DISTRIBUTION: 72

MINNESOTA DEPARTMENT OF TRANSPORTATION	TRANSMITTAL LETTER NO. 00-01
DEVELOPED BY: Office of Bridges and Structures	**MANUAL:** Drainage Manual
ISSUED BY: Office of Bridges and Structures	**DATE:** August 30, 2000

SUBJECT:

Chapter 1	Introduction
Chapter 2	Materials and Structural Design
Chapter 3	Hydrology
Chapter 4	Channels
Chapter 5	Culvert
Chapter 6	Energy Dissipator
Chapter 7	Storage Facilities
Chapter 8	Storm Drain Systems
Appendix A	Risk Assessment
Appendix B	TP-40 Rainfall Intensity Curves
Appendix C	Pipe Flow Design Charts
Appendix D	Open Channel Flow Charts
Appendix E	Computer Programs and References

The Drainage Manual has been revised to add more information and show current Mn/DOT procedures and practices. This manual replaces the previous manual in its entirety.

INSTRUCTIONS:

1. Place the transmittal record sheet in the front of the manual and record the transmittal letter number, date, and subject.

2. Insert in the new manual: The index and the above listed chapters.

3. Any technical questions regarding this transmittal should be directed to John Boynton, State Hydraulic Engineer at (651) 747-2162.

4. Any questions concerning missing manual sheets or extra transmittal letters should be directed to Map and Manual Sales, Room G-19, M.S. 260, (651) 296-2216. Please furnish in writing any address changes to: Mail Room G-21 Transportation Building, M.S. 275, 395 John Ireland Blvd., St. Paul, MN 55155. Any questions concerning mailing of this material should be directed to the Mail Room G-21 (651) 296-2420.

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State Bridge Engineer

DRAINAGE
MANUAL

MINNESOTA DEPARTMENT OF TRANSPORTATION

Developed by

Office of Bridges and Structures

DRAINAGE MANUAL INDEX

Chapter 1 INTRODUCTION

1.1 INTRODUCTION

The Minnesota Department of Transportation (Mn/DOT) Drainage Manual is based on the Model Drainage Manual (AASHTO, 1991 and 1999) produced by the American Association of State Highway and Transportation Officials (AASHTO) Task Force on Hydrology and Hydraulics. The manual has been developed to give basic working knowledge of hydrology and hydraulic design so that typical highway drainage features can be designed with minimal assistance. However, this manual cannot provide guidance on all hydrologic or hydraulic problems and is no substitute for experience or engineering judgment.

1.1.1 Updates

Comments and or recommendations, which may lead to revision of the manual, should be submitted in writing to the State Hydraulics Engineer. These comments and/or recommendations will be reviewed by the Office of Bridges and Structures. It is anticipated that revisions will be developed periodically and issued under a transmittal letter. All transmittal letters should be recorded on the Transmittal Record Sheet in the front of the manual. More complete reviews will be performed approximately biannually and revisions to the manual may be issued.

1.1.2 Drainage Manual Organization

The Drainage Manual contains eight chapters and five appendices.

Chapter 1	Introduction
Chapter 2	Materials and Structural Design
Chapter 3	Hydrology
Chapter 4	Channels
Chapter 5	Culvert
Chapter 6	Energy Dissipator
Chapter 7	Storage Facilities
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1.1.3 User Instruction

The purpose of the Drainage Manual is to provide a basis for uniform design practice for typical roadway drainage. Unfortunately, it is impractical to provide precise rules and methods for all design situations. Situations will arise where these standards do not apply. The use and adherence to standards does not exempt the engineer from the professional responsibility of developing an appropriate design. The engineer is responsible for identifying standards that are inappropriate for a particular project, and for obtaining the necessary variances to achieve proper design.

Each page shows the date of issuance and the page number. The pages are numbered according to the first index number appearing on that page. Revisions and supplements will be accompanied by a numbered Transmittal Letter. The number of the Transmittal Letter, date and subject should be recorded in the Transmittal Record Sheet located in the front of the manual.

References to specific computer programs, AASHTO guidelines, manuals and regulations are noted within the manual. It is assumed that the designer is knowledgeable in the use of the referenced items. Designers are responsible for obtaining computer program user manuals and keeping up-to-date with these programs and the latest drainage-related Federal regulations. Designers and agencies are responsible for keeping themselves up-to-date on changes to hydraulic design policy, procedures or practices.

The Drainage Manual is intended to be used in conjunction with other Mn/DOT design aids. Standard Specifications for Construction, Standard Plates, CADD standards, Technical Memorandums and other design manuals all provide information that will influence hydraulic design.

1.2 METRIFICATION

This manual is based on English units with all equations, tables and figures provided in English units. Two systems of units are currently used in transportation design, English and SI. SI is the symbol for the International System of Units (modern metric). Since some of the information sent to the hydraulic designer may be in metric units, or the plan is being developed as a metric plan, it is necessary to be able to convert between the two sets of units. Although this manual is in English, each engineer will decide to perform calculations in English or SI based on the format of input data (surveys, roadway design files, Quad maps); format of design aids (manuals, charts and programs); the preferences of the engineer; and the required format of the drainage output, (English or Metric).

This section is intended to assist with the conversion of units so that the designer can perform computations in English, and convert the final hydraulic design into metric for inclusion in a metric plan. For designers performing computations in metric units, SI versions of the equations, tables and figures are available in the metric version of the *Model Drainage Manual, Metric Edition* (AASHTO, 1999). In addition, many of Federal Highway Administration's (FHWA) Hydraulic Engineering Circular (HEC) and Hydraulic Design Series (HDS) manuals have been converted to metric or dual units and are available for reference.

