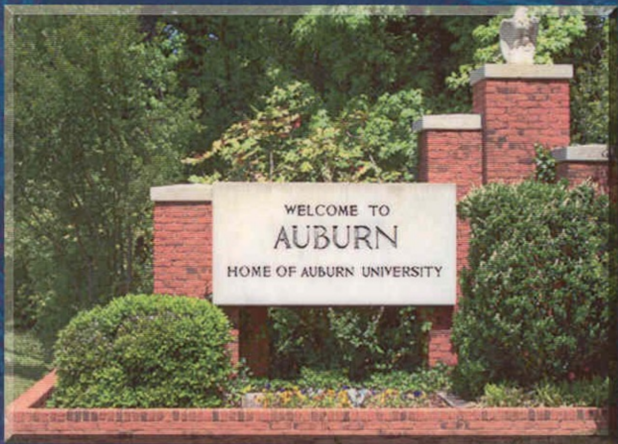
City of Auburn Storm Water Management Manual



Prepared for

City of Auburn

Prepared by

CH2MHILL

Montgomery, Alabama

April 2003
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City of Auburn Storm Water Management Manual

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Contents

Appendix A

Tables

Figures

Acronyms

ADEM	Alabama Department of Environmental Management
AHW	Allowable headwater
ALDOT	Alabama Department of Transportation
cfs	Cubic feet per second
CMP	Corrugated metal pipe
CN	Curve number
DOT	U.S. Department of Transportation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
fps	Feet per second
ft	Feet
ft/ft	Feet per foot
ft/sec	Feet per second
ft/sec^2	Feet per second squared
ft^2	Square feet
ft^3	Cubic feet
HDPE	High-density polyethylene
HDS	Hydraulic Design Services
HEC	Hydraulic Engineering Circular
HW	Headwater
L/W	Length and width
mg	Million gallons
mm	Millimeter
msl	mean sea level
NGVD	National geodetic vertical datum
NRCS	Natural Resource Conservation Service
SCS	Soil Conservation Service (former name of NRCS)
SWMM	Storm water management manual

Definitions

Abstractions	The portion of a storm's total precipitation that does not become storm water runoff.
Access Spacing	The point in the pipeline where access is available from the surface, like a manhole or inlet.
Allowable Headwater Depth	Maximum depth of flow allowed at any point along the ditch profile, measured from the invert, minus the required freeboard.
Antecedent Soil Moisture Conditions	Soil moisture at the onset of a rainfall event.
Apron	A platform below a storm drain outlet to protect against erosion.
Backwater	Water backed up or retarded in its course, compared with its normal or natural condition of flow. In stream gaging, a rise in stage produced by a temporary obstruction such as ice or weeds, or by the flooding of the stream below. The difference between the observed stage and that indicated by the stage-discharge relation is reported as backwater.
Baffle Wall	A flat board or plate, deflector, guide, or similar device constructed or placed in flowing water or storm water storage systems to cause more uniform flow velocities, and to divert or guide liquids.
Bank Storage	The water absorbed into the banks of a stream, lake, or reservoir, when the stage rises above the water table in the bank formations, then returns to the channel as effluent seepage when the stage falls below the water table. Bank storage may be returned in whole or in part as seepage back to the water body when the level of the surface water returns to a lower level.
Base Flood Elevation	The height of the base flood, usually in feet, in relation to the National Geodetic Vertical Datum of 1929, the North American Vertical Datum of 1988, or other datum, or depth of the base flood, usually in feet, above the ground surface.
Berm	1) A narrow ledge or path as at the top or bottom of a slope or stream bank. 2) A horizontal step or bench in the upstream or downstream face of an embankment dam.

"C" Factor	The runoff coefficient C is a critical element in that it serves the function of converting the average rainfall rate of a particular recurrence interval to the peak runoff intensity of the same frequency. The magnitude will be affected by antecedent moisture condition, ground slope, ground cover, depression storage, soil moisture, shape of drainage area, overland flow velocity, intensity of rain, etc. If the area contains multiple types of surfaces, a composite coefficient is determined by estimating the fraction of each type of surface within the total area, multiplying each fraction by the appropriate coefficient for that type of surface, and then summing up the product for all types of surfaces.
Channel Capacity	The maximum rate of flow that may occur in a stream without causing overbank flooding; the maximum flow that can pass through a channel without overflowing the banks.
Channel Storage Volume	The volume of water at a given time in the channel or over the floodplain of the streams in a drainage basin or river reach. Channel storage is sometimes significant during the progress of a flood event.
Constructed Wetland	Wetlands constructed specifically for the purpose of treating wastewater effluent before re-entering a stream or other body of water or being allowed to percolate into the groundwater.
Control Elevation	A location in the receiving drainage system where the water surface elevation is known.
Crest	The highest elevation reached by flood waters flowing in a channel, as in crest stage or flood stage.
Critical Depth	The depth of water flowing in an open channel or conduit under conditions of critical flow at which specific energy is a minimum for a given discharge.

"Critical Flow"	Critical flow occurs when the flow velocity in a channel equals the wave velocity generated by a disturbance or obstruction. In this condition, the Froude number (Fr) = 1. When the wave velocity exceeds the flow velocity (Fr is less than 1), waves can flow upstream, water can pond behind an obstruction, and the flow is said to be subcritical or tranquil. When Fr is greater than 1, waves cannot be generated upstream and the flow is said to be supercritical, rapid, or shooting. In this condition, a standing wave is formed over obstructions in the river bed. In nature, supercritical flow is found only in rapids and waterfalls, but it is often created artificially by weirs and flumes with the aim of measuring discharge.
Crown	The vertex of an arch or arched surface. Center of roadway elevated above the sides.
Curve Number (CN)	A number between 0 and 100 that indicates the runoff-producing potential of a soil/vegetation combination when the ground is not frozen.
Deflectors	A plate, baffle, or the like that diverts the flow of a forward-moving stream.
Design Flood Conditions	The flood magnitude selected for use as a criterion in designing flood control works. The largest flood that a given project is designed to pass safely. In dam design and construction, the reservoir inflow-outflow hydrograph used to estimate the spillway discharge capacity requirement and corresponding maximum surcharge elevation in the reservoir.
Design Storm	The rainfall or precipitation amount and distribution adopted over a given drainage area, used in determining the Design Flood.
Design Storm Flows	A storm whose magnitude, rate, and intensity do not exceed the design load for a storm drainage system or flood protection project.
Detention Area	An area for the slowing and storage of storm water runoff.
Detention Basin (Pond)	A relatively small storage lagoon for slowing storm water runoff, generally filled with water for only a short period of time after a heavy rainfall.
Dry Detention Basin (Pond)	Offers temporary storage accompanied by the controlled release of stored water.

Energy Dissipation	Any loss of energy due to change in flow paths, generally by conversion into heat; quantitatively, the rate at which this loss occurs.
Erosion	The process by which rain, running water, waves, moving ice, and wind dislodge the upper layers of soil. As usually employed, the term includes weathering, solution, corrosion, and transportation.
Excess Precipitation	That portion of total precipitation that becomes storm water runoff during a storm event.
Extended Wet Detention Basin	Combines the treatment concept of the dry detention pond and the wet pond. The treatment volume is divided between the permanent pool and detention storage provided above the permanent pool.
Falling Limb	The portion of the hydrograph immediately following the peak and reflecting the decreasing production of storm flow.
Flood Elevation Profile	The height of flood waters above an elevation datum plane.
Flood Routing	The process of determining progressively downstream the timing and stage of a flood at successive points along a river. Also, the determination of the attenuating effect of storage on a flood passing through a valley, channel, or reservoir.
Forebay	The water behind a dam. A storage basin for regulating water for percolation into groundwater basins.
Freeboard	The vertical distance between a design maximum water level and the top of a structure such as a channel, dike, floodwall, dam, or other control surface. The freeboard is a safety factor intended to accommodate the possible effect of unpredictable obstructions, such as ice accumulations and debris blockage, that could increase stages above the design water surface.
Grade	The slope of a stream bed, road, curb, gutter, etc.
Gravity Flow	The downhill flow of water through a system of pipes, generated by the force of gravity.
Headwater Depth	The water level upstream of a culvert or bridge.

Hydraulic Gradient	The gradient or slope of a water table or piezometric surface in the direction of the greatest slope, generally expressed in feet per mile or feet per feet. Specifically, the change in static head per unit of distance in a given direction, generally the direction of the maximum rate of decrease in head. The difference in hydraulic heads ($h_1 - h_2$), divided by the distance (L) along the flowpath, or expressed in percentage terms: $I = (h_1 - h_2)/L \times 100$.
Hydraulic Radius	The cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter. Also referred to as hydraulic mean depth.
Hydrograph	A graph showing stage, flow, velocity, or other hydraulic properties of water with respect to time for a particular point on a stream.
Hydrologic Soil Group	The classification of soils by their reference to the intake rate of infiltration of water, which is influenced by texture, organic matter content, stability of the soil aggregates, and soil horizon development.
Hyetograph	A chart showing the distribution of rainfall over a particular period of time or a particular geographic area.
Imperviousness	The portion of a subbasin, sub-watershed, or watershed, expressed as a percentage, that is covered by surfaces such as roof tops, parking lots, sidewalks, driveways, streets, and highways. Impervious surfaces are important because they will not absorb rainfall, and therefore, cause almost all of the rainfall to appear as surface runoff.
Infiltration Capacity Rate	The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow.
Interception Infiltration	The process whereby the downward movement of precipitation is interrupted and redistributed.
Interceptor Ditch	A ditch that collects rainfall, allowing it to evaporate without contributing to runoff.
Invert	The floor or bottom of a conduit.
Jet	A forceful stream of fluid discharged from a narrow opening or a nozzle.
Lag Time	The time from the center of mass of the rainfall excess to the runoff hydrograph peak.

Land Use	The primary or primary and secondary uses of land, such as cropland, woodland, pastureland, etc. The description of a particular land use should convey the dominant character of a geographic area, and thereby establish the types of activities that are most appropriate and compatible with primary uses.
Littoral Zone	The region along the shore of a non-flowing body of water, corresponding to riparian for a flowing body of water.
Natural Resource Conservation Service (NRCS)	An agency of the U.S. Department of Agriculture, the Natural Resources Conservation Service (NRCS) was started in 1929 as an emergency act of Congress in response to the famous Dust Bowl when land practices caused extensive soil erosion and threatened the food production of the United States. Currently, the NRCS works in 3 areas: 1) soil and water conservation; 2) resource inventories; and 3) rural community development.
Nomograph	A chart that represents an equation containing three variables by means of three scales so that a straight line cuts the three scales in values of the three variables, thus satisfying the equation.
NRCS Curve Number (CN) Method	Relates soil type, soil cover, land use type, and antecedent moisture conditions to a curve number. Used to determine the depth of runoff for a given area.
Open Channel	A system of conveyance channels in which the top flow boundary is a free surface (e.g., canal systems).
Ordinate	The perpendicular distance of a point (x,y) of the plane from the x-axis.
Orifice	As used in water studies, an opening with a closed perimeter, usually sharp edged, and of regular form in a plate wall or partition through which water may flow. An orifice is used for the measurement or control of water.
Overtopping	To rise above; exceed in height; tower over.
Parapets	A solid wall built along the top of a dam for ornament, for the safety of vehicles and pedestrians, or to prevent overtopping.
Partially Full Subcritical Flow	A flow condition where the velocity is less than the critical velocity and the depth is greater than the critical depth.
Peak Flow	The maximum flow rate of the hydrograph.

Peat Bog	A wet overwhelmingly vegetative substratum that lacks drainage and where humic and other acids give rise to modifications of plant structure and function. Bogs depend primarily on precipitation for their water source, and are usually acidic and rich in plant residue with a conspicuous mat of living green moss. Only a restricted group of plants, mostly mychorrhizal (fungi, heaths, orchids, and saprophytes), can tolerate bog conditions.
Point of Inflection	Assumed to mark on the recession curve of a hydrograph when surface inflow to the channel system ceases.
Ponding	The natural formation of a pond in a stream by an interruption of the normal streamflow.
Principle of Linearity	The property whereby a mathematical system is well behaved (in the context of the given system) with regard to addition and scalar multiplication.
Principle of Superposition	A general principle applying to many physical systems that states that if a number of independent influences act on the system, the resultant influence is the sum of the individual influences acting separately.
Principle of Time Invariance	A system in which all quantities governing the system's behavior remain constant with time, so that the system's response to a given input does not depend on the time it is applied.
Rainfall Excess Hyetograph	A single block of rainfall excess over duration, D.
Rational Method	A simple procedure for calculating the direct precipitation peak runoff from a watershed, using the rainfall intensity, the area of the watershed, and the runoff coefficient appropriate for the type of watershed runoff surface.
Recession Curve	A hydrograph that shows the decreasing rate of runoff following a period of rain or snowmelt. Because direct runoff and base runoff recede at different rates, separate curves, called direct runoff recession curves, are generally drawn.
Retention Pond	A permanent pond used to slow storm water runoff and promote infiltration into the groundwater.

Return Period	In statistical analysis of hydrologic data, based on the assumption that observations are equally spaced in time with the interval between two successive observations as a unit of time, the return period is the reciprocal of 1 minus the probability of a value equal to or less than a certain value; it is the mean number of such time units necessary to obtain a value equal to or greater than a certain value one time. For example, with a 1-year interval between observations, a return period of 100 years means that, on average, an event of this magnitude or greater is not expected to occur more often than once in 100 years.
Riser	A vertical pipe used for drainage.
Rising Limb	The increasing portion of the storm hydrograph. Contrast to Falling Limb.
Scupper	An opening for draining off water, as from a floor or the roof of a building.
Sedimentation	Strictly, the act or process of depositing sediment from suspension in water. Broadly, all the processes whereby particles of rock material are accumulated to form sedimentary deposits. Sedimentation, as commonly used, involves not only aqueous but also glacial, aeolian, and organic agents.
Sheet Flow	An overland flow or downslope movement of water taking the form of a thin continuous film over relatively smooth soil or rock surfaces and not concentrated into channels.
Shoring	Providing temporary support with shores to a building or an excavation.
Siltation	The deposition of finely divided soil and rock particles upon the bottom of stream and river beds and in reservoirs.
Skew Angle	Deviating from rectangularity or a straight line.
Spread	The width of water transported on the pavement measured from the face of the curb.
Storm Water Phase II	The federal regulations requiring smaller communities to address storm water management and requiring coverage by a National Pollutant Discharge Elimination System (NPDES) permit.
Subbasin	A portion of a sub-region or basin drained by a single stream or group of minor streams.
Subdivision	A piece of land resulting from this; especially, a large tract subdivided into small parcels for sale.

Subgrade	The soil or rock leveled off to support the foundation of a structure.
Substrate Surface	Any naturally occurring immersed or submersed solid surface, such as a rock or tree, upon which an organism lives.
Sump	A low-lying place, such as a pit, that receives drainage.
Surface Detention	That part of the rain that remains on the ground surface during rain and either runs off or infiltrates after the rain ends; surface detention does not include Depression Storage.
Surface Storage	The part of precipitation retained temporarily at the ground surface as interception or depression storage so that it does not appear as infiltration or surface runoff either during the rainfall period or shortly thereafter.
Tailwater	Water in a river or channel, immediately downstream from a structure.
Time Base	The total time from when runoff begins to the estimated peak flow rate.
Time Lag	The time it takes a flood wave to move downstream.
Time of Concentration	The time required for water to flow from the hydraulically farthest point on the watershed to the gauging station, culvert, or other point of interest.
Time to Peak	The time from the start of the hydrograph to the peak flow.
Turning Line	A temporary line whose elevation is determined by additions and subtractions of backsights and foresights respectively.
Turnout	A structure that diverts water from a drainageway to a distribution system or delivery point. Turnouts are used at the head of laterals.
Unit Duration	The time over which 1 inch of surface runoff is distributed for unit hydrograph theory.
Unit Hydrograph	A runoff hydrograph that is produced by 1 inch (25.4 millimeters [mm]) of excess rainfall distributed uniformly over a watershed and occurring at a uniform rate during a specified period of time.
Watershed	An area that, because of topographic slope, contributes water to a specified surface water drainage system, such as a stream or river. An area confined by a topographic divide that drains a given stream or river.

Weir	A device for determining the quantity of water flowing over it from measurements of the depth of water over the crest or sill and known dimensions of the device.
Wet Detention Basin (Pond)	Constructed basins that have a permanent pool of water throughout the year or wet season and generally are found in locations where groundwater is high and/or percolation is poor.

Conversion Factors

To convert	Into	Multiply By
Acres	square feet	43,560.0
Acres	square miles	1.562 EE-3
Cubic feet	gallons	7.4805
Cubic feet/sec	gallons/min.	448.831
Days	seconds	86,400.0
Feet	miles (statute)	1.894 EE-4
Gallons	cubic feet	0.1337
Gallons/min.	cubic feet/sec	2.228 EE-3
Miles (statute)	feet	5,280.0
Seconds	days	1.157 EE-5
Square feet	acres	2.296 EE-5
Square miles	acres	640

1. Manual Purpose

1. Manual Purpose

The City of Auburn, Alabama, has been identified by the Alabama Department of Environmental Management (ADEM) as a Storm Water Phase II entity. One requirement of the Phase II program is to develop and implement a storm water management program. The City of Auburn has decided to develop an *Auburn Storm Water Management Manual* (SWMM) as a component of its storm water management program.

The purpose of the SWMM is to provide storm water design information for local agencies, engineers, developers, or others whose activities affect storm water management in the City of Auburn. The manual will serve as a guide for city staff, consultants, and citizens to achieve consistency in the design and compliance of storm water projects so that both growth and environmental guidelines can be followed effectively. This manual has been prepared to document the following items:

- Runoff estimation
- Hydraulic design of storm water conveyance systems
- Hydraulic design of storm water storage systems

This manual should assist all parties involved in development projects to achieve an understanding of the requirements, guidelines, and benefits of a detailed storm water management manual. Incorporating the guidelines contained in this manual into applications and permits will aid in obtaining approval for grading, erosion and sedimentation control, construction, and subdivision development permits from the Public Works Department.

1.1 Related Documents and Guidance

The following documents are incorporated into this manual as references:

- Chapter 7 of the Auburn City Code, *Drainage and Flood Control*
- *City of Auburn 2000 Zoning Ordinance* (most recent version)
- Most recently adopted version of *Subdivision Regulation for Auburn, Alabama*
- Most recently adopted version of *ALOA Erosion and Sediment Control Policy*
- City of Auburn Standard Details and Specifications (most recent version)
- State of Alabama Highway Department–*Hydraulic Manual*

These references are to be used as guides in conjunction with this manual to fulfill the requirements and guidelines set by the City of Auburn, Lee County, and ADEM.

1.2 Use of Manual

As stated, this manual is to be used to streamline the permit application and design process, with a typical permit application review time for the City of about 2 weeks. Other design approaches may be used and submitted with the understanding that the proposed approach may be denied and/or review times prolonged.

1.3 Submittal Requirements

The information listed below represents the level of information that is usually required to evaluate an application. The level of information required for a specific project will vary depending on the nature and location of the site and activity proposed. Providing a greater level of detail will reduce the need to submit additional information at a later date. If an item does not apply to your project, proceed to the next item. Please submit all information on paper no larger than 24 inches x 36 inches.

1.3.1 Environmental Considerations

a. Describe how boundaries of wetlands or other surface waters were determined. If there has ever been a jurisdictional declaratory statement, a formal wetland determination, a formal determination, a validated informal determination, or a revalidated jurisdictional determination, provide the identifying number.

b. Provide a description of how water quantity, quality, hydroperiod, and habitat will be maintained in onsite wetlands and other surface waters that will be preserved or will remain undisturbed.

c. Provide a narrative description of any proposed mitigation plans, including purpose, maintenance, monitoring, construction sequence and techniques, and estimated costs.

d. Provide impact summary tables showing the amount of onsite wetlands, affected wetlands, and mitigation areas.

1.3.2 Plans

Provide a map(s) of the project area and vicinity at a scale of no greater than 1″=400′. Provide clear, detailed plans for the system including plan (overhead) views, cross sections (with the locations of the cross sections shown on the corresponding plan view), and profile (longitudinal) views of the proposed project as required by the City Engineer. If requested by the City Engineer, specifications also will be provided. The plans must be signed and sealed by an appropriate registered professional as required by law. Plans must include a scale and a north arrow. These plans should show the following:

a. Project area boundary and total land area, including distances and orientation from roads or other landmark.

b. Existing land use and land cover (acreage and percentages), and onsite natural communities, including wetlands and other surface waters, aquatic communities, and uplands.

c. The existing topography extending at least 100 feet (ft) off the project area, including adjacent wetlands and other surface waters. All topography shall include the location and a description of known benchmarks, referenced to national geodetic vertical datum (NGVD).

d. If the project is in the known floodplain of a stream or other water course, identify the floodplain boundary and approximate flooding elevation. Identify the 100-year flood elevation and floodplain boundary of any lake, stream, or other watercourse located on or adjacent to the site. Minimum floor elevations must be provided for lots adjacent to floodplains.

e. The boundaries of wetlands and other surface waters within the project area. Distinguish those wetlands and other surface waters that have been delineated by any binding jurisdictional determination.

f. Proposed land use, land cover, and natural communities (acreage and percentages), including wetlands and other surface waters, undisturbed uplands, aquatic communities, impervious surface, and detention ponds.

g. Proposed buffer zones.

h. Pre- and post-development drainage patterns and basin boundaries showing the direction of flows, including any offsite runoff being routed through or around the system; and connections between wetlands and other surface waters.

i. Location of all detention ponds with details of size, side slopes, and designed water depths.

j. Location and details of all water control structures, control elevations, any seasonal water level regulation schedules, and the location and description of benchmarks (minimum of one per structure).

k. Location, dimensions, and elevations of all proposed structures, including utility lines, roads, and buildings.

l. Location, size, and design capacity of the internal storm water conveyance facilities.

m. Rights-of-way and easements for the system, including all onsite and offsite areas to be reserved for storm water management purposes, and rights-of-way and easements for the existing drainage system, if any.

n. Receiving waters or surface water management systems into which storm water runoff from the developed site will be discharged.

o. Location and details of the erosion, sediment, and turbidity control measures to be implemented during each phase of construction and all permanent control measures to be implemented in post-development conditions.

p. Location, grading, design water levels, and planting details of all mitigation areas.

q. Site grading details, including perimeter site grading.

r. Disposal site for any excavated material, including temporary and permanent disposal sites.

s. Dewatering plan details.

t. Location and description of any offsite existing features (such as wetland and other surface waters, detention ponds, and building or other structures) that might be affected by or affect the proposed construction or development.

u. For phased projects, provide a master development plan.

Upon completion of the project, the owner shall submit the final as-built plan and details certified by an appropriate registered professional as detailed in the City's Subdivision Regulations.

1.3.3 Drainage Information

a. The City of Auburn has standard forms to provide design pre-development, and post-development calculations (copies shown in Figure 1-1) that need to be completed, signed, and sealed by an appropriate registered professional, as follows:

   1. Runoff characteristics, including area, runoff curve number or runoff coefficient, and time of concentration for each drainage basin.
   2. Water table elevations (normal and seasonal high), including area extent and magnitude of any proposed water table drawdown.
   3. Receiving water elevations (normal, wet season, and design storm).
   4. Design storms used including rainfall depth, duration, frequency, and distribution.
   5. Runoff hydrograph(s) for each drainage basin, for all required design storm event(s).
   6. Stage-storage computations for any area such as a reservoir, closed basin, detention area, or channel used in storage routing.
   7. Stage-discharge computations for any storage areas at a selected control point, such as control structure or natural restriction. Emergency spillway sizing computations (based on 100-year storm event).
   8. Flood routing through onsite conveyance and storage areas and through the first City structure.
   9. Water surface profiles in the primary drainage system for each required design storm event(s).
   10. Runoff peak rates and volumes discharged from the system for each required design storm event(s).
   11. Tail water history and justification (time and elevation).

b. Provide the results of any percolation tests, where appropriate, and soil borings that are representative of the actual site conditions, if requested by the City Engineer.

c. Provide the acreage, and percentages of the total project, of the following:

   1. Impervious surfaces, excluding wetlands.
   2. Pervious surfaces (green areas not including wetlands).

3. Lakes, retention areas, other open water areas.
4. Wetlands.

d. Provide an engineering analysis of floodplain storage and conveyance (if applicable), including the following:
   1. Hydraulic calculations for all proposed traversing works.
   2. Backwater water surface profiles showing upstream effects of traversing works.
   3. Location and volume of encroachment within regulated floodplain(s).
   4. Plan for compensating floodplain storage, if necessary, and calculations required for determining minimum building and road flood elevations.

e. Provide a description of the engineering methodology, assumptions, and references for the parameters listed above, and a copy of all such computations, engineering plans, and specifications used to analyze the system. If a computer program is used for the analysis, provide the name of the program, description of the program, input and output data, and justification for model selection.

1.3.4 Operation and Maintenance and Legal Documentation

a. Describe the overall maintenance and operation schedule for the proposed system.

b. Identify the entity that will be responsible for operating and maintaining the system in perpetuity.

c. Provide copies of all proposed conservation easements, storm water management system easements, property owner's association documents, and plats for the property containing the proposed system.

d. Provide a copy of the boundary survey and/or legal description and acreage of the total land area of contiguous property owned or controlled by the applicant.

1.4 Example Problem

An example problem was created to act as a supplement to the manual. The example problem has been divided into separate calculations and placed within the manual in the sections dealing with each particular component of the problem. The entire set of calculations have been summarized and are provided in Appendix A. The general overview of the problem is provided below and illustrated in Figure 1-2.

A developer plans on converting a 10-acre parcel in the Auburn area into a subdivision. The parcel, as it stands, is in pasture (good condition), has Soil Type B, and a 1 percent average landscape slope from the northern end of the parcel to the outlet at the southern end–a distance of approximately 1,000 ft.

The developer's site plan for the subdivision is shown in Figure 1-2. Nine acres will be used for housing and roads, while 1 acre will contain a dry detention pond and common area. Housing density will be 4 houses per acre on the 9 acres containing housing. As illustrated, each acre, or subbasin, will be served by an inlet, with the main drainage line running along the road to the dry detention pond. The average subbasin grade toward each inlet will be 1 percent. The pond will discharge (free outlet) into an adjacent creek at the southern end of the property. The pond's bottom elevation should be set at elevation 90 ft[1]. In accordance with the City's storm water management guidelines, the drainage system should be sized to handle the 25-year, 24-hour storm. The dry detention pond must be able to attenuate the 2-, 5-, 10-, and 25-year, 24-hour storm events to pre-development levels. The emergency spillway shall be designed to pass the 100-year, 24-hour storm event as defined in this manual.

[1] Arbitrary reference elevation for illustration purposes.

DESIGN INFORMATION

Date: ________________________________

Project: ________________________________

Project Owner: ________________________________
Submitted by: ________________________________

	Pond #1	Pond #2	Other
Bottom Pond Elevation			
Maximum Pond Elevation during 25-yr Storm			
Top of Pond Embankment			
Pond Volume at Peak Pond Elevation (25-yr storm)			
Description of Outlet			
Outlet diameter/dimensions			
Orifice Invert elevation(s)			
Peak Outlflow 2-yr 5-yr 10-yr 25-yr			
Peak Pond Elevation 2-yr 5-yr 10-yr 25-yr			
Emergency Spillway Elevation			
Stage-Storage Relation *			
Routing*			
100-yr storm provision			
Other			

* Attach additional information if necessary

FIGURE 1-1A
STANDARD SUBMITTAL FORMS
DESIGN INFORMATION FORM
Auburn Storm Water Management Manual

PRE-DEVELOPMENT CONDITIONS

Sub-Basin*: ____________________
Total no. of Basins: ____________________ **Date:** ____________________
Runoff Method:
NRCS ____________________ **Project:** ____________________
Rational ____________________
Project Owner: ____________________
Submitted by: ____________________

	2-year storm 24-hour	5-year storm 24-hour	10-year storm 24-hour	25-year storm 24-hour
Rainfall**				
Area(acres/miles^2)				
Curve Number/ Runoff Coefficient				
Average Slope over Critical path				
Total Length of Flow Path (ft)				
T_c***				
Intensity				
Peak flow (cfs)				
Runoff (inches)				
Street Drainage Checked				
Other				

* Use separate sheet for each sub-basin.
** If storms other than 24-hr event are used, attach pertinent information and justification.
*** Attach all assumptions made in determining these values.

FIGURE 1-1B
STANDARD SUBMITTAL FORMS
PRE-DEVELOPMENT CONDITIONS FORM

POST-DEVELOPMENT CONDITIONS

Sub-Basin*: ______________
Total no. of Basins: ______________ **Date:** ______________
Runoff Method:
NRCS ______________ **Project:** ______________
Rational ______________
Project Owner: ______________
Submitted by: ______________

	2-year storm 24-hour	5-year storm 24-hour	10-year storm 24-hour	25-year storm 24-hour
Rainfall (in)**				
Area(acres/miles^2)				
Curve Number/ Runoff Coefficient				
Average Slope over Critical path				
Total Length of Flow Path (ft)				
T_c***				
Intensity				
Peak flow (cfs)				
Runoff (inches)				
Street Drainage Checked				
Other				

* Use separate sheet for each sub-basin.
** If storms other than 24-hr event are used, attach pertinent information and justification.
*** Attach all assumptions made in determining these values.

FIGURE 1-1C
STANDARD SUBMITTAL FORMS
POST-DEVELOPMENT CONDITIONS FORM
Auburn Storm Water Management Manual

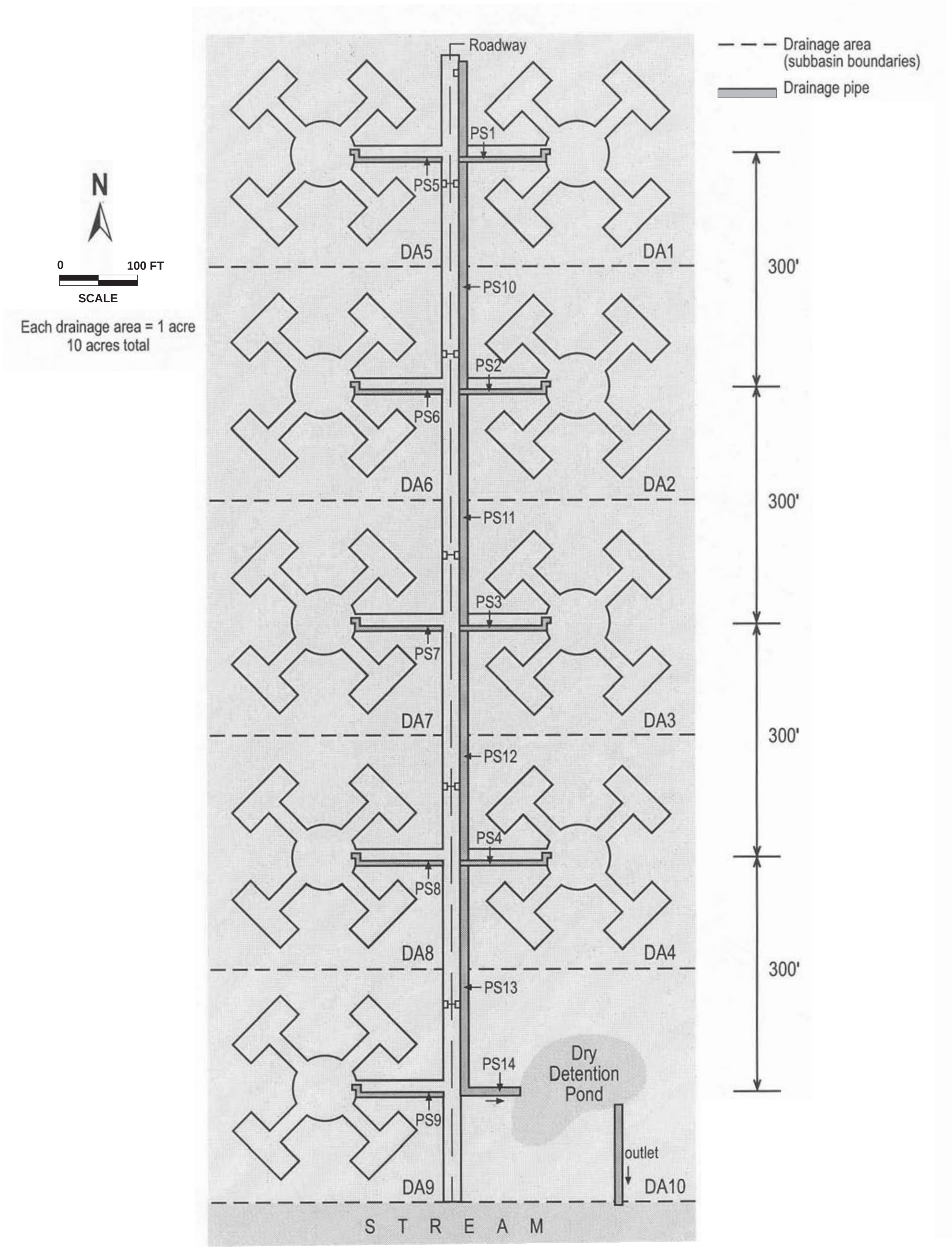


FIGURE 1-2
SW DRAINAGE SYSTEM AND SUBDIVISION LAYOUT
Auburn Storm Water Management Manual

2. Runoff Estimation

2. Runoff Estimation

Foremost in designing storm water conveyance and storage systems is to determine the volume and rate of runoff. The hydraulic design of a storm water conveyance system generally requires an estimate of the peak rate of runoff generated by the design event. In the following subsections, information will be provided to determine design storm information for the Auburn area, to predict excess precipitation, and to determine peak discharge rates.

2.1 Design Storm Information

To determine the volume of runoff to be used for sizing facilities, design storms for the Auburn area must be determined. Design storms represent conservatively high rainfall intensities and volumes that can be expected during a storm event. Table 2-1 displays the rainfall intensities in inches per hour for a given return period and time of concentration for the Auburn area. Table 2-2 represents the depth of rainfall in inches that can be expected over a 24-hour period for a given return period.

In addition to the total storm volume and peak intensities listed above, there is also a need to specify a time distribution of the intensities during the storm event. The City of Auburn uses the Natural Resource Conservation Service (NRCS) Type II dimensionless design storm. This information is to be used when computing runoff hydrographs utilizing computer models. It is widely used and typically is included in these types of models.

2.2 Excess Precipitation Calculation

Excess precipitation is that portion of total precipitation that becomes storm water runoff during a storm event. The portion of a storm's total precipitation that does not become storm water runoff is called abstractions. Abstractions include interception infiltration, surface storage, surface detention, and bank storage. When a storm water system is designed using a single design storm event, initial abstractions need to be considered. Initial abstractions include interception and surface storage before runoff can begin. Once the initial abstractions are placated, then runoff occurs. The depth of runoff during a fixed time interval is the difference between the rainfall and infiltration capacity rates. Initial abstractions are insignificant compared to infiltration losses, and for practical hydrologic design, initial abstractions can be considered negligible.

The infiltration rate is dependant on the physical properties of the soil, vegetative cover, antecedent soil moisture conditions, rainfall intensity, and slope of the infiltrating surface. This combination of factors that affects the infiltration rate for a particular watershed interacts to cause a generally complex spatial distribution of infiltration capacity. The NRCS Curve Number (CN) method may be used to determine the depth of runoff for a given area.

2.2.1 NRCS Curve Number

The NRCS CN method relates soil type, soil cover, land use type, and antecedent moisture conditions to a CN. Excess rainfall calculations are based on the CN and the event precipitation. The runoff volume is estimated by the following equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad \text{(Equation 2-1)}$$

where:

Q = accumulated rainfall excess or runoff, inches

P = accumulated rainfall, inches (refer to Table 2-2)

S = maximum watershed rainfall retention factor

The rainfall retention factor, S, includes the initial abstraction and infiltrated volume and is related to the watershed CN by the following relationship:

$$S = \frac{1{,}000}{CN} - 10 \qquad \text{(Equation 2-2)}$$

The watershed CN is a dimensionless coefficient that reflects watershed cover conditions, hydrologic soil group, land use, and antecedent moisture conditions.

CN values for urban areas are provided in Table 2-3. Nearly all soils in Lee County are B-Type soils (*Soil Survey of Lee County*). B soils have fine to coarse texture moderately suited for infiltration and drainage. These values are to be used unless the applicant can prove to the City that the soils are different.

Example 2.1: Pre-development Volume and Peak Discharge Determination

Use the NRCS CN Method (Section 2.2.1) to determine the depth of runoff over the parcel. Table 2-3 indicates that pasture in good condition has a CN of 60. Using Equation 2-2:

S = (1,000/60) –10 = 6.7

Using Equation 2-1 and the Design Storm Volumes from Table 2-2, the depth of runoff in inches can be calculated for each storm event. Multiply by the acreage to obtain the volume of runoff. For example:

From Table 2-2 for a 2-year storm event, P = 4.2 inches.

Q= [4.2 – (0.2 * 6.7)]^2/[4.2 + (0.8)6.7] = (2.86)^2/9.56 = 0.86 inches

Volume of Runoff = 0.86 inches * (ft/12 inches) * 10 acres * (43,560 ft^2/acre) = 31,100 ft^3

Volume of Runoff = 31,100 ft^3 * (7.4805 gallons/ft^3) * (mg/1,000,000 gallons) = 0.232 mg

The calculations for each storm frequency are summarized in the following chart:

Storm	P (inches)	Weighted CN	S (inches)	Q (inches)	Acreage (acres)	Volume of Runoff	
						ft^3	mg
2-yr	4.2	60	6.7	0.86	10.0	31,300	0.234
5-yr	5.4	60	6.7	1.54	10.0	55,900	0.418
10-yr	6.3	60	6.7	2.12	10.0	77,000	0.653
25-yr	7.2	60	6.7	2.75	10.0	99,700	0.746

Notes:
Runoff inches / 12 x Acreage x 43,560 ft^2/ac = Volume ft^3
ft^3 x 7.4805 gallons/ft^3 x $1/10^6$ = Volume mg
CN = Curve number
mg = Million gallons
ft^3 = Cubic feet

2.3 Storm Water Runoff Determination

This subsection deals with estimating the peak storm water runoff rate that will flow from the watershed. An important step in determining the peak discharge is to also determine the distribution of all flows during the design storm, which will be discussed, as well.

2.3.1 Time of Concentration

The time of concentration, t_C, is the time required for water to travel from the most hydraulically distant point in the watershed to the watershed outlet. The TR-55 method for determining time of concentration will be used in this manual, with the following equation for time of concentration:

$$t_c = \frac{D}{3600 \times V} \qquad \text{(Equation 2-3)}$$

where:

t_c = time of concentration, minutes

D = distance runoff travels in watershed, ft

V = average runoff velocity, feet per second (ft/sec)

The average velocities can be computed using the following equations:

$$V_{up} = 16.1345(s)^{0.5} \qquad \text{(Equation 2-4)}$$

$$V_p = 20.3282(s)^{0.5} \qquad \text{(Equation 2-5)}$$

where:

V_{up} = average velocity for unpaved areas or channel flow, ft/sec

V_p = average velocity for paved areas, ft/sec

s = slope of hydraulic grade line, feet per foot (ft/ft). Assume this equals the landscape slope.

