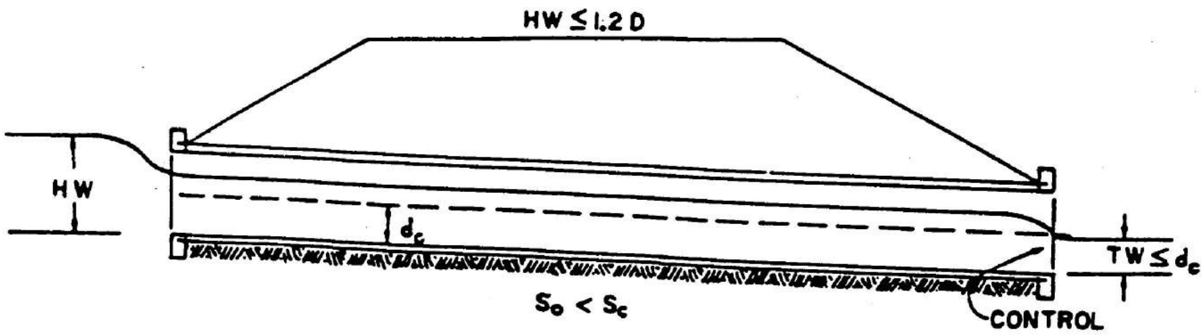

Figure 8-1

Conditions

The entrance is unsubmerged (HW < 1.2D), the slope at design discharge is sub-critical ($S_o < S_c$), and the tailwater is below critical depth (TW < d_c).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat flood plains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet the culvert is said to have "Outlet Control." A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D or entrance is submerged and Type 4 operation will result.

d_c = Critical depth of flow in feet, refer to nomograph

D = Diameter of pipe or height of box.

q = Discharge in cfs per foot.

V_c = Critical velocity in feet per second occurring at critical depth.

h_e = Entrance head loss in feet.

$$h_e = K_e \frac{v_c^2}{2g}$$

K_e = Entrance loss coefficient (See Table C-6).

h_f = Friction head loss in feet = S_fL.

S_f = Friction slope or slope that will produce uniform flow. For Type I operation the friction slope is based upon 1.1 d_c (See Figures C-16 and C-22)

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

Type II

Culvert Flowing Part Full

With Outlet Control And Tailwater Depth

Above Critical Depth

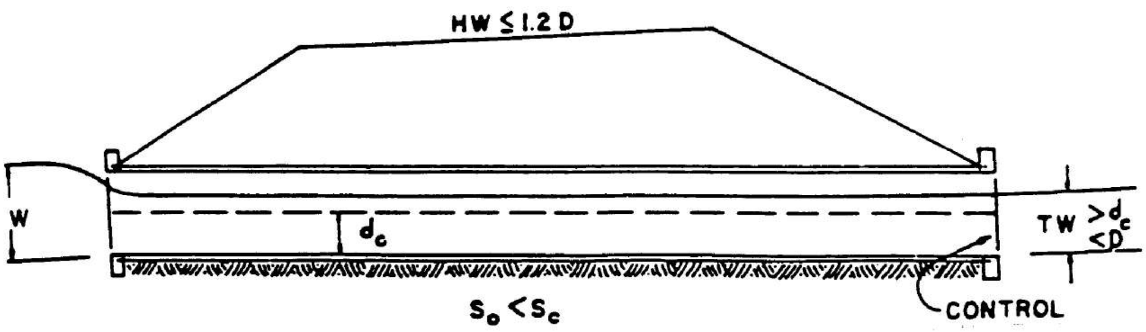

Figure 8-2

Conditions

The entrance is unsubmerged (HW < 1.2 D), the slope at design discharge is subcritical ($S_o < S_c$), and the tailwater is above critical depth (TW > dc).

The above condition is a common occurrence where the channel is deep, narrow and well defined.

If the headwater pool elevation does not submerge the culvert inlet, the slope at design discharge is subcritical, and the tailwater depth is above critical depth the control is said to occur at the outlet; and the capacity of the culvert shall be governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

TW = Tailwater depth above the invert of the downstream end of the culvert in feet.

V_{TW} = Culvert discharge velocity in feet per second at tailwater depth.

he = Entrance head loss in feet.

$$h_e = K_e \frac{V_{Tc}^2}{2g}$$

K_e = Entrance loss coefficient (See TAble C-6).

h_f = Friction head loss in feet = S_fL

S_f = Friction slope or slope that will produce uniform flow. For Type II operation the friction slope is based upon TW depth.

S_o = Slope culvert in feet per foot.

L = Length of culvert in feet.

Type III

Culvert Flowing Part Full With Inlet Control

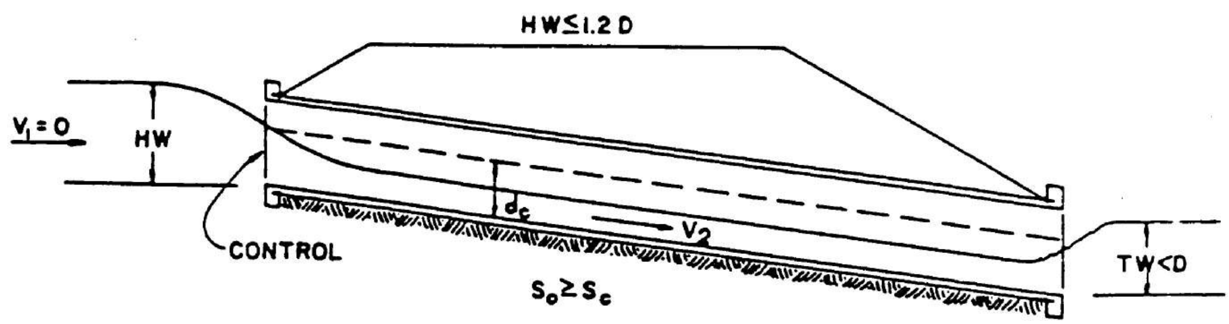

Figure 8-3

Conditions

The entrance is unsubmerged ($HW < 1.2D$) and the slope at design discharge is equal to or greater than critical (Supercritical) ($S_o > S_c$).