1.2.1 Hard and Soft Conversion

Soft conversion is an exact conversion and hard conversion is a rounded off conversion. When using soft conversion, the English unit is converted to an exact metric equivalent. However, for hard conversion, the English unit is rounded off to a rationalized metric equivalent that is easier to work with and is appropriate for the particular conversion at hand. Soft conversion shall be used for proprietary items that do not yet have standard metric sizes and for values such as discharge and drainage area. Appropriate rounding should comply with the Mn/DOT Technical Manual. Hard conversion is used for items such as concrete pipe, and catch basin grates.

1.2.2 Conversion Procedure

Step 1 Make a decision to work in Metric or English units. The decision is based on units of input, design aids, and desired output units.

Step 2 Convert any information or equations that is not in the correct units. Maintain correct number of significant digits during conversion.

Step 3 Perform calculations.

Step 4 Convert discharges, flood elevations or any other necessary information to go on the plan to the correct format. Use correct type of conversion, soft or hard.

1.2.3 Conversion Tables

Expression	SI prefixes	Symbol	Example
.001	milli -	m	mm
1	-		m
1000	kilo -	k	km
1,000,000	mega -	M	Mg

Table 1.1 Unit Conversion

Unit Conversion Table				
Quantity	**Common Variables**	**English (Symbol)**	**Multiply by to get**	**SI (Symbol)**
Length	Pipe Diameter Precipitation Infiltration	inches (in)	25.4	millimeters (mm)
	Depth Head Length Elevation Headwater Tailwater Cross sections Profile plots Contour intervals Friction losses	feet (ft)	0.3048	meters (m)
	Distance	miles (mi)	1.609344	Kilometers (km)
Area	Drainage Area	square feet (ft^2) acres square miles (mi^2)	0.0929 0.4047 2.59	square meters (m^2) hectares (ha) square kilometers (km^2)
Volume	Storage Volume	cubic feet (ft^3)	0.0283	cubic meters (m^3)
Mass (weight)		pound (lb) ton	0.4536 0.9072	kilogram (kg) metric ton (t)
Force		pound force (lb_f)	4.45	newton (N)
Pressure		pounds per square foot (psf) poundforce per square inch (psi)	47.880 6.895	pascal (Pa) = N/m^2 kilopascal (kPa)
Rate	Flow Rate Discharge	cubic feet per second (cfs)	0.02832	cubic meters per second (m^3/s)
	Rainfall Intensity	inches per hour	25.4	millimeter per hour
	Flow Velocity	feet per second (fps)	0.3048	meters per second (m/s)
Acceleration	Acceleration Due to Gravity	32.174 ft/s^2	equals	9.807 m/s^2
Specific Gravity		dimensionless	no change	dimensionless
Angle		degree - minute - second (° ' ")	no change	degree - minute - second (° ' ")
Temperature		degree Fahrenheit (°F)	5/9(°F-32)	degree Celcius (°C)

Table 1.2 Pipe Hard Conversion

Hard Conversion of Pipe Diameters

Round Pipe
Mn/DOT Standard Plate 3000 & M3000

Inner Diameter Inches	Metric Size Nominal Millimeters
12	300
15	375
18	450
21	525
24	600
27	675
30	750
33	825
36	900
42	1050
48	1200
54	1350
60	1500
66	1650
72	1800
78	1950
84	2100
90	2250
96	2400
102	2550
108	2700
114	2850
120	3000

nominal diameters of 27" and 33" not provided for corrugated metal pipe

Pipe Arch, Span x Rise
Mn/DOT Standard Plate 3040 & M3040
2 2/3" x ½" (68 mm x 13 mm) corrugations

Span x Rise Inches	Metric Size Nominal Millimeters
17 x 13	430 x 330
21 x 15	530 x 380
24 x 18	610 x 460
28 x 20	710 x 510
35 x 24	885 x 610
42 x 29	1060 x 740
49 x 33	1240 x 840
57 x 38	1440 x 970
64 x 43	1620 x 1100
71 x 47	1800 x 1200
77 x 52	1950 x 1320
83 x 57	2100 x 1450

Reinforced Concrete Pipe Arch
Mn/DOT Standard Plate 3014 & M3014

Nominal Span Inches	Nominal Span Millimeters
22	560
28	725
36	920
44	1110
51	1300
58	1485
65	1650
73	1855
88	2235
102	2590
115	2920
122	3100
138	3505
154	3910
169	4285

Pipe Arch, Span x Rise
Mn/DOT Standard Plate 3041 & M3041
3" x 1" (75 mm x 25 mm) corrugations

Span x Rise Inches	Metric Size Nominal Millimeters
40 x 31	1010 x 790
46 x 36	1160 x 920
53 x 41	1340 x 1050
60 x 46	1520 x 1170
66 x 51	1670 x 1300
73 x 55	1850 x 1400
81 x 59	2050 x 1500
87 x 63	2200 x 1620
95 x 67	2400 x 1720
103 x 71	2600 x 1820
112 x 75	2840 x 1920
117 x 79	2970 x 2020
128 x 83	3240 x 2120
137 x 87	3470 x 2220
142 x 91	3600 x 2320

1.3 DATA COLLECTION

Efficient design requires the ability to identify possible sources of data, experience as to which sources will most likely yield desired data and knowledge of procedures for acquiring it. The effort necessary for data collection and compilation shall be tailored to the importance of the project. Not all of the data discussed in this chapter will be needed for every project.