Figure 2-1 is a worksheet for determining the time of concentration that can be calculated for a watershed using the TR-55 method. The TR-55 method is commonly used to predict peak runoff for small watershed areas. This form also includes a section for computing the travel time for larger channels. If needed, the equations in Figure 2-1 can be applied to estimate channel velocity and the subsequent travel time. In general, larger channels will be modeled separately and would not be included in the subbasin drainage.

To accurately determine the time of concentration, the flow path should be segmented as appropriate into sheet or overland flow (most recently determined not to exceed 100 ft, as documented in the new release of TR-55), shallow concentrated flow, and channelized flow. The time of concentration in each segment in the watershed should be calculated and then added together to get the total time of concentration. Pre- and post-development time of concentrations may differ significantly depending on the extent of development. In most cases, increased imperviousness and more direct flow routes decrease the time of concentration.

$$t_c = t_{c1} + t_{c2} + t_{c3} + \ldots\ldots t_{cn} \qquad \text{(Equation 2-6)}$$

where:

t_{cn} = time of concentration for section n

2.3.2 Peak Runoff Rate

The preferred method for determining the peak runoff rate in urban settings is the Rational Method. However, use of the method will be limited to small contributing areas and t_c values of 15 minutes and less. Otherwise, the TR-55 Method will be used. Refer to the flow chart in Figure 2-2 to determine which method to use. The well-known formula for the Rational Method is:

$$Q_T = CI_T\ A \qquad \text{(Equation 2-7)}$$

where:

Q_T = peak runoff rate, cubic feet per second (cfs), for the design storm return period, T

C = rational method runoff coefficient expressed as a dimensionless ratio (Table 2-4)

I_T = average rainfall intensity, inches/hour (from Table 2-1), during a period of time equal to t_c for the design storm return period, T.

A = site drainage area, acres, tributary to the design point

t_c = watershed time of concentration, minutes, defined in Section 2.3.1.

The basic assumptions for application of the Rational Method include the following:

1. Runoff is linearly related to rainfall.
2. The rainfall occurs uniformly over a given site.
3. The peak runoff rate occurs when the entire area is contributing flow.
4. The excess rainfall hyetograph is one of constant intensity for duration equal to t_C.
5. The frequency of the peak runoff rate is the same as the frequency of the average rainfall intensity.

The following procedure is recommended for using the Rational Method:

1. Collect site data.
2. Calculate time of concentration using information in Section 2.2.1. If the calculated t_c is less than 5 minutes, the site will be assigned a t_c value of 5 minutes.
3. Determine the average rainfall intensity for the return period from Table 2-1 and t_c from Step 2.
4. Obtain a runoff coefficient for the land use, using the information provided in Table 2-4.
5. Compute the peak runoff rate for the return period using Equation 2-7.

Example 2.2: Determining Rate Discharge

Use the worksheet in Figure 2-1 to determine the time of concentration (t_c). The first 100 ft of flow will be considered sheet flow; thereafter, it will be considered shallow concentrated flow. The Manning's roughness coefficient for pasture is 0.15. The 2-year, 24-hour rainfall is taken from Table 2-2. The slope is given as 0.01 ft/ft. From Figure 2-1, the t_c is 20.0 minutes (0.334 hours).

$$t_c(\text{sheet flow}) = \frac{0.007(0.15*100)^{0.8}}{4.2^{0.5}*0.01^{0.4}} = 0.18 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{shallow concentrated flow})} = \frac{900}{3600 \times 1.613} = 0.154 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{total})} = (0.18 + 0.154) * 60 \frac{\text{min}}{hr} \approx 20 \text{ min}$$

where V is calculated from

$$V_{up} = 16.1345(0.01)^{0.5} = 1.613 \text{ ft/sec} \qquad \text{(from Equation 2-4)}$$

This result exceeds the criteria for the use of the Rational Method (20 minutes is more than the maximum of 15 minutes) to determine peak discharge. Therefore, use the TR-55 model to estimate the pre-development discharge rate (public domain software from NRCS).

TR-55 Input file:

Name	Description	Reach	Area(ac)	RCN	T_c
sub-div	pre-development	Outlet	10	60	0.334

Total area: 10 (ac)

TR-55 Output File:

Auburn SW Example
Lee County, Alabama
Hydrograph Peak/Peak Time Table
Peak Flow and Peak Time (hr) by Rainfall Return Period

2-Yr	5-Yr	10-Yr	25-Yr
(cfs)	(cfs)	(cfs)	(cfs)
7.82	15.52	22.13	29.10

Example 2.3: Determine Post-development Drainage Characteristics of Each Subbasin

It is presumed that the post-development peak flow rate will exceed the pre-development rate, so proceed with estimating the data required for the flow rates for designing the drainage facilities and detention pond.

Determine the time of concentration for the nine subbasins containing four houses per acre (or each house sits on approximately ¼ acre). The maximum hydraulic distance is interpreted from the site plan to be 210 ft for each cul-de-sac. The first 100 ft will be sheet flow, followed by shallow concentrated flow as the runoff enters the street/gutters and flows toward the inlets. The slope is given as 0.01 ft/ft.

$$t_{c(\text{sheet flow})} = \frac{0.007(0.15*100)^{0.8}}{4.20^{0.5}*0.010^{0.4}} = 0.18 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{shallow concentration flow})} = \frac{110}{3600 \times 2.032} = 0.015 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{total})} = (0.18 + 0.015) * 60 \frac{\text{min}}{hr} \approx 11 \text{ min}$$

where V is calculated from

$$V_{up} = 20.3282(0.01)^{0.5} = 2.032 \text{ft/sec} \qquad \text{(from Equation 2-4)}$$

Using Figure 2-1, the t_c for each subbasin is approximately 11 minutes. This result indicates that the Rational Method can be used to determine the peak discharge to each inlet. Using Equation 2-7, C is determined from Table 2-4 as 0.60 for a ¼-acre residential lot. Storm drain inlets are designed for a 25-year storm event, so the I_T for a 25-year storm with an 11 min t_c is interpolated from Table 2-1 as 7.18 inches/hour.

Q_T = 0.60 * 7.18 inches/hr * 1 acre = 4.3 cfs (from Equation 2-7)

The peak discharge for the 25-year storm for each cul-de-sac is 4.3 cfs.

Subbasin 5, containing the dry pond and nature area, is assigned a t_c of 5 minutes according to the minimum requirements. From Table 2-4, C is 0.35 for open space. From Table 2-1, the I_T for a 25-year storm event with a 5-minute t_c is interpolated as 8.7 inches/hour.

Q_T = 0.35 * 8.7 inches/hr * 1 acre = 3.04 cfs (from Equation 2-7)

Using the Rational Method, the peak discharge for the dry detention basin is 3.04 cfs.

2.3.3 Runoff Hydrographs

A runoff hydrograph is a continuous plot of the surface runoff flow rate versus time. Although historical stream flow data are often studied, they are not generally used for designing facilities in a watershed. Synthetic methods for developing runoff hydrographs from design storms are the standard of practice for designing storm water facilities.

A typical hydrograph resulting from an isolated period of rainfall has the following major components, as illustrated in Figure 2-3[1]:

1. Rising limb
2. Crest segment
3. Falling limb or recession curve

The shape of the rising limb is influenced primarily by the characteristics of the storm that produced the surface runoff. The point of inflection on the recession curve of the hydrograph is commonly assumed to mark when surface inflow to the channel system ceases. Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession curve is largely independent of the storm and is influenced instead by watershed characteristics, such as channel slope and storage availability.

The hydrograph terminology used throughout this manual is presented in Figure 2-3, along with appropriate NRCS hydrograph equations. A rainfall excess hyetograph, which in this case is a single block of rainfall excess over duration, D, is shown in the upper part of Figure 2-3. The runoff hydrograph is presented directly below the rainfall excess hyetograph.

The area enclosed by the hyetograph and by the runoff hydrograph represents the same volume, Q, of direct runoff. The maximum flow rate of the hydrograph is the peak flow, q_p, while the time from the start of the hydrograph to q_p is the time to peak, t_p. The total time duration of the hydrograph is known as the time base, t_b. The watershed lag time, t_L, is defined as the time from the center of mass of the rainfall excess to the runoff hydrograph peak.

2.3.3.1 Unit Hydrograph Theory

A unit hydrograph is defined as a runoff hydrograph that is produced by 1 inch (25.4 millimeters [mm]) of excess rainfall distributed uniformly over a watershed and

[1] SCS is the U.S. Department of Agriculture Soil Conservation Service, which is now called the NRCS.

occurring at a uniform rate during a specified period of time. The following assumptions constitute the basis of unit hydrograph theory:

1. The excess rainfall is uniformly distributed within its unit duration or specified period of time.
2. The excess rainfall is uniformly distributed in space over a particular drainage area (or subarea).
3. The time base of a direct runoff hydrograph due to an excess rainfall of unit duration for a given drainage area is constant.
4. The ordinates of the direct runoff hydrographs, when a common base time is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (i.e., principle of linearity or superposition).
5. For a given drainage basin, the hydrograph from events of a given duration and volume of excess rainfall is invariable (i.e., principle of time invariance).

The above assumptions cannot precisely apply to natural rainfall and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results that are sufficiently accurate for most practical problems in storm water management.

Two fundamental assumptions that must be kept in mind when applying unit hydrograph theory are the principle of linearity (Assumption 4) and the principle of time invariance (Assumption 5). Theoretically, each increment of excess rainfall can be routed through the subject watershed in accordance with the principle of linearity. In practice, however, linearity means that the product of an excess rainfall volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cfs per inch of excess rainfall) produce an estimate of the runoff hydrograph for that volume of excess rainfall. In addition, the principle of linearity allows individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (e.g., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) to be superimposed and added when estimating a total runoff hydrograph. The principle of time invariance requires that the condition of the drainage basin be fixed or specified for a particular unit hydrograph. Temporary effects during construction for land development and channel improvements are typical activities that violate the principle of time invariance. However, over time, the landscape does stabilize enough to approach steady conditions.

2.3.3.2 Synthetic Unit Hydrographs

A unit hydrograph for an ungauged urban stream can be estimated if the information derived from the analysis of gauging records is related to measurable watershed characteristics such as channel slope, channel length, and drainage area. The NRCS has a standardized approach to modifying a dimensionless unit hydrograph. The procedure generally requires that the unit hydrograph time parameters, peak flow, and shape or time distribution be estimated.

The NRCS dimensionless unit hydrographs were derived from a large number of observed unit hydrographs for watersheds varying widely in size and geographic locations. Two types of dimensionless unit hydrographs have been developed by the NRCS; the first has a curvilinear shape and the second is a triangular approximation to that curvilinear shape (Figure 2-4). The City of Auburn requires the curvilinear shape to be used.

Once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of that unit hydrograph can be estimated using the appropriate dimensionless unit hydrograph ratios and mass data, as shown in Figure 2-5 and listed in Table 2-5.

The procedure for using the NRCS curvilinear dimensionless unit hydrograph is as follows:

1. Estimate the time of concentration, t_c (Section 2.3.1).
2. Calculate the incremental duration of runoff production rainfall, DD, using the equation:

$$DD = 0.133\ t_c \qquad \text{(Equation 2-8)}$$

where:

DD = Incremental duration of runoff producing rainfall, minutes
t_c = Time of concentration, minutes

3. Calculate the time to peak, t_p, using the equation:

$$t_p = \frac{\Delta D}{2} + 0.6 t_c \qquad \text{(Equation 2-9)}$$

where:

t_p = Time to peak, minutes

4. Calculate the peak flow rate of the unit hydrograph, q_p, as follows:

$$q_p = 60 \frac{BA}{t_p} \qquad \text{(Equation 2-10)}$$

where:

q_p = Peak flow rate, cfs

B = Hydrograph shape factor, ranging from 300 for flat swampy areas to 600 in steep terrain. **Use NRCS standard B value = 484.**

A = Drainage area, square miles

5. List the hydrograph time, t, increments of ΔD and calculate t/t_p.
6. Using Table 2-5 or Figure 2-5, find the q/q_p ratio for the appropriate t/t_p ratios calculated in Step 5.

7. Calculate the appropriate unit hydrograph ordinates by multiplying the q/q_P ratios by q_P.
8. Determine the volume under the unit hydrograph to ensure that it is equal to 1 inch. If not, then adjust one or more increments so the volume is equal to 1 inch.

2.3.3.3 Unit Hydrograph Procedure

A runoff hydrograph is developed through the following steps using the unit hydrograph theory:

1. Develop an excess rainfall hyetograph using the NRCS CN and rainfall time distribution.
2. Route the excess rainfall hyetograph through the subject watershed by multiplying the ordinates of the unit hydrograph by the respective excess rainfall increments. Thus, each increment of excess rainfall produces an incremental hydrograph. Each routed incremental hydrograph begins at the same time interval as the rainfall increment.
3. Develop the composite synthetic runoff hydrograph by summing the ordinates of each routed incremental hydrograph from Step 2, at each time interval of the hydrograph.
4. Check to ensure that the volume of the synthetic runoff hydrograph is equal to the volume of excess rainfall.

$$V = \frac{12\Delta t \sum q_i}{A(43{,}560)} \qquad \text{(Equation 2-11)}$$

where:

V = volume under the hydrograph, inches

Dt = time increment of the runoff hydrograph ordinates, seconds

$\sum q_i$ = sum of the runoff hydrograph ordinates, cfs, for each time increment, i

A = drainage area, acres

2.3.4 Routing of Design Storm

The “routing” of a design storm entails predicting how runoff from a subbasin is attenuated as it moves from the subbasin to a designated point downstream in the watershed. Having developed a synthetic runoff hydrograph for a particular watershed, it may be necessary to route that hydrograph to another point in the drainage system without adding additional flow. This process is generally known as flood routing. Two categories of flood routing techniques are available to quantify the peak flow attenuation and the time lag that is likely to occur as this runoff hydrograph travels through a drainage system. These two categories of flood routing techniques are as follows:

1. Hydrologic routing techniques
2. Hydraulic routing techniques

Hydrologic routing considers only the conservation of mass, whereas hydraulic routing considers both the conservation of mass and the equations of motion. In practice, hydrologic routing techniques usually are adequate for storm water design purposes. The scope of this section will be limited to hydrologic routing techniques, because hydraulic techniques generally require a sophisticated computer solution beyond the scope of this guidance manual.

Hydrologic or hydraulic flood routing techniques may be further categorized depending on the type of drainage system being designed. The two categories of drainage systems that require unique flood routing techniques are 1) channel systems, and 2) reservoir systems.

This section is limited to channel drainage systems and hydrologic routing techniques. The Muskingum Method of hydrologic channel routing is recommended when computer-based procedures are not used. Pipe flow computations are discussed in Section 3.

2.3.4.1 Muskingham Method

The Muskingham Method is applied with the following steps:

1. Select a representative flow rate for evaluating the parameters K and X. Use 75 percent of the inflow hydrograph peak. If this flow exceeds the channel capacity, use the channel capacity as representative.

2. Find the velocity of a small kinematic wave in the channel using the equation:

$$v = \frac{1}{B}(\frac{Q(Y + \Delta Y) - Q(Y)}{\Delta Y} \qquad \text{(Equation 2-12)}$$

where:

v = Velocity of a small kinematic wave, ft/sec

Q(Y) = A representative flow rate for channel routing at representative depth Y, cfs

ΔY = A small increase in the representative depth of flow in the channel

Q(Y + ΔY) = Flow rate at the new depth Y + ΔY, cfs

B = Top of bank width of water surface, ft

3. Estimate the minimum channel length allowable for the routing using the following equation, and make sure that ΔL is greater than ΔL_{min}:

$$\Delta L_{min} = \frac{Q}{BS_o v} \qquad \text{(Equation 2-13)}$$

where:

ΔL_{min} = Minimum channel length for routing calculations, ft

Q = Flow rate, cfs

S_o = Slope of channel bottom, ft/ft

4. Estimate a value of K using the following equation (make sure that K is less than the time of rise for the inflow hydrograph, t_p):

$$K = \frac{\Delta L}{v} \qquad \text{(Equation 2-14)}$$

where:

K = Muskingum channel routing time constant for a particular channel segment

ΔL = Channel routing segment length, ft

5. Estimate the value of X using the equation:

$$X = 0.3\left(1 - \frac{Q}{BS_o v \Delta L}\right) \qquad \text{(Equation 2-15)}$$

where:

X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume

6. Select a reasonable channel routing time period, Δt, using the criteria expressed by the following inequality:

$$\frac{K}{3} \le \Delta t \le K \qquad \text{(Equation 2-16)}$$

7. Determine coefficients C_0, C_1, and C_2 using the following equations (make sure that $C_0 + C_1 + C_2 = 1.0$):

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{(Equation 2-17)}$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{(Equation 2-18)}$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{(Equation 2-19)}$$

where:

Δt = Routing time period, hours

8. Determine an initial outflow, O_1, then calculate an ending outflow, O_2, using the equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad \text{(Equation 2-20)}$$

where:

O_2 = Outflow rate at the end of routing time period Δt, cfs

I_2 = Inflow rate at the end of routing time period Δt, cfs

I_1 = Inflow rate at the beginning of routing time period Δt, cfs

O_1 = Outflow rate at the beginning of routing time period Δt, cfs

The routing is then performed by repetitively solving Equation 2-20, assigning the current value of O_2 to O_1, and determining a new value of O_2. This sequence of calculations continues until the entire inflow hydrograph is routed through the channel. Although this could be done with a spreadsheet, there are computer models available to complete this similarly.

Rainfall Intensity for Auburn, Alabama (in/hr)

Return Period (yr)	Time of Concentration (min)										
	5	6	7	8	9	10	11	12	13	14	15
2	6.2	5.9	5.8	5.6	5.4	5.2	5.1	4.9	4.7	4.5	4.5
5	7.1	6.8	6.6	6.5	6.3	6.0	5.9	5.7	5.5	5.3	5.1
10	7.7	7.5	7.3	7.1	6.9	6.5	6.4	6.3	6.1	5.8	5.6
25	8.7	8.5	8.2	8.0	7.8	7.4	7.3	7.1	6.8	6.6	6.3
50	9.4	9.1	8.9	8.6	8.4	8.1	7.9	7.7	7.4	7.2	6.9
100	10.2	9.8	9.6	9.3	9.0	8.8	8.5	8.2	8.0	7.7	7.6

Each value is the corresponding intensity in/h.

Design Storm Volume for Auburn, Alabama

Return Period (yrs)	24-Hour Rainfall (in.) (P)
2	4.2
5	5.4
10	6.3
25	7.2
50	8.1
100	9.0

TABLES 2-1 & 2-2
RAINFALL INTENSITY AND DESIGN STORM VOLUME

SCS Curve Numbers

Land Use			Curve Number (CN) B-Type Soil
Full Impervious			98
Gravel Parking			85
Clay/Gravel Road			89
Urban District			92
Commercial/Industrial			88
Woods			60
Pasture, Lawns, Open Spaces w/good grass cover			60
Residential			
	Size	Typical % Impervious	
	1 acre	20	68
	½ acre	25	70
	1/3 acre	30	72
	¼ acre	40	76
	1/8 acre	65	85

TABLE 2-3
SCS CURVE NUMBERS
Auburn Storm Water Management Manual

Rational Method

Land Use			Runoff Coefficient
Full Impervious			0.90
Gravel Parking			0.70
Clay/Gravel Road			0.75
Urban District			0.82
Commercial/Industrial			0.75
Woods			0.35
Pasture, Lawns, Open Spaces w/good grass cover			0.35
Residential			
	Size	Typical % Impervious	
	1 acre	20	0.45
	½ acre	25	0.49
	1/3 acre	30	0.53
	¼ acre	40	0.60
	1/8 acre	65	0.71

The general equation for determining all other C values is as follows:

C=(%Area Impervious * Full Impervious Values) + (%Area Pervious * Pasture Value)

TABLE 2-4
RATIONAL METHOD

SCS Dimensionless Unit Hydrograph Ratios And Mass Data

Time Ratios (t/t_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.055	.999
5.0	.000	1.000

Reference: USDA, SCS, NEH-4 (1972).

TABLE 2-5
SCS DIMENSIONLESS UNIT HYDROGRAPH RATIOS AND MASS DATA

Worksheet

Project ____________________________ By __________ Date _______________

Location _________________________________ Checked _____ Date _______________

Circle one : Present Developed ___________________________________

Circle one: t_c t_t through subarea ___________________________________

NOTES: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to t_c only)	Segment ID		
1. Surface description			
2. Manning's roughness coeff., n (Table 3-1 and 3-2)			
3. Flow length, L (total L ≤ 300 ft)	ft		
4. Two-yr 24-hr rainfall, P_2	in		
5. Land slope, s	ft/ft		
6. $t_{sheet\ flow} = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}} * 60\ \frac{min}{hr}$ Compute $t_{sheet\ flow}$	min		=

Shallow concentrated flow	Segment ID		
7. Surface description (paved or unpaved)			
8. Flow length, L	ft		
9. Watercourse slope, s	ft/ft		
10. Average velocity, V Eq (2-4 or 2-5)	ft/s		
11. $t_{shallow\ concentrated\ flow} = \frac{D}{3600V} * 60\ \frac{min}{hr}$ Compute $t_{shallow\ concentrated\ flow}$	min		=

Channel flow	Segment ID		
12. Cross sectional flow area, a	ft^2		
13. Wetted perimeter, P_w	ft		
14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r	ft		
15. Channel slope, s	ft/ft		
16. Manning's roughness coeff., n			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s		
18. Flow length, L	ft		
19. $t_{channel\ flow} = \frac{D}{3600\ V} * 60\ \frac{min}{hr}$ Compute $t_{channel\ flow}$	min		=
20. Watershed or subarea t_c or t_t (add t in steps 6, 11, and 19)		min	

FIGURE 2-1
WORKSHEET: TIME OF CONCENTRATION (t_c) OR TRAVEL TIME (t_t)

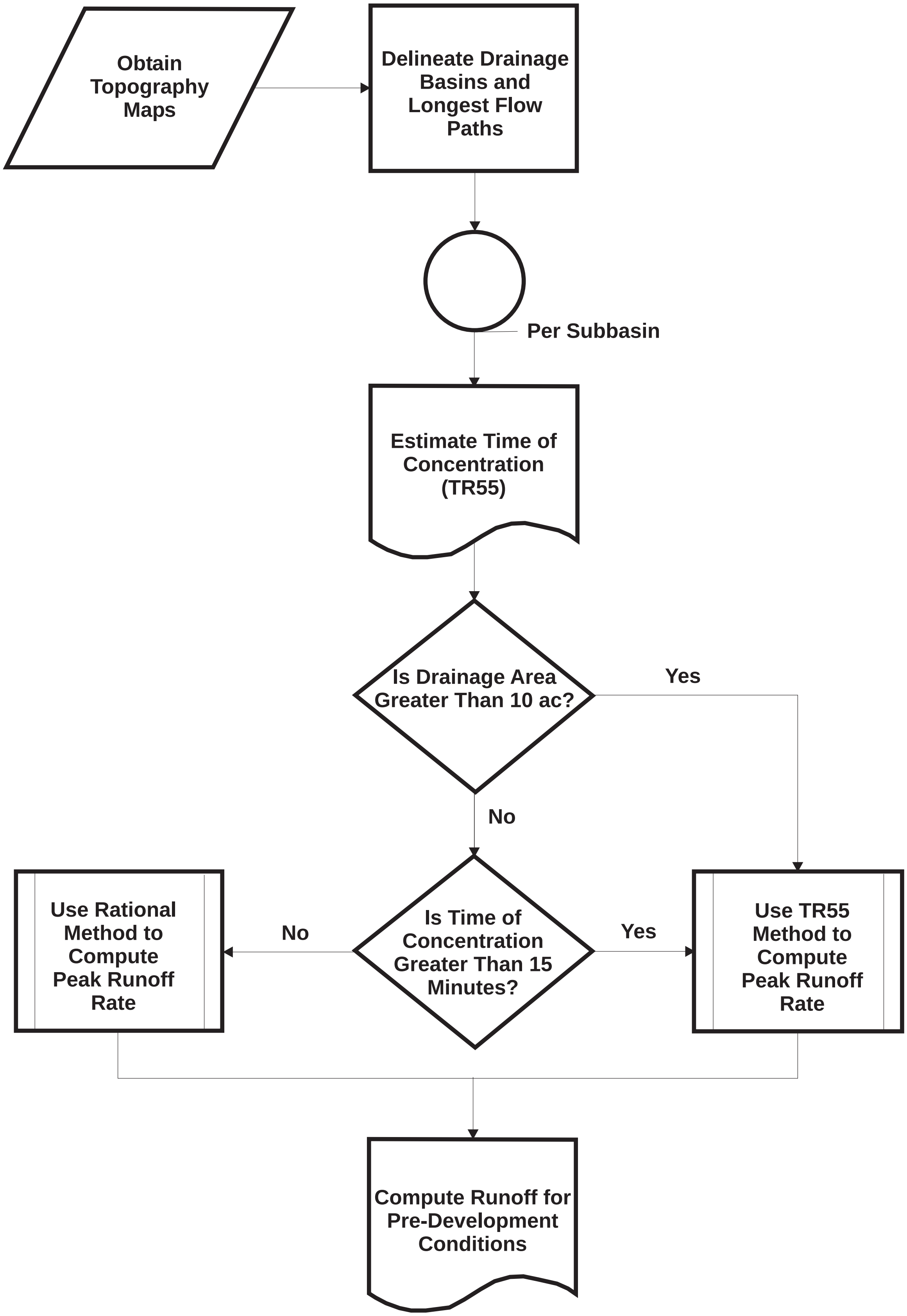


FIGURE 2-2
PEAK RUNOFF RATE METHODOLOGY FLOW CHART
Auburn Storm Water Management Manual

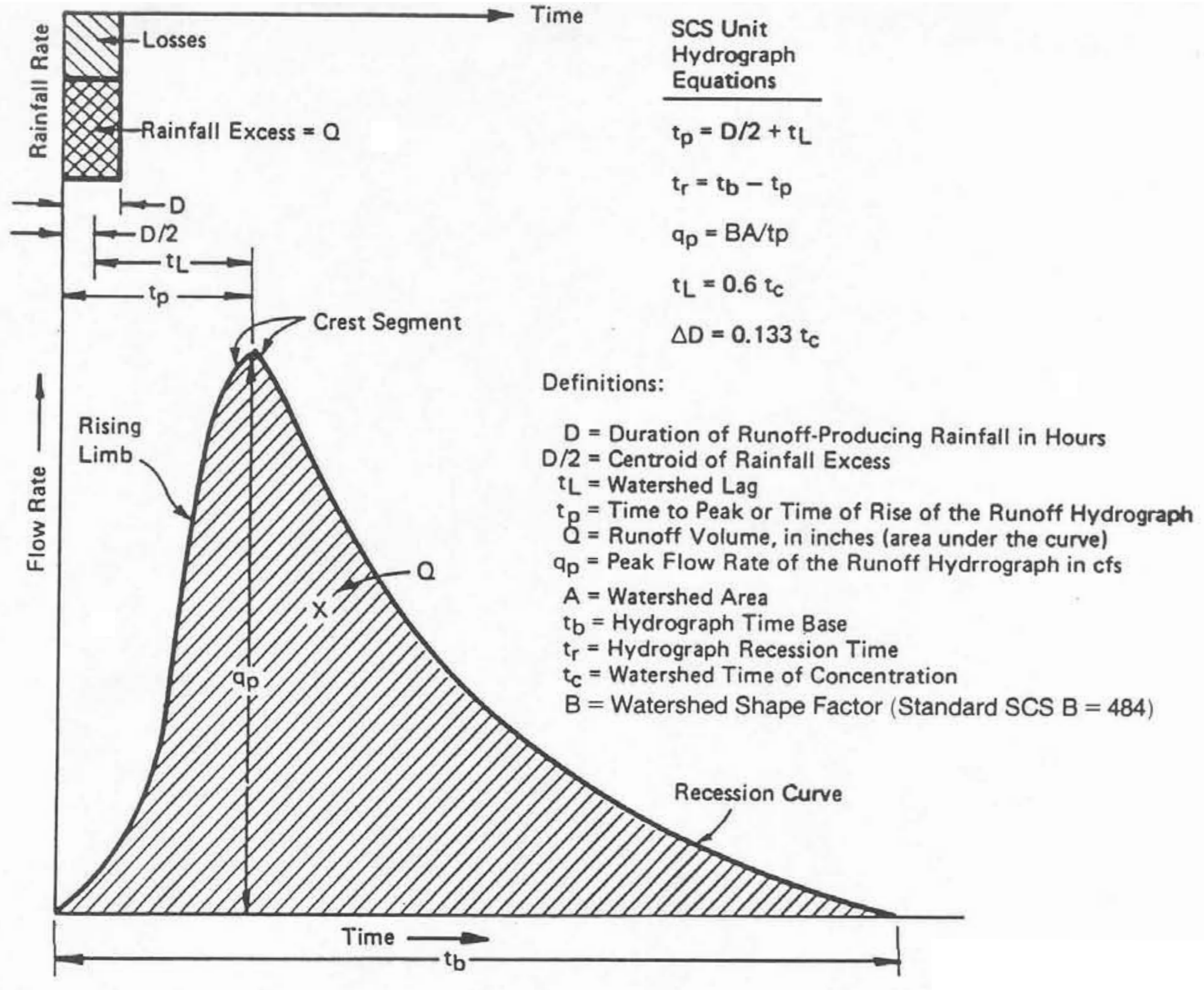
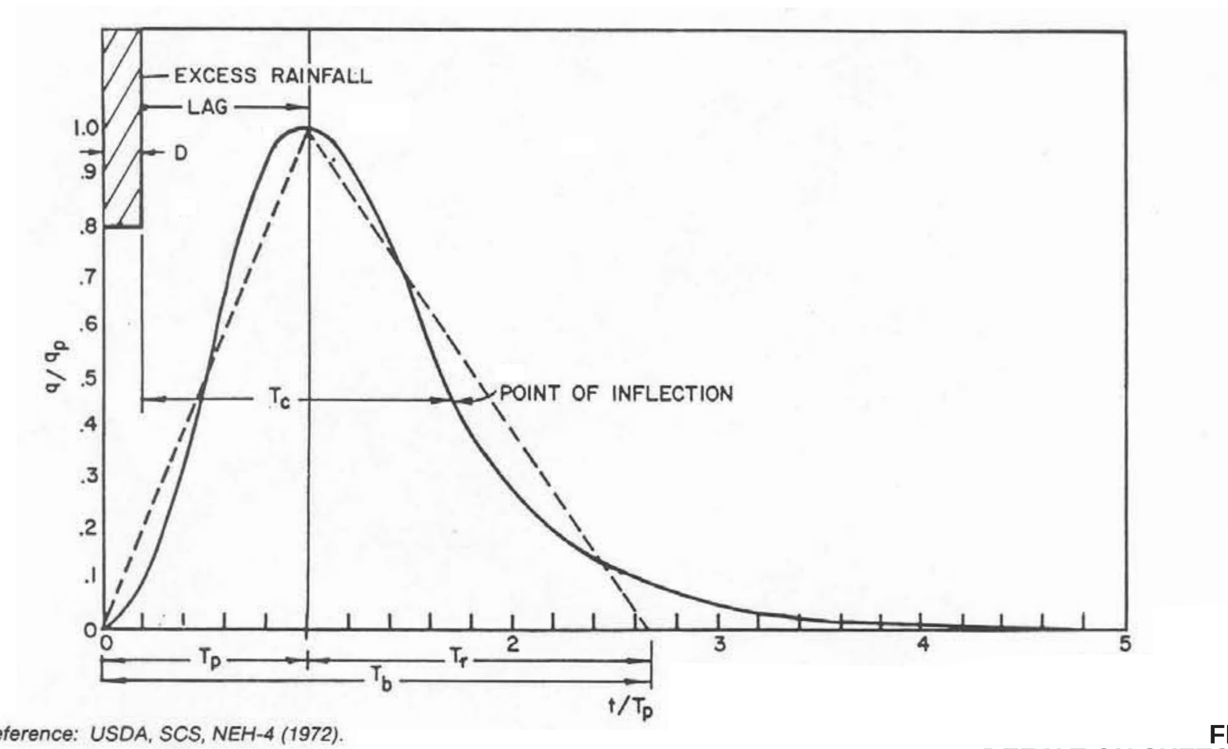
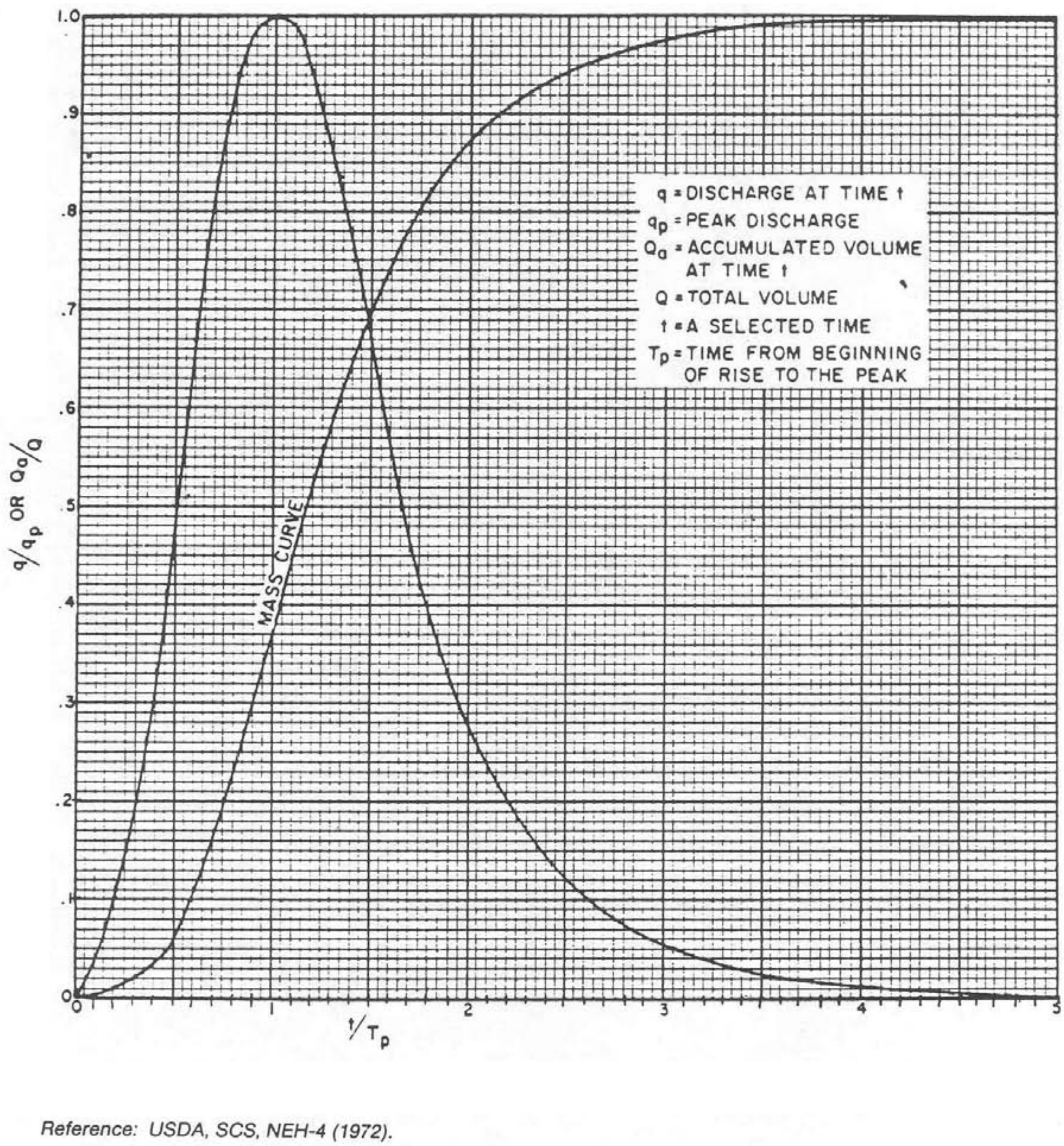


FIGURE 2-3
GENERAL HYDROGRAPH TERMINOLOGY
Auburn Storm Water Management Manual



Reference: USDA, SCS, NEH-4 (1972).

FIGURE 2-4
DEFINITION SKETCH OF THE
SCS DIMENSIONLESS UNIT HYDROGRAPH
Auburn Storm Water Management Manual



Reference: USDA, SCS, NEH-4 (1972).

FIGURE 2-5
SCS DIMENSIONLESS UNIT HYDROGRAPH AND MASS CURVE
Auburn Storm Water Management Manual

3. Hydraulic Design of Storm Water Conveyance System

3. Hydraulic Design of Storm Water Conveyance Systems

The hydraulic design of a storm water conveyance system requires that water surface or flood elevation profiles be determined for a given design storm return period. A storm water conveyance system should then be designed to prevent or mitigate the flood damage to the extent possible with the financial resources available. In addition to flooding considerations, factors concerning erosion and sedimentation, such as scour velocities, within the conveyance system must be considered during the hydraulic design process.

The purpose of this section is to identify primarily desktop procedures, which are applicable in Auburn, for the hydraulic design of open channels, pipes, streets, inlets, and culverts. The procedures developed require appropriate hydrologic data as an input to the hydraulic design process. The appropriate hydrologic data should be developed according to the procedures presented in Section 2. Generally, an estimate of the peak storm water flow rate expected to occur during a design storm is all that will be required. However, if channel routing is required, then a complete runoff hydrograph should be developed.

For procedures, processes, and designs outlined in this section, calculations must be conducted all the way downstream to the first City-maintained storm water management facility. During design storm flows, the structures and system currently in place are not flooded. If the added volume will compromise the current structures or system, necessary steps must be taken to resolve flooding problems.

3.1 Open Channels

This subsection presents standards for the design of artificial or manmade open channels, including roadside ditches, median ditches, interceptor ditches, outfalls, and canals.

Ditches, outfalls, detention areas, and other drainage-related features must be provided with berms and other physical access devices that facilitate maintenance activities. Consideration must be given to future expansion of the facilities and to possible increased maintenance requirements. Absolute minimum tolerances (e.g., bank full conditions) should only be used in extremely stable areas, in areas requiring infrequent maintenance, or in areas where existing physical constraints require their use. Berms should be based at the narrowest point; the right-of-way should be reasonably uniform.

The maximum allowable side slopes for open channels are as follows:

- Vegetated Residential–4:1
- Vegetated Nonresidential–3:1
- Rip-rap–2:1

For ditches where positive flow conditions are required (i.e., water must flow downstream), a minimum physical slope of 1.5 percent will be used unless otherwise authorized by the City Engineer.

Design documentation for open channels will include the hydrologic analysis and the hydraulic analysis, including analysis of channel lining requirements.

Open channels will be designed to convey, without damage, peak storm water flow based on a design frequency of 25 year.

The basic equation for design of open channels is Manning's Equation:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad \text{(Equation 3-1)}$$

or

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad \text{(Equation 3-2)}$$

where:

v = average channel velocity, ft/sec

Q = average channel flow, cfs

n = Manning's roughness coefficient (provided in Tables 3-1 and 3-2)

R = hydraulic radius of the channel, ft

S = slope of the channel bottom, ft/ft

The hydraulic design process for any man-made open channel can be visualized as a two-step process. First, an optimum channel cross section is determined. Second, this optimum channel configuration is adjusted as necessary such that the average channel velocity is less than the maximum permissible velocity for the channel material.