The condition is a common occurrence for culverts in rolling or mountainous country where the flow does not submerge the entrance. The control is critical depth at the entrance.

If critical flow occurs near the inlet, the culvert is said to have "Inlet Control". The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + K_e \frac{(V_2)^2}{2g}$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = Critical depth of flow in feet,

q = Discharge in cfs per foot.

V_2 = Velocity of flow in the culvert in feet per second.

The velocity of flow varies from critical velocity at the entrance to uniform velocity at the outlet provided the culvert is sufficiently long. Therefore, the outlet velocity is the discharge divided by the area of flow in the culvert.

K_e = Entrance loss coefficient (See Table C-6).

Type IV-A

Culvert Flowing Full With Submerged Outlet

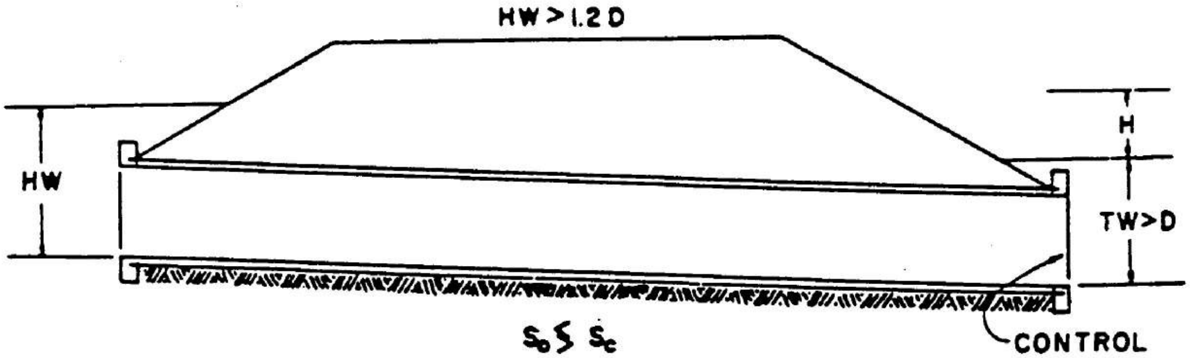

Figure 8-4

Conditions

(Submerged Outlet)

The entrance is submerged (HW > 1.2D). The tailwater completely submerges the outlet.

Most culverts flow with free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installation. Generally, these will be considered at the outlet. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe of height of box. The capacity of a culvert flowing full with a submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet Velocity is based on full flow at the outlet.

$HW = H + TW - S_oL$

HW = Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = Head for culvert flowing full.

TW = Tailwater depth in feet.

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

Type IV-B

Culvert Flowing Full
With Partially Submerged Outlet

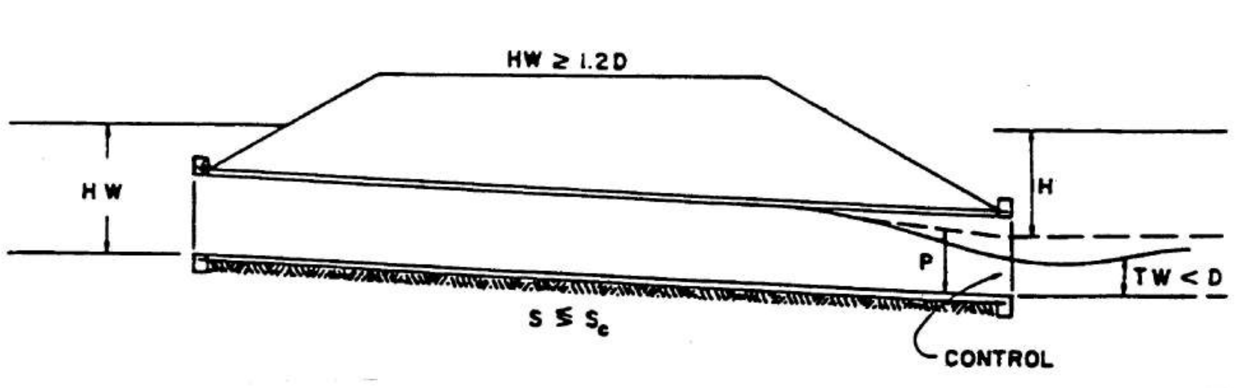

Figure 8-5

Conditions

(Partially Submerged Outlet)

The entrance is submerged (HW > 1.2D). The tailwater depth is less than D (TW < D).

The capacity of a culvert flowing full with a partially submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$HW = H + P - S_o L$

HW = Headwater Depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = Head for culverts flowing full.

P = Pressure line height = $\frac{d_c + D}{2}$

d_c = Critical depth in feet.

D = Diameter or height of structure in feet.

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.