1.3.1 Types and Sources of Data

The following items may be required depending on the nature of the project:
- watershed characteristics,
- site characteristics,
- geomorphic characteristics,
- hydrologic and meteorologic information, and
- environmental considerations and regulations.

Specific design parameters are often obtained from field reconnaissance. Moreover, it is easier to interpret published sources of data after an on-site inspection. Only after a thorough study of the area and a complete collection of all required information should the designer proceed with the final design of a hydraulic facility.

Watershed Characteristics

Data needed to define watershed characteristics includes:
- Contributing Drainage Area - Determine the boundaries and size of the contributing drainage area. Check for changes in the topographic contributing area.
- Slope - The slope of streams and ditches and the average slope of the watershed (basin slope), should be determined. Hydrologic and hydraulic procedures in other chapters of this manual are dependent on watershed and channel slopes.
- Watershed Land Use - Define and document present and expected future land use, particularly the location, and degree of anticipated urbanization.
- Soils - Determine soil type and classification.

Sources of watershed information include:
- field reviews;
- photos;
- field survey with conventional survey instruments;
- aerial photographs (conventional and infrared);
- zoning maps and master plans;
- land use maps;
- topographic, drainage area and other maps;
- soil and geological maps;
- municipal planning agencies;
- public ditch maps;
- landsat (satellite) images; and
- files.

Watershed data is available from agencies such as:
- State Agencies,
- Municipal Agencies,
- Local Agencies,
- County Agencies,
- Public Ditch Authorities,
- Watershed Districts and Watershed Management Organizations (WMO),
- Metropolitan Council or other local planning agencies,
- United States Geologic Society (USGS),
- Local Developers,
- Private Citizens,
- Private Industry, and
- Natural Resources Conservation Service (NRCS).

Site Characteristics

Any work being performed, proposed or completed, that changes the hydraulic characteristics of a stream or watercourse must be evaluated to determine its effect on the stream flow. The designer should be aware of plans for channel modifications, and any other changes which might affect the facility design.

- Pavement Alignment and Cross-sections
- Channel Geometry
  - Stream Profile - Stream bed profile should extend sufficiently upstream and downstream to determine the average slope and to encompass any proposed construction or aberrations.
  - Stream Cross-sections - Stream cross-sections shall be obtained that represent the conditions at proposed and existing structures. Stream cross-section data should also be obtained at other locations where stage-discharge and related calculations will be necessary.
  - Channel modifications
- Overbank or Floodplain Conditions
  - Roughness Coefficients - Roughness coefficients, ordinarily in the form of Manning's "n" values should be estimated for the flood limits of the stream.
  - Acceptable Flood Levels - Development and property use adjacent to the proposed site, both upstream and downstream, may determine acceptable flood levels. Floor elevations of structures or fixtures should be noted. In the absence of upstream development, acceptable flood levels may be based on tailwater and freeboard requirements of the highway itself. In these instances, the presence of downstream development may determine flood levels when an overtopping design of the highway is considered.
- Obstructions
  - Existing Structures - The location, size, description, condition, observed flood stages, and channel section relative to existing structures on the stream reach and near the site should be obtained in order to determine their capacity and effect on the stream flow. Any structures, downstream or upstream, which may cause backwater or retard stream flow should be investigated. In addition, the manner in which existing structures have been functioning with regard to such things as scour, overtopping, debris and ice passage, fish passage, etc. should be noted. For bridges, these data should include span lengths, type of piers, and substructure orientation which usually can be obtained from existing structure plans. Necessary culvert data includes size, inlet and outlet geometry, slope, end treatment, culvert material, and flow line profile.
  - Controls Affecting Design Criteria - Determine what natural or man-made controls should be considered in the design. Any ponds or reservoirs, along with their spillway elevations and design levels of operation, should be noted as their effect on backwater and/or stream bed aggradation may directly influence the proposed structure. Conservation and/or flood control reservoirs in the watershed may reduce peak discharges. Capacities and operation designs for these features should be obtained.
  - Debris and Ice -The quantity and size of debris and ice carried or available for transport by a stream during flood events should be investigated and such data obtained for use in the design of structures. In addition, the times of occurrence of debris and ice in relation to the occurrence of flood peaks should be determined; and the effect of backwater from debris and ice jams on recorded flood heights should be considered in using stream flow records.
- Storage Potential - Define parameters of all streams, rivers, ponds, lakes, and wetlands that will affect or may be affected by the proposed structure or construction. These data are essential for stream hydrology and may be needed for regulatory permits.
  - Outline the boundary (perimeter) of the water body for the ordinary highwater.
  - Elevation of normal as well as high water for various frequencies.
  - Detailed description of any natural or manmade spillway or outlet works including dimensions, elevations, and operational characteristics.
  - Determine classification of state waters.
  - Description of adjustable gates, soil and water control devices.
  - Profile along the top of any dam and a typical cross section of the dam.
  - Determine the use of the water resource (stock water, fish, recreation, power, irrigation, municipal or industrial water supply, etc.).
  - Note the existing conditions of the stream, river, pond, lake or wetlands as to turbidity and silt.
  - Determine riparian ownership(s).
- Flood History - Research the history of past floods and their effect on existing structures. Changes in channel and watershed conditions since the occurrence of the flood shall be evaluated in relating historical floods to present conditions.