The design of open channels will consider the need for channel linings. Maximum velocities for the various forms of channel linings are provided in Tables 3-3 and 3-4. When design flow velocities do not exceed the maximum value for bare earth as given in Table 3-3, standard treatment of ditches consists of grassing and mulching. For higher design velocities, sodding (with staking or netting), ditch paving, or other forms of lining consistent with Tables 3-3 and 3-4 will be provided. However, grassing and sodding should not be used if there is continuous standing or flowing water in the canal that prohibits grass growth. Drainage designs will be reviewed to evaluate whether some form of protective treatment will be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Protective treatment for open channels in the form of fencing will be considered when a potential hazard exists. The design and location of open channels will comply with roadside safety and clear zone requirements.

3.2 Pipes

3.2.1 Design Criteria

3.2.1.1 Return Periods

For all calculations, a 25-year storm return period should be used unless facilities are under the jurisdiction of the Alabama Department of Transportation (ALDOT). ALDOT requires a 50-year storm return period to be used for calculations.

3.2.1.2 Manning's n Values

All storm sewer pipes will be reinforced concrete or corrugated metal pipe (CMP) in main channels and reinforced concrete, CMP, or high-density polyethylene (HDPE) in side channels unless the Public Works Department approves alternative materials. Values for Manning's roughness coefficient for concrete pipe, concrete box culvert, HDPE, and CMP are given below:

Concrete Pipe	n = 0.012
Box Culvert (cast-in-place)	n = 0.013
CMP (non-spiral flow, annular corrugations)	n = 0.024
CMP (full pipe spiral flow, helical corrugations)	
sizes 15-24 inches	n = 0.017
sizes 30-54 inches	n = 0.021
sizes 60-96 inches	n = 0.024
HDPE (24″ or smaller)	n = 0.020

Full spiral flow occurs only for a pipe with a length 20 times its diameter, operating under full flow and free of sediment buildup. Conditions where full spiral flow would be appropriate are down drains and free outlet or gravity storm sewer systems with a design velocity above 4 ft/sec.

3.2.1.3 Slopes and Hydraulic Gradient

The maximum and minimum slopes for storm sewers will conform to the following criteria:

1. The maximum hydraulic gradient will not produce a velocity that exceeds 15 ft/sec. Higher velocities require approval from Public Works.

2. The minimum desirable physical slope will be that which will produce a velocity of 2.5 ft/sec when the storm sewer is flowing full.

When hydraulic calculations do not consider minor energy losses, the elevation of the hydraulic gradient for design flood conditions will be at least 1 foot below the gutter elevation. Minor losses will be considered when the velocity exceeds 6 ft/sec. If all minor energy losses are calculated, it is acceptable for the hydraulic gradient to reach the gutter elevation.

3.2.1.4 Pipe Size and Length and Access Spacing

Access spacing is the point in the pipeline where there is access available from the surface, like at a manhole or inlet. The minimum pipe size will be 15 inches when access spacing is 200 ft or less. When access spacing exceeds 200 ft, the minimum pipe size will be 18 inches. Standard pipe size increments of 6 inches will be used for pipes larger than 18 inches. Access spacing will not exceed 500 ft for conduits less than 54 inches in diameter and will not exceed 800 ft under any circumstances.

The minimum box culvert size will be 3 by 3 ft for pre-cast units and 4 by 4 ft for cast-in-place units. Increments of 1 foot in the height or width will be used above this minimum. The "span by height" format is used for reporting box culvert dimensions. For example, for the dimension 10 x 7, the span is 10 ft wide and the height is 7 ft.

3.2.1.5 Minimum Clearances

Minimum clearances for storm sewer pipe will comply with the following criteria:

1. For the minimum spacing required between road base material and the outside crown of the storm sewer pipe, refer to the City Standard Details.

2. For utility conflicts that involve crossing a storm sewer alignment, the minimum design clearance between the outside of the pipe and the outside of any conflicting utility should be 0.5 foot if the utility has been accurately located at the point of conflict. If the utility has been approximately located, the minimum design clearance should be 1 foot. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

3. Storm sewer systems should not be placed parallel to or below existing utilities in a manner that could cause utility support problems. The horizontal clearance between pipes should be 2 ft (i.e., extending from each side of the storm sewer and 1:1 side slopes from the trench bottom). Where right-of-way limitations are inadequate, shoring of trenches is acceptable.

4. When a sanitary line or other utility must pass through a manhole, a minimum 1-foot clearance should be maintained between the bottom of the utility and the flow line of the storm main, and greater clearance is recommended. Flow will be less obstructed when the utility is placed above or as close as possible to the crown of the pipe and shall be no lower than the top one-third of the pipe. The head loss caused by an obstruction should be taken into account. (Note: Gas mains must not pass through inlet and manhole structures.)

3.2.2 General Procedure

The design of storm sewer systems is usually an iterative process involving the following four steps:

1. System Layout: Selection of inlet locations and development of a preliminary plan and profile configurations consistent with the design criteria in Section 3.2.1

2. Hydrologic Calculations: Determination of design flow rates and volumes (Section 3.2.3)

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 3.2.4
4. Outfall Design: Outlet protection or detention may be required because of downstream constraints and compliance with City ordinances and Storm Water Phase II requirements

3.2.3 Hydrologic Calculations

The hydrologic determination of a peak runoff rate for sizing a storm sewer system will be made using procedures presented in Section 2. In general, storm sewer systems are sized to carry storm water intercepted by appropriate inlet facilities. However, if the intercepted runoff is transported through an extensive pipe network, channel storage within the storm sewers can modify the peak rate of runoff as it travels along the system. The peak flow modification can be evaluated with hydrologic channel routing procedures (Section 2.3.4).

For many small projects, the Rational Method is well suited to performing hydrologic calculations for storm sewer systems. In general, as the time of concentration, drainage area, and variability in land use increase, more complex procedures are warranted. In addition, the size and complexity of the storm sewer system should be considered.

To apply the Rational Method at each design point, the following data are required:

1. Tributary area
2. Time of concentration (Section 2.3.1)
3. Rainfall intensity (Table 2-1)
4. Runoff coefficient (Table 2-5)

The Rational Method presented in Section 2.3.2 implicitly assumes that all runoff from the tributary area is intercepted by the storm sewer system. Design the inlet so that sufficient capacity is available to capture peak flow.

The proper selection of the time of concentration indirectly accounts for situations in which peak runoff arrives at individual inlets at different points in time. However, this approach does not explicitly account for channel storage that can become important in the upstream portions of a large storm sewer system.

In some cases, only a portion of the area draining to a particular location in a storm sewer system will cause the peak discharge. In systems where a small subbasin of the total area has an unusually long travel time, another larger subbasin of the contributing area with a shorter time of concentration (and therefore, higher average rainfall intensity) can cause a higher peak discharge.

3.2.4 Hydraulic Calculations

Hydraulic calculations are used to size pipes to handle the design flows determined from hydrologic calculations (Section 3.2.3). The hydraulic capacity of a storm sewer pipe can be calculated for the two types of conditions typically referred to as open channel and pressure flow. The results should provide a balanced system in which all segments will be used to their full capacity, consistent with the flood protection criteria for the project site.

3.2.4.1 Pressure Versus Open Channel Flow

Open channel flow occurs when a free water surface is exposed to the atmosphere as a boundary (Figure 3-1). When the conduit is flowing full, the pipe is considered to be flowing under pressure (Figure 3-2).

Guidance is presented in Figure 3-3 for determining whether pressure or open channel flow conditions occur in a storm sewer system. In general, if the hydraulic grade line is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of the pipe, open channel flow calculations are appropriate. The assumption of straight hydraulic grade lines illustrated in Figure 3-3 is not entirely correct, but is generally reasonable for typical pipe sizes.

A special concern with storm sewers designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned, as well, including an evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system.

As hydraulic calculations are performed, the existence of the desired flow condition should be verified. Storm sewer systems can alternate between pressure and open channel flow conditions from one section to another.

The discharge point of the storm sewer system usually establishes a starting point for evaluating flow conditions. If the discharge is submerged, as when the tailwater established elevation is above the crown of the storm sewer, the exit loss should be added to the tailwater and calculations for head loss in the storm sewer system started from this point (Figure 3-3). If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream manhole.

When the discharge point is not submerged, a flow depth should be determined at some control section to allow calculations to proceed upstream. As shown in Figure 3-3, the hydraulic grade line is then projected to the upstream manhole. Pressure flow calculations may be used at the manhole if the hydraulic grade is above the pipe crown. It also is usually appropriate to assume that the hydraulic grade calculations begin at the crown of the outlet pipe for simple non-submerged systems. If additional accuracy is needed, as with large conduits or where the result will have a significant effect on design, backwater and drawdown curves should be developed.

3.2.4.2 Energy Losses

The following energy losses will be considered for storm sewer systems:

1. Friction
2. Entrance
3. Exit

Additional energy loss parameters should be evaluated for complex or critical systems. The following losses are especially important when failure to handle the design flood creates the potential to flood offsite areas:

1. Expansion
2. Contraction
3. Bend
4. Junction and manhole

Friction Loss

The energy loss required to overcome friction caused by conduit roughness generally is calculated as:

$$H_f = \left[\frac{29n^2L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad \text{(Equation 3-3)}$$

where:

H_f = Energy loss due to friction, ft

n = Manning's roughness coefficient

L = Conduit length, ft

R = Hydraulic radius of conduit, ft

v = Average velocity, feet per second squared (ft/sec^2)

g = Acceleration due to gravity, 32.2 ft/sec^2

Entrance, Exit, Expansion, Contraction, and Bend Losses

Head losses due to pipe form conditions generally are calculated as:

$$H_L = K\left(\frac{v^2}{2g}\right) \qquad \text{(Equation 3-4)}$$

where:

H_L= Head loss due to pipe form conditions, ft

K = Loss coefficient for pipe form conditions

v = Average velocity, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

The loss coefficient, K, is different for each category of pipe form loss and should be based on operating characteristics of the system being considered. Values for the entrance loss coefficient are the same as those developed for culverts (Section 3.4). The exit loss coefficient is generally assigned a value of 1. Expansion and contraction loss coefficients for circular pipes can be selected based on the data from Brater and King (1976), presented in Tables 3-5 and 3-6.

Junction and Manhole Losses

If losses associated with junctions and manholes are evaluated, procedures presented in a report by the University of Missouri (1958) or Marsalek (1985) should be used. Although details of the procedures are not duplicated, the implications of laboratory test results are discussed below, and head loss coefficients for typical manholes and junctions are presented in Table 3-7.

For straight flow-through conditions, the pipes should be positioned vertically with the inverts aligned or crowns aligned. An offset in the plan is allowable, provided that the projected area of the smaller pipe falls within that of the larger. It is most effective to align the pipe inverts, because the manhole bottom will then support the bottom of the jet issuing from the upstream pipe.

When two laterals intersect at a manhole, pipes should not be oppositely aligned, because the jets could impinge upon each other. If directly opposing laterals are necessary, the installation of a deflector (as shown in Figure 3-4) will significantly reduce losses. The research conducted on this type of deflector is limited to the ratios of outlet pipe to lateral pipe diameters equal to 1.25. In addition, lateral pipes should be located such that their centerlines are separated laterally by at least the sum of the two lateral pipe diameters.

Figure 3-4 depicts several types of deflectors that can be efficient in reducing losses at junctions and bends for full flow conditions.

3.2.4.3 Open Channel Flow

Under non-pressure conditions, the capacity of a closed conduit can be analyzed by applying Manning's Equation to evaluate frictional losses for uniform flow. As shown in Figure 3-1, the hydraulic grade line is the free water surface elevation and is parallel to the energy grade line under uniform flow conditions. For circular conduits flowing full, Manning's Equation can be expressed as:

$$v = \frac{0.592}{n} d_s^{2/3} s_o^{1/2} \qquad \text{(Equation 3-5)}$$

or

$$Q = \frac{0.465}{n} d_s^{8/3} s_o^{1/2} \qquad \text{(Equation 3-6)}$$

where:

v = Full flow velocity, ft/sec

Q = Design discharge, cfs

n = Manning's roughness coefficient

d_s = Diameter of the circular conduit, ft

s_o = Pipe slope, ft/ft

Non-circular and non-full flow conditions can be evaluated using the standard form of Manning's Equation, as discussed in Section 3.1.

Given the appropriate peak runoff rate for the design point in question, the conduit is sized such that the capacity as computed for an open channel, using Manning's Equation, exceeds the required design flow, thus ensuring that open channel conditions exist at that rate. Equation 3-6 was used to develop the following flow rates for some combinations of pipes and sizes:

So = 0.03 ft/ft
n = 0.012 RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	12.2
18	1.50	19.8
24	2.00	42.6
30	2.50	77.3
36	3.00	125.6

So = 0.02 ft/ft
n = 0.012 RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	9.9
18	1.50	16.2
24	2.00	34.8
30	2.50	63.1
36	3.00	102.6

So = 0.01 ft/ft
n = 0.012 RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	7.0
18	1.50	11.4
24	2.00	24.6
30	2.50	44.6
36	3.00	72.5

So = 0.005 ft/ft
n = 0.012 RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	5.0
18	1.50	8.1
24	2.00	17.4
30	2.50	31.5
36	3.00	51.3

Ref. Concrete Pipe Manual

For a condition in which pressure is allowed to develop in storm sewers with a design based on open channel flow conditions, the design capacity of the system will be greater than that determined using Equation 3-6, and can be evaluated as discussed below.

3.2.4.4 Pressure Flow

If the hydraulic grade line, as illustrated in Figure 3-2, can be increased above the crown of the pipe, pressure flow occurs. The capacity of storm sewers designed to operate under pressure flow conditions can be sized using inlet and outlet control nomographs developed for the evaluation of culverts (Section 3.4). A more general procedure involves the application of the energy and continuity equations, which can be developed to consider unsteady flow conditions.

The energy equation between upstream and downstream locations can be evaluated by considering velocity head, pipe form, and friction losses expressed as:

$$H = H_v + H_t + H_f \qquad \text{(Equation 3-7)}$$

or

$$H = \left[1 + K_L + \frac{29n^2L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad \text{(Equation 3-8)}$$

where:

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, ft

H_v = Velocity head, ft

H_l = Head loss due to pipe form conditions, ft

H_f = Head loss due to friction, ft

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, ft

R = Hydraulic radius, ft

v = Average velocity of flow, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

If H can be determined, the storm sewer capacity is calculated by rearranging Equation 3-8 as follows:

$$v = \left[\frac{2gH}{1 + K_L + \frac{29n^2L}{R^{1.33}}}\right]^{1/2} \qquad \text{(Equation 3-9)}$$

or

$$Q = A\left[\frac{2gH}{1 + K_L \frac{29n^2L}{R^{1.33}}}\right]^{1/2} \qquad \text{(Equation 3-10)}$$

where:

v = Average velocity of flow, ft/sec

Q = Storm sewer capacity, cfs

g = Acceleration due to gravity, 32.2 ft/sec^2

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, ft

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, ft

R = Hydraulic radius, ft

The determination of H will generally involve an evaluation of energy losses to establish the hydraulic and energy gradients. Because the velocity is a required input to energy loss calculations, an iterative trial-and-error procedure generally is required.

3.3 Streets and Inlets

3.3.1 Design Criteria

The following design criteria are factors to consider for gutter and inlet capacity calculations:

1. Return period
2. Spread
3. Manning's n values
4. Longitudinal slope
5. Cross slope
6. Curb and gutter sections
7. Inlet spacing
8. Roadside ditches
9. Bridge decks

3.3.1.1 Return Period

The design storm return period for pavement drainage will be consistent with the value selected for other components of the drainage system and thus will be 25 years.

3.3.1.2 Spread

Pavement flooding will not exceed the spread for design storm conditions. Spread is defined as the width-of water transported on the pavement measured from the face of the curb. For all roadways, the spread will not exceed one half of the travel lane adjacent to the curb.

3.3.1.3 Manning's n Values

Curb and gutter flow characteristics, including spread, will be calculated using a minimum Manning's n value of 0.016.

3.3.1.4 Longitudinal Slope

Curb and gutter longitudinal slopes will not exceed 15 percent for Local, 12 percent for Collectors, or 8 percent for Minor Arterials or fall below 1 percent for City streets and 15 percent or fall below 1 percent in ALDOT streets, without approval from the Public Works Department.

3.3.1.5 Cross Slope

The design of pavement cross slope on residential streets will conform to the standard roadway sections provided in the City's standard drawings or the ALDOT standard drawings, as applicable. Shoulders generally should be sloped to drain away from the pavement, except with raised, narrow medians. The Department of Public Works must approve deviations from these standards.

3.3.1.6 Curb and Gutter Sections

A concrete curb and gutter at the outside edge of pavements is normal practice for urban roadway facilities. Traditional combination curb and gutter sections for the City may be 2.0 wide with a 1-inch-per-18 inch cross slope and for the ALDOT 2.5 ft wide with a 1-inch-per-foot cross slope for the gutter portion of the section, consistent with the standard details. Curb and gutter capacities for conveying pavement drainage to the storm water inlets must be considered in the design of the roadway. Allowable gutter flow rates will be based on a longitudinal gutter slope of 0.5 inch per foot (0.042 ft/ft) to account for possible debris and sediment accumulation.

Rolled curbs are sometimes used in residential areas because they avoid curb cuts for driveways and allow runoff to flow onto adjacent pervious areas. These gutters must convey the required design storm, also.

3.3.1.7 Inlet Spacing

Curb inlets will be located to facilitate the entrance of water from gutters into the storm sewer system. Inlets will be located or spaced so that at least 80 percent of the gutter flow is intercepted (i.e., no more than 20 percent of the gutter flow may bypass the inlet). The procedures in Section 3.3.2 will be used to establish inlet spacing such that a maximum of 20 percent bypass is allowed. The last inlet will have no bypass. The maximum length of longitudinal curb and gutter section without an inlet will be 500 ft. The typical inlet type used in Auburn is the Type S.

Consideration also must be given to the movement of vehicles to and from adjacent property on turnouts and to the maintenance of safe pedestrian walkways. No flow will be allowed to cross intersecting streets unless approved by the Public Works Department. In addition, curb and gutter inlets will not be placed directly at the corner of intersections. Curb inlets will not be located within handicap ramps and will not be located within a 25-foot radius of turns. Variations from these criteria are subject to approval by the Public Works Department.

Yard inlets will be designed to intercept the total design flow approaching the inlet. Refer to the City's Standard Details for the preferred grated inlet design.

3.3.1.8 Roadside Ditches

All new roadways will require curbs and gutters that conform to the Subdivision Regulations of the City of Auburn, unless exempted by the Subdivision Regulations. Where the Public Works Department approves roadside ditches along the roadway, the design criteria will conform to the standard roadway sections as shown in the City's Standard Detail drawings. Where practicable, the flow from upgradient areas draining toward curbed highway pavements should be intercepted by ditches.

Roadside ditches can be used with uncurbed roadway sections to convey pavement runoff and upgradient area runoff that drain toward the pavement. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Roadside ditches will be sized using the open channel hydraulic procedures presented in Section 3.1. Roadside drainage structures will be designed to minimize hazards to vehicles that leave the traveled roadway.

3.3.1.9 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect debris, and small inlets on scuppers have a higher potential for clogging than curb-opening inlets. Bridge deck constructibility usually requires a constant cross slope, so the guidelines in Section 3.3.5 do not apply. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways will be intercepted before it reaches a bridge.

3.3.2 Gutter Flow Calculations

Generally, gutter flow calculations are used to make sure that water will not cover the street. This type of flooding can be controlled by including more inlets (and maybe pipes). Often, only a few typical calculations are required to illustrate sufficient capacity. Also note that inlets often do not capture 100 percent of the flow, and downstream gutters must include this bypass flow. The following form of Manning's Equation will be used to evaluate gutter flow hydraulics:

$$Q = \frac{0.56}{n} S_x^{5/3} s^{1/2} T^{8/3} \qquad \text{(Equation 3-11)}$$

where:

Q = Gutter flow rate, cfs

n = Manning's roughness coefficient, use 0.016

S_x = Pavement cross slope, ft/ft

s = Longitudinal slope, ft/ft

T = Width of flow or spread, ft

A nomograph for solving Equation 3-11 is presented in Figure 3-5. Rolled curbs can be approximated with a V-shape.

3.3.2.1 Uniform Cross Slopes

The nomograph in Figure 3-5 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_x, gutter flow, Q, and Manning's n.
2. Draw a line between the S and S_x scales and note where it intersects the turning line.
3. Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.
4. Read the value of the spread, T, at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_x, spread, T, and Manning's n.
2. Draw a line between the S and S_x scales and note where it intersects the turning line.
3. Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.
4. For Manning's n values of 0.016, the gutter capacity, Q, from Step 3 is selected. For other Manning's n values, the gutter capacity times n, Qn, is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

3.3.2.2 Standard Gutter Sections

Allowable capacity data for Auburn standard curb and gutter for residential pavement sections are presented in Table 3-8. Table 3-8 provides allowable gutter capacities for a 2.0-foot curb and gutter section using a residential pavement cross slope of 0.0208 ft/ft and a maximum spread of 8 ft. When the roadway or longitudinal slope is known, the allowable gutter capacity can be estimated directly from Table 3-8.

3.3.2.3 Composite Gutter Sections

Figures 3-5 and 3-6 can be used to find the flow in a composite gutter section with width, W, less than the total spread, T. The following steps are used to evaluate any composite gutter section:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_x, depressed section slope, S_w, depressed section width, W, Manning's n, gutter flow, Q, and a trial value of the gutter capacity above the depressed section, Q_s.
2. Calculate the gutter flow in W, Q_w using the equation:

$$Q_w = Q - Q_s \qquad \text{(Equation 3-12)}$$

3. Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3-6 to find an appropriate value W/T.
4. Calculate the spread, T, by dividing the depressed section width, W, by the value of W/T from Step 3.
5. Find the spread above the depressed section, T_s by subtracting W from the value of T obtained in Step 4.
6. Use the value of T_s from Step 5 along with Manning's n. S, and S_x to find the actual value of Q_x from Figure 3-6 (Section 3.3.2.1, Condition 2).
7. Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including spread, T, spread above the depressed section (i.e., gutter), T, cross slope, S_x, longitudinal slope, S, depressed section slope, S_w, depressed section width, W, Manning's n, and depth of gutter flow, d.
2. Use Figure 3-5 to determine the capacity of the gutter section above the depressed section, Q_s. Use the procedure in Section 3.3.2.1, Condition 2, substituting T_s for T.
3. Calculate the ratios W/T and S_w/S_x, and, from Figure 3-6, find the appropriate value of E_o (the ratio of Q_w/Q).
4. Calculate the total gutter flow using the equation:

$$Q = Q_s/(1 - E_0) \qquad \text{(Equation 3-13)}$$

where:

Q = Gutter flow rate, cfs

Q_s = Flow capacity of the gutter section above the depressed section, in cfs

E_0 = Ratio of frontal flow to total gutter flow (Q_w/Q), obtained from Figure 3-6.

5. Calculate the gutter flow in W, Q_w, with Equation 3-12.

3.3.3 Curb Opening Inlets

Curb-opening inlets are relatively free of clogging problems and offer little interference to traffic operation. They are preferable to grates in traffic lanes or where grates would be hazardous for pedestrians or bicyclists.

For the flow capacity of inlets, refer to Table 3-9. These values were computed for nearly flat conditions and represent the maximum allowable capacity expected from a clean S-Type inlet.

3.3.4 Grate Inlets

Grates are efficient for intercepting pavement drainage if clogging by debris is properly controlled through maintenance. Grate inlets will intercept the gutter flow passing over the front of the grate if the gutter flow does not splash over the grate. The portion of side flow intercepted will depend on the cross slope of the pavement, length of grate, and flow velocity.

Procedures to determine the capacity of grate inlets placed on continuous grade and at sump locations are available from HEC-12 (U.S. Department of Transportation [DOT], Federal Highway Administration [FHWA], 1984).

Example 3.1: Sizing Gutters and Inlets for Drainage System

A standard 2-foot curb and gutter will be used for the roadway. For this example, place an inlet every 250 ft on either side of the road to carry street runoff. By applying the Rational Method, peak runoff from a section of street is 0.45 cfs.

$$Q_T = 0.9 * 8.7 * .06 = 0.45 \text{ cfs} \qquad \text{(from Equation 2-7)}$$

(area = 1/2 street width [10 ft] multiplied by street length [250 ft], t_c = 5 minutes, C = 0.9 from Table 2-4, intensity = 8.7 inches/hour from Table 2-1).

Gutter flow rates listed in Table 3-8 indicate that the hydraulic capacity is 1.00 cfs for a roadway slope 0.5 percent. Therefore, the gutter flow will not exceed the capacity or spread limitations. Each gutter inlet will interconnect with the main storm water drainage line. As stated in Section 3.2.1.4, the minimum pipe size will be 15 inches.

Table 3-9 indicates that an S-type inlet can handle the necessary hydraulic load (4.3 cfs) for all of the subbasins. Each inlet will be sized according to City standard details. Inlet location and spacing should meet the criteria identified in Sections 3.2.1.6 and 3.3.1.7. The 250-foot spacing between inlets meets the City's minimum requirements, so there is not a problem with street drainage.

3.4 Culverts

Culvert hydraulics can be classified and analyzed on the basis of a control section. A control section is a location where a unique relationship exists between the rate of flow and depth of flow or water surface elevation. The two basic types of control sections are inlet and outlet control.

3.4.1 Inlet Control

Inlet control exists when the culvert barrel is capable of conveying more flow than the inlet will accept. The control sections for this condition are located just inside the entrance.

Critical depth occurs at or near this location and the flow in the culvert is supercritical. The following variables influence culvert performance at the inlet:

1. Headwater elevation
2. Inlet area
3. Inlet edge configuration
4. Inlet shape

Flow under inlet control may be described mathematically by either the weir formula (Section 4.3.2) or the orifice formula (Section 4.3.3), depending on the headwater depth. It is important to note that the tailwater elevation has no influence on capacity.

3.4.2 Outlet Control

Outlet control occurs when the culvert barrel or outlet has less capacity than the inlet. The control section for this situation is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur. In addition to the variables influencing inlet performance listed above, the following factors can affect outlet performance:

1. Barrel roughness
2. Barrel area
3. Barrel shape
4. Barrel length
5. Barrel slope
6. Tailwater elevation

For outlet control, the difference between headwater and tailwater elevation represents the energy (a.k.a., head) that conveys the flow through the culvert.

3.4.3 Culvert Selection

In most situations, the hydraulic sizing of a culvert is a trial-and-error process. A trial culvert size is assumed and inlet and outlet performance is evaluated to determine if they will satisfy the conditions prevailing at the proposed location. A culvert system is selected by choosing the following items:

1. Inlet structure
2. Barrel material

3. Shape
4. Size
5. Outlet structure

The inlet and outlet structures are usually the same to achieve a symmetrical installation.

3.4.4 Design Criteria

The following parameters will be considered when culvert hydraulic calculations are performed:

1. Discharge
2. Headwater elevation
3. Tailwater elevation
4. Manning's n values
5. Length and slope
6. Velocity limitations

3.4.4.1 Discharge

The design discharge for culvert design is for the 25-year return period for culverts handling drainage from primarily internal facilities. For culverts designed to accept drainage from upstream areas outside of the development, the City requires that the applicant predict the design flows for the entire upstream area. Furthermore, if the design storm is increased by the development, then the applicant must conduct calculations further downstream to, at least, the first City-maintained storm water facility. New development must not affect City facilities.

In addition to the design flow, the culvert capacity should be checked for the 100-year return period to ensure that overtopping flood conditions do not exceed 1 foot above the top of curb. If ponding occurs at the culvert entrance and a reduction in discharge attributable to storage is appropriate, reservoir routing calculations can be used to estimate the reduction. Furthermore, this guidance manual does not supercede Federal Emergency Management Agency (FEMA) or the City's subdivision rules; however, if this manual is more stringent, then it takes precedence.

3.4.4.2 Headwater

The allowable headwater elevation is determined from an evaluation of conditions upstream of the culvert and the proposed or existing roadway elevation. The following criteria will be analyzed:

1. Non-damaging or permissible upstream flooding should be identified. Headwater should be kept below these elevations.

2. Headwater depth for the design discharge should not exceed a height greater than 1.5 ft below the edge of the shoulder of a road.

3. Headwater depth for the design discharge should not cause water to rise above the top of approach channels adjacent to improved land and should not have an effect on the 100-year flood elevation (i.e., should not cause the water level to rise above the established 100-year floodplain, Q_{100}).
4. Other site-specific design considerations should be addressed as required.

The constraint that gives the lowest allowable headwater elevation will establish the basis for hydraulic calculations.

3.4.4.3 Tailwater

The hydraulic conditions downstream of the culvert site will be evaluated to determine a tailwater depth for the design discharge. The following conditions are typical:

1. If the culvert outlet is operating in a free fall condition (e.g., a cantilever pipe), the critical depth and equivalent hydraulic grade line should be determined using the procedures presented in Section 3.4.5.
2. For culverts that discharge to an open channel, tailwater depth is established by evaluating the normal depth of flow in the outlet channel.
3. If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
4. If the culvert discharges to a lake, pond, or other major waterbody, the expected high water elevation of the particular waterbody may establish the culvert tailwater.

3.4.4.4 Manning's n Values

All culverts will be constructed using reinforced concrete pipe or CMP in main channels and reinforced concrete pipe, CMP, or HDPE in side channels as detailed in the City's Standard Details unless prior approval for alternate materials has been received from the Public Works Department. Manning's n values for culvert capacity calculations will be as follows:

Material	n Value
Circular and Arch Concrete Pipe	0.012
Box Culvert (Concrete)	0.013
Oval Concrete	0.012
HDPE	0.020
CMP (non-spiral flow, annular corrugations)	0.024
CMP (full pipe spiral flow, helical corrugations)	0.017-0.024
Structural Plate CMP	0.0328 - 0.0302
Arch CMP	0.024
Structural Plate Arch	0.0327 – 0.0306

3.4.4.5 Length and Slope

The length and slope of a culvert should consider the following factors:

1. Channel bottom of the stream being conveyed
2. Geometry of the roadway embankment
3. Skew angle of the culvert

In general, the culvert slope should be chosen to approximate existing topography.

3.4.4.6 Velocity Limitations

A minimum velocity of 2.5 ft/sec when the culvert is flowing full is recommended to ensure a self-cleaning condition during partial depth flow. When velocities below this minimum are anticipated, the installation of a sediment trap upstream of the culvert is required.

The maximum velocity will be consistent with channel stability requirements at the culvert outlet. Rip-rap is required at all culvert outlets, unless otherwise approved by the Public Works Department. If velocities exceed permissible velocities for the outlet lining material (Section 3.1), energy dissipation is required.

3.4.5 Design Calculations

A flow chart for performing culvert design calculations is provided in Figure 3-7. A worksheet for performing calculations for standard culvert design is provided in Figure 3-8.

The capacities for standard culvert designs for the following inlet and outlet control charts are found in the Hydraulic Design Series (HDS) 5 (DOT, FHWA, 1985) and duplicated as the figures shown below:

	Figure Numbers	
Standard Culvert Type	**Inlet Control**	**Outlet Control**
Circular Concrete Pipe	3-9	3-10
Circular Corrugated Metal Pipe (CMP)	3-11	3-12
Structural Plate CMP	3-11	3-13
Concrete Box	3-14	3-15
Oval Concrete Pipe–Long Axis Horizontal	3-16	3-17
Oval Concrete Pipe–Long Axis Vertical	3-18	3-19
CMP Arch	3-19	3-20
Structural Plate CMP Arch (18-inch Corner Radius)	3-19	3-21
Circular Pipe with Beveled Ring	3-22	3-10 or 3-12
Concrete Pipe Arch	3-23	3-24
HDPE	3-11	3-12

Metal and HDPE culverts are not permitted except by special allowance of the Public Works Department. The figures shown above are provided only for reference. Note: The HDS-5 charts are valid for headwater depths of about twice the culvert height. For greater depths, a hydrodynamic model will be used.

3.4.5.1 General Procedure

The following procedure, illustrated in Figure 3-7, will be used to select a culvert size with the charts from HDS-5:

1. Perform hydrologic calculations (Section 2).
2. List the following design data (a suggested tabulation form is provided in Figure 3-8, which includes a drawing labeled with design variables):
    a. Design discharge, Q, cfs, with average return period (e.g., Q_{25}). When more than one barrel is used, show Q divided by the number of barrels.
    b. Approximate length, L, of culvert, ft.
    c. Slope of culvert (if grade is given in percent, convert to slope in ft/ft).
    d. Allowable headwater (AHW) depth, ft; that is, the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert (Section 3.4.4.2).
    e. Mean and maximum flood velocities in natural stream (optional).
    f. Types of culvert, including barrel material, barrel cross-sectional shape, and inlet configuration.
3. Determine a trial culvert size by choosing one of the following options:
    a. Arbitrary selection with a flow area similar to the stream at a reasonable depth.
    b. An approximating equation such as:

$$A = \frac{Q}{v} \qquad \text{(Equation 3-14)}$$

where:

A = Culvert area, square ft

Q = Design discharge, cfs

v = Average velocity,-ft/second, pick a target value (e.g, 7 ft per second [fps]) to start

    c. Inlet control nomographs for the culvert type selected (appropriate figures in this section). A trial size is determined by assuming HW/D, e.g. HW/D = 1.5, and using the given Q.

    If the trial size selected is larger than available standard culvert sizes or its use is prohibited by other physical limitations (such as limited embankment height), multiple culverts may be used by dividing the discharge equally between the number of barrels. It also may be possible to consider raising the embankment height or using arch pipe and box culverts with width greater than the height.

4. Find inlet and outlet control headwater, HW, depths for the trial culvert size as follows:

   a. For inlet control, perform the following calculations (Section 3.4.5.2 for additional details):

      (1) Use an appropriate inlet control chart from this chapter or HDS-5 and the trial size from Step 3 to find the HW. Tailwater, TW, conditions are neglected in this determination. HW is found by multiplying HW/D, obtained from the nomographs, by the height of culvert, D.

      (2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control, before computing HW for outlet control.

   b. For outlet control, perform the following calculations (Section 3.4.5.3 for additional details):

      (1) Approximate the depth of TW, ft, above the invert at the outlet for the design flood condition in the outlet channel.

      (2) If the TW elevation determined above is equal to or greater than the top of the culvert at the outlet, set design tailwater, DTW, equal to TW and find HW, using the following equation and the appropriate outlet control nomograph from this section or HDS-5:

$$HW = H + DTW - LS_o \qquad \text{(Equation 3-15)}$$

where:

HW = Headwater depth for outlet control, ft

H = Total head loss, obtained from the appropriate outlet control nomograph from this section or HDS-5, ft

DTW = Design tailwater, ft

L = Barrel length, ft

S_0 = Barrel slope, ft/ft

      (3) If the TW elevation determined above is less than the top of the culvert at the outlet, find HW using Equation 3-15, except that DTW is the greater of the following two parameters:

$$h_o = \frac{d_c + D}{2} \qquad \text{(Equation 3-16)}$$

or

TW

where:

h_o = Equivalent hydraulic elevation at outlet, ft

d_c = Critical depth, ft (from HDS-5 charts, duplicated as figures listed below). Note: d_c cannot exceed D.

D = Height of culvert opening, ft

TW = Downstream tailwater elevation, ft

For standard culvert design, the following critical depth charts, found in HDS-5 and duplicated as the figures shown below, are required culvert capacity calculations:

Culvert Type	Figure
Rectangular	3-25
Circular	3-26
Oval – Long Axis Vertical	3-27
Oval – Long Axis Horizontal	3-28
Standard CMP Arch	3-29
Structural Plate CMP Arch	3-30
Concrete Pipe Arch	3-31, 3-32

Note: Headwater depth determined for this condition becomes increasingly less accurate as the headwater computed by this method falls below the value:

$$HW \leq D + \left(1 + k_e \frac{v^2}{2g}\right) \qquad \text{(Equation 3-17)}$$

where:

D = Height of culvert opening, ft

k_e = Entrance loss coefficient

v = Average velocity of flow, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

5. Compare the HW values from Step 4a (inlet control) and Step 4b (outlet control). The higher HW governs and indicates the type of flow control existing under the given conditions for the trial size and inlet configuration selected.

6. If outlet control governs and the HW is higher than the acceptable AHW, select a larger trial size and find HW as instructed under Step 4b. (Inlet control does not need to be

checked, because the smaller size should be satisfactory for this control, as determined under Step 4a.)

7. If desired, select an alternate culvert type or shape and determine size and HW by the above procedure.

8. If the culvert operates under inlet control, design a tapered inlet following procedures in HDS-5, if an improved inlet is desirable.

9. If roadway overtopping occurs, calculate the capacity following the procedures in Section 3.4.5.4.

10. If storage routing is considered important, follow the procedures in Section 4.4.

11. Compute outlet velocities for the culvert size and types to be considered in selection:

    a. If outlet control governs in Step 5, calculate the outlet velocity using Equation 3-6. If d_c or TW is less than the height of the culvert barrel, use the cross-sectional area corresponding to d_c or TW depth, whichever gives the greater area of flow. The total cross-sectional area, A, of the culvert barrel should not be exceeded.

    b. If inlet control governs in Step 5, the outlet velocity can be assumed to equal the mean velocity for open channel flow conditions in the barrel, computed by Manning's Equation (Section 3.1) for the rate of flow, barrel size, roughness, and slope of culvert selected.

12. Determine to what extent channel protection is required downstream of the outlet. Properly sized rip-rap is required as a minimum.

13. Record final selection of culvert with size, type, required headwater, outlet velocity, channel protection, and economic justification in the spreadsheet in Figure 3-8.

3.4.5.2 Inlet Control

Inlet control charts are presented in this manual for the following standard culvert types:

Standard Culvert Type	Figure Number for Inlet Control Charts
Circular Concrete Pipe	3-9
Circular Corrugated Metal Pipe (CMP)	3-11
Structural Plate CMP	3-11
Concrete Box	3-14
Oval Concrete Pipe – Long Axis Horizontal	3-16
Oval Concrete Pipe – Long Axis Vertical	3-18
CMP Arch	3-19
Structural Plate CMP Arch (18-inch Corner Radius)	3-19
Circular Pipe with Beveled Ring	3-22
Concrete Pipe Arch	3-23
HDPE	3-11

The following three types of calculations can be performed using the inlet control charts:

1. To determine the HW, given Q and size for selected culvert type and inlet configuration:
   a. Use a straightedge to connect the culvert diameter or height, D, scale and the discharge, Q, scale, or Q/B for box culverts. Note the point of intersection of the straightedge on the HW/D scale marked (1).
   b. If the HW/D scale marked (1) represents the inlet configuration used, read HW/D on this scale. When either of the other two inlet configurations listed on the nomograph is used, extend the point of intersection obtained in Step 1b horizontally to scale (2) or (3) and read HW/D.
   c. Compute the HW by multiplying HW/D by D.

   Note: The approach velocity is assumed to be zero by this procedure. If the approach velocity is considered significant, subtracting the velocity head can decrease the HW.

2. To determine Q per barrel, given HW and size for selected culvert type and inlet configuration:
   a. Compute HW/D for given conditions.
   b. Locate HW/D on the scale for appropriate inlet configuration. If scale (2) or (3) is used, extend the HW/D point horizontally from scale (1).
   c. Use a straightedge to connect the point on HW/D scale (1) obtained above with the culvert size on the far left scale. Read Q or Q/B at the intersection of this line with the middle discharge scale.
   d. If Q/B is read in Step 2c, multiply by B (span of box culvert) to find Q.

e. To determine the culvert size, given Q, AHW, and type of culvert with desired inlet configuration:
   a. Using a trial size, compute HW/D.
   b. Locate HW/D on the scale for the appropriate inlet configuration. If scale (2) or (3) is used, extend the HW/D point horizontally to scale (1).
   c. Use a straightedge to connect the point on HW/D scale (1) obtained above with the given discharge on the middle scale. Read diameter, height, or size of culvert required at the intersection of this line with the culvert size scale on the far left.
   d. If D is not as originally assumed, repeat procedure with a new D.

3.4.5.3 Outlet Control

Outlet control charts are presented in this manual for the following standard culvert types:

Standard Culvert Type	Figure Number for Inlet Control Charts
Circular Concrete Pipe	3-10
Circular Corrugated Metal Pipe (CMP)	3-12
Structural Plate CMP	3-13
Concrete Box	3-15
Oval Concrete Pipe – Long Axis Horizontal	3-17
Oval Concrete Pipe – Long Axis Vertical	3-17
CMP Arch	3-20
Structural Plate CMP Arch (18-inch Corner Radius)	3-21
Circular Pipe with Beveled Ring	3-10 or 3-12
Concrete Pipe Arch	3-24
HDPE	3-12

The following steps outline the use of the outlet control charts:

1. To determine H for a given culvert and Q:

   a. Locate the appropriate nomograph for the type of culvert selected. Find the entrance loss coefficient, k_e , for the inlet configuration using data from Table 3-10.

   b. Begin nomograph solution by locating the proper starting point on the length scale:

      (1) If the n value of the nomograph corresponds to that of the culvert being used, select the proper length curve for an assigned k_e value and locate the starting point at the given culvert length. If a curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph chart, see (3) below.

      (2) For the n value of the nomograph and an intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two scales in proportion to the k_e values.

      (3) For a different roughness coefficient, n_1, than that of the chart n, use the length scales shown with an adjusted length, L_1, calculated as:

$$L_1 = L \frac{{n_1}^2}{n} \qquad \text{(Equation 3-18)}$$

(Step 2 for n values)

c. Use a straightedge to connect the point on the length scale to the size of the culvert barrel and mark the point of crossing on the turning line. See Step 3 for size considerations for a rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect with the given discharge rate. For multiple barrels, divide Q by the number of barrels before using the nomograph. Read head in ft on the R scale located on the far right. For values beyond the limit of the printed scales, find H by solving the equation:

$$H = \left[1 - k_e + \frac{29n^2 L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad \text{(Equation 3-19)}$$

where:

H = Total head loss, or the elevation difference between HW and DTW, ft (Figure 3-9 for sketch)

k_e = Entrance loss coefficient (Table 3-10)

n = Manning's roughness coefficient (appropriate culvert nomograph)

L = Barrel length, ft

R = Hydraulic radius of the culvert, ft

v = Average velocity of flow, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

2. Values of n, which are the basis for the nomographs, are presented on each nomograph.

3. To use the box culvert nomograph (Figure 3-15) for full flow for other than the configurations shown:

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

   b. Use a straightedge to connect the proper point (Step 1) on the length scale to the barrel area and mark the point on the turning line. Note that the area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with the area of square boxes ensures that it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions, use Equation 3-18 for more accurate results

3.4.5.4 Roadway Overtopping

The overall performance curve for roadway overtopping can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.
3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and the following equation to calculate flow rates across the roadway:

$$Q_o = C_d L HW_r^{1.5} \qquad \text{(Equation 3-20)}$$

where:

Q_o = overtopping flow rate, cfs

$C_d = K_t\, C_r$ = Discharge coefficient

K_t = Submergence factor (from Figure 3-33, Part C)

C_r = Unsubmerged discharge coefficient (from Figure 3-33, Part A or B)

L = Length of roadway crest, ft

HW_r = Upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown, ft

4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Example 3.2: Sizing Pipes for Drainage System

Pipe sizing starts at the most distant hydraulic point, or Subbasin 1. As stated in Section 3.2.1.4, the minimum pipe size shall be 15 inches. Begin with the most upstream unit, pipe segment 1 (PS1) to start estimating the flow to be carried by the storm drain pipes. Pipe segment 5 will be sized the same way. The flow rate in the pipe segment can be determined from Manning's equation (Equation 3-5), assuming that the pipe will be sized to operate under gravity flow. In this manner, each pipe section serving an individual sub basin (pipe segments 1 through 9) will need a capacity to handle 4.3 cfs. The following table was computed from Equation 3-6:

Q = 0.465/n * Ds^(8/3) * So^(1/2)

So =	0.01	ft/ft
n =	0.012	RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	7.0
18	1.50	11.4
24	2.00	24.6
30	2.50	44.6
36	3.00	72.5

Therefore, each pipe segment from the *cul-de-sacs* will be 15 inches in diameter. Pipe segment 10 must carry the peak discharge from pipe sections 1 and 5, or 8.6 cfs. Using the results from the above table, a pipe size of 18 inches in diameter is required.

As one moves further downstream, the flow will be somewhat attenuated by the storage in the pipe. Because this is a small area, flow in pipe segment 11 will be based on the Rational Method, calculated as follows:

t_c = 11 minutes + 0.95 minutes = 11.95 minutes

This accounts for the travel time through Subbasin 1 (and 5) and the travel time in pipe segment 10 calculated as time = distance/velocity.

The intensity from interpolation of Table 2-1 values is 7.0 inches per hour. The peak discharge from Equation 2-7, with C = 0.6 and area = 4 acres is 16.7 cfs.

$$Q_T = 0.6 * 7.0 * 4 = 16.7 \text{ cfs} \qquad \text{(from Equation 2-7)}$$

Using the table above, the pipe diameter is sized at 24 inches. This process continues for sizing each of the pipes segments (12 through 14):

Pipe segment 12 = 30 inches

Pipe segment 13 = 36 inches

Pipe segment 14 = 36 inches

Alternatively, a storm water modeling program could be used to evaluate various pipe sizes. The output from the program needs to demonstrate the input data and assumptions.

Recommended Manning "n" Values For Channels With Bare Soil And Vegetative Linings

Channel Linings	Description	Design "n"
Bare Earth, Fairly Uniform	Clean, recently completed	0.022
Bare Earth, Fairly Uniform	Short grass and some weeds	0.028
Dragline Excavated	No Vegetation	0.030
Dragline Excavated	Light Brush	0.040
Channels not Maintained	Dense weeds to flow depth	0.10
Channels not Maintained	Clear bottom, brush sides	0.08
Maintained Channels or Sodded Ditches	Good stand, well maintained 2 – 6"	0.06*
Maintained Channels or Sodded Ditches	Fair stand, length 12 – 24"	0.20*

Note:
*Decrease 30 percent for flows > 0.7-foot depth (max. flow depth 1.5-feet)

Recommended Manning "n" Values For Channels With Rigid And Semi-Rigid Lining

Channel Linings	Description	Design "n"
Concrete Paved	Broom	0.016
Concrete Paved	"Roughened" – Standard	0.020
Concrete Paved	Gunite	0.020
Concrete Paved	Over Rubble	0.023
Asphalt Concrete	Smooth	0.013
Asphalt Concrete	Rough	0.016
Riprap	Placed	0.030
Riprap	Dumped	0.035
Gabions		0.028

TABLES 3-1& 3-2
RECOMMENDED MANNING "n" VALUES FOR CHANNELS WITH BARE SOIL AND VEGETATIVE LININGS AND FOR CHANNELS WITH RIGID LININGS

Allowable Velocities for Different Soils

Soil Type	Allowable Velocity (ft/sec)
Sandy Loam	1.75
Silt Loam	2.00
Firm Loam	2.50
Stiff Clay	3.75

Maximum Velocities for Various Lining Types

Lining Type	Maximum Velocity (ft/sec)
Grass with Mulch	Bare Soil (Table 3-3)
Sod	4
Riprap (Rubble) (Ditch Lining)	6
Geotextile Grid	4-8*
Rigid	10**

Notes:
*Varies with grid
**Higher velocities acceptable with provisions for energy dissipation

TABLES 3-3 & 3-4
ALLOWABLE VELOCITIES FOR DIFFERENT SOILS AND MAXIMUM VELOCITIES FOR VARIOUS LINING TYPES

d_2/d_1 = Ratio of larger pipe to smaller pipe

v_1 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_1, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Reference: Brater and King (1976).

TABLE 3-5
VALUES OF K_2 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN EXPANSION IN PIPES, FROM THE FORMULA $H_2 = K_2 (V_1^2/2g)$

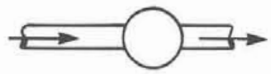
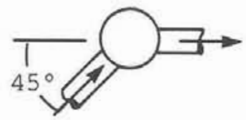
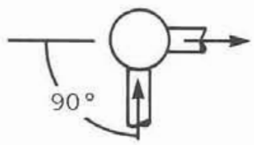
d_2/d_1 = Ratio of larger to smaller diameter

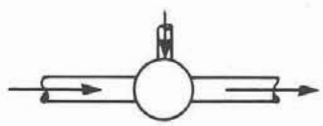
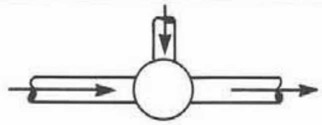
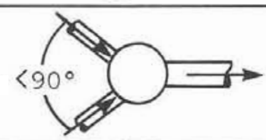
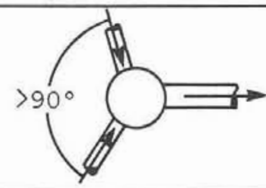
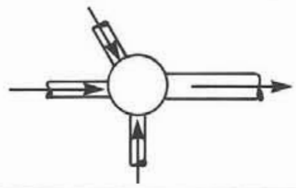
v_2 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_2, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04.	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41.	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Reference: Brater and King (1976).

TABLE 3-6
VALUES OF K_3 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION IN PIPES, FROM THE FORMULA $H_3 = K_3 (V_2^2/2g)$

Single Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline only with no bend at junction 	0.5
Trunkline only with 45° bend at junction 	0.6
Trunkline only with 90° bend at junction 	0.8

Multiple Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline with one small lateral 	0.6
Trunkline with one large lateral 	0.7
Two roughly equivalent entrance lines with angle of <90° between lines 	0.8
Two roughly equivalent entrance lines with angle of >90° between lines 	0.9
Three or more entrance lines 	1.0

Reference: Golding (1987).

Note: Above values of K are to be used to estimate energy or head losses through surcharged junctions/manholes in pressure flow portions of a storm sewer system. The energy loss equation is

$$h_j(ft)=K\,\frac{[v(ft/sec)]^2}{64.4}$$

with v = larger velocity in main entrance or exit line of junction/manhole.

TABLE 3-7
HEAD LOSS COEFFICIENTS FOR MANHOLES/JUNCTIONS

Roadway Slope (%)	Allowable Flow (cfs)	Roadway Slope (%)	Allowable Flow (cfs)
0.50	1.00	6.50	3.59
0.75	1.22	6.75	3.66
1.00	1.41	7.00	3.73
1.25	1.58	7.25	3.80
1.50	1.73	7.50	3.86
1.75	1.86	7.75	3.92
2.00	1.99	8.00	3.99
2.25	2.11	8.25	4.05
2.50	2.23	8.50	4.11
2.75	2.34	8.75	4.17
3.00	2.44	9.00	4.23
3.25	2.54	9.25	4.29
3.50	2.64	9.50	4.34
3.75	2.73	9.75	4.40
4.00	2.82	10.00	4.46
4.25	2.91	10.25	4.51
4.50	2.99	10.50	4.57
4.75	3.07	10.75	4.62
5.00	3.15	11.00	4.67
5.25	3.23	11.25	4.73
5.50	3.31	11.50	4.78
5.75	3.38	11.75	4.83
6.00	3.45	12.00	4.88
6.25	3.52		

Notes:
1. Pavement cross slope is 0.0208 ft/ft
2. Manning's n = 0.016
3. Storm water spread is 8 feet

TABLE 3-8
HYDRAULIC CAPACITIES OF AUBURN STANDARD 2.0-FOOT CURB AND GUTTER FOR RESIDENTIAL PAVEMENT SECTIONS

Hydraulic Intake For Inlets

Hydraulic Intake for Inlets

Inlet Type	Grade Consideration	Hydraulic Intake (cfs)
S – Single wing	Continuous	2.2
	Sag	11.1
S – Double wing	Continuous	3.5
	Sag	16.8

The hydraulic intake values are based on a 0.2% slope. To determine the hydraulic intake value for other slopes, use the table below to find the slope, and then multiply the values above by the adjustment factor to determine the hydraulic intake values at the noted slope

Slope	Adjustment Factor
0.2%	1.00
0.5%	0.85
0.75%	0.78
1.0%	0.73
1.5%	0.66
2.0%	0.62
3.0%	0.56
5.0%	0.49
7.0%	0.45
10.0%	0.42

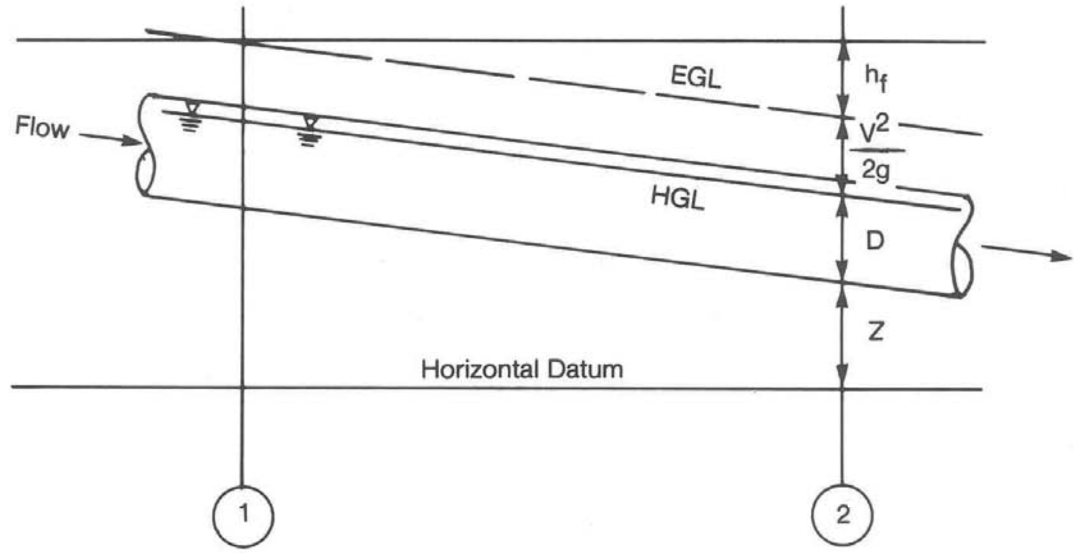
TABLE 3-9
HYDRAULIC INTAKE FOR INLETS

Type of Structure and Design of Entrance	Entrance Coefficient, k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-section conforming to fill slope[a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe or Pipe Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-section conforming to fill slope[a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Reference: USDOT, FHWA, HDS-5 (1985).

[a] "End section conforming to fill slope," made of either metal or concrete, is the section commonly available from manufacturers. From limited hydraulic tests, the sections are equivalent in operation to a headwall in both inlet and outlet control. End sections that incorporate a closed taper in their design have a superior hydraulic performance.

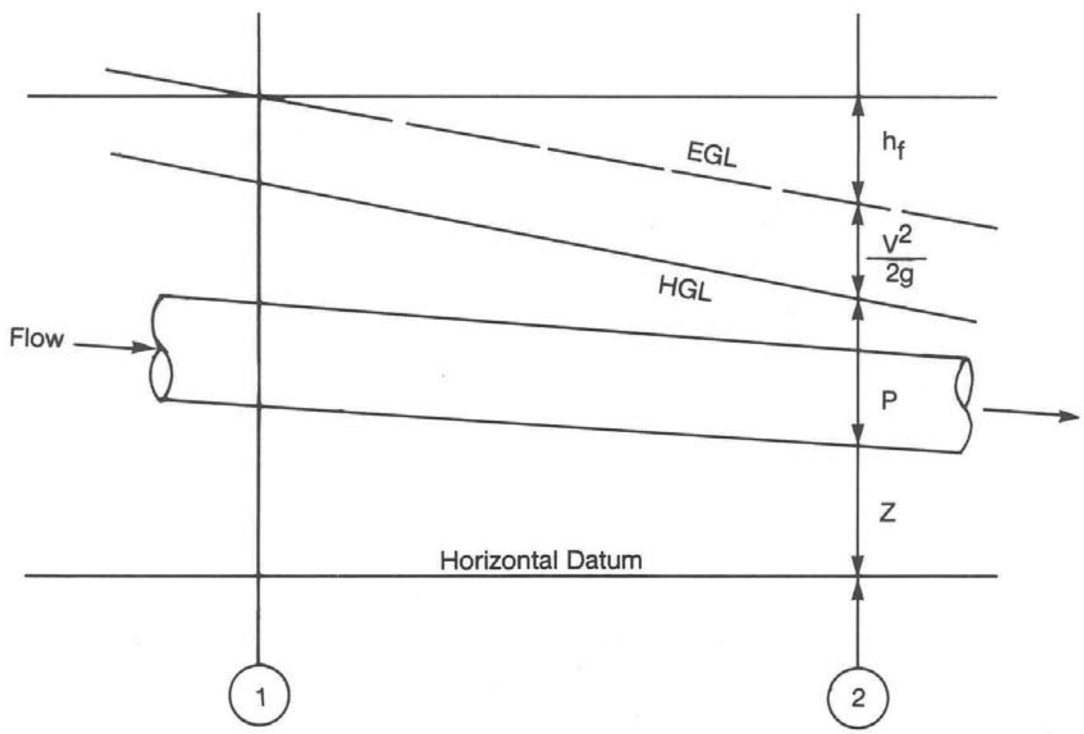
TABLE 3-10
CULVERT ENTRANCE LOSS COEFFICIENTS



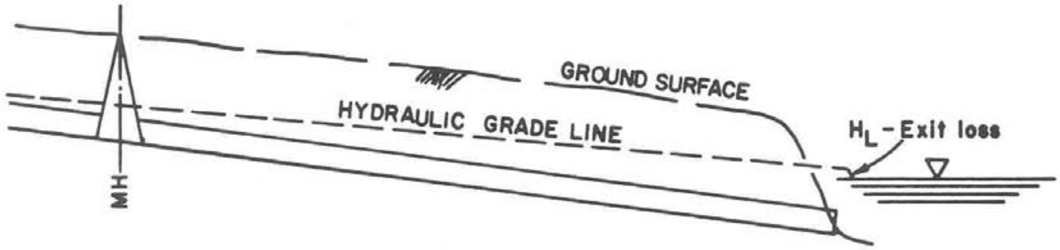
Open Channel Flow In A Closed Conduit

Z = Distance above horizontal datum
D = Depth of flow
P = Pressure head
$\frac{V^2}{2g}$ = Velocity Head
h_f = Friction loss between Section 1 and Section 2
EGL = Energy Grade Line
HGL = Hydraulic Grade Line

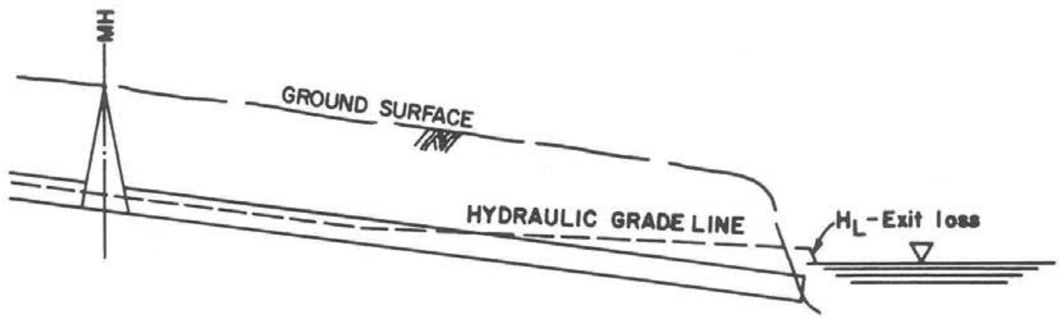
Pressure Flow In A Closed Conduit



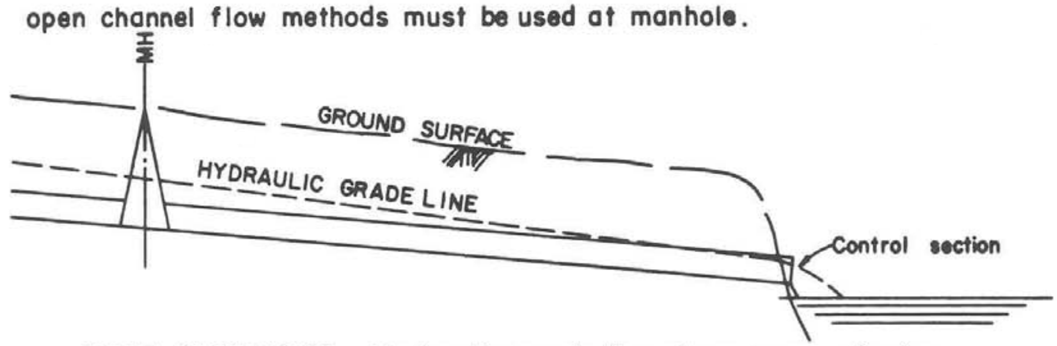
FIGURES 3-1 & 3-2
OPEN CHANNEL FLOW IN A CLOSED CONDUIT AND PRESSURE FLOW IN A CLOSED CONDUIT
Auburn Storm Water Management Manual



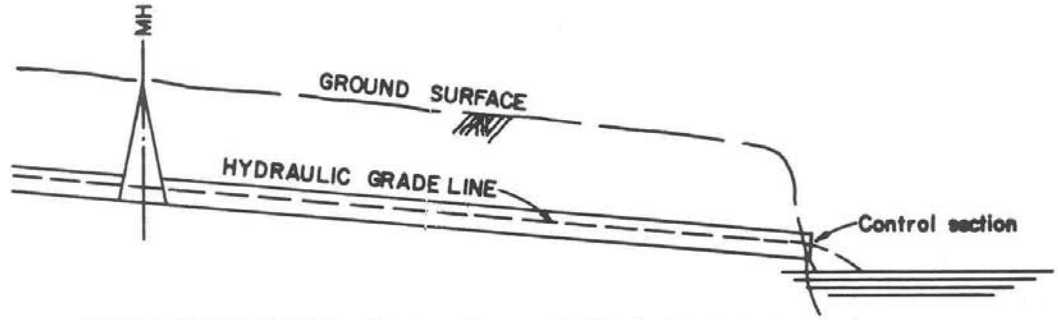
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.



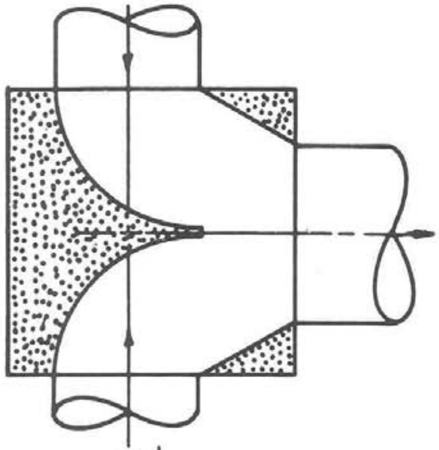
FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



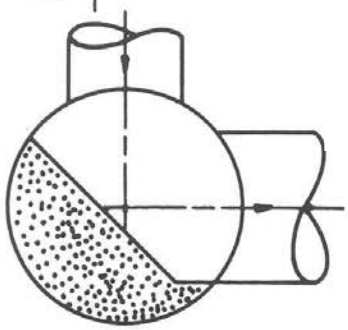
FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

Reference: Wright-McLaughlin Engineers (1969).

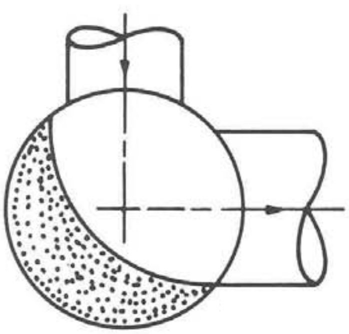
FIGURE 3-3
DETERMINATION OF PRESSURE VS. OPEN CHANNEL FLOW CONDITIONS IN STORM SEWER SYSTEMS



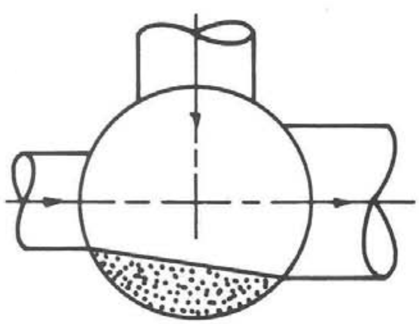
Directly opposed lateral with deflector
(head losses are still excessive with this method, but are significantly less than when no deflector exists.)



Bend with straight deflector



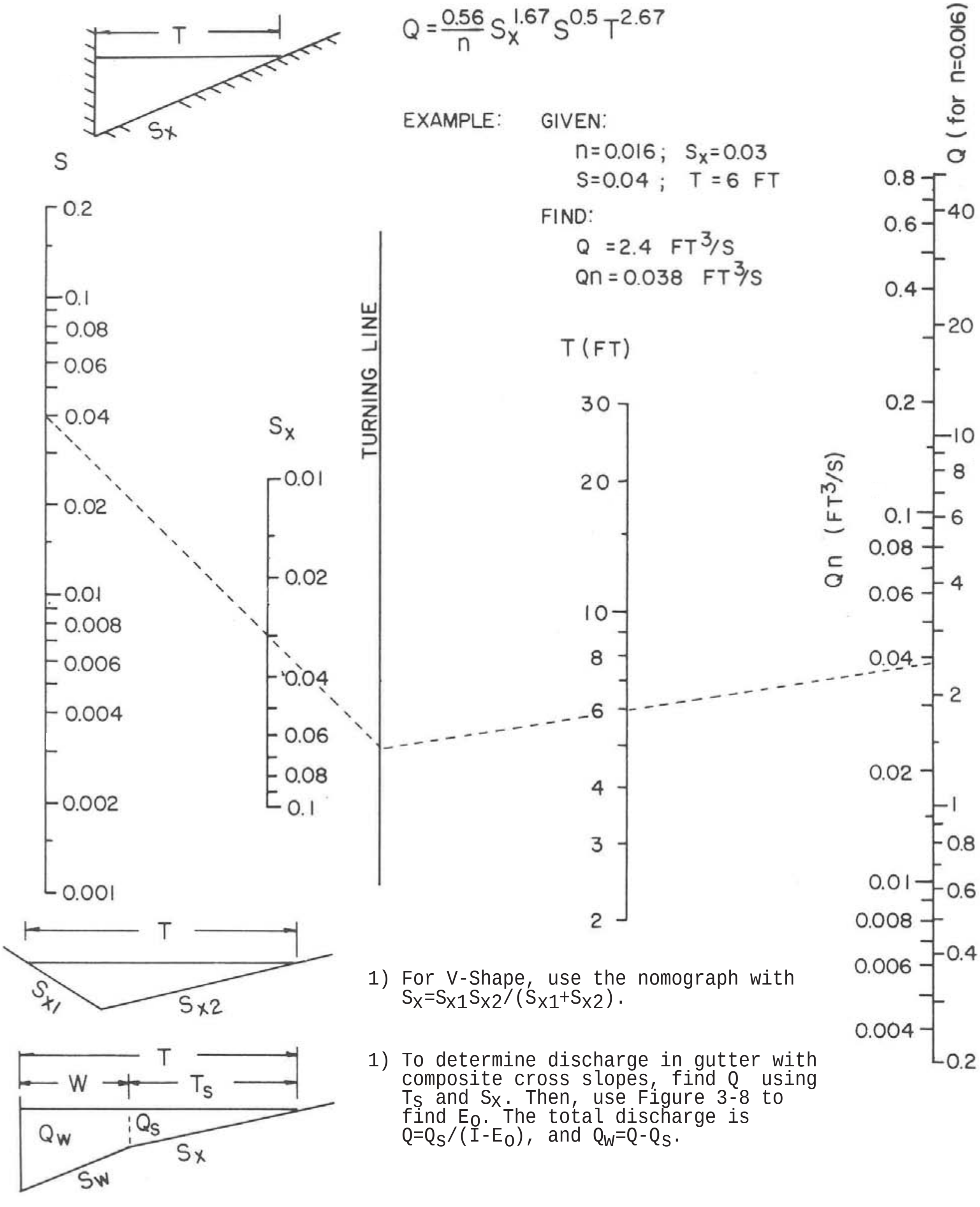
Bend with curved deflector



Inline upstream main & 90° lateral with deflector

Reference: Wright-McLaughlin Engineers (1969).

FIGURE 3-4
EFFICIENT MANHOLE SHAPING
Auburn Storm Water Management Manual

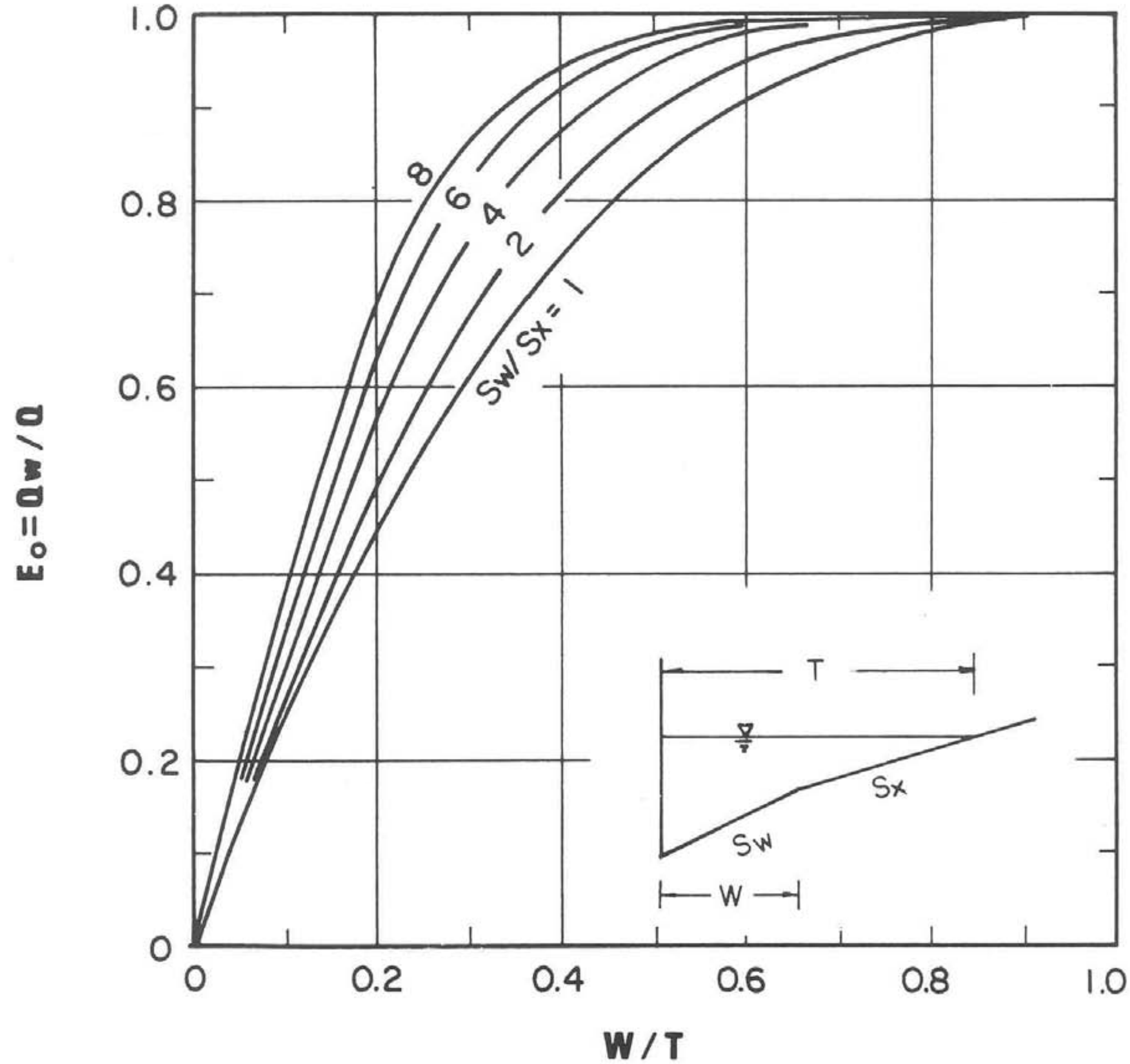


1) For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2}/(S_{x1}+S_{x2})$.

1) To determine discharge in gutter with composite cross slopes, find Q using T_s and S_x. Then, use Figure 3-8 to find E_o. The total discharge is $Q=Q_s/(1-E_o)$, and $Q_w=Q-Q_s$.

Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 3-5
NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTER SECTIONS



Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 3-6
RATIO OF FLOW TO TOTAL GUTTER FLOW
Auburn Storm Water Management Manual

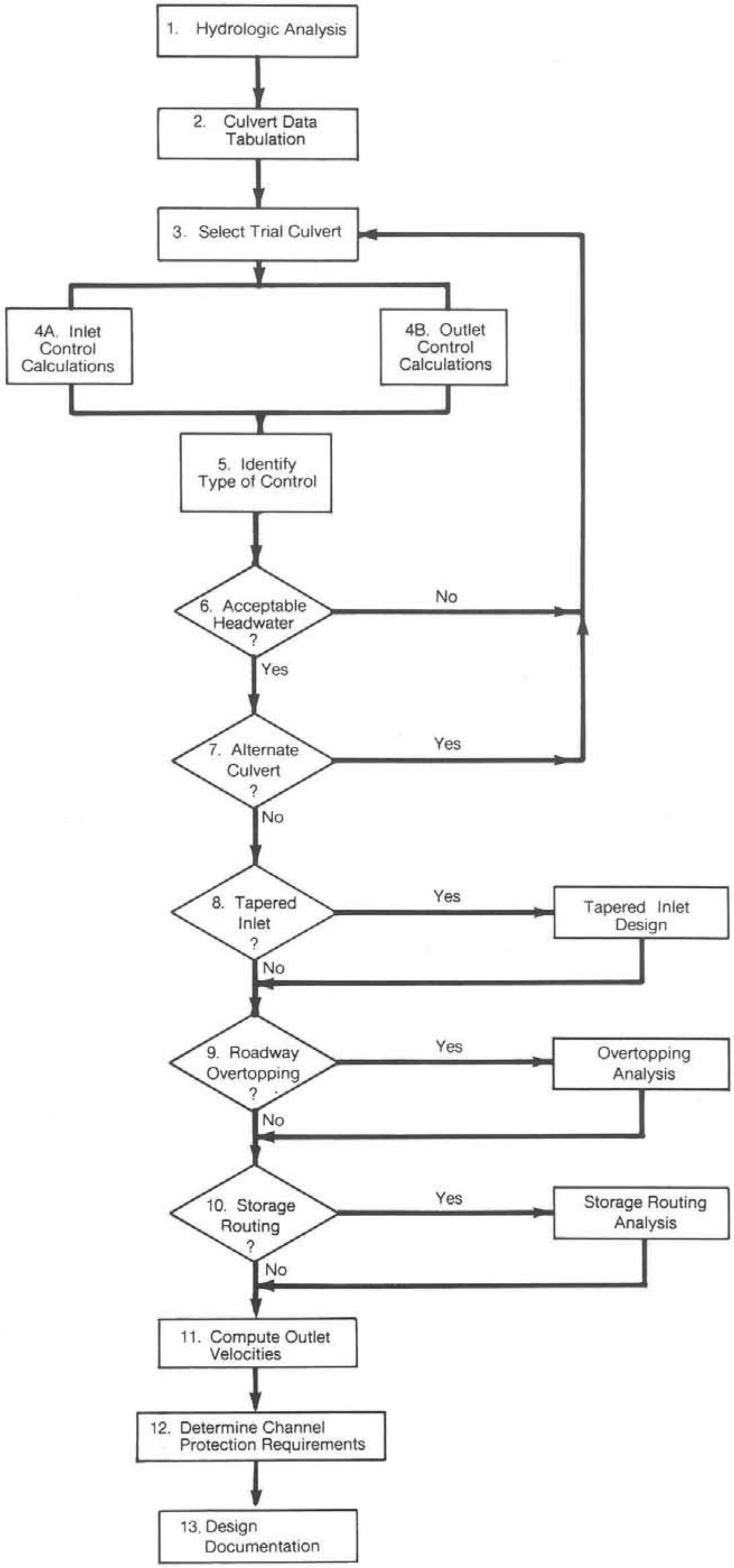


FIGURE 3-7
GENERAL CULVERT DESIGN FLOW CHART
Auburn Storm Water Management Manual

PROJECT: ______________________________

STATION: ______________________

SHEET _______ OF _______

CULVERT DESIGN FORM

DESIGNER / DATE: __________ / __________

REVIEWER / DATE: __________ / __________

HYDROLOGICAL DATA

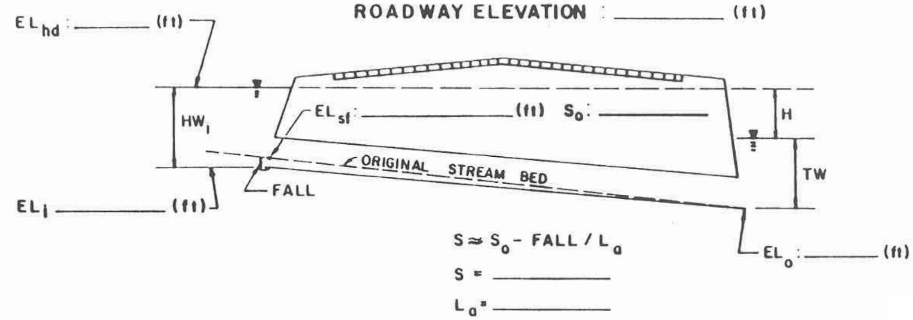
SEE ADD'L SHTS.