Sources of site data include:
- site visit;
- photographs, aerial photographs (conventional and infrared), landsat (satellite) images;
- field survey with conventional survey instruments;
- highwater or flood data;
- flood insurance studies (FIS);
- zoning maps and master plans;
- topographic, and USGS quadrangle maps;
- site and location maps;
- soil and geological maps;
- municipal planning agency reports;
- plansheets (existing, as-built and proposed);
- existing conservation and/or flood control reservoirs design or structure operation records;
- files, plans, design data and hydraulic recommendations for existing and nearby structures;
- past flood scour data;
- maintenance records; and
- project files and correspondences (planning, budgeting, previous recommendations and scoping documents).

Site data is available from agencies such as:
- State Agencies,
- Local Agencies and Municipal Agencies,
- County Agencies,
- Surveyor,
- Field Review,
- U.S. Army Corps of Engineers (USACE),
- Natural Resources Conservation Service (NRCS),
- Federal Emergency Management Agency (FEMA),
- Private Citizens,
- U.S. Geological Survey (USGS),
- Federal Highway Administration (FHWA),
- Watershed Districts and Watershed Management Organizations (WMO),
- Public Ditch Authorities, and
- Reservoir Sponsors.

Geomorphological Data

Geomorphology is important in the analysis of channel stability and scour. The use of an alluvial river computer model will increase the need for geomorphological data to calibrate the model. Also, geomorphological data is needed to determine the presence of bed forms so a reliable Manning's n as well as bed form scour can be estimated. The need for geomorphological data will be determined by the designer.
- Stream Classification - Define stream classification parameters.
- Sediment Transport - Investigate degradation, aggradation and sediment movement.
- Form Stability - Define stability of stream form over time.
- Scour Potential - Research scour history and evidence of scour. Scour potential is a function of the stability of the natural materials at the facility site, tractive shear force exerted by the stream and sediment transport characteristics of the stream.
- Bed and Bank Material - Identify bed and bank material.

Sources of Geomorphological information include:
- bed and bank material samples,
- geotechnical study, and
- soil maps.

Geomorphological data is available from:
- Natural Resources Conservation Service (NRCS), and
- Site Investigations.

Hydrologic Data

Hydrologic data is collected to determine the runoff rate or volume for which a hydraulic facility must be designed.

- Rainfall
- Discharge

Sources of hydrologic data include:

- meteorological data (precipitation records),
- gaging station data (peak and continuous),
- regional regression studies,
- site studies,
- flood insurance studies (FIS),
- TP-40,
- rainfall-intensity-duration curves,
- hydrologic studies for nearby structures,
- watershed districts overall plan, and
- regional and local flood studies.

Hydrologic data is available from agencies such as:

- National Oceanic and Atmospheric Administration (NOAA),
- Climatic Data Center,
- State Agencies,
- National Weather Service (NWS),
- Department of Natural Resources (DNR),
- U.S. Corps of Engineers (USACE),
- U.S. Geological Survey (USGS),
- Federal Highway Administration (FHWA), and
- Natural Resources Conservation Service (NRCS).

Environmental Considerations

The need for environmental information stems from the need to investigate and mitigate potential impacts. The environmental sensitivity of a site is a function of the design configuration and surface water attributes (water use, water quality and standards, aquatic and riparian wildlife biology, and wetlands). Environmental data needs may be categorized as follows.

- Regulatory and Permits
- Environmentally Compatible Design - Physical, chemical and biological information may be required to identify environmental impacts relevant to the facility design. For instance, a culvert may be designed to accommodate fish migration by creating a low flow channel. Other factors that should be considered include: fish habitat, sediment transport, water supply, recreational use, water velocity, and water quality.
- Wetlands - Each wetland is unique and evaluation should be coordinated with the appropriate permitting authorities.

Sources of environmental information include:

- flood plain delineations and studies,
- FHWA design criteria and practices,
- Minnesota state laws,
- local ordinances and master plans,
- Corps of Engineers Section 404 permit program,
- boat passage,
- fish migration,
- national wetland inventory (NWI),
- U.S. Geological Survey maps ("Quad" sheets),
- wetland mitigation plans,
- protected waters (DNR),
- right of way limitations,
- water quality studies (MPCA),
- permit applications, and
- project files and correspondences (planning, previous commitments and scoping documents).

Environmental, permitting or regulatory information is available from agencies such as:
- Department of Natural Resources (DNR),
- Federal Emergency Management Agency (FEMA),
- Federal Highway Administration (FHWA),
- U.S. Environmental Protection Agency (EPA),
- Board of Soil and Water Resources (BWSR),
- Watershed Districts or Watershed Management Organization (WMO),
- U.S. Fish and Wildlife Service (USFWS),
- U.S. Forest Service (USFS),
- Natural Resources Conservation Service (NRCS),
- U.S. Corps of Engineers (USACE), and
- Minnesota Pollution Control Agency (MPCA).

1.3.2 Survey Information

Complete and accurate survey information is necessary to develop a design that will best serve the requirements of a site. The project surveyor is not likely to have a good knowledge of drainage design, and will need to coordinate with the drainage designer to identify the extent of the survey. The drainage designer should inspect the site and its contributing watershed to determine the required field and/or aerial drainage survey needed for the hydraulic analysis and design. Survey requirements for small drainage facilities such as 36 inch culverts are less extensive than those for major facilities such as bridges. However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility. Data collection should be as complete as possible during the initial survey in order to avoid repeat visits.