- ☐ METHOD: ______________________
- ☐ DRAINAGE AREA: __________ ☐ STREAM SLOPE: __________
- ☐ CHANNEL SHAPE: ______________________
- ☐ ROUTING: __________ ☐ OTHER: __________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: __________ (ft)



$S \approx S_o - FALL / L_a$

S = __________

L_a = __________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______________________

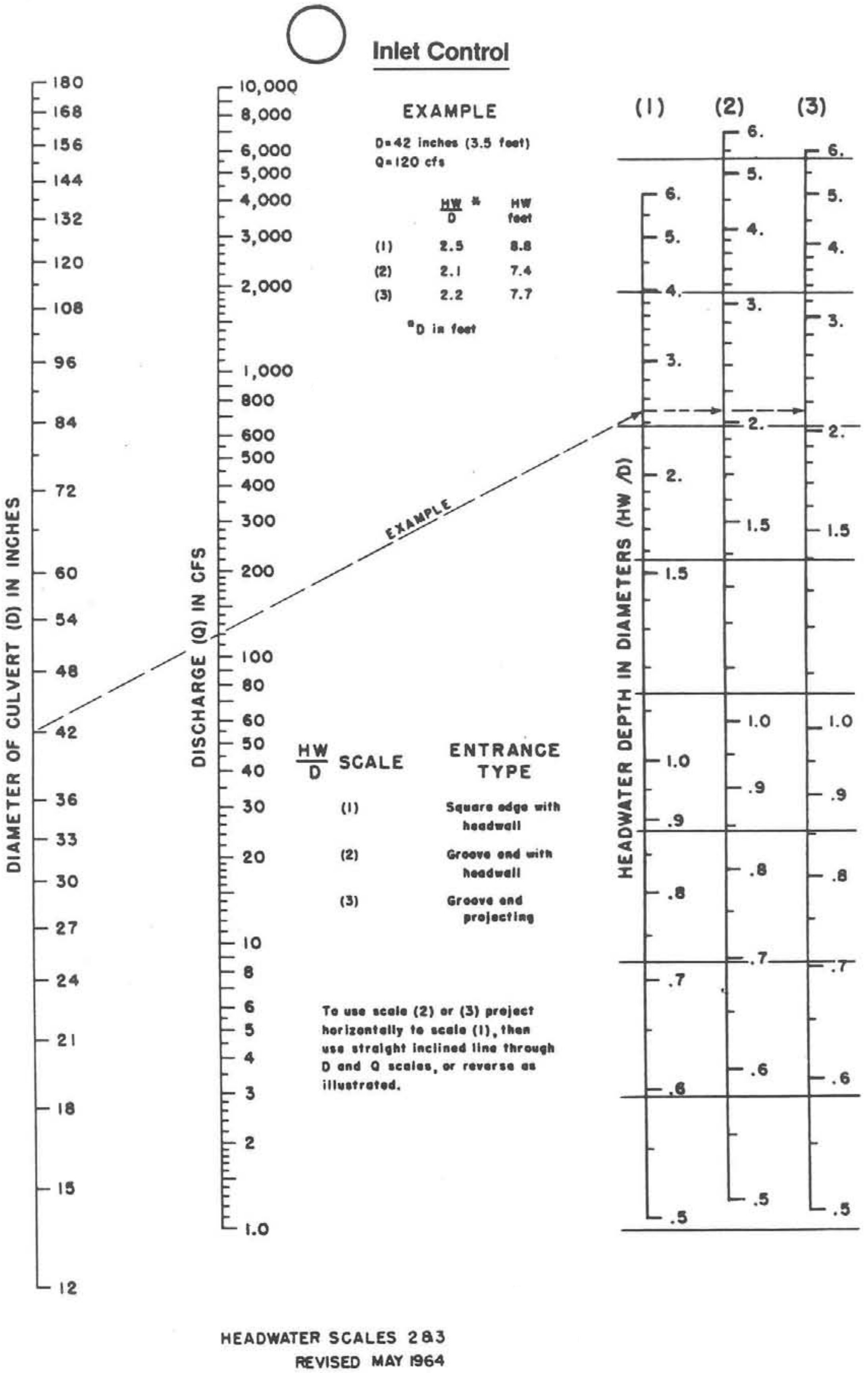
SHAPE: ______________________

MATERIAL: ______________ n: __________

ENTRANCE: ______________________

Reference: USDOT, FHWA, HDS-5 (1985)

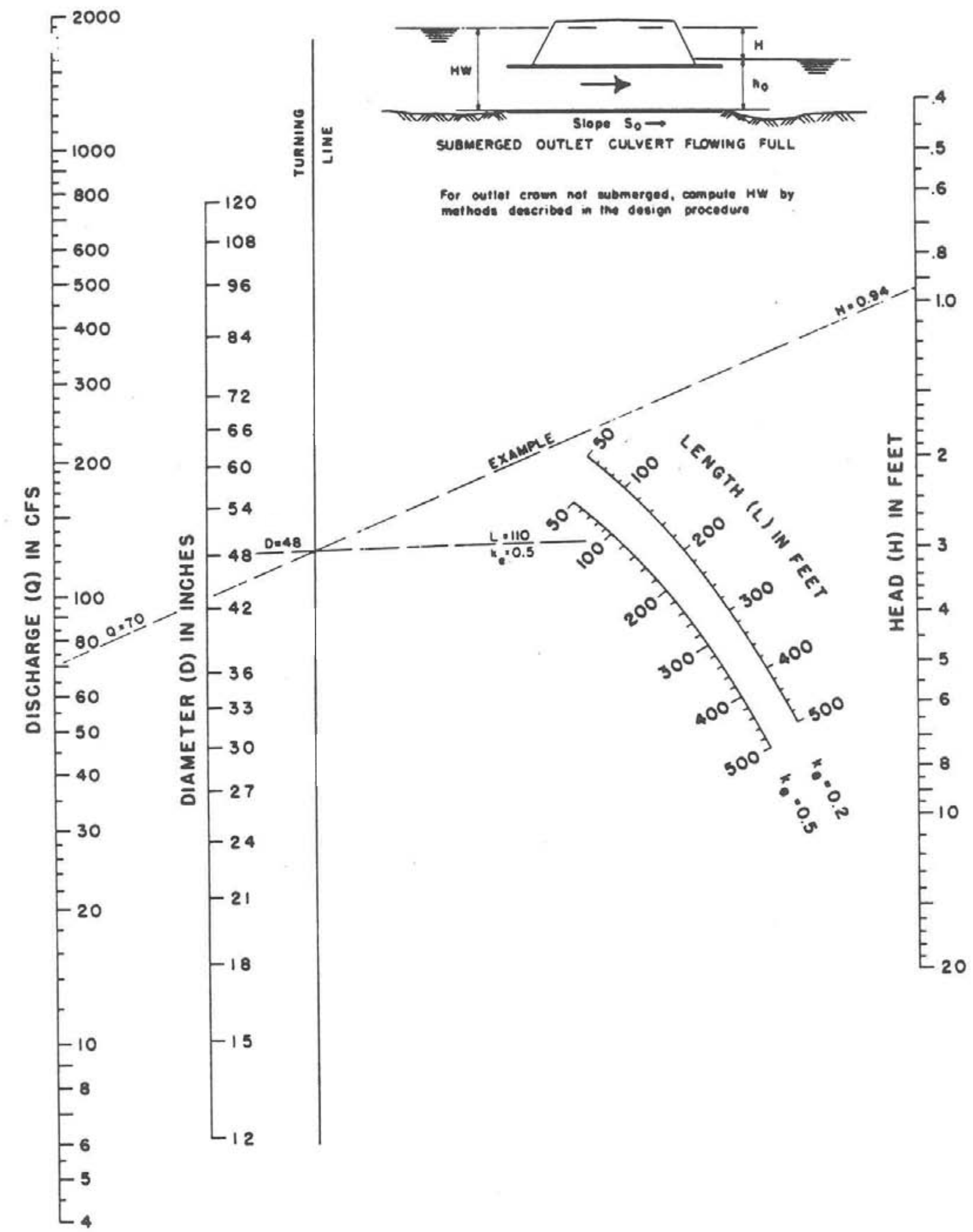
FIGURE 3-8
CULVERT DESIGN FORM



Reference: USDOT, FHWA, HDS-5 (1985).

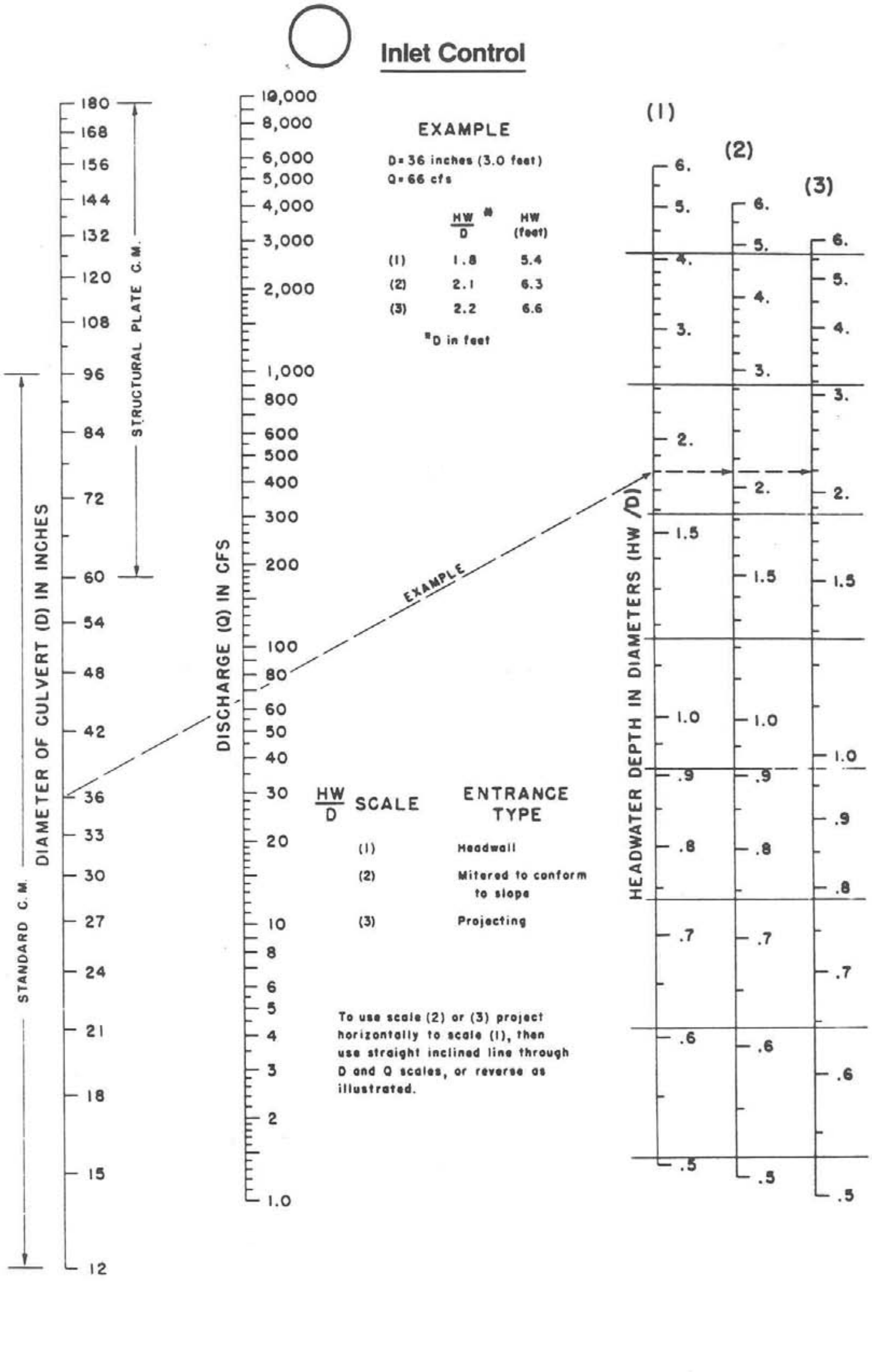
FIGURE 3-9
INLET CONTROL CHART FOR CONCRETE PIPE CULVERTS

Outlet Control—Flowing Full, n = 0.012



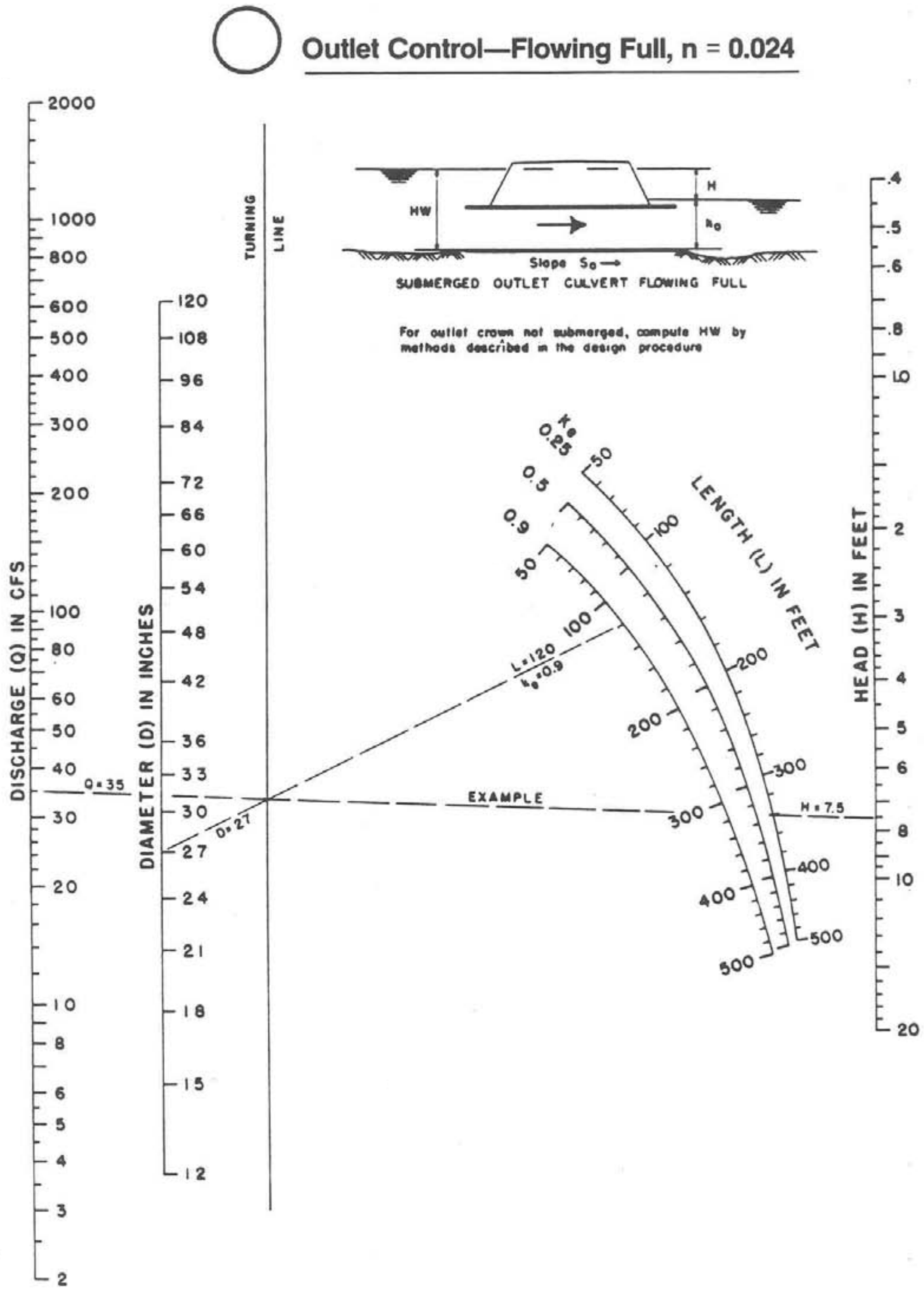
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-10
OUTLET CONTROL CHART FOR CIRCULAR CONCRETE
Auburn Storm Water Management Manual



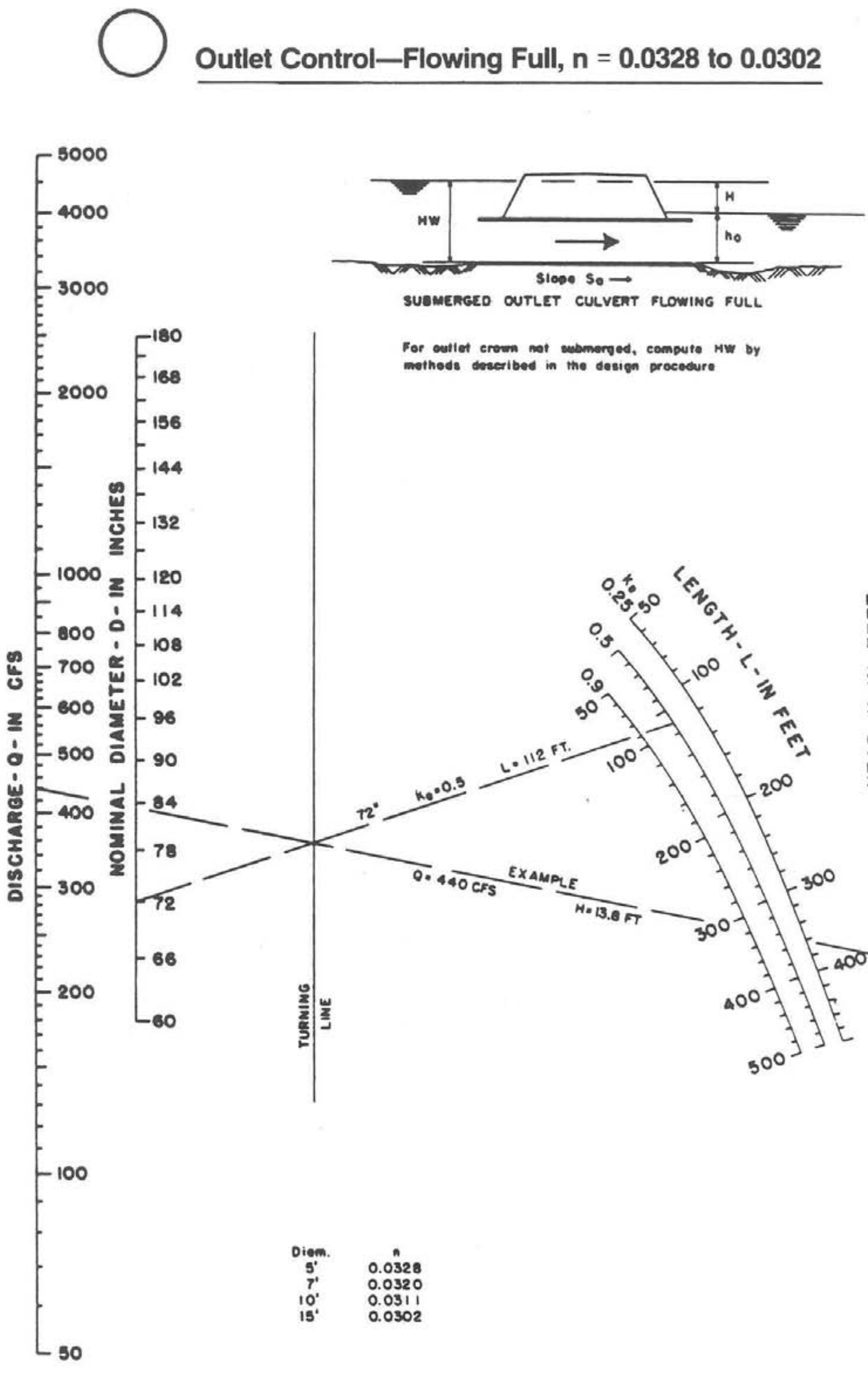
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-11
INLET CONTROL CHART FOR CIRCULAR CMP CULVERTS
Auburn Storm Water Management Manual



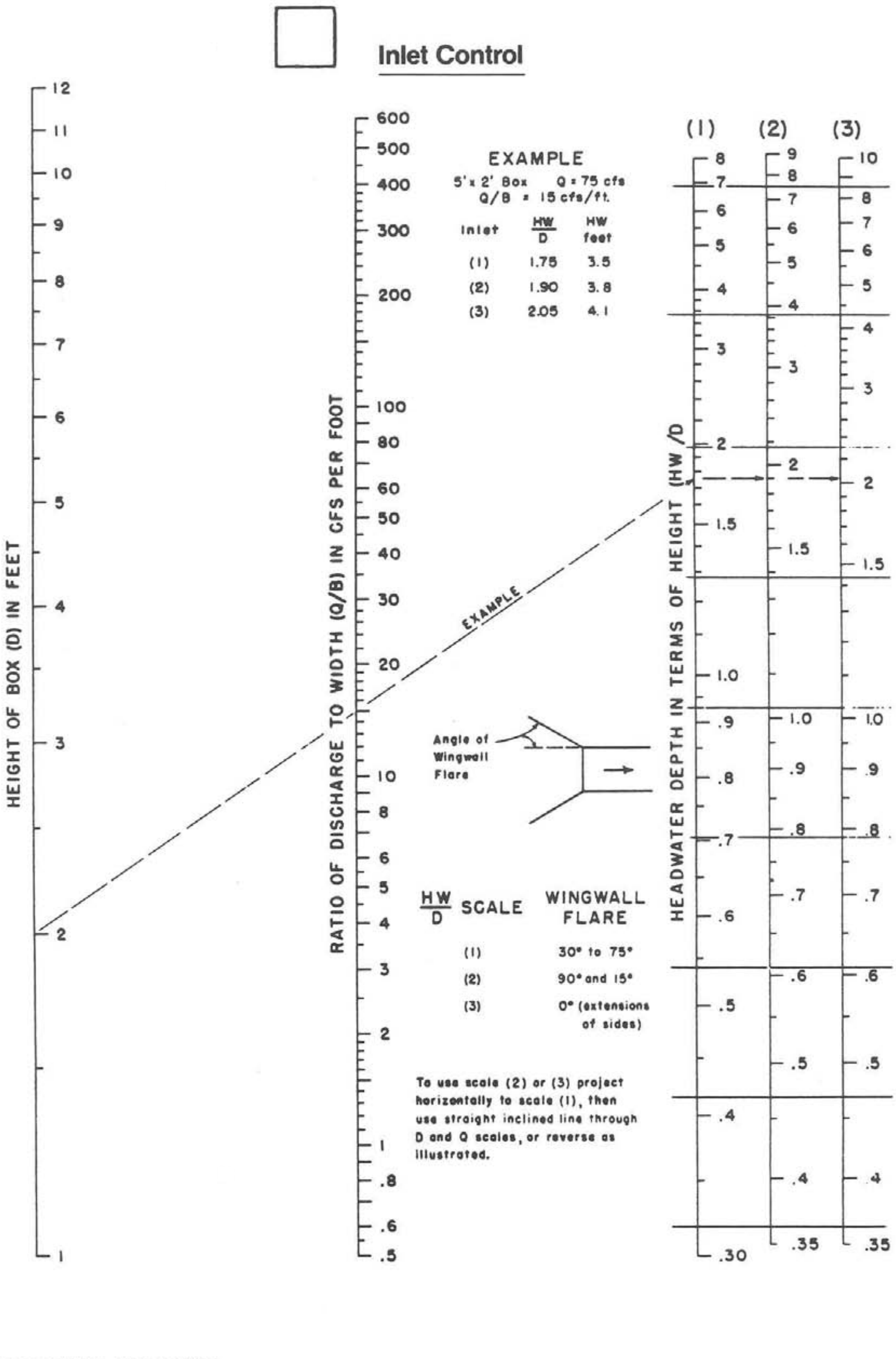
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-12
OUTLET CONTROL CHART FOR CIRCULAR CMP CULVERTS FLOWING FULL



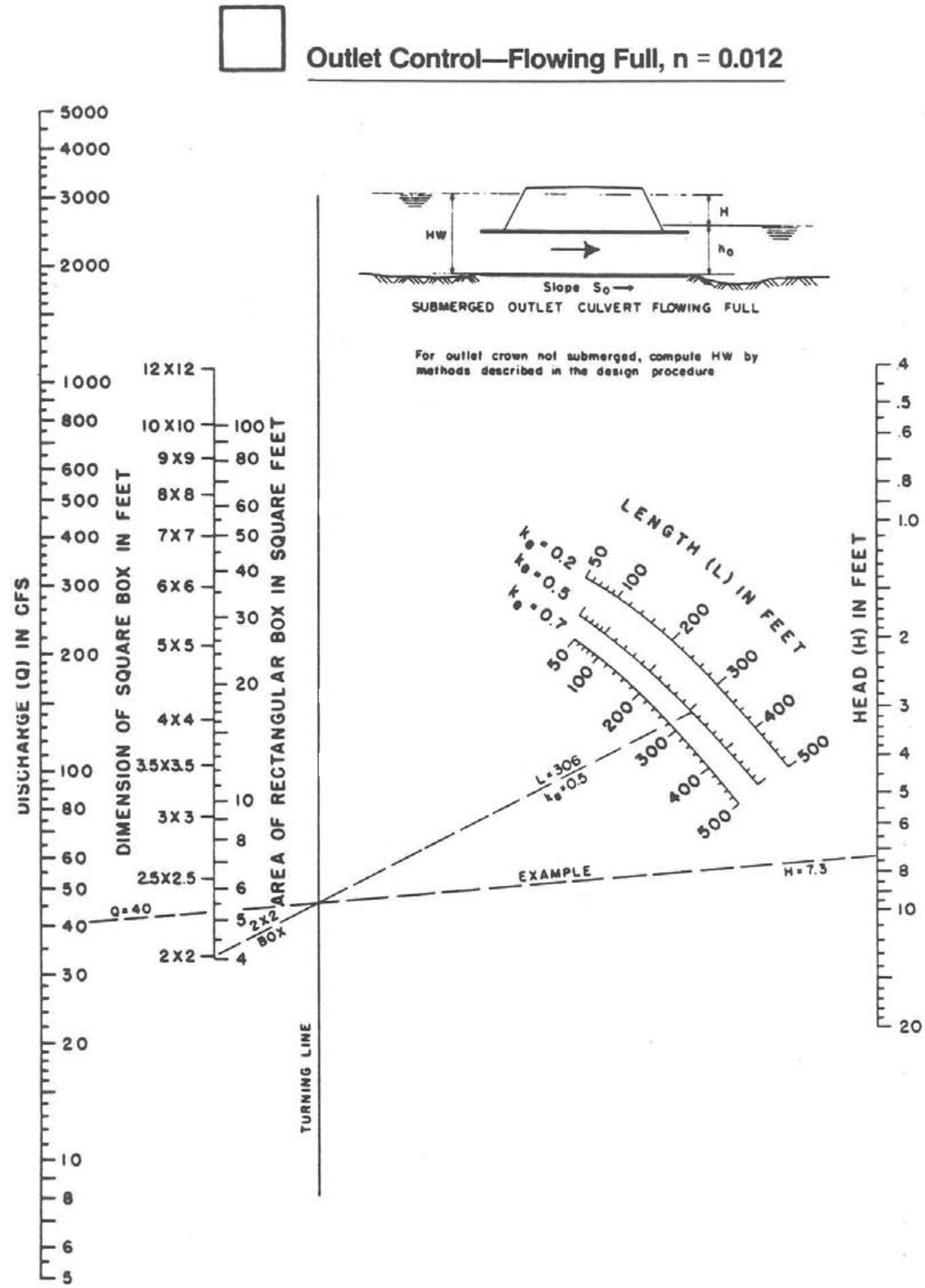
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-13
OUTLET CONTROL CHART FOR STRUCTURAL PLATE CMP CULVERTS FLOWING FULL



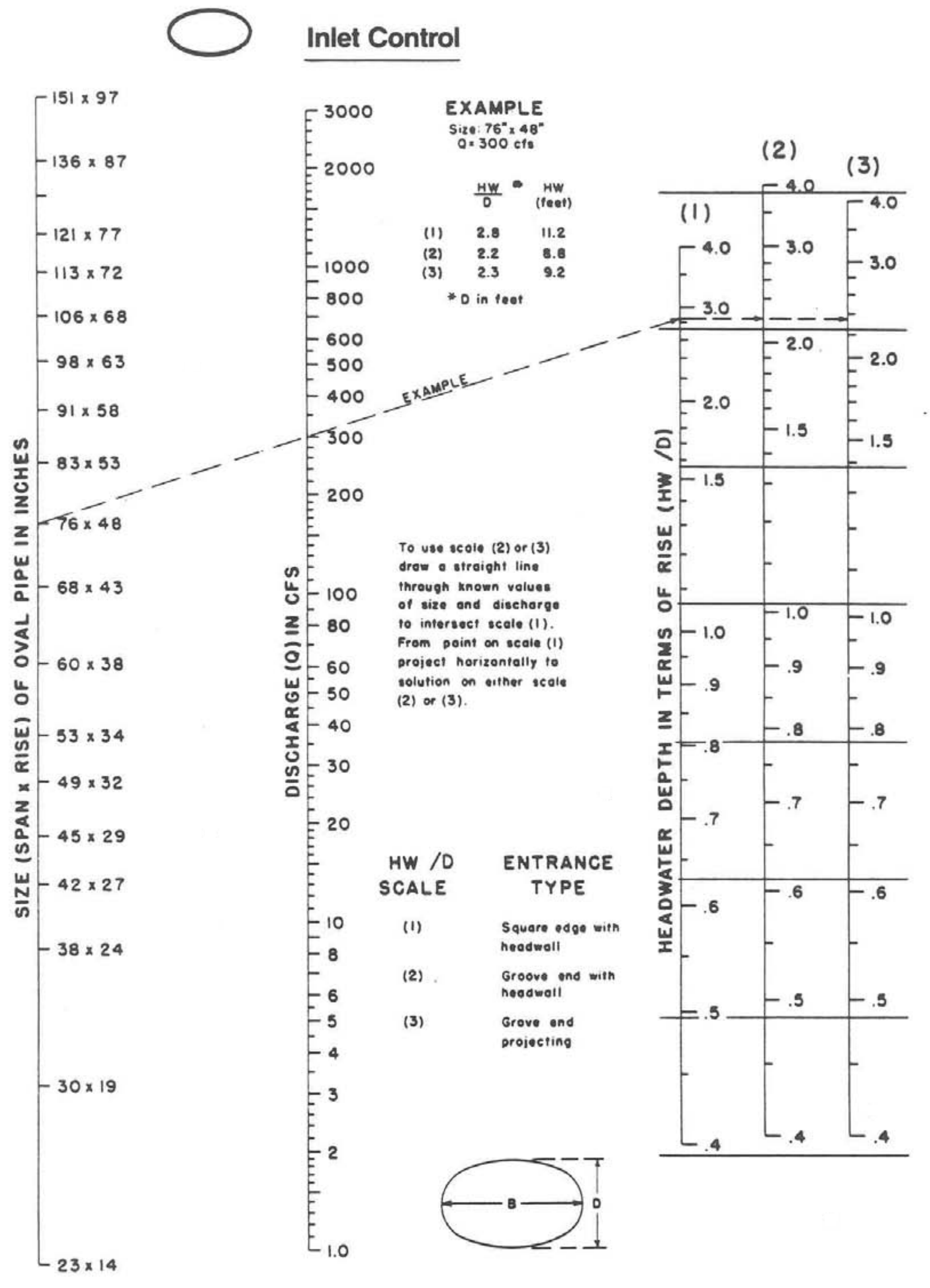
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-14
INLET CONTROL CHART FOR CONCRETE BOX CULVERTS



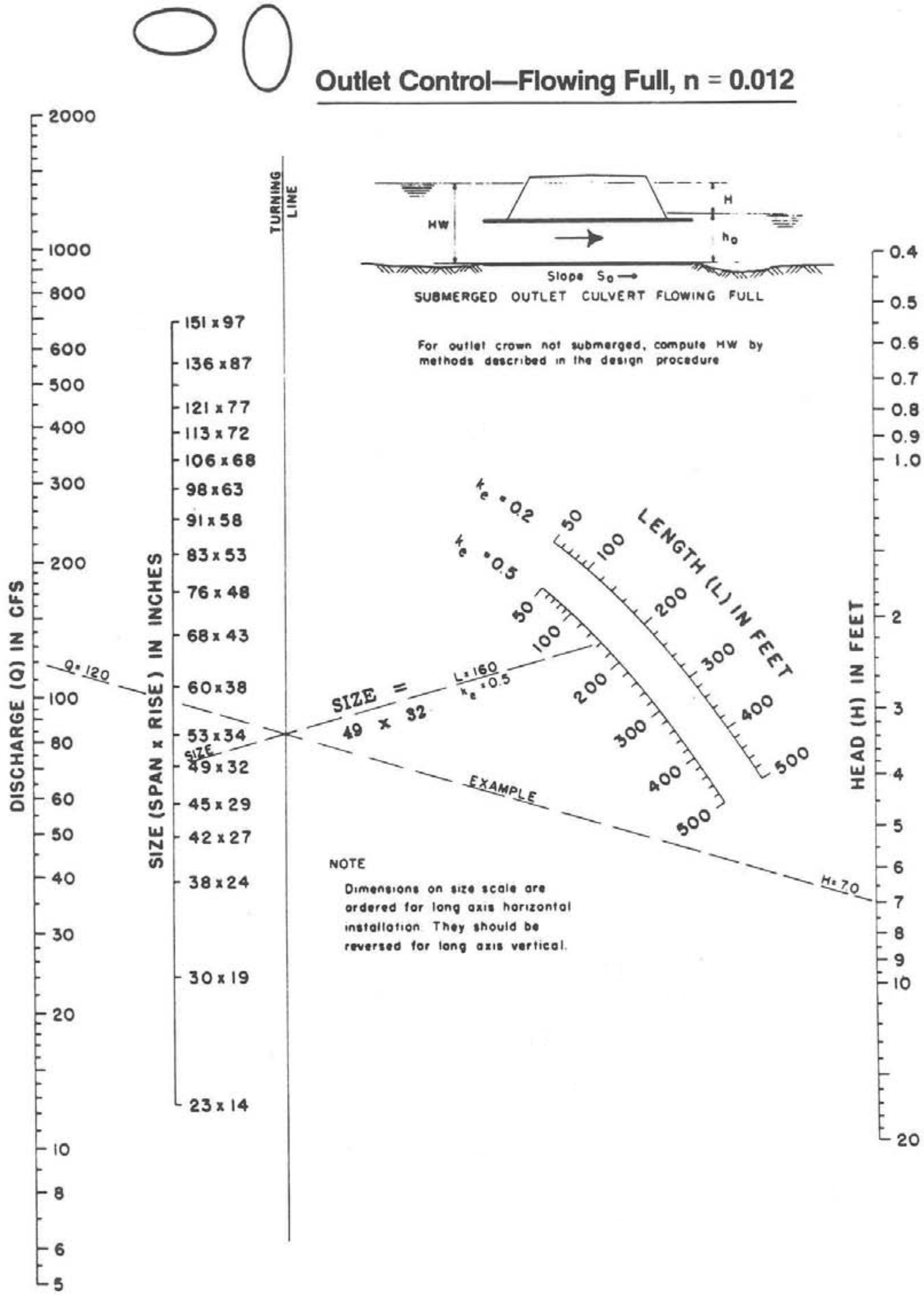
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-15
OUTLET CONTROL CHART FOR CONCRETE BOX CULVERTS FLOWING FULL



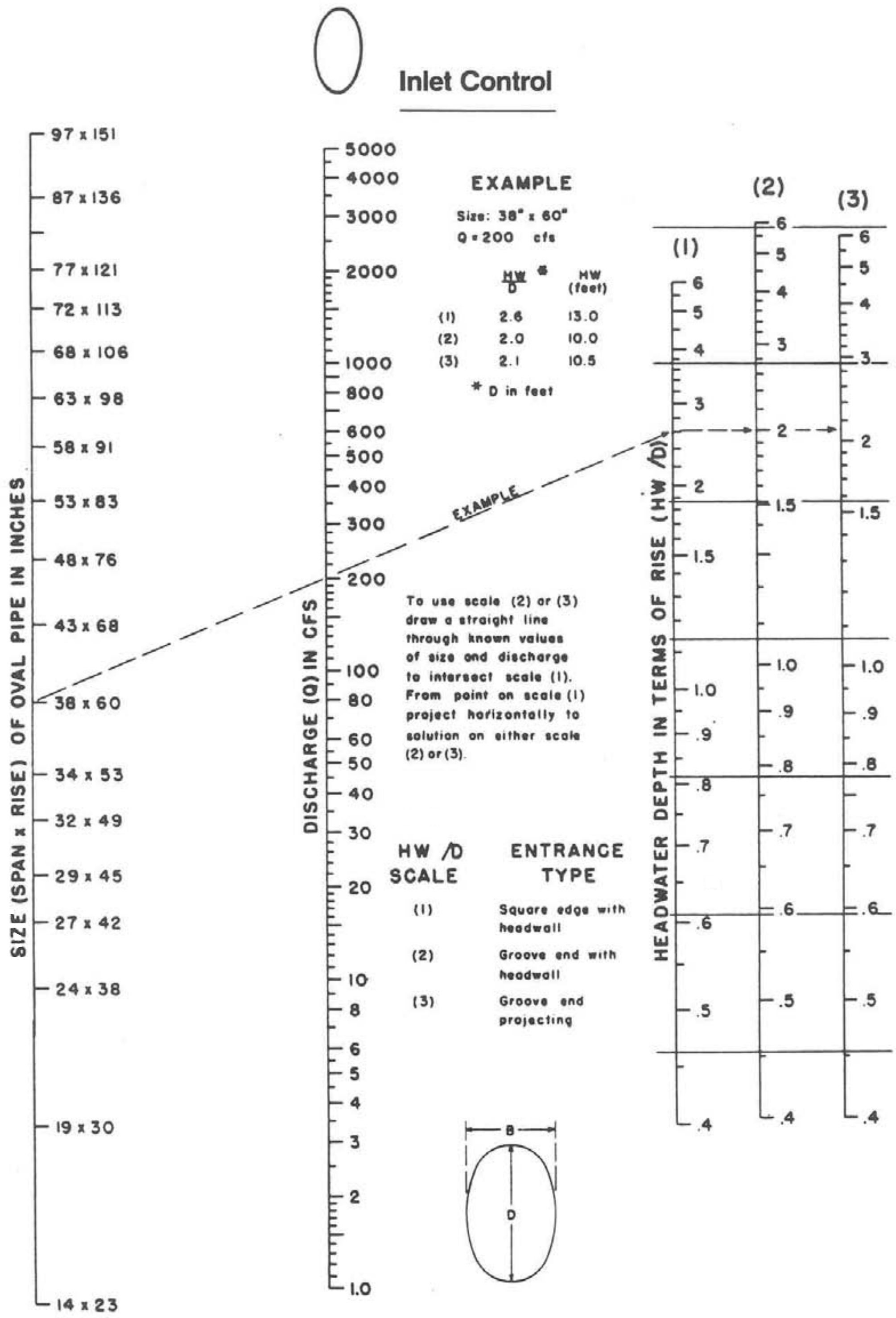
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-16
INLET CONTROL CHART FOR OVAL CONCRETE PIPE CULVERTS - LONG AXIS HORIZONTAL



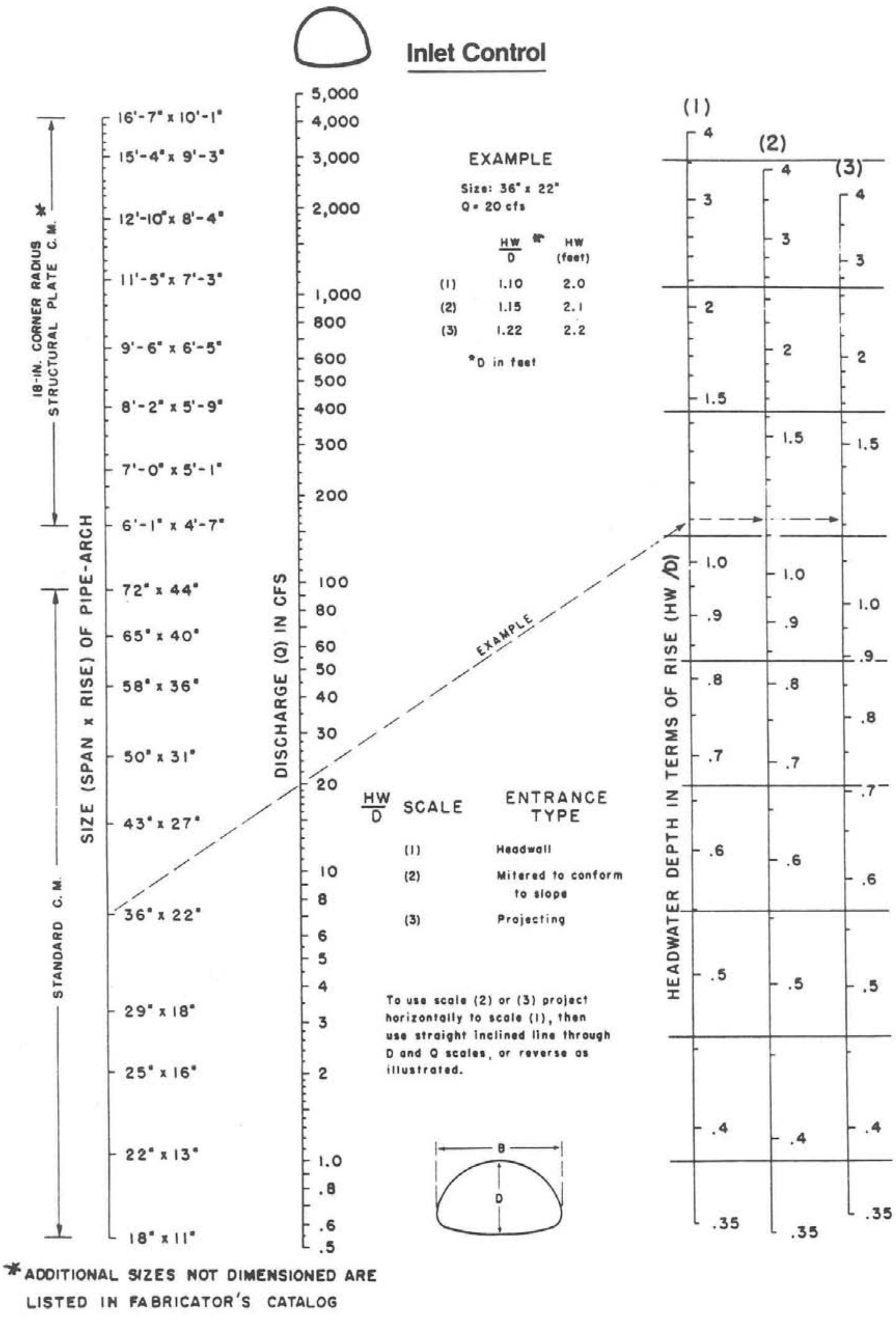
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-17
OUTLET CONTROL CHART FOR OVAL CONCRETE PIPE CULVERTS - LONG AXIS HORIZONTAL OR VERTICAL



Reference: USDOT, FHWA, HDS-5 (1985).