The following data can be obtained or verified during a field survey:
- contributing drainage area characteristics,
- stream reach cross sections and thalweg profile,
- location and survey of existing structures,
- elevation of flood prone property,
- general ecological information about the drainage area and adjacent lands,
- high water elevations including the date of occurrence, and
- ordinary highwater elevation.

At many sites photogrammetry (aerial photo survey) is an excellent method of securing the topographical components of a drainage survey. Planimetric and topographic data covering a wide area are easily and cost effectively obtained . A supplemental field survey is required to provide data in areas obscured on the aerial photos (underwater, under trees, etc.).

1.3.3 Field Reviews

Field reviews are usually initiated for one of several reasons, data is needed for a design project, documentation of performance during a flood event, or operation and maintenance review.

<u>Project field review</u>

Field reviews allow the drainage designer to become familiar with the site. The most complete topographic survey cannot adequately depict all site conditions or substitute for personal inspection by someone experienced in drainage design. Factors that most often need to be confirmed by field inspection are:
- selection of roughness coefficients,
- evaluation of apparent flow direction and diversions,
- flow concentration,
- observation of land use and related flood hazards,
- geomorphic relationships,
- highwater marks or profiles and related frequencies, and
- existing structure size and type.

Photographs are taken during a field review. For centerline culverts and bridges, photos should be taken looking upstream and downstream from the site as well as along the contemplated highway centerline in both directions. Details of the stream bed and banks should also be photographed along with structures in the vicinity both upstream and downstream. Close up photographs complete with a scale or grid should be taken to facilitate estimates of the stream bed gradation.

When drainage areas are checked in the field, or when the drainage survey is conducted, the size of all inplace structures in the vicinity should be noted, and highwater information should be obtained from local citizens or agencies. The performance of structures, which have been conveying runoff from a watershed for a long time, is an excellent source of information for estimating design discharges. Upstream structures may throttle the runoff and reduce the peak discharge. Downstream channels or structures may cause a backwater effect on the structure being designed, and thereby affect its operation and design.

Flood field review

Each District should be interested in maintaining a permanent file of highwater or flood data. Every year there are claims for flood damages caused or partially caused by highway drainage structures, new and old. Many times it can be proven that the Department is not at fault, but there also are instances where the complaints are justified or where claims cannot be disproven. Therefore, it is strongly recommended that each District established a permanent file on highwater data which will be available for design purposes as well as for settlement of claims.

During a flood or major runoff event, measure flow parameters so the discharge can be calculated and hydraulic models calibrated. A suggested flood analysis form is shown at the end of this section. All of the information called for on this form is important for determination of the discharge which occurred, the adequacy of the structure, and the possible liability of the State for damages. The following is information that should be collected during a flood event.

- Locate the structure precisely with enough accuracy and detail for positive identification.
- The location of highwater marks must be documented.
- Record of the date of the highwater is important.
- Record the date of the observation or measurements which may be taken some time after the peak has passed the structure.
- General information on the size and type of structure is important for descriptive purposes and for estimating the discharge. In the case of culverts, every one of the items listed (size, type, length, inlet and outlet elevations and type of inlet) have a direct influence on the hydraulic capacity.
- Presence of ice or debris at the structure is important in determining whether or not the structure was operating at capacity. Sometimes remedial measures, such as debris barriers or splitter walls, are needed rather than additional waterway opening to alleviate flooding.
- Silt deposits or erosion should be noted where significant, so that proper maintenance work can be done to protect the efficiency and stability of the structure and to prevent erosion from progressing onto private property and becoming a basis for a claim.
- Information on the drainage area characteristics is important for hydrological reasons and also for estimating the general severity of flood damage.
- Where water flows over the highway, a profile of the road which will cover more than the inundated section should be taken after the flood has subsided. With this profile, and the upstream and downstream highwater elevations, the discharge over the road can be estimated.
- Record other factors, such as the influence of dikes, dams, or water overflowing the road, which need elaboration. If the form lacks sufficient space for a thorough description of these effects, such information may be recorded on the back of the form or on extra sheets.

When recording elevations, it may not always be convenient to use mean sea level datum. A local benchmark set to assumed datum will suffice. The objective is to have all levels at a site on the same datum. Such benchmarks should be recorded on the form so they can be found in the future for subsequent measurements or for tying into sea level datum. District personnel can mark the highwater levels if properly informed where to make the marks in relation to the inlet and outlet of the structure. Later, survey crews can determine the elevations and complete the form. Both dates should be accurately recorded.

Highwater elevations upstream and downstream of a structure or flooded roadway are the most important data and are often incorrectly obtained. When water enters a structure, the water surface is drawn down due to the increased velocity, and there is turbulence at the outlet. Elevations should not be taken in these disturbed areas. Water level elevations should be taken a short distance away from the inlet and outlet, where the water level is representative of the general pool or flow before it is affected by the drawdown or after the turbulence has dissipated. This distance may be as little as five or ten feet for small pipe culverts and up to one bridge length away from the opening for larger structures. In case of uncertainty, make several highwater marks so they can be checked later with a level. Usually the highest water levels will occur on both sides at the same time, so it is possible to use visual highwater marks (such as trash lines) with fair reliability if the peak flow cannot be observed. It is important that elevations be taken both upstream and downstream of structures so the restrictive effects and discharge can be estimated. One elevation at the inlet end is of little real value either for discharge measurements or for determination of damages.

Return to District Hydraulics Unit

FIELD OBSERVATION INFORMATION FOR FLOOD ANALYSIS

1. Location of Structure: S.P.________________________ T.H. ________________________

   Name of Stream:____

   Sta., Mile Point, Sec. etc.:____

2. Date of Highwater:________________________ Date of Observation:____________________

3. General Information:

   A. Bridge No.__________________ Low Beam Elev.______________________________

   B. Culvert Size___________ Type: ☐ CMP ☐ CMPA ☐ RCP ☐ RCPA ☐ BOX

   ☐ Other: ________________________________

   Length_______________ Inlet Elev.________________ Outlet Elev.______________

   Type of Inlet: ☐ Apron ☐ Headwall ☐ Sloped or Beveled ☐ Square End

   ☐ Other: ____

4. Highwater Information:

   A. Elev. upstream of structure or roadway ____

   B. Elev. downstream of structure or roadway ______________________________________

   C. Elevation at centerline when roadway is flooded __________________________________

5. Give statement as to effect of ice, debris, silting, or erosion.

   ____

   ____

   ____

6. If structures upstream or downstream affect this site give same information as above on separate sheet and attach.

7. Information on Drainage Area:

   A. Type of Soil: ☐ Granular ☐ Semi Granular ☐ Plastic ☐ Exposed Ledge Rock

   B. Land Use: ☐ Swamp ☐ Forest ☐ Cultivated ☐ Pasture ☐ Urbanized ☐ Other: ___________

   C. Topography: ☐ Steep Rugged ☐ Hilly ☐ Moderately Rolling ☐ Gently Rolling ☐ Flat

8. Record any other pertinent information here and on the reverse side as necessary.

Note: During unusually high water conditions contact the Hydraulics Unit at once. See Drainage Manual Section 1.3.3 for instructions on obtaining data.

1.3.4 Data Evaluation

Once the needed data and information have been collected, the next step is to compile it into a usable format. The designer must ascertain whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations or results. The main reason for analyzing the data is to draw all of the various pieces of collected information together, and to fit them into a comprehensive and accurate representation of the hydrologic and hydraulic characteristics of a particular site.

Experience, knowledge, and judgment are important parts of data evaluation. Historical information should be reviewed to determine whether significant changes have occurred in the watershed and whether the data can be used. Data acquired from the publications of established sources such as the USGS can usually be considered as valid and accurate. Data should always be subjected to careful study by the designer for accuracy and reliability. Basic data, such as stream flow data derived from non-published sources, should be evaluated and summarized before use. Maps, aerial photographs, satellite images, and land use studies should be compared with one another and with the results of the field survey and any inconsistencies should be resolved. General reference material should be consulted to help define the hydrologic character of the site or region under study and to aid in the analysis and evaluation of data.

Often sensitivity studies can be used to evaluate data and the importance of specific items to the final design. Sensitivity studies consist of conducting a design with a range of values for specific data items. The effect on the final design can then be established. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Time and effort should then be spent on the more sensitive data items making sure these data are as accurate as possible. This does not mean that inaccurate data are accepted for less sensitive data items, but it allows prioritization of the data collection process given a limited budget and time allocation. The results of this type of data evaluation should be used so that as reliable a description as possible of the site can be made within the allotted time and the resources committed to this effort. The effort of data collection and evaluation should be commensurate with the importance and extent of the project and/or facility.

1.4 DOCUMENTATION

The design of highway drainage facilities must be well documented. Frequently, it is necessary to refer to plans and specifications after construction is completed. Information will be needed in the case of litigation, failure, or just for future reference. Documentation should be easy to understand, and include all design assumptions and enough data and computations to allow someone to understand why the facility was designed as it was. Documentation should include: engineering calculations and analysis, drainage area and other maps, filed survey information, project correspondence relative to hydraulic considerations, and permit information. These documents should be in the appropriate format.

The major purpose of providing good documentation is to define the design procedure that was used and to show how the final design and decisions were arrived at. Often there is expressed the myth that avoiding documentation will prevent or limit litigation losses as it supposedly precludes providing the plaintiff with incriminating evidence. This is seldom if ever the case and documentation should be viewed as the record of reasonable and prudent design analysis based on the best available methodology. Thus, good documentation can provide the following:

- protection for the Agency by proving that reasonable and prudent actions were, in fact, taken, (such proof should certainly not increase the potential court award and may decrease it by disproving any claims of negligence by the plaintiff);
- identifying the situation at the time of design which might be very important if legal action occurs in the future;
- documenting that rationally accepted procedures and analysis were used at the time of the design which were commensurate with the perceived site importance and flood hazard, (this should further disprove any negligence claims);
- providing a continuous site history to facilitate future reconstruction;
- providing the file data necessary to quickly evaluate any future site problems that might occur during the facility's service life; and
- expediting plan development by clearly providing the reasons and rationale for specific design decisions.

Documentation should not be considered as occurring at specific times during the design or as the final step in the process which could be long after the final design is completed. Documentation should rather be an ongoing process and part of each step in the hydrologic and hydraulic analysis and design process. This will increase the accuracy of the documentation, provide data for future steps in the plan development process, and provide consistency in the design even when different designers are involved at different times of the plan development process.