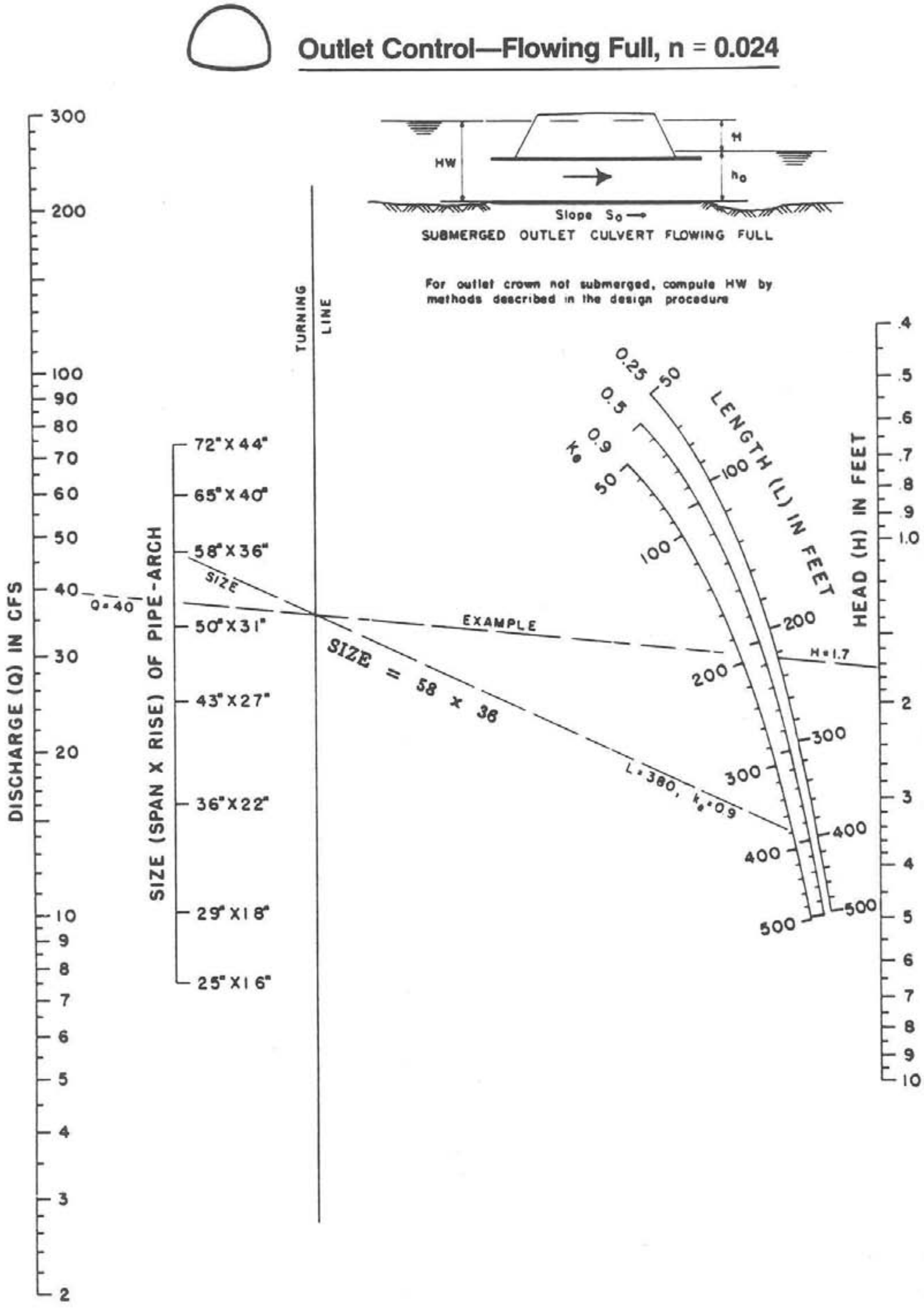
FIGURE 3-18
INLET CONTROL CHART FOR OVAL CONCRETE PIPE CULVERTS - LONG AXIS VERTICAL



Reference: USDOT, FHWA, HDS-5 (1985).

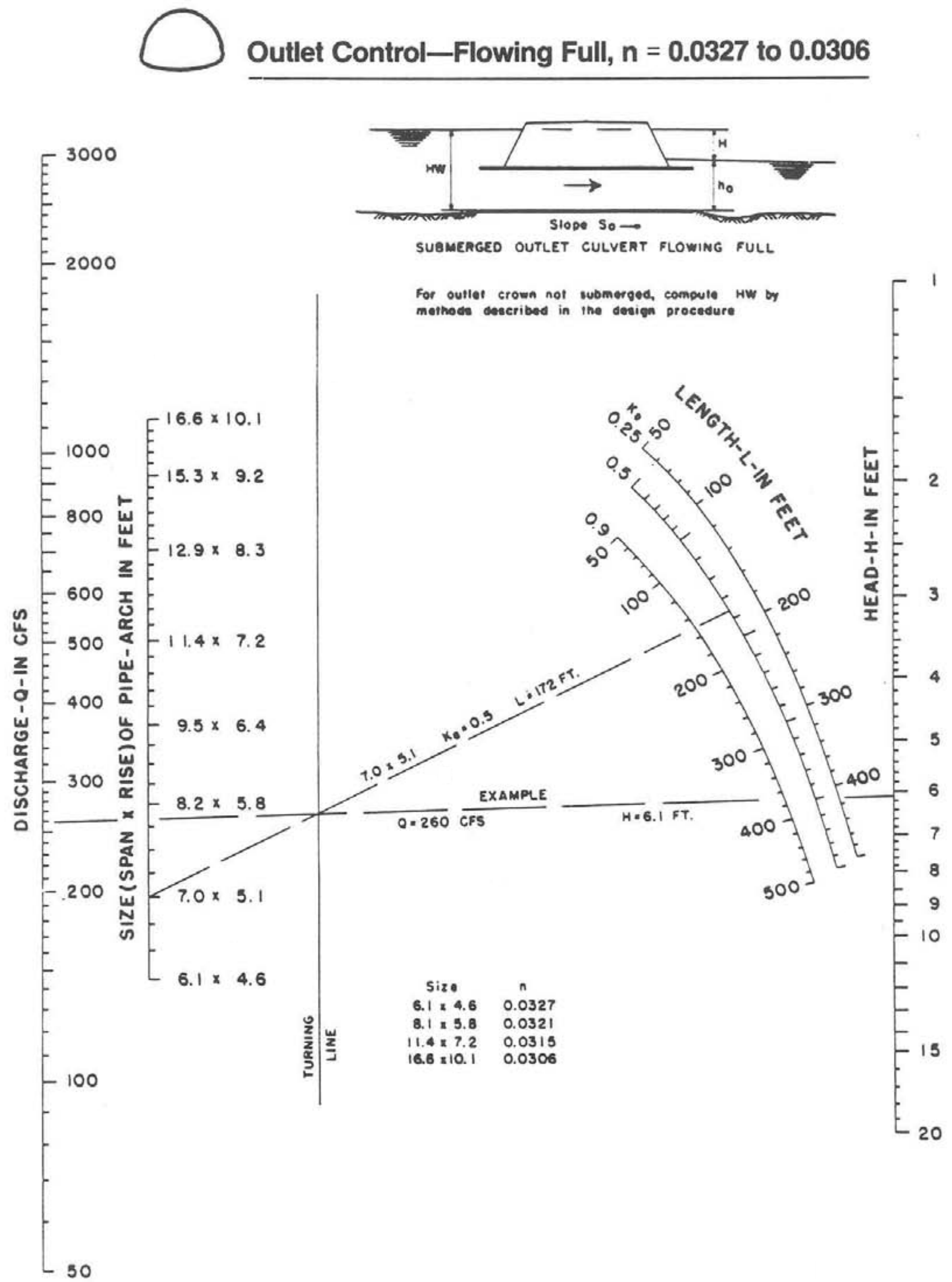
FIGURE 3-19
INLET CONTROL CHART FOR CMP ARCH CULVERTS

Outlet Control—Flowing Full, n = 0.024



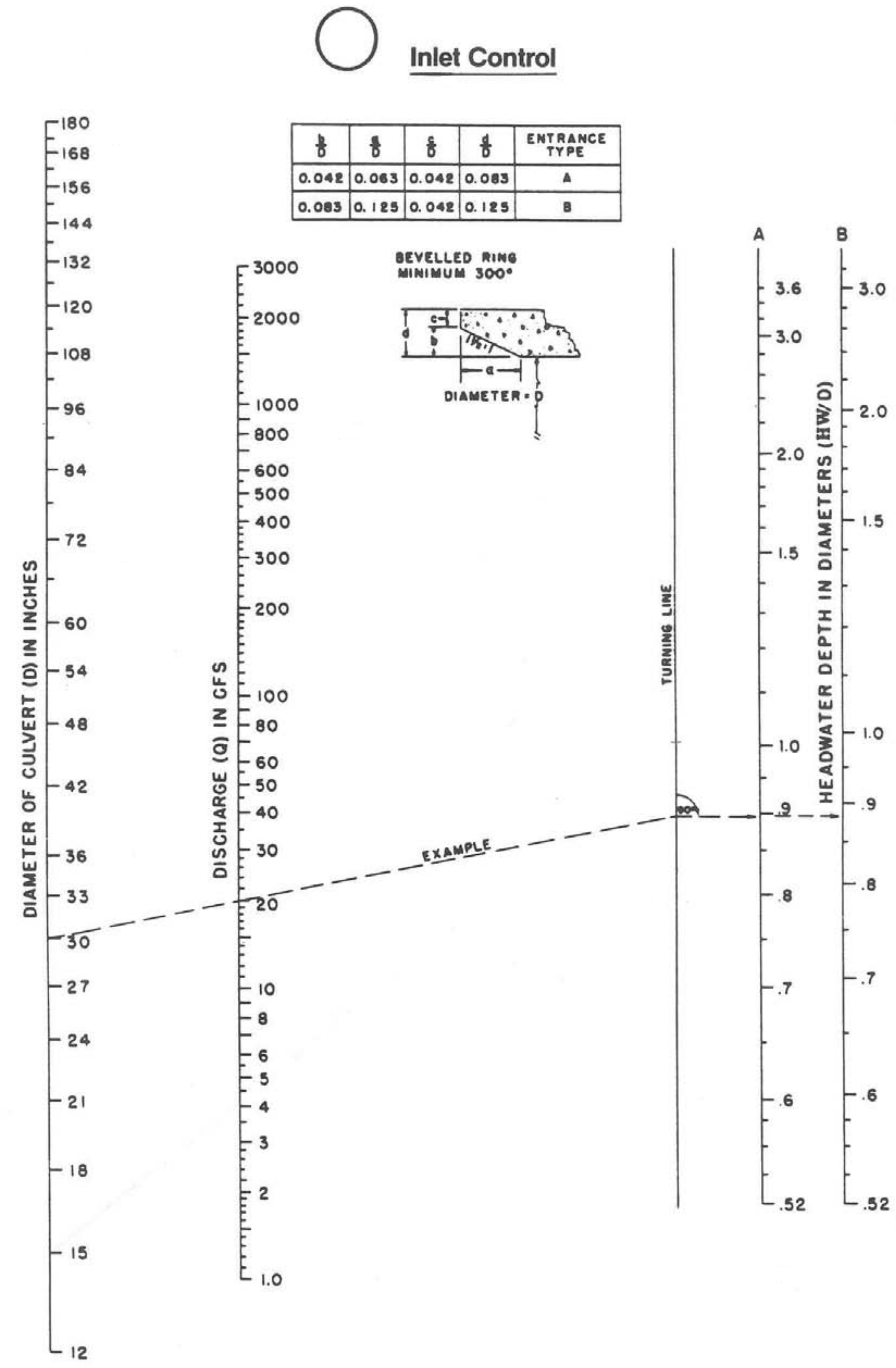
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-20
OUTLET CONTROL CHART FOR CMP ARCH CULVERTS FLOWING FULL



Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-21
OUTLET CONTROL CHART FOR STRUCTURAL PLATE CMP ARCH CULVERTS (18-INCH CORNER RADIUS) FLOWING FULL



Reference: USDOT. FHWA, HDS-5 (1985).

FIGURE 3-22
INLET CONTROL CHART FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING
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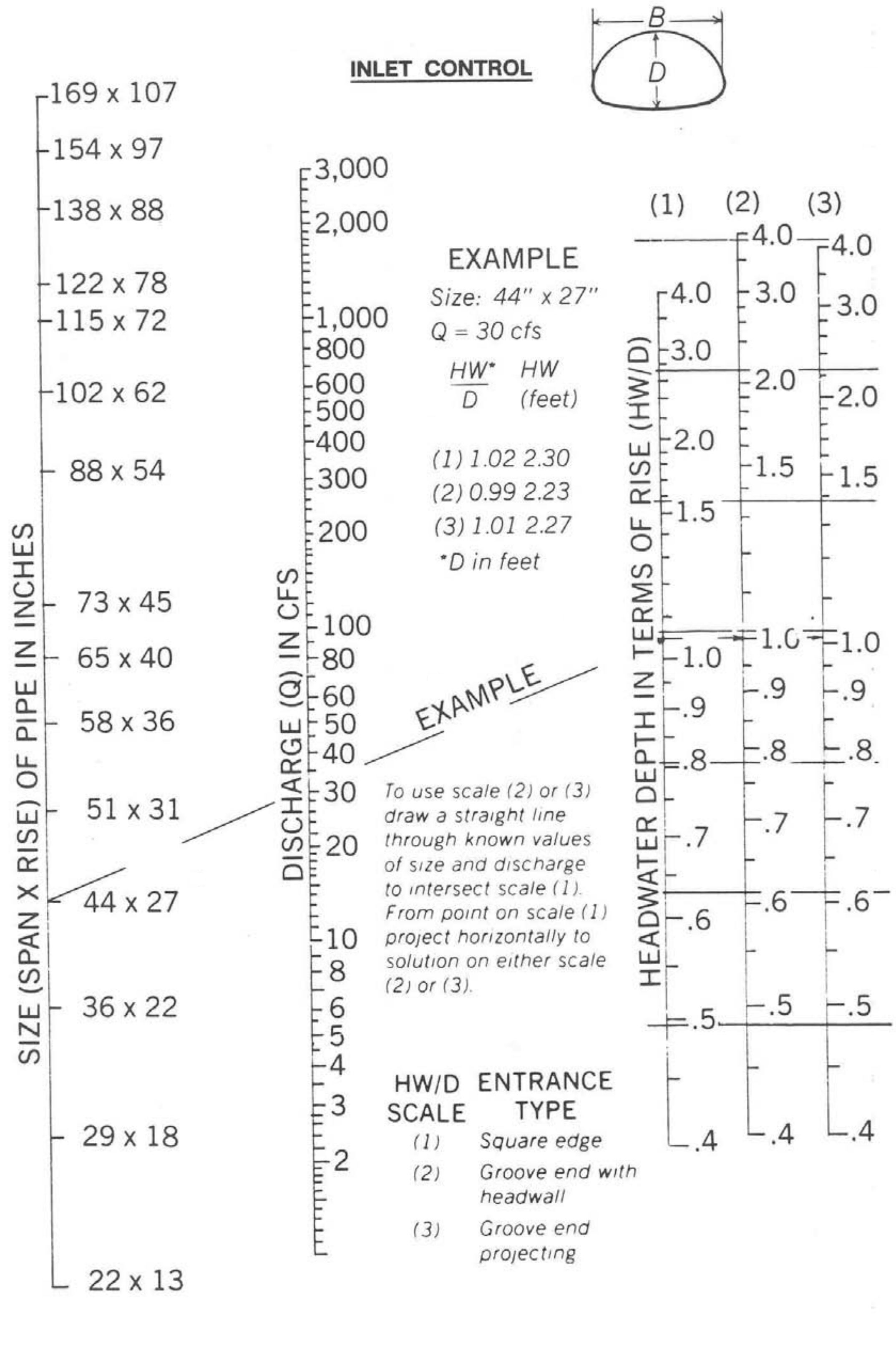
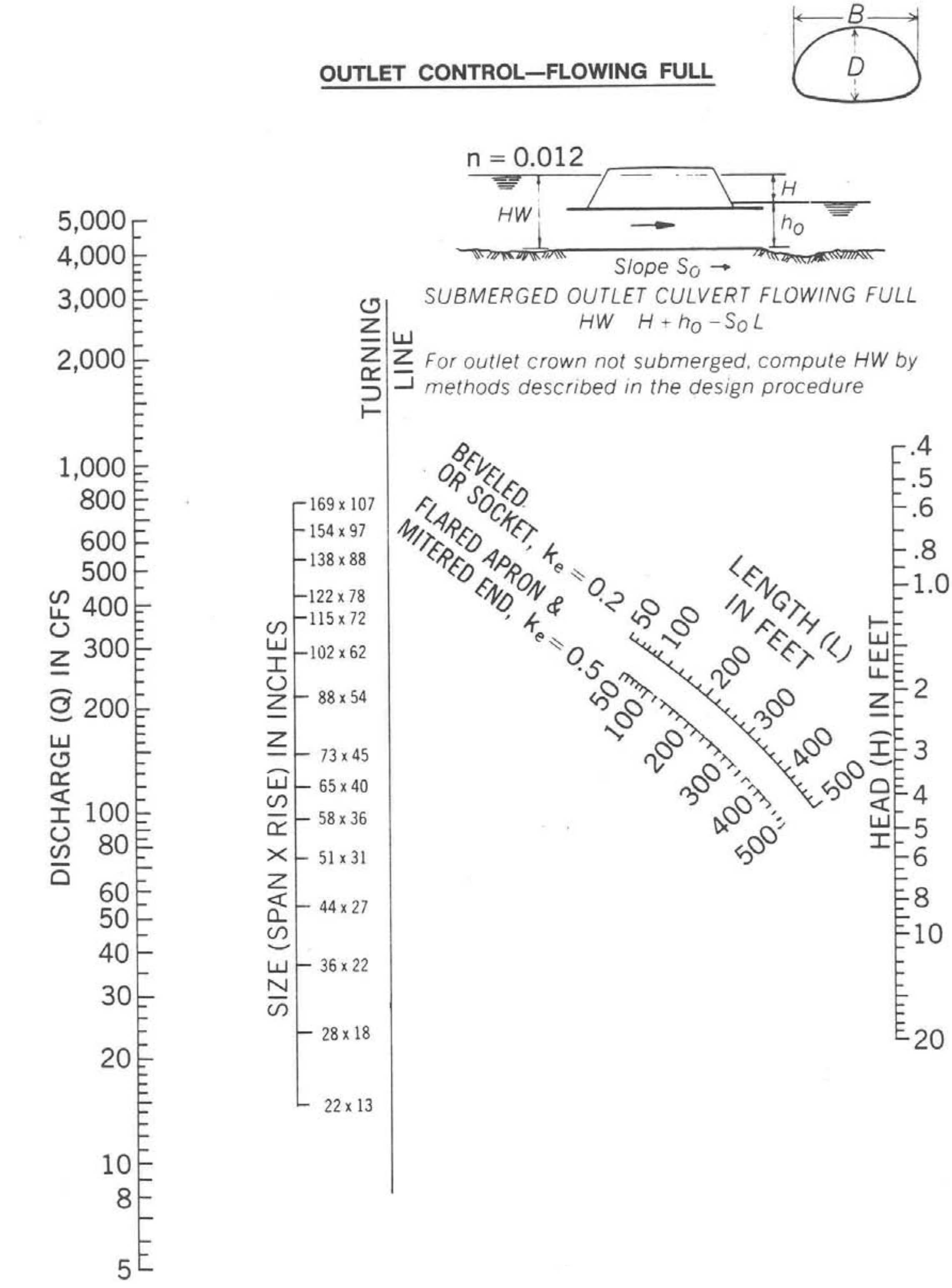


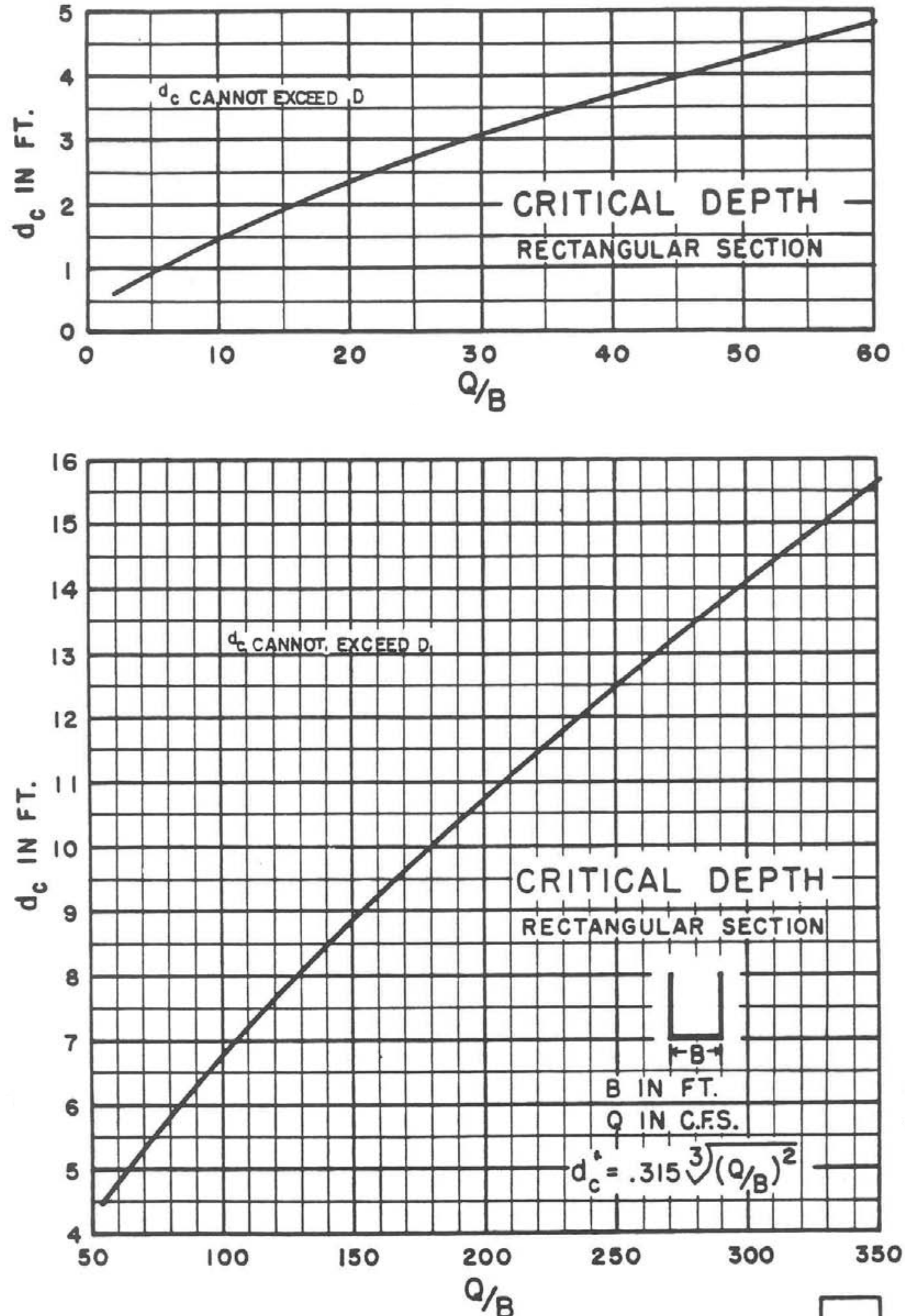
Reference: American Concrete Pipe Association (1980).

FIGURE 3-23
INLET CONTROL CHART FOR ARCH CONCRETE PIPE CULVERTS



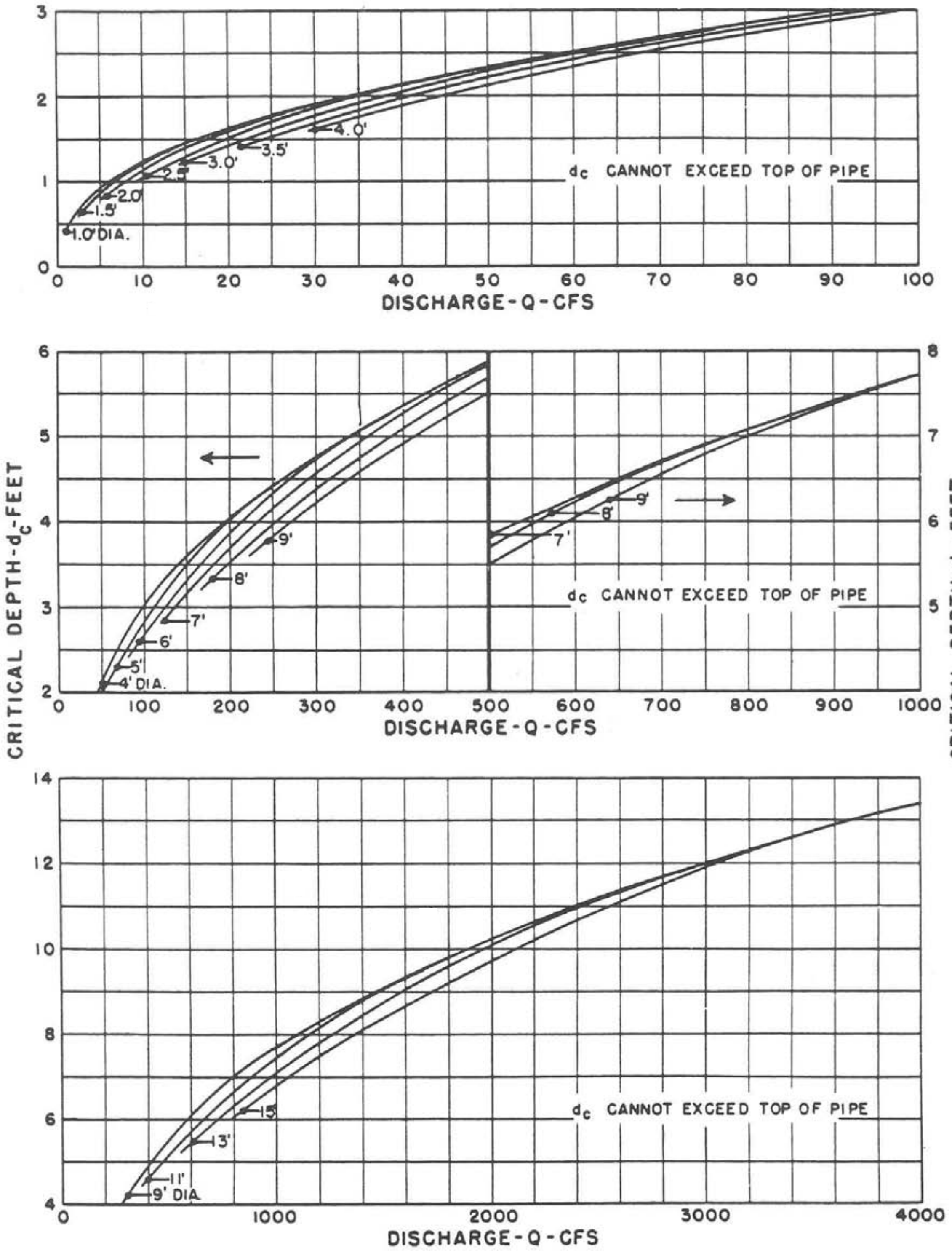
Reference: American Concrete Pipe Association (1980).

FIGURE 3-24
OUTLET CONTROL CHART FOR ARCH CONCRETE PIPE CULVERTS FLOWING FULL



Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-25
CRITICAL DEPTH CHART FOR RECTANGULAR SECTIONS



Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-26
CRITICAL DEPTH CHART FOR CIRCULAR PIPE
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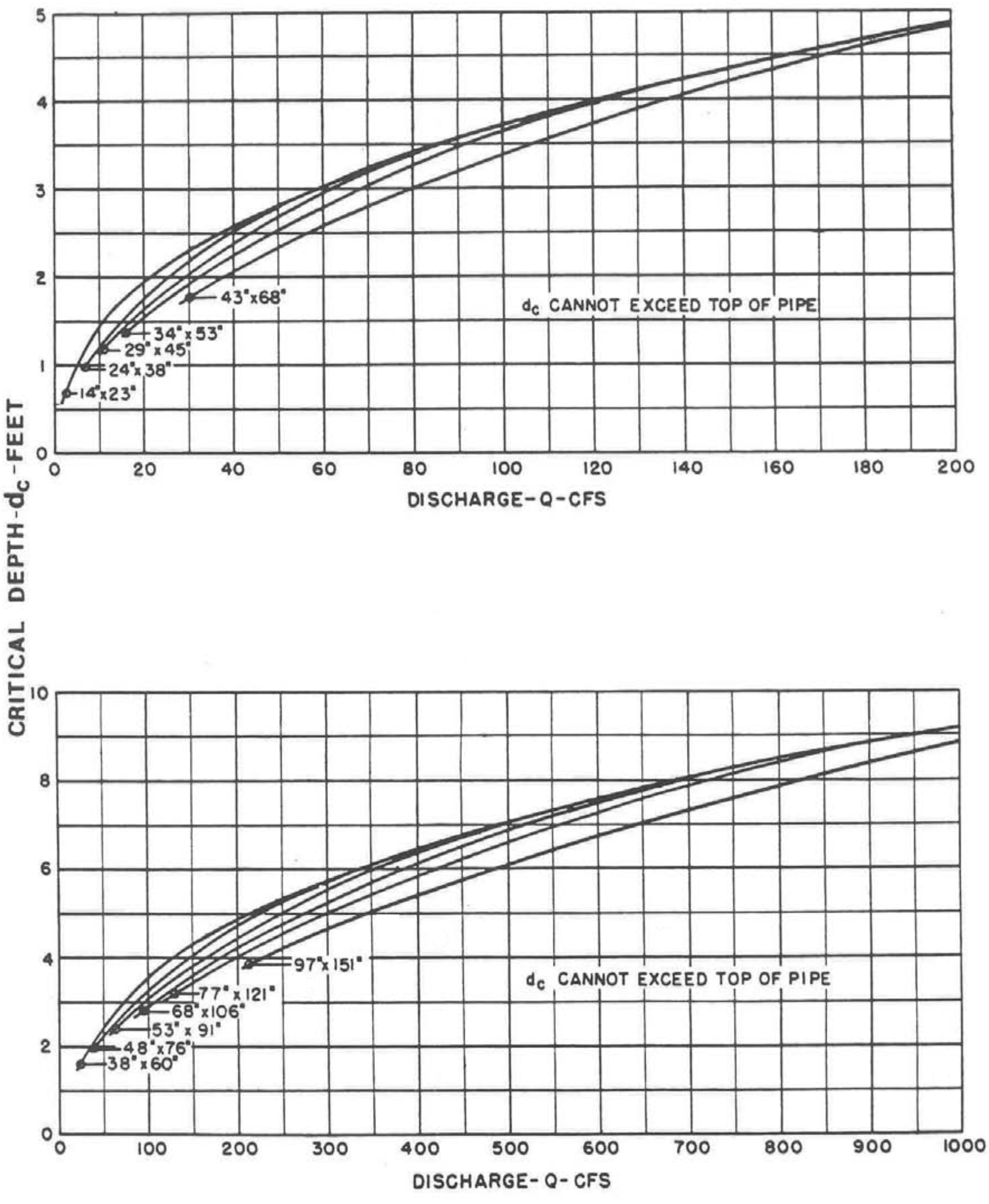
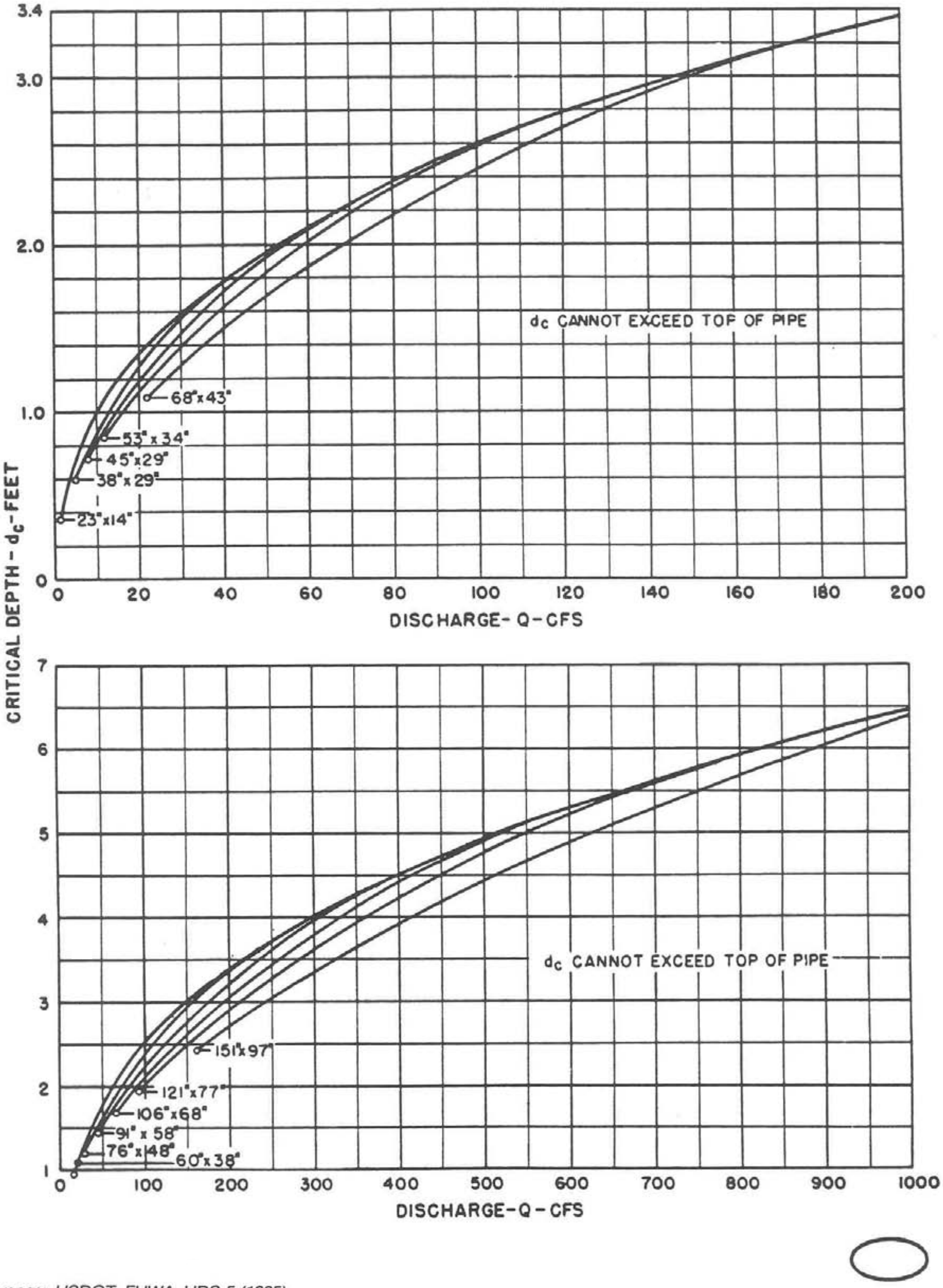


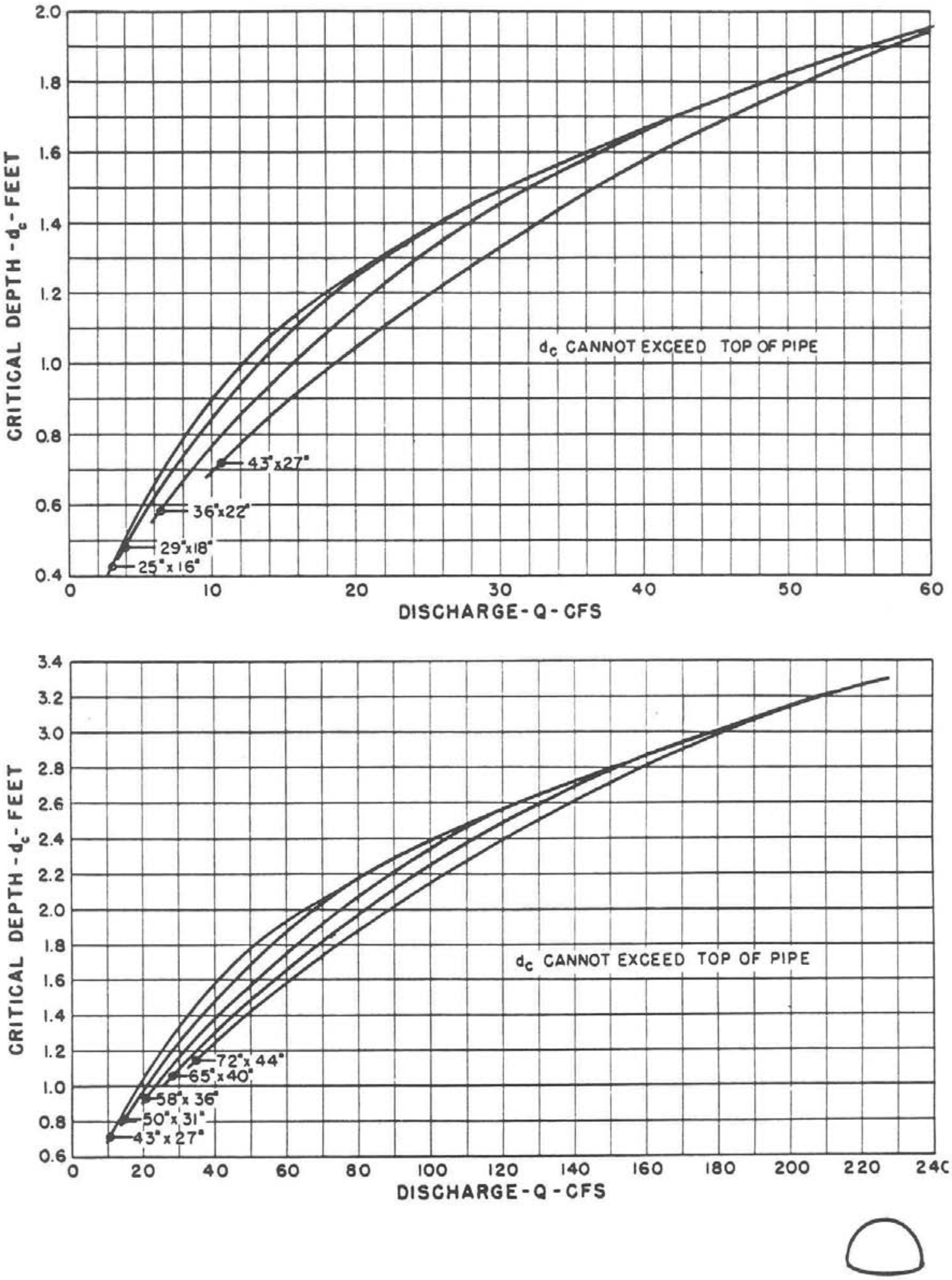
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-27
CRITICAL DEPTH CHART FOR OVAL CONCRETE PIPE - LONG AXIS VERTICAL
Auburn Storm Water Management Manual



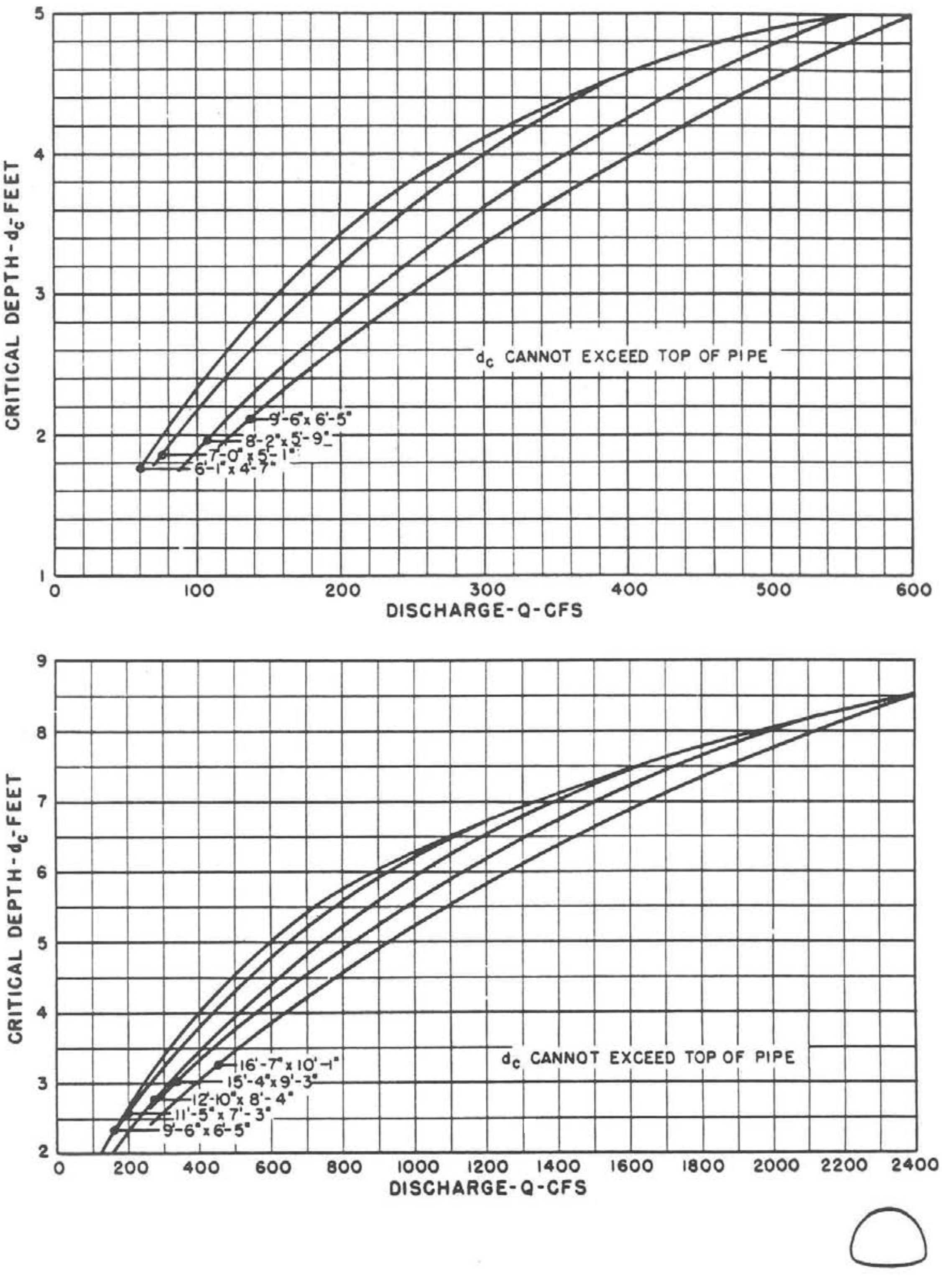
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-28
CRITICAL DEPTH CHART FOR OVAL CONCRETE PIPE - LONG AXIS HORIZONTAL



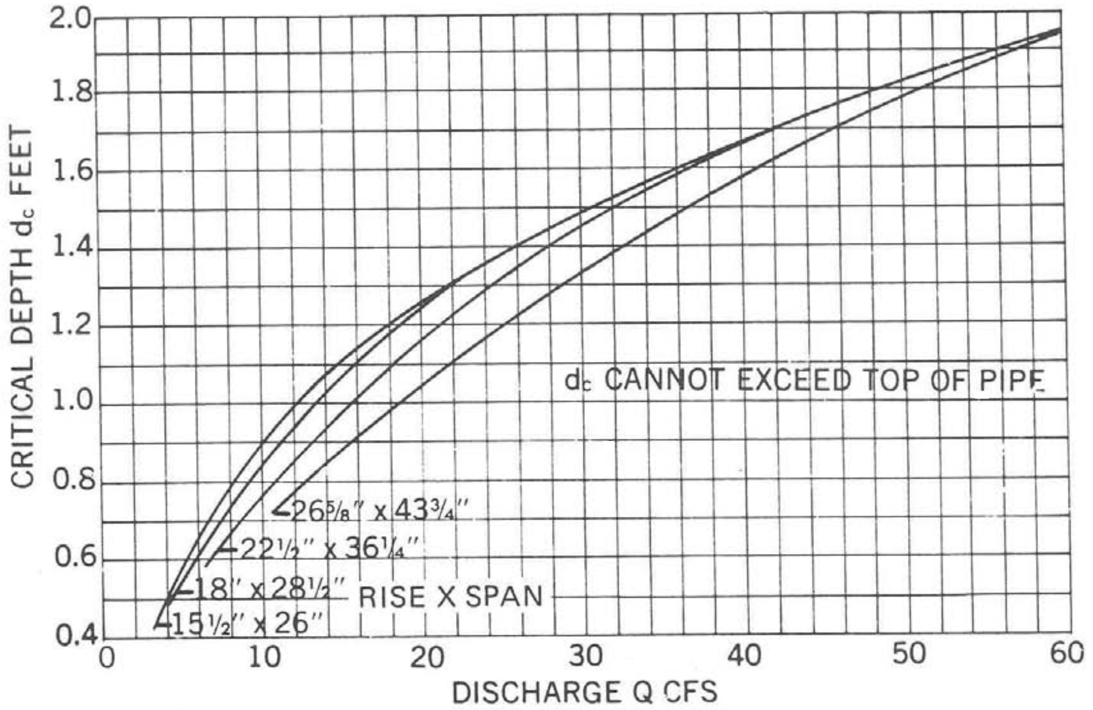
Reference: USDOT, FHWA, HDS-5 (1985).

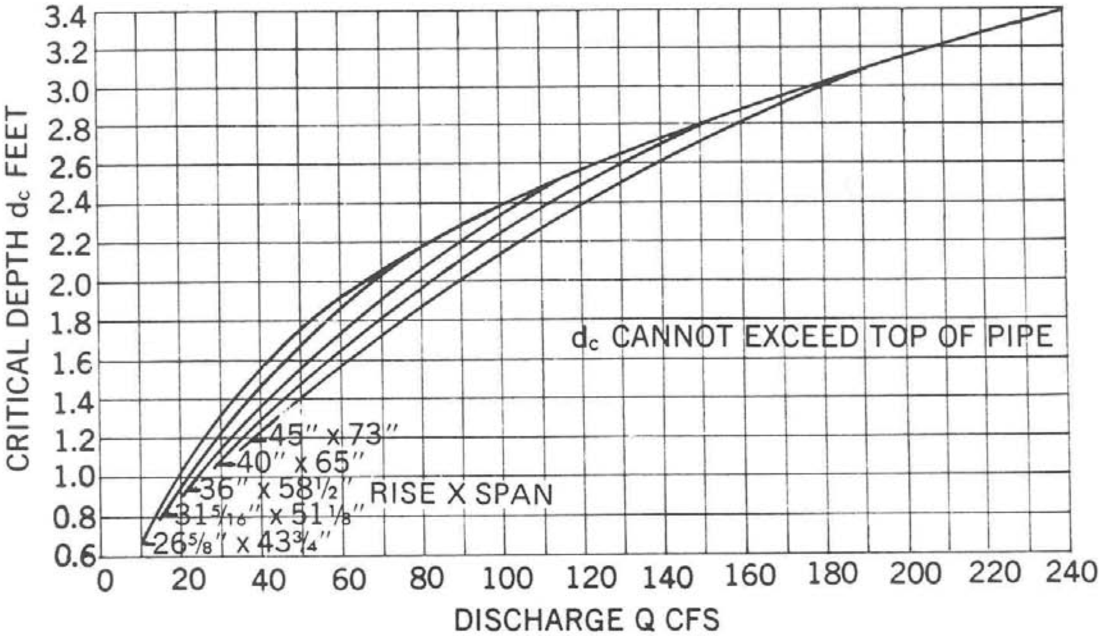
FIGURE 3-29
CRITICAL DEPTH CHART FOR STANDARD CMP ARCH

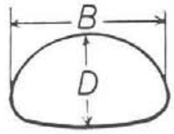


Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 3-30
CRITICAL DEPTH CHART FOR STRUCTURAL PLATE CMP ARCH

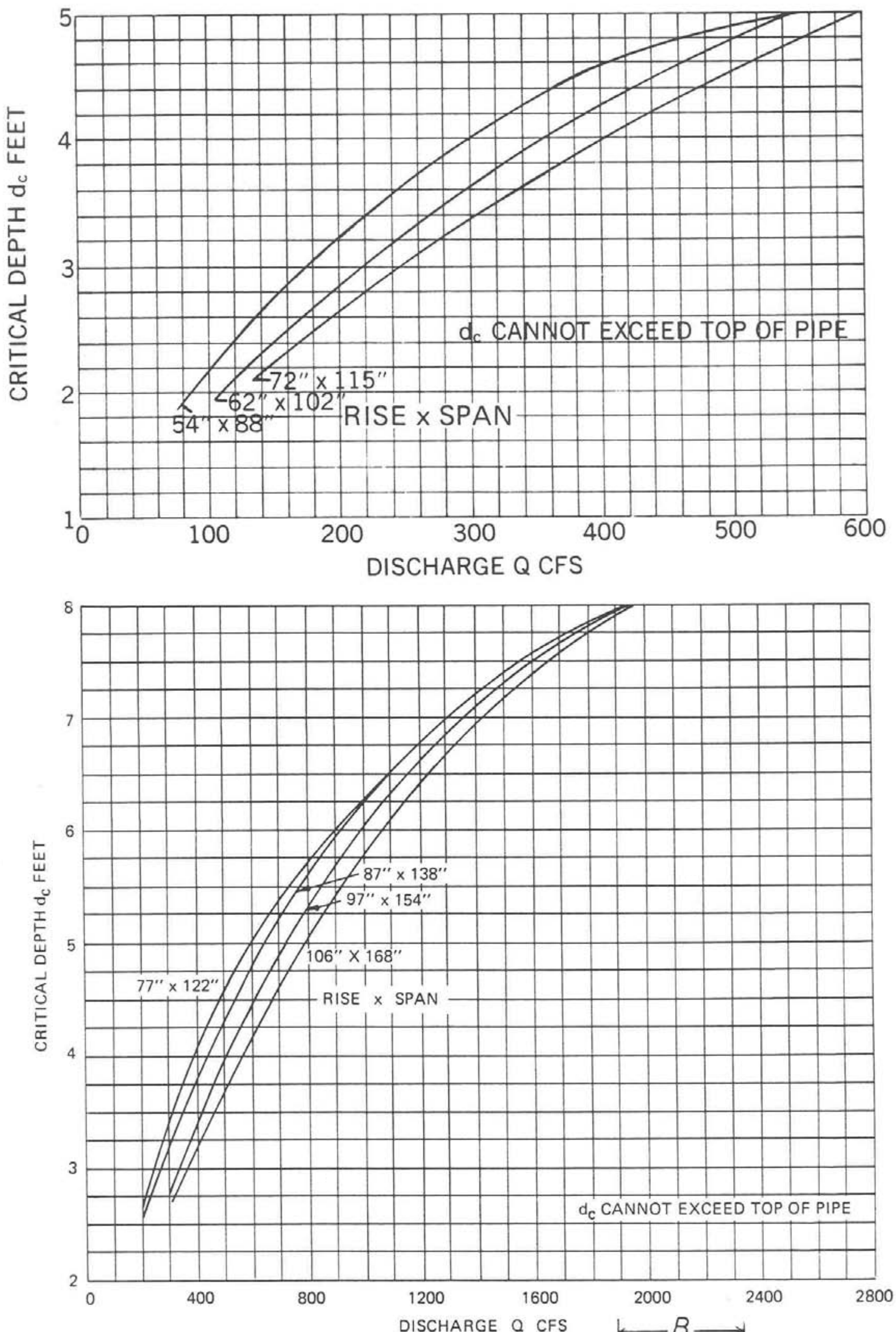


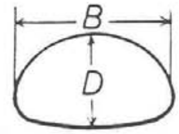




Reference: American Concrete Pipe Association (1980).

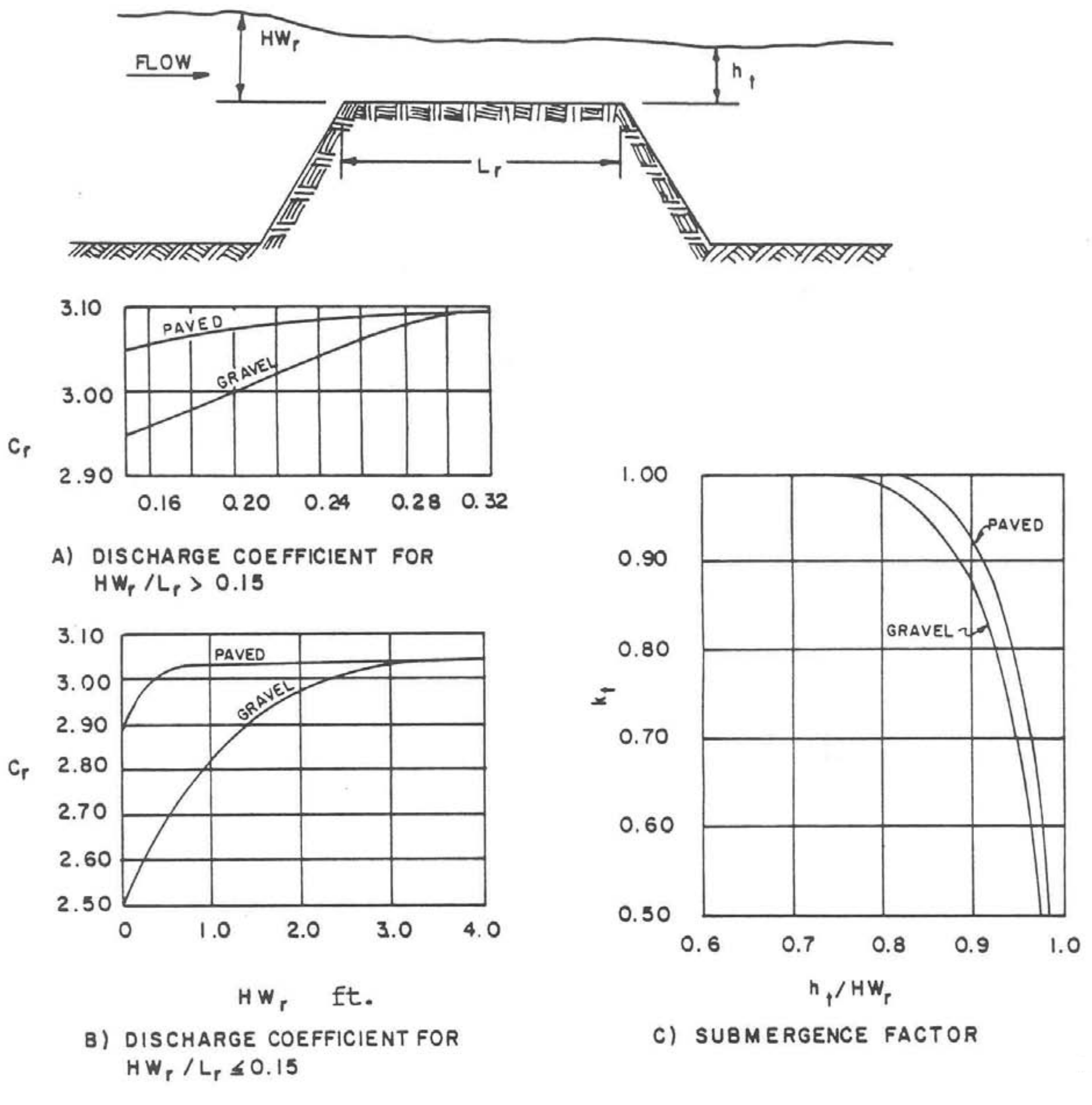
FIGURE 3-31
CRITICAL DEPTH CHART FOR ARCH CONCRETE PIPE SIZES 15 1/2"x26" TO 45"x73"
Auburn Storm Water Management Manual





Reference: American Concrete Pipe Association (1980).

FIGURE 3-32
CRITICAL DEPTH CHART FOR ARCH CONCRETE PIPE SIZES 54"x88" TO 106"x168"



Reference: *USDOT, FHWA, HDS-5 (1985).*

FIGURE 3-33
DISCHARGE COEFFICIENTS FOR ROADWAY OVERTOPPING

4. Hydraulic Design of Storm Water Storage Systems

4. Hydraulic Design of Storm Water Storage Systems

A non-disturbed watershed generally has storm water storage widely distributed in small-volume components throughout the watershed (shallow depressions, porous soils, etc.). This natural storage usually is reduced when urbanization occurs. If the reduction is significant, onsite storm water storage measures are required to offset the increase in storm water peak discharge and the reduction in water quality. Types of storm water storage that are appropriate in the Auburn area are identified in this section. In addition to the types of storage, general design considerations, flow control structures, and information for conducting a storage reservoir routing are presented.

The calculation of offsite discharges must be determined to the first downstream City-maintained storm water management facility so that during design storm flows, the structures and system currently in place are not flooded. If the added volume will compromise the current structures and system, necessary steps must be taken to resolve flooding problems.

Because the City of Auburn has been designated by ADEM as a Storm Water Phase II community, the City is required to show that storm water runoff into local streams does not degrade the stream's water quality. One way to document compliance with the Storm Water Phase II requirements is to have the pre- and post-hydrographs for altered land be similar. To meet this requirement, storm water storage may be needed.

Except as otherwise provided by the subdivision regulations or the City Code, the detention pond provisions of this manual do not apply to developments of less than one acre.

4.1 Types of Storage Systems

Two types of storm water storage systems are described in this section: detention basins and constructed wetlands. Detention basins slow the runoff rates, with no or little percolation. For Auburn, nearly all storage systems will be detention basins. Underground storage, if desired, will be reviewed by the City Engineer on a case-by-case basis.

The outlet structure is what controls the discharge from the storage system. Simple outlets may be a circular riser, a rectangular grate, or open pipe. More complex inlets may consist of a series of items; for example, a grated inlet over a weir leading to an open pipe.

4.1.1 Dry Detention Basins

Dry detention basins offer temporary storage accompanied by the controlled release of the stored water. Dry detention basins typically are placed with the basin bottom above the seasonal high water table and the outlet near the bottom of the pond so that there will be no standing water left after the design storm is routed through the structure. When designing detention basins, the following items should be considered:

1. Release rate
2. Detention volume
3. Grading and depth requirements
4. Outlet works

4.1.2 Wet Detention Basin

Wet detention basins are constructed basins that have a permanent pool of water throughout the year or wet season and generally are found in locations where groundwater is high and/or percolation is poor. Wet detention basins may be created in areas that do not contain these conditions by effectively sealing the bottom of the basin by compacting the subgrade and placing layers of clay above the compacted fill. Additional storage is provided above the permanent pool for peak discharge attenuation and treatment volume. Littoral zones (i.e., shallow planting areas) are used for water quality enhancement.

Wet detention basins can be part of a multi-use recreation facility that provides community benefits other than storm water management.

Generally, there are two types of basins, excavated or embankment. Excavated basins are normally the simplest to construct and maintain. Embankment basins deal with more complex issues such as earthen dams and possible destruction of property if these structures fail. Therefore, all embankment ponds must be designed and certified by a professional engineer before construction. Risks typically are controlled by limiting the height of embankments to less than 10 ft.

4.1.3 Extended Wet Detention Basin

Extended wet detention basins are an effective, low-cost means of removing sediments. The extended wet detention basin combines the treatment concepts of both the dry detention basin and the wet detention basin. The treatment volume is divided between the permanent pool and detention storage provided above the permanent pool. The benefit of the extended wet detention is that it takes up less space (overall, because the dry detention basin area is usable) than a traditional wet detention basin, but has similar pollutant removal efficiencies.

If storm water is detained for at least 24 hours, as much as 90 percent removal of sediments can be achieved. Two common methods for extended wet detention basins are to attach a slotted standpipe for shallow wet basins or to include a negatively sloped pipe for other wet detention basins (Figure 4-1) (Schueler, 1987).

4.1.4 Constructed Wetlands

Constructed wetlands, or treatment wetlands, are natural or artificially constructed wetlands used for the purpose of water treatment. The ability of the constructed wetland to transform common pollutants into harmless by-products or nutrients that can be used for biological production makes it an attractive alternative to conventional treatment systems. Constructed wetlands may be surface flow wetlands, shallow where water travels across the substrate surface; or subsurface wetlands, where water passes through a shallow substrate (below ground, like a gravel pack). Many of the same plants can be planted in both types of wetlands. Constructed wetlands behave similarly to extended wet-detention basins. Wetlands are established in the shallow parts of the basin, as well as other natural-based features. Examples of constructed wetlands include the following:

- Alternating shallow and deep ponding areas
- Hardwood swamp
- Reed bed on top of a nearly saturated soil layer
- Cattail marsh
- Peat bog

The incoming flow is slowed down, causing temporary detention. The amount of storage and detention are determined by the size, nature, and geometry (Urbonas and Stahre, 1993).

4.2 General Design Considerations

This subsection provides guidelines about the design of storm water storage systems. These storage systems have to be designed by a licensed engineer in the State of Alabama, and the design drawings, specifications, and design calculations need to be submitted to the City for review. Once the storm water storage system is constructed, this same engineer must certify that the system has been constructed properly and that it will function and operate for the purpose that it was designed.

4.2.1 Public Safety

4.2.1.1 Basin Side Slopes

Normally, dry basins designed to impound more than 2 ft of water or permanently wet basins must contain side slopes that are no steeper than 3H:1V out to a depth of 2 ft below the normal water level. As an alternative when space constraints exist, the basin can be fenced or otherwise restricted from public access. A dry basin or sediment forebay will have provisions for access and maintenance (e.g., ramp with 3H:IV or flatter).

4.2.1.2 Control Structures Safety

Control structures that are designed to contain more than 2 ft of water under the design storm and have openings of greater than a 1-foot minimum dimension must be restricted from public access. This restriction will include fencing and posting of a warning sign or installing culvert grating. The grating will be specified by the City.

4.2.2 Release Rate

Release rates for managing storm water peak discharge by detention will be based on limiting peak runoff rates to match the more restrictive of the two following conditions:

1. Post-development peak discharge is less than or equal to pre-development peak discharges for the following design storms: 2-, 5-, 10-, and 25-year, 24-hour.

2. Limit release rate based on the discharge capacity to the first City-maintained storm water management facility downstream from the project.

4.2.3 Detention Volume

The detention volume will be adequate to provide attenuation of the post-development peak discharge rates to the allowable release rate set according to provisions in Section 4.2.2. Drainage features of the outlet control structure will be designed to drain the detention volume between 24 and 72 hours.

The routing calculations will be consistent with the procedures outlined in Section 4.4. If siltation during construction causes a loss of detention volume in the basin, the design dimensions will be restored before submitting as-built certification.

4.2.4 Depth

The maximum depth of storm water detention facilities should be consistent with public safety and aesthetic considerations for the system, in addition to structural and geotechnical considerations for the facility. In general, if the facility provides a permanent pool of water (wet detention basin), a depth sufficient to discourage the growth of attached weeds (without creating undue potential for anaerobic bottom water) should be considered. A depth of from 6 to 8 ft is generally reasonable. Furthermore, a minimum freeboard of 2 ft above the high water elevation of the 25-year design flood will be provided. A minimum freeboard of 0.5 ft shall be provided for the 100-year design flood.

4.2.5 Outlet Works

Outlet works will be designed to prevent overtopping of the embankment based on a 100-year discharge rate, considering total watershed development. Outlet works can take the following forms:

1. Drop inlets with pipes
2. Weirs
3. Orifices

The principal spillway will convey the 25-year discharge rate for total watershed development without allowing flow to enter the emergency outlet. The emergency outlet shall be sized to safely pass the 100-year design storm. A smooth and stable transition to the first City-maintained storm water management facility will be provided. The sizing of a particular outlet work will be based on the results of the storage routing calculations described in Section 4.4.

4.2.6 Sediment Storage

Sediment storage will be provided in basins to contain 3,600 cubic ft (ft^3) per contributing drainage acre per the City of Auburn standard for erosion control. This volume will be maintained in the permanent facility, also. It is recommended that a sediment sump (a forebay or trap) be installed at the inlet of the pond structure to allow for sediment accumulation and clean out.

4.2.7 Outlet Protection and Rip-rap

Erosion protection is required for all outlets, including but not limited to, culverts, pipes, and basin outfalls. Such protection can be accomplished using a stilling basin or concrete apron based on DOT standards.

The preferred method of slope and channel protection is the use of vegetation. If vegetation cannot withstand the design flows, then there are other alternatives that use control blankets where vegetation can grow (e.g., fiber reinforced mats, geoweb, or other plastic reinforcement). There are also other types of stabilization such as reinforced articulated concrete mats, which can accommodate vegetation. Rip-rap is most often used at storm drain outlets, at the toe of slopes, and in transitions from concrete channels to vegetated channels.

Rip-rap is a lining of aggregates consisting of large, loose, angular stone used to protect the soil from the erosive forces of flowing water and to dissipate the energy of the water. A rip-rap reinforcement consists of the rock overlying a geotech filter fabric to prevent undermining of the rock. There are two primary uses for rip-rap outlet protection and erosion and scour protection for channels.

Rip-rap is often described in Alabama in three classes, ranging from smaller to larger:

Class 1: Graded stone ranging from 10 to 100 pounds, with no more than 10 percent having a weight over 100 pounds and at least 50 percent having a weight over 50 pounds and not more than 10 percent having a weight under 10 pounds.

Class 2: Graded stone ranging from 10 to 200 pounds, with no more than 10 percent having a weight over 200 pounds and at least 50 percent having a weight over 80 pounds and not more than 10 percent having a weight under 10 pounds.

Class 3: Graded stone ranging from 25 to 500 pounds, with no more than 10 percent having a weight over 500 pounds and at least 50 percent having a weight over 200 pounds and not more than 15 percent having a weight under 25 pounds.

In all instances, geotextile filter cloth will be placed between the rip-rap and underlying soil. The filter cloth shall be based on the ALDOT standards. The cloth and rip-rap will be buried so the top of the rip-rap is at the bottom grade of the channel. The end of the filter cloth will extend approximately 1 foot beyond the minimum dimension of the rip-rap and be buried (i.e., keystoned) into the ground. The buried cloth shall be compacted and stabilized so it will not be unearthed under normal use.

Rip-rap is often used beneath a storm drain outlet to protect the downstream channel from high velocities. In no case shall rip-rap be used when the slope of the outlet channel is greater than 10 percent. In these cases, a concrete channel will lead to the bottom of the slope and rip-rap will be used at the transition. Aprons should be aligned without bends. Figures 4-2 and 4-3 can be used to select the appropriate rip-rap class, with Class 1 being used most often. The minimum depth of rip-rap shall be 1.5 times the size of the median stone.

For well-defined channels, the apron width will extend along the wetted perimeter of the channel and be terminated approximately 1 foot above the design flow depth. The rip-rap and filter cloth will be buried so that there is no difference in the downstream channel invert of the banks and the end of the protection. The width of the channel will be designed to carry the design flow as described elsewhere in this guidance document.

For flat areas, at the upstream end of the apron, the apron width will be three times the diameter of the pipe. The apron length shall be six times the diameter of the outlet pipe. The width of the apron should spread at approximately a 1.5:1 angle, such that the width at the most downstream point is as wide as the apron is long (plus the diameter of the pipe).

4.2.8 Inlet Protection and Rip-rap

Erosion protection may be required for inlets, including but not limited to, culverts and pipes. Erosion protection will be as described in Section 4.2.7.

4.2.9 Operation Maintenance Considerations

In the documentation submitted to the City, the applicant shall identify the entity responsible for the perpetual care, operation, maintenance, and associated liabilities of the system. If the entity is to be a public body, such as a county, municipality, or special district, a letter or other evidence of acceptance must be included. If the entity is a non-public body such as a homeowners' association or private corporation or person, documentation of its existence, fiscal and legal ability, and willingness to accept the responsibility must be included.

The operational and maintenance phase will not become effective until the registered professional engineer that designed the system certifies that all facilities have been constructed in accordance with the design permitted by the City, and the City has inspected the system and found it acceptable. The engineer's certification shall be submitted to the City within 30 days after the completion of construction of the system.

4.3 Flow Controls

One of the important components of a storm water storage system is proper flow control. Without proper control, the system will become overloaded or not function as designed, possibly creating more problems than were initially present. To prevent this situation from occurring, flow control devices such as weirs and orifices are used to limit the discharge rates to desired levels.

4.3.1 Length to Width Ratio

The inlet and the outlet of the basin will be located to prevent short-circuiting and to maximize flow through the basin to improve water quality benefits. The ratio between the length and width (L/W) will be 2:1 or larger. If this ratio cannot be attained because of topographic constraints, then additional provisions, such as a baffle wall, will be incorporated into the basin to create a L/W of 2:1.

4.3.2 Weir

Weirs are often used as flow-measuring devices in open channels, but additionally are used as flow control devices. Weirs allow flow to occur across some span, called a crest. If the crest spans the entire width of the channel, the weir is referred to as a suppressed weir. If not, the weir is referred to as partially or fully contracted. The crest could be rectangular or triangular (V-notch) in shape. The height of water flowing over the crest of the weir is used to measure the discharge. Weirs commonly are used as outlet structures in storm water storage systems, maintaining the water in the system until a desired elevation is reached. Once this elevation is exceeded, the discharge may be easily calculated using a weir equation.

Two different weirs and their relative equations for discharge are presented below.

Broad-Crested Weirs

$$Q = CLH^{1.5} \qquad \text{(Equation 4-1)}$$

where:

C = Broad-crested weir coefficient (values between 2.6 and 3.1 are normal)

L = Broad-crest weir length, ft

H = Head above weir crest, ft

V-Notch Weirs

$$Q = 2.5\tan\left(\frac{\theta}{2}\right)H^{2.5} \qquad \text{(Equation 4-2)}$$

where:

θ = Angle of V-notch, degrees

H = Head on vertex of notch, ft

4.3.3 Orifices

As with weirs, orifices also are a good flow control device. The discharge through an orifice can be evaluated using the following equation:

$$Q = CA(2gH)^{0.5} \qquad \text{(Equation 4-3)}$$

where:

C = Orifice coefficient (a value of 0.6 is usually appropriate)

A = Area of orifice, square feet (ft^2)

g = Acceleration due to gravity, 32.2 ft/sec^2

H = Head above orifice centroid, ft

The outlet structure configuration shall be as shown in Figure 4-4, with a 3-inch orifice near the bottom of the structure, a V-notch weir near the top, and a grated inlet to pass the 25-year storm event. The outlet structure needs to be designed by a professional engineer registered in the State of Alabama.

Example 4.1: Sizing the Basin for Storm Water Attenuation

The post-development storm water sizing for the detention structure requires the use of computer modeling software. In this example, the contributing watershed was divided into two subbasins: the natural common area surrounding the detention basin and the contributing housing development.

The first step is to estimate the maximum basin surface area based on a reasonable percent of the total project area. A rule of thumb, which was used for this problem, is that a detention basin area be at least 3 percent of the area draining to it. The maximum detention basin depth is 8 ft (Section 4.2.4). Two feet of freeboard above the design depth will be used to establish the berm height. Thus, the bottom detention basin elevation will be set at 90 ft and the maximum allowed water surface elevation during the 25-year design storm will be 97 ft, with 2 ft of freeboard to the top of the berm at 99 ft.

The detention basin's surface area at elevation 98 ft is set at 3 percent of 10 acres = 0.3 acre. Using a side slope of 3:1 (Section 4.2.1.1), the surface area at the bottom of the detention basin will be 0.1 acre. The detention basin will be kidney shaped, with the length to width ratio maintained at 2:1 (Section 4.3.1) to make it more aesthetic while using the full length of the detention basin. It is presumed that the detention basin will be partially excavated, with a berm along the most downstream edge near the stream.

The outfall will be sized to meet the target maximum depth during the 25-year design storm. Calculations also will be performed for the 2-, 5-, and 10-year design storms. Also, the 100-year storm runoff will be accommodated by installing an emergency overflow weir at an elevation above the 25-year peak depth. Furthermore, outlet structures should be set at elevations and sized such that post-development discharge is less than the pre-development peak discharge and the detention basin is drawn down between 24 and 72 hours. A computer program was used to estimate an appropriate outlet structure. Several iterations were conducted given the general sizes listed below. Acceptable computer programs include Pondpack, ICPR, and SWMM. If there is difficulty in achieving all of the target area, maximum depth, or post-development flow rates, then the detention basin may have to be made larger or additional storage may have to be provided in the development.

For this example, the outfall selected was a concrete box inlet with a smaller orifice near the bottom and a V-shaped weir near the top. After several iterations, the box outfall was sized as follows. The box was a standard pre-cast 4-foot x 4-foot box. The V-shape weir had a 90-degree opening and a depth of 6 inches. The crest of the riser is set at elevation 95 ft and the bottom of the V-weir was 94.5 ft. In addition, the detention basin is dewatered by a 3-inch circular opening set at elevation 90.1 ft. The box is drained by an 18-inch culvert pipe through the berm with an invert elevation of 90 ft. This pipe may seem small when compared to the pipe calculations above, but the pipe operates under pressure flow under the highest conditions, and the computer model confirmed its capacity. Figure 4-4 shows a schematic design drawing of the outfall structure discussed here.

4.4 Storage Routing

For the purposes of this manual, the primary function of a storm water storage system is to reduce the peak flow of a hydrograph to a desired value. To determine the peak flow reduction obtained by a storm water storage system, a reservoir routing procedure is required.

The design engineer is expected to use a standard computer software program to estimate the performance of the detention basin. These programs must base the routing on a storage-continuity approach, such as the level-pool routing method. Documentation will include the input variables and peak flow rates and stages for each design storm. The configuration of the outfall will be clearly described. Furthermore, stage (or elevation) versus storage and discharge table(s) will be provided. Additional design criteria discussed in this section, but not necessarily contained in the routing procedures, will be addressed in the application as supporting information and/or contained within the design documents. The engineer also may use manual calculations. An example of a manual approach is presented in Section 4.4.1.

Example 4.1 Continued: Routing of Design Storm

The routing of the post-development design storm can be accomplished using standard storm water modeling software. In this case, PondPak v7.0, a detention basin and watershed model developed by Haestad Methods, was used. Basin routing was accomplished by the level-pool routing method. The sample output from the PondPak simulation of the final configuration is as follows:

Storm Event	Peak Discharge from Pond (cfs)	Peak Elevation for Storm (ft)
2-year	1.90	95.07
5-year	15.11	95.57
10-year	21.97	96.10
25-year	23.45	96.96

The post-development discharge from every design storm is less than the pre-development rate. The 25-year storm is less than the target maximum elevation of 97 ft, but the 100-year storm also must be examined. Additional simulations were conducted to size a broad-crested weir over the embankment. The final weir was selected to be 10 ft wide, with a bottom elevation set at 98.0 ft. The peak 100-year flood elevation in the pond was 98.5 ft, which is 0.5 foot under the berm height. This level is the minimum allowable freeboard (0.5 ft) for the emergency spillway, so the berm height is adequately sized. However, the emergency overflow weir could have been set at elevation 97 to provide more freeboard under the 100-year design flood.

PondPak also calculates the minimum drain time from the maximum water surface elevation (97 ft) until empty. This calculation is accomplished by routing the full volume of the pond through the outlet structure without any inflow. The minimum drain time was approximately 12 hours.

4.4.1 Manual Routing Calculations

The Storage Indication Method is recommended for performing reservoir routing calculations manually for final design of detention facilities. The following procedure is used to route the storm:

1. Develop an inflow hydrograph, a stage-discharge curve, and a stage-storage curve for the proposed detention facility. Example stage-storage and stage-discharge curves are presented in Figures 4-5 and 4-6, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristic curves that provide values of $S \pm \Delta t/2$ versus stage. An example tabulation of storage characteristics curve data is presented in Table 4-1 and Figure 4-7.

4. For a given time interval, the inflow points I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - O_1\,\Delta t/2$ can be determined from the appropriate storage characteristics curve (Figure 4-7).