1.4.1 Documentation Procedures

A complete hydrologic and hydraulic design and analysis documentation file for each waterway encroachment or crossings should be maintained by the hydraulic section. The documentation file should contain design/analysis data and information which influenced the facility design and which may not appear in other project documentation. Following are the Agency's practices related to documentation of hydrologic and hydraulic designs and analyses.

- Hydrologic and hydraulic data, preliminary calculations and analyses and all related information used in developing conclusions and recommendations related to drainage requirements, including estimates of structure size and location, should be compiled in a documentation file.
- The drainage designer should document all assumptions and selected criteria including the decisions related thereto.
- The amount of detail of documentation for each design or analysis shall be commensurate with the risk and the importance of the facility.
- Organize documentation to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done by predecessors.
- Provide all related references in the documentation file to include such things as published data and reports, memos and letters, and interviews. Include dates and signatures where appropriate.
- Documentation should include data and information from the conceptual stage of project development through service life so as to provide successors with all information.
- Organize documentation to logically lead the reader from past history through the problem background, into the findings, and through the performance.
- An executive summary at the beginning of the documentation will provide an outline of the documentation file to assist users in finding detailed information.
- Hydrologic/Hydraulic documentation should be retained in the project plans or other permanent location at least until the drainage facility is totally replaced or modified as a result of a new drainage study.

1.4.2 Documentation Content

The following items should be included in the documentation file. Figure 1.1 is a example of a documentation project check list. The intent is not to limit the data to only those items listed, but rather establish a minimum requirement consistent with the hydraulic design procedures as outlined in this manual. If circumstances are such that the drainage facility is sized by other than normal procedures or if the size of the facility is governed by factors other than hydrologic or hydraulic factors, a narrative summary detailing the design basis should appear in the documentation file. Additionally, the designer should include in the documentation file items not listed below, but which are useful in understanding the analysis, design, findings, and final recommendations.

It is very important to prepare and maintain in a permanent file the as-built plans for every drainage structure to document subsurface foundation elements such as footing types and elevations, pile types and (driven) tip elevations, etc. There may be other information which should be included or may become evident as the design or investigation develops. This additional information should be incorporated at the discretion of the designer.

<u>Drainage Design</u>

The following list recommends file contents for all types of drainage projects including bridges, culverts and storm drain systems. Design documentation should include all the information used to justify the design, including:

- photographs (ground and aerial);
- contour mapping;
- vicinity maps and topographic maps;
- watershed map or plan including:
  - flow directions,
  - watershed boundaries,
  - watershed areas,
  - natural storage areas;
- surveyed data reduced to include:
  - existing hydraulic facilities,
  - existing controls,
  - profiles - roadway, channel, driveways, cross sections - roadway, channels, faces of structures;
- flood insurance studies and maps by FEMA;
- soil maps;
- field trip report(s) which may include:
  - video cassette recordings,
  - audio tape recordings,
  - still camera photographs,
  - movie camera films,
  - written analysis of findings with sketches;
- reports from other agencies (local, State or Federal);
- newspapers clippings;
- interviews (local residents, adjacent property owners, and maintenance forces);
- reports from other agencies;
- hydrological investigations and report;
- hydraulic report; and
- approvals.

<u>Hydrology</u>

The following items used in the design or analysis should be included in the documentation file:

- contributing watershed area size and identification of source (map name, etc.);
- design frequency and decision for selection;
- hydrologic discharge and hydrograph estimation method and findings;
- flood frequency curves to include design flood, check floods, discharge hydrograph, and any historical floods; and
- expected level of development in upstream watershed over the anticipated life of the facility (include sources of and basis for these development projections).

Storm Drains

The following items should be included in the documentation file:

- computations for inlets and pipes, including hydraulic grade lines;
- complete drainage area map;
- design frequency;
- information concerning outfalls, existing storm drains, and other design considerations;
- a schematic indicating storm drain system layout;
- design notes, and correspondence relating to design decisions;
- history of performance of existing structure(s); and
- assumptions.

Open Channels

The following items should be included in the documentation file:

- stage discharge curves for the design, 100-year and any historical water surface elevation(s);
- cross-section(s) used in the design water surface determinations and their locations;
- roughness coefficient assignments ("n" values);
- information on the method used for design water surface determinations;
- observed highwater, dates, and discharges;
- channel velocity measurements or estimates and locations;
- water surface profiles through the reach for the design event, 100 year event, and any historical floods;
- design or analysis of materials proposed for the channel bed and banks;
- energy dissipation calculations and designs; and
- copies of all computer analyses.

Bridges

The following items should be included in the documentation file:

- design and highwater for undisturbed, existing and proposed conditions;
- stage-discharge curve for undisturbed, existing and proposed conditions;
- cross-section(s) used in the design and 100 year highwater determination;
- roughness coefficient ("n" value) assignments;
- information on the method used for design highwater determination;
- observed highwater, dates, and discharges;
- velocity measurements or estimates and locations (include both the through-bridge and channel velocity) for design and 100-year floods;
- performance curve to include calculated backwater, velocity and scour for design floods, 100 year flood, and 500-year flood for scour evaluation;
- magnitude and frequency of overtopping flood;
- copies of all computer analyses;
- complete hydraulic study report;
- economic analysis of design and alternatives;
- risk assessment;
- bridge scour results;
- roadway geometry (plan and profile);
- potential flood hazards to adjacent properties;
- identification and location of the facility;
- design notes, and correspondence relating to design decisions;
- history of performance of existing structure(s); and
- assumptions.