5. Determine the value of $S_2 + O_2\,\Delta t/2$ from the following relationship:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \qquad \text{(Equation 4-4)}$$

   where:

   S_2 = Storage volume at time 2, in cubic feet

   O_2 = Outflow rate at time 2, in cfs

   Δt = Routing time period, in seconds

   S_1 = Storage volume at time 1, in cubic feet

   O_1 = Outflow rate at time 1, in cfs

   I_1 = Inflow rate at time 1, in cfs

   I_2 = Inflow rate at time 2, in cfs

6. Enter the appropriate storage characteristics curve (Figure 4-7) at the value of $S_2 + O_2\,\Delta t/2$ (from Step 5) and read off a new stage, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 (Step 6) using the stage discharge curve (Figure 4-6).

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, and so forth. Then pick the next inflow point to be I_2. The process is continued until the entire inflow hydrograph has been routed through the storage basin.

Example 4.2 Detention Pond Routing Using Manual Method

Using the data presented in Figures 4-5 through 4-7 and Table 4-1, the computations in Table 4-2 were developed to route a storm through a pond. This inflow hydrograph is presented in columns 1 and 2 of Table 4-2. A step-by-step discussion is presented below.

1. Using the data tabulated in Column 2 of Table 4-2 to calculate:

$$\frac{I_1 + I_2}{2} \Delta t$$

   and tabulate these values in Column 3.

2. Given that $S_1 - O_1 \Delta t/2 = 0.05$ acre-foot for $H_1 = 0$ foot, find $S_2 + O_2 \Delta t/2$ by adding $0.05 + 0.01 = 0.06$ (Column 5 plus Column 3) for Column 6.

3. Enter the $S + O \Delta t/2$ storage characteristic curve in Figure 4-7 and read the stage at the value of 0.06 acre-foot. This value is 100.10 ft and is tabulated as stage H_2 in Column 7.

4. Using the stage of 100.10 ft from Step 3, enter the stage-discharge curve (Figure 4-6) and find the discharge corresponding to that stage. In this case, O is approximately 1 cfs and is tabulated in Column 8.

5. Assign the value of H_2 to H_1, find a new value of $S_1 - O_1 \Delta t/2$ from Figure 4-7, and repeat the calculations for Steps 2, 3, and 4. Continue this cycle until the entire hydrograph is routed through the storage basin. This may be until O = 0.0 cfs, unless there is no need to continue past a certain point.

Example Tabulation Of Storage Characteristics Curves

Stage (ft)	Storage (AF)	Discharge (cfs)	Discharge (AF/hr)	S – OÄt/2 (AF)	S + OÄt/2 (AF)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.16	2.04
104	2.8	95	7.85	2.14	3.46
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10	275	22.73	8.10	11.90

AF is acre-feet
Storage is obtained from Figure 4-5
Discharge is obtained from Figure 4-6
Ät = 10 min. = 0.167 hour
1 cfs = 0.0826 AF/hr

TABLE 4-1
EXAMPLE TABULATION OF STORAGE CHARACTERISTICS CURVES

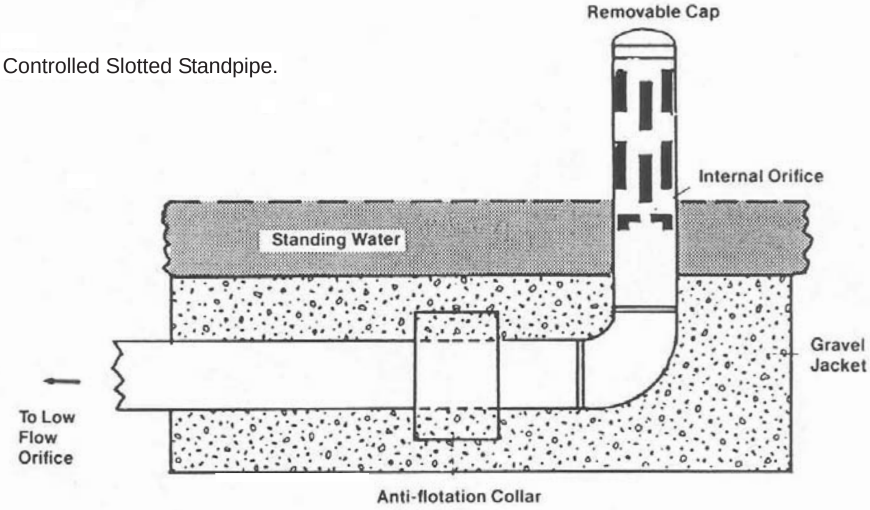
Storage Indication Method Example 4.2 Calculations

1	2	3	4	5	6	7	8
Time (min)	**Inflow (cfs)**	**(I1+I2)Ät/2 (AF)**	**H1 (ft)**	**S1 - O1 Ät/2 (AF)**	**S2 + O2 Ät/2 (AF)**	**H2 (ft)**	**Outflow, O (cfs)**
0	0						
10	2	0.01	0	0.05	0.06	101.10	1
20	27	0.2	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.97	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.48	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.26	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.68	105.10	3.60	4.27	105.50	116
120	20	0.39	105.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.30	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.01	101.40	0.35	0.35	100.70	10

Column 3 is computed from Column 2
Column 4 is from Column 7’s previous time step
Column 5 is determined from Figure 4-7 (based on Table 4-1)
Column 6 = Column 5 + Column 3
Column 7 is determined from Figure 4-7 (based on Table 4-1)
Column 8 is determined from Figure 4-6

TABLE 4-2
STORAGE INDICATION METHOD EXAMPLE 4.2 CALCULATIONS

A. Internally Controlled Slotted Standpipe.



A. Negatively Sloped Pipe from Riser.

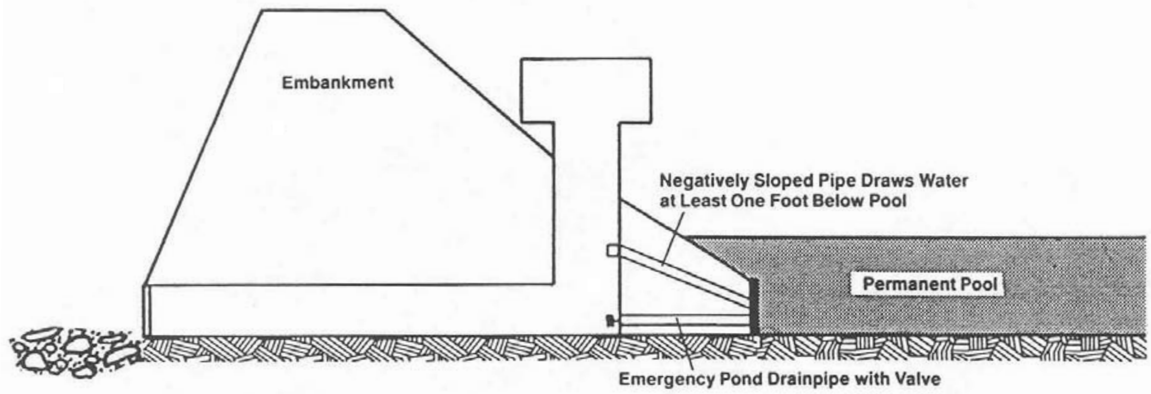


FIGURE 4-1
METHODS FOR EXTENDING DETENTION TIMES IN WET PONDS
Auburn Storm Water Management Manual

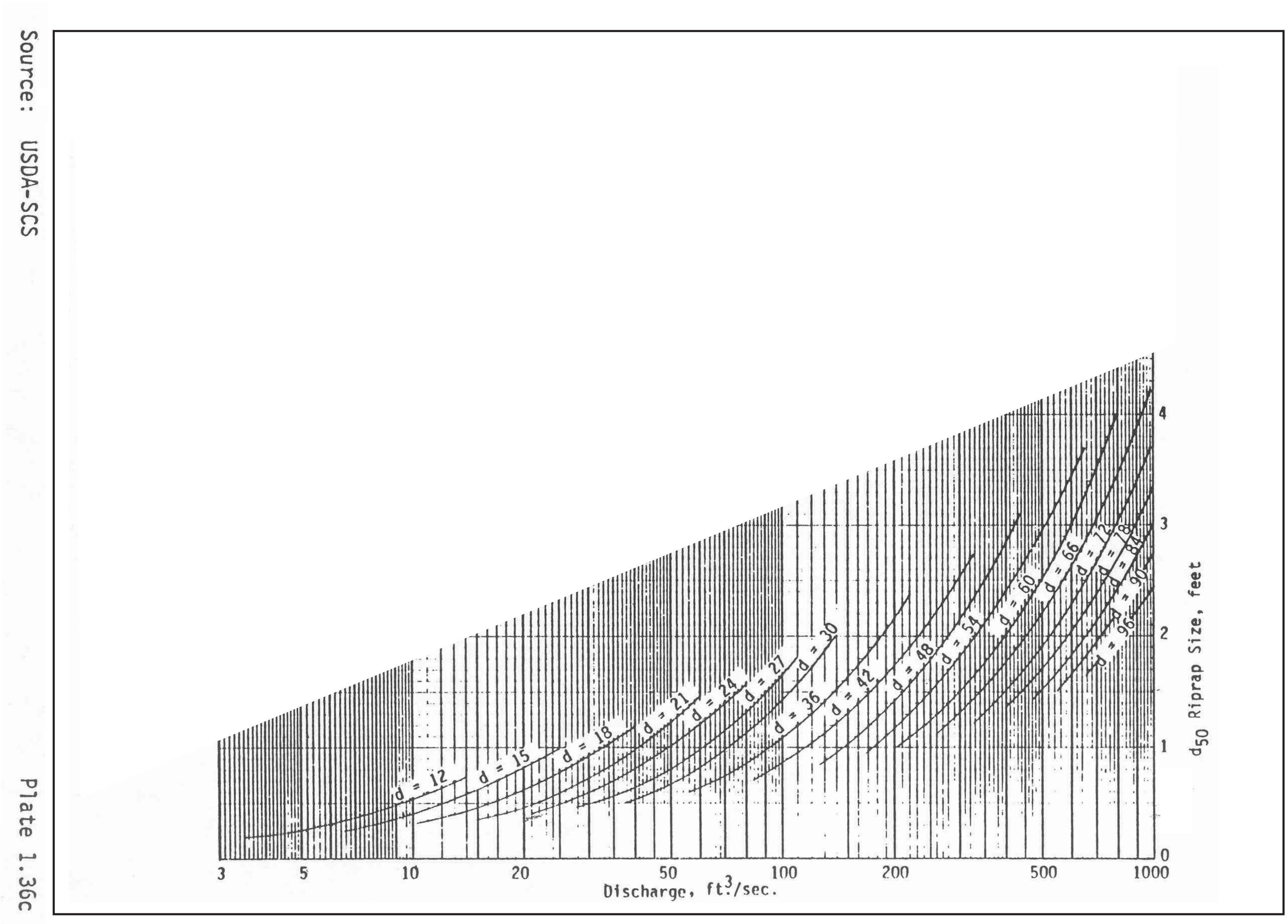


FIGURE 4-2
DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MINIMUM TAILWATER CONDITION (T_W<0.5 DIAMETER)
Auburn Storm Water Management Manual

Source: USDA-SCS

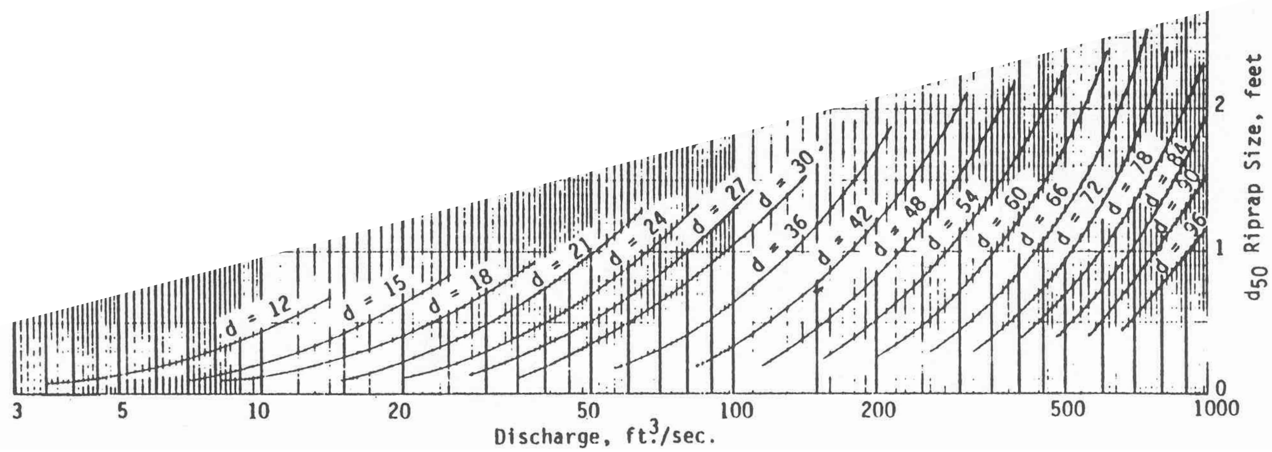


Plate 1.36d

FIGURE 4-3
DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MAXIMUM TAILWATER CONDITION ($T_W \geq 0.5$ DIAMETER)

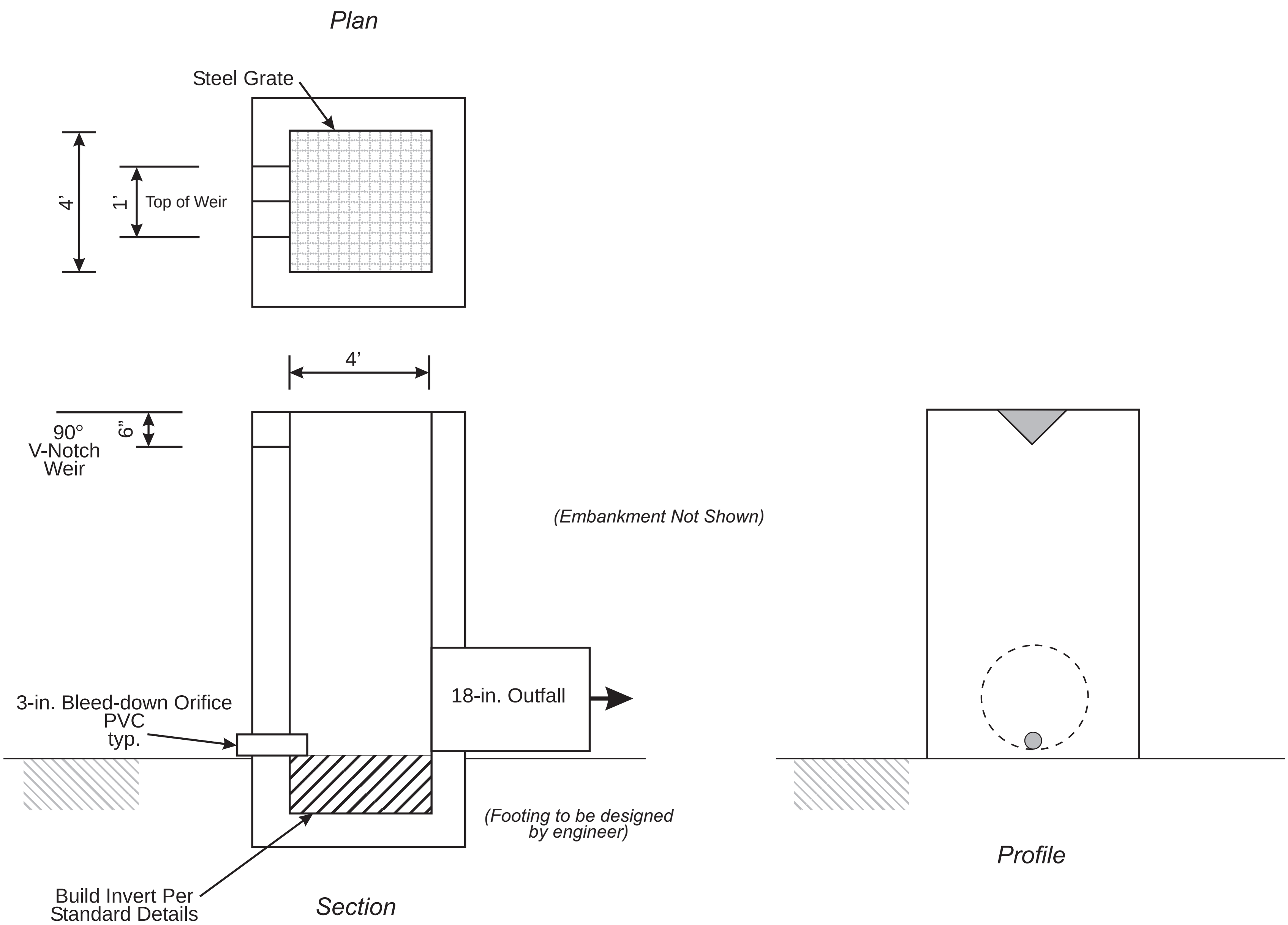


FIGURE 4-4
RECOMMENDED OUTFALL STRUCTURE EXAMPLE 4.1
Auburn Storm Water Management Manual

Example Stage-Storage Curve

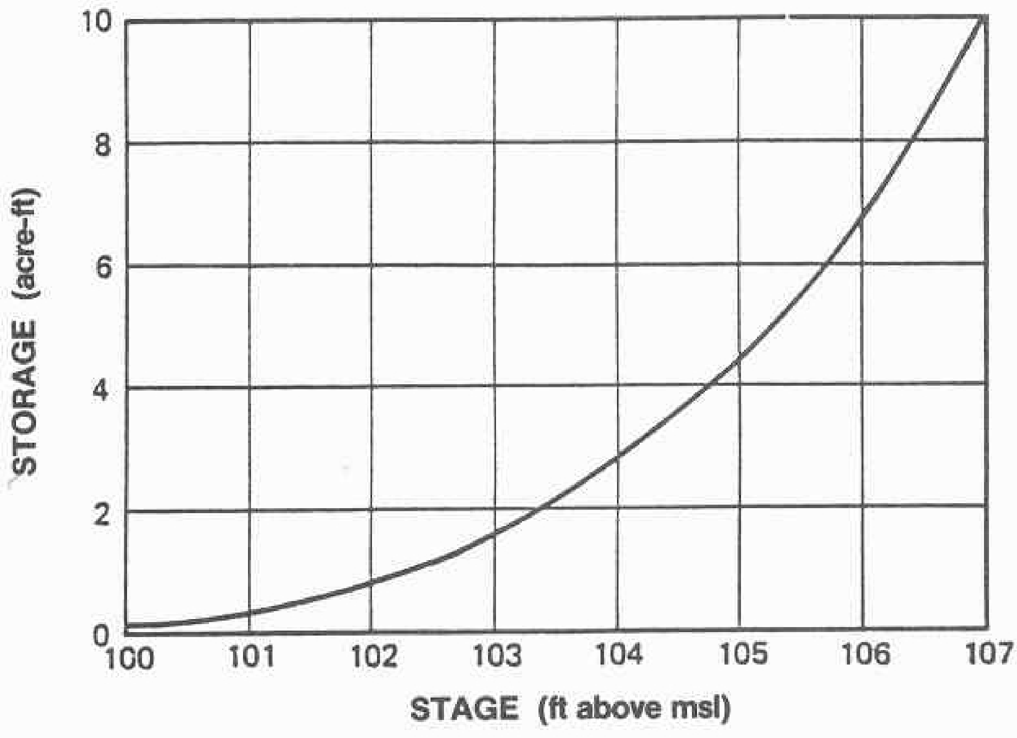


FIGURE 4-5
EXAMPLE STAGE-STORAGE CURVE
Auburn Storm Water Management Manual

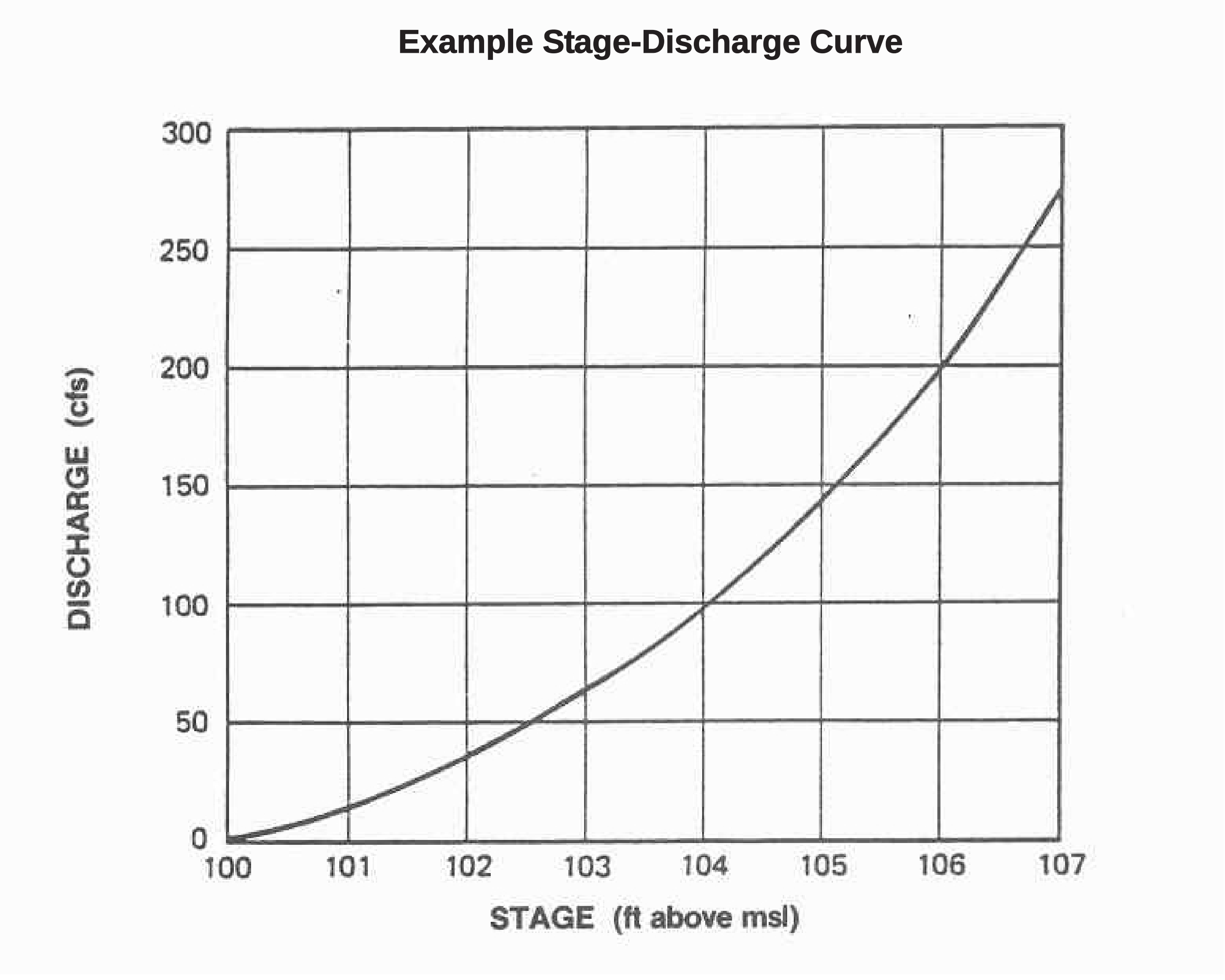


FIGURE 4-6
EXAMPLE STAGE-DISCHARGE CURVE
Auburn Storm Water Management Manual

Example Storage Characteristics Curve

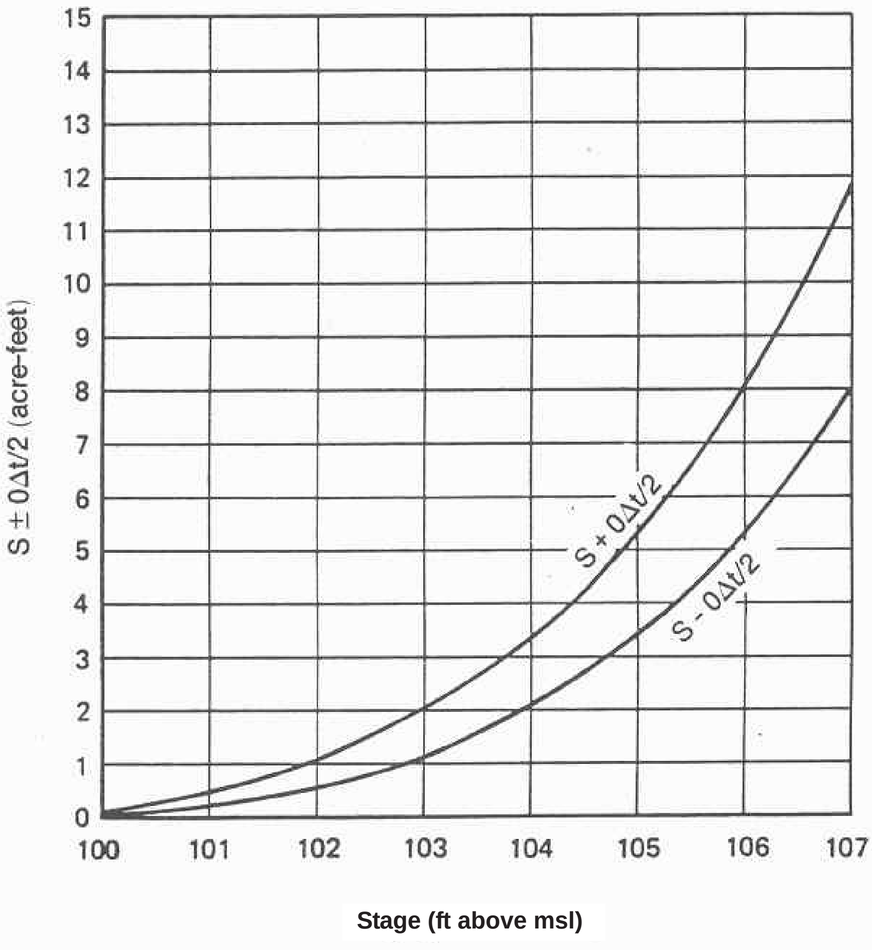


FIGURE 4-7
EXAMPLE STORAGE CHARACTERISTICS CURVE
Auburn Storm Water Management Manual

5. References

5. References

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APPENDIX A

Example Problem

APPENDIX A

Auburn Example Problem

A developer plans on converting a 10-acre parcel in the Auburn area into a subdivision. The parcel, as it stands, is in pasture (good condition), has Soil Type B, and a 1 percent average landscape slope from the northern end of the parcel to the outlet at the southern end–a distance of approximately 1,000 feet (ft).

The developer's site plan for the subdivision is shown in Figure A-1. Nine acres will be used for housing and roads, while 1 acre will contain a dry detention pond and common area. The housing density will be 4 houses/acre on the 9 acres containing housing. As illustrated, each acre or subbasin will be served by an inlet, with the main drainage line running along the road to the dry detention pond. The average subbasin grade toward each inlet will be 1 percent. The pond will discharge (free outlet) into an adjacent creek at the southern end of the property. The pond's bottom elevation should be set at elevation 90 ft[1]. In accordance with the City's storm water management guidelines, the drainage system should be sized to handle the 25-year, 24-hour storm. The dry detention pond must be able to attenuate the 2-, 5-, 10-, and 25-year, 24-hour storm events to pre-development levels.

Solution

Step 1: Pre-development Volume and Peak Discharge Determination

Use the Natural Resource Conservation Service (NRCS) Curve Number (CN) Method (Section 2.2.1) to determine the depth of runoff over the parcel.

Table 2-3 indicates that a pasture in good condition has a CN of 60.

Using Equation 2-2:

$S = (1000/60) - 10 = 6.7$

Using Equation 2-2 and the Design Storm Volumes from Table 2-2, the depth of runoff in inches can be calculated for each storm event. Multiply by the acreage to obtain volume of runoff.

[1] Arbitrary reference elevation for illustration purposes.

Storm Event	24-hour Rainfall inches	Weighted CN	S inches	Depth of Runoff inches	Acreage acres	Volume of runoff	
						ft^3	mg
2-year	4.2	60	6.7	0.86	10.0	31,300	0.234
5-year	5.4	60	6.7	1.54	10.0	55,900	0.418
10-year	6.3	60	6.7	2.12	10.0	77,000	0.653
25-year	7.2	60	6.7	2.75	10.0	99,700	0.746

Notes:
Runoff inches / 12 x Acreage x 43,560 ft^2/ac = Volume ft^3
ft^3 x 7.4805 gallons/ft^3 x $1/10^6$ = Volume MG
CN = Curve number
ft^3 = Cubic feet
mg = Million gallons

Use the worksheet in Figure 2-1 to determine the time of concentration (t_c). The first 100 ft of flow will be considered sheet flow, and thereafter, will be considered shallow concentrated flow. The Manning's roughness coefficient for pasture is 0.15. The 2-year, 24-hour rainfall is gathered from Table 2-2. The slope is given as 0.01 foot per feet (ft/ft). From Figure 2-1, the t_c is 20.0 minutes (0.334 hr).

$$t_c(\text{sheet flow}) = \frac{0.007(0.15 * 100)^{0.8}}{4.2^{0.5} * .01^{0.1}} = 0.18 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{shallow concentration flow})} = \frac{900}{3600 \times 1.613} = 0.154 \text{ hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{total})} = (0.18 + 0.154) * 60 \frac{\text{min}}{\text{hr}} \approx 20\text{min}$$

where V is calculated from

$$V_{up} = 16.1345(0.01)^{0.5} = 1.613 \text{ ft/sec} \qquad \text{(from Equation 2-4)}$$

This result exceeds the criteria for use of the Rational Method to determine peak discharge (Section 2.3.2). Use the TR-55 model to estimate the pre-development discharge rate (public domain software from NCRS).

TR-55 Input file:

```
Name       Description        Reach     Area(ac)   RCN   Tc
---------------------------------------------------------------------------
sub-div    pre-development    Outlet    10         60    0.334

Total area: 10 (ac)
```

TR-55 Output File:

Auburn SW Example
Lee County, Alabama
Hydrograph Peak/Peak Time Table
Peak Flow and Peak Time (hr) by Rainfall Return Period
2-Year 5-Year 10-Year 25-Year

(cfs)	(cfs)	(cfs)	(cfs)
7.82	15.52	22.13	29.10

Step 2: Determine Post-development Drainage Characteristics of Each Subbasin

It is presumed that the post-development peak flow rate will exceed the pre-development rate, so proceed with estimating the data required to calculate the flow rates for designing the drainage facilities and detention pond.

Determine the time of concentration for the 9 subbasins containing 4 houses/acre (or each house sits on approximately ¼ acre). The maximum hydraulic distance is interpreted from the site plan to be 210 ft for each *cul-de-sac*. The first 100 ft will be sheet flow, followed by shallow concentrated flow as the runoff enters the street/gutters and flows toward the inlets. The slope is given as 0.01 ft/ft.

$$t_{c(\text{sheet flow})} = \frac{0.007(0.15 * 100)^{0.8}}{4.2^{0.5} * .01^{0.4}} = 0.18\ \text{hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{shallow concentration flow})} = \frac{110}{3600 \times 2.032} = 0.015\ \text{hours} \qquad \text{(from Figure 2-1)}$$

$$t_{c(\text{total})} = (0.18 + 0.015) * 60 \frac{\text{min}}{\text{hr}} \approx 11\ \text{min}$$

where V is calculated from

$$V_{up} = 20.3282(0.01)^{0.5} = 2.032\text{ft/sec} \qquad \text{(from Equation 2-4)}$$

Using Figure 2-1, the t_c for each subbasin is approximately 11 minutes. This result indicates that the Rational Method can be used to determine the peak discharge to each inlet. Using Equation 2-8, C is determined from Table 2-4 as 0.60 and the Rainfall Intensity for a 25-year storm (11 min t_c) is interpolated from Table 2-1 as 7.18 inches/hour:

$$Q_T = 0.6 * 7.18 * 1 = 4.3\ \text{cfs} \qquad \text{(from Equation 2-7)}$$

The peak discharge for the 25-year storm for each *cul-de-sac* is 4.3 cubic feet per second (cfs).

Subbasin 5, containing the dry pond and nature area, will have a t_c of 5 minutes, according to the manual minimum requirements (Section 2.3.2):

$$Q_T = 0.35 * 8.7 * 1 = 3.04\ \text{cfs} \qquad \text{(from Equation 2-7)}$$

Using the Rational Method, the peak discharge for the dry detention basin is 3.04 cfs.

Step 3: Sizing the Drainage System

Gutters and Inlets

A standard 2-foot curb and gutter will be used for the roadway. For this example, place an inlet every 250 ft on either side of the road to carry street runoff. By applying the Rational Method, peak runoff from a section of street is 0.45 cfs:

$$Q_T = 0.9 * 8.7 * .06 = 0.45 \text{ cfs} \qquad \text{(from Equation 2-7)}$$

(area = 1/2 street width [10 ft] multiplied by street length [250 ft], t_c = 5 minutes, C = 0.9 from Table 2-4, intensity = 8.7 inches/hour from Table 2-1).

The gutter flow rates listed in Table 3-8 indicate that the hydraulic capacity is 1.00 cfs for a roadway slope of 0.5 percent. Therefore, the gutter flow will not exceed the capacity nor spread limitations. Each gutter inlet will interconnect with the main storm water drainage line. As stated in Section 3.2.1.4, the minimum pipe size will be 15 inches.

Table 3-9 indicates that an S-type inlet can handle the necessary hydraulic load (4.3 cfs) for all of the subbasins. Each inlet will be sized according to the City Standards details. Inlet location and spacing should meet the criteria identified in Section 3.2.1.6 and Section 3.3.1.7. The 250-foot spacing between inlets meets the City's minimum requirements, so there is not a problem with street drainage.

Pipes

Pipe sizing starts at the most distant hydraulic point, or Subbasin 1. As stated in Section 3.2.1.4, the minimum pipe size will be 15 inches. Start with the most upstream unit, pipe segment 1 (PS1), to start estimating the flow to be carried by the storm drain pipes. Pipe segment 5 will be sized the same way. The flow rate in pipe segment can be determined from Manning's equation (Equation 3-5), assuming that the pipe will be sized to operate under gravity flow. In this manner, each pipe section serving an individual subbasin (pipe segments 1 through 9) will need a capacity to handle 4.3 cfs. The following table was computed from Equation 3-6:

Q = 0.465/n * Ds^(8/3) * So^(1/2)

So =	0.01	ft/ft
n =	0.012	RCP pipe

Ds (in)	Ds (ft)	Q (cfs)
15	1.25	7.0
18	1.50	11.4
24	2.00	24.6
30	2.50	44.6
36	3.00	72.5

Therefore, each pipe segment from the *cul-de-sacs* will be 15 inches in diameter. Pipe segment 10 must carry the peak discharge from pipe section 1 and 5, or 8.6 cfs. Using the results from the above table, a pipe size of 18 inches diameter is required.

As one moves further downstream, the flow will be somewhat attenuated by the storage in the pipe. Because this is a small area, flow in pipe segment 11 will be based on the Rational Method, calculated as follows:

t_c = 11 minutes + 0.95 minutes = 11.95 minutes

This accounts for the travel time through Subbasin 1 (and 5) and the travel time in Pipe segment 10 calculated as time = distance/velocity.

The intensity from the interpolation of Table 2-1 values is 7.0 inches/hr. The peak discharge from Equation 2-8, with C = 0.6 and area = 4 acres, is 16.7 cfs:

$Q_T = 0.6 * 7.0 * 4 = 16.7 \text{ cfs}$ (from Equation 2-7)

Using the table above, the pipe diameter is sized at 24 inches. This process continues for sizing each of the pipes segments (12 through 14).

Pipe segment 12 = 30 inches

Pipe segment 13 = 36 inches

Pipe segment 14 = 36 inches

Alternatively, a storm water modeling program could be used to evaluate various pipe sizes. The output from the program needs to demonstrate the input data and assumptions.

Step 4: Sizing the Pond for Storm Water Attenuation

The post-development storm water sizing for the detention structure requires the use of computer modeling software. In this example, the contributing watershed was divided into two subbasins: the natural common area surrounding the pond, and the contributing housing development.

The first step is to estimate the maximum pond surface area based on a reasonable percent of the total project area. A pond area of 3 percent was selected for this problem. The maximum pond depth is 8 ft (Section 4.2.2). One foot of freeboard above the design depth will be used. Thus, the bottom pond elevation will be set at 90 ft and the maximum allowed water surface elevation will be 98 ft, with 1 foot of freeboard to the top of the berm at 99 ft.

The pond's surface area at elevation 98 ft is set at 3 percent of 10 acre = 0.3 acres. Using a side slope of 3:1 (Section 4.2.1.1), the surface area at the bottom of the pond will be 0.1 acre. The pond will be kidney shaped, with the length-to-width ratio maintained at 2:1 (Section 4.3.1) to make it more aesthetic while using the full length of the pond. It is presumed that the pond will be partially excavated with a berm along the most downstream edge near the stream.

The outfall will be sized to meet the target maximum depth during the 25-year design storm. Also, the 100-year storm runoff will be accommodated by installing an emergency overflow weir at an elevation above the 25-year peak depth. Furthermore, outlet structures should be set at elevations and sized such that post-development discharge is less than pre-development peak discharge and the pond is drawn down within 72 hours. A computer program was used to estimate an appropriate riser. Several iterations were conducted given the general sizes listed below. If there is difficulty in achieving all of the target area, maximum depth, or post-development flow rates, then the pond may have to be made larger, or additional storage provided in the development.

For this example, the outfall selected was a concrete box inlet with a smaller orifice near the bottom and a V-shaped weir near the top. After several iterations, the box outfall was sized as follows. The box was a standard pre-cast 2-foot x 3-foot box. The V-shape wier had a 90-degree opening and a depth of 6 inches. The crest of the riser is set at elevation 95 ft and the bottom of the V-weir was 94.5 ft. In addition, the pond is dewatered by a 3-inch circular opening set at elevation 90.1 ft. The box is drained by an 18-inch culvert pipe through the berm with an invert elevation of 90 ft. This pipe may seem small when compared to the pipe calculations above, but the pipe operates under pressure flow under the highest conditions, and the computer model confirmed its capacity. Figure 4-4 shows a standard design drawing of the outfall structure discussed here.

The routing of the post-development design storm can be accomplished using standard storm water modeling software. In this case, PondPack v7.0, a detention pond and watershed model developed by Haestad Methods, was used. Pond routing was accomplished by the level pool routing method. The output from the Pondpak simulation of the final configuration is as follows:

Storm Event	Peak Discharge from Pond (cfs)	Peak Elevation for Storm (ft)
2-year	1.90	95.07
5-year	15.11	95.57
10-year	21.97	96.10
25-year	23.45	96.96

The post-development discharge from every design storm is less than the pre-development rate. The 25-year storm is less than the target maximum elevation of 98 ft, but the 100-year storm also must be examined. Additional simulations were conducted to size a broad-crested weir over the embankment. The final weir was selected to be 10 ft wide, with a bottom elevation set at 98.0 ft. The peak 100-year flood elevation in the pond was 98.5 ft, which is 0.5 ft under the berm height. This level is the minimum allowable freeboard (0.5 ft) for the emergency spillway, so the berm height is adequately sized.

Pondpak also calculates the minimum drain time from the maximum water surface elevation (98 ft) until empty. This step is accomplished by routing the full volume of the pond through the outlet structure without any inflow. The minimum drain time was approximately 12 hours.

City of Auburn Forms

Once the design is completed, the City requires that the pertinent information be summarized on a set of forms. These forms were completed for this example.