Culverts

The following items should be included in the documentation file:

- culvert performance curves;
- allowable headwater elevation and basis for its selection;
- cross-section(s) used in the design highwater determinations;
- roughness coefficient assignments ("n" values);
- observed highwater, dates, and discharges;
- stage discharge curve for undisturbed, existing and proposed conditions to include the depth and velocity measurements or estimates and locations for the design event, 100 year event , and check floods;
- performance curves showing the calculated backwater elevations, outlet velocities and scour for the design event, 100 year event, and any historical floods;
- type of culvert entrance condition;
- culvert outlet appurtenances and energy dissipation calculations and designs;
- copies of all computer analyses and standard computation sheets;
- roadway geometry (plan and profile);
- potential flood hazard to adjacent properties;
- identification and location of the facility;
- design notes, and correspondence relating to design decisions;
- history of performance of existing structure(s); and
- assumptions.

Construction and Operations

Construction or operation documentation should include:

- plans,
- revisions,
- as-built plans and subsurface borings,
- photographs,
- record of operation (during flooding events, complaints and resolutions),
- engineering cost estimates, and
- actual construction costs.

Computer Files

The following items should be included in the documentation file and be clearly labeled:

- input data listing, and
- output results of selected alternatives.

Figure 1.1 Documentation Project Check List
(Check Appropriate Items)

Engineer ______________________________ Project ______________________________

City/County ___________________________ Description ___________________________

REFERENCE DATA

Maps:
- ☐ USGS Quad Scale: _________ Date: _________
- ☐ Mn/DOT
- ☐ Local Zoning Maps
- ☐ Flood Hazard Delineation (Quad.)
- ☐ Flood Plain Delineation (HUD)
- ☐ Local Land Use
- ☐ Soils Maps
- ☐ Geologic Maps
- ☐ Aerial Photos Scale: _________ Date: _________

Studies By External Agencies:
- ☐ USACE Flood Plain Information Report
- ☐ SCS Watershed Studies
- ☐ Local Watershed Management
- ☐ USGS Gages & Studies
- ☐ Interim Flood Plain Studies
- ☐ Water Resource Data
- ☐ Regional Planning Data
- ☐ Forestry Service
- ☐ Utility Company Plans

Studies By Internal Sources:
- ☐ Hydraulics Section Records
- ☐ District Drainage Records
- ☐ Flood Records (High Water, Newspaper)

Technical Aids:
- ☐ Mn/DOT Drainage Manual
- ☐ Mn/DOT & FHWA Directives
- ☐ Technical Library

HYDROLOGIC-HYDRAULIC REPORTS

Data Reports:
- ☐ Agency Data
- ☐ Other Agency Data
- ☐ Environmental Reports
- ☐ Surface Water Environmental Study
- ☐ Surface Water Environmental Revisions
- ☐ Reconnaissance Report
- ☐ Reconnaissance Revisions Report
- ☐ Location Report
- ☐ Location Revisions Report
- ☐ Drainage Survey Inspection Report
- ☐ Drainage Survey Inspection Report Revisions
- ☐ Hydraulic Design Report
- ☐ Hydraulic Design Report - Revisions
- ☐ Construction Report
- ☐ Construction Report - Revisions
- ☐ Hydraulic Operation Report
- ☐ Hydraulic Operation Report - Revisions

HYDRAULIC DESIGN

Calibration Of High Water Data:
- ☐ Discharge and Frequency of H.W. elevation
- ☐ Influences Responsible for H.W. elevation

Analyze Hydraulic Performance
- ☐ Existing Facility for Minimum Flow through design event
- ☐ Proposed Facility for Minimum Flow through design event

Design Appurtenances:
- ☐ Dissipators
- ☐ Rip Rap
- ☐ Erosion & Sediment Control
- ☐ Fish & Wildlife Protection

Computer Programs:
- ☐ Culvert Design: HY8, CDS
- ☐ Water Surface Profile: HEC-2, HEC-RAS
- ☐ WSPRO Bridge Backwater
- ☐ HYDRAIN module: _________
- ☐ FESWMS-2DH
- ☐ Geopak Drainage
- ☐ Other: _________________

HYDROLOGY

Flood History:
- ☐ External Sources
- ☐ Personal Reconnaissance
- ☐ Maintenance Records
- ☐ Photographs

High Water Elevations:
- ☐ Mn/DOT Survey
- ☐ External Sources
- ☐ Personal Reconnaissance

Discharge Calculations:
- ☐ Drainage Areas
- ☐ Rational Formula
- ☐ SCS: peak flow, or hydrograph
- ☐ Gaging Data
- ☐ Regression Equations
- ☐ Area-Discharge Curves
- ☐ Log-Pearson Type III Gage Rating
- ☐ Computer Programs: HYDRO, TR 20, TR 55, HEC-1
  Other: _________________

1.5 REFERENCES

American Association of State Highway and Transportation Officials. 1982. Highway Drainage Guidelines.

American Association of State Highway and Transportation Officials (AASHTO), 1991. *Model Drainage Manual*. ISBN I-56051-010-2.

American Association of State Highway and Transportation Officials (AASHTO), 1999. *Model Drainage Manual, Metric Edition*. ISBN I-56051-106-0.

Chapter 2 MATERIALS AND STRUCTURAL DESIGN

2.1 INTRODUCTION

This chapter provides information regarding pipe service life, policy regarding material selection for pipe, and load tables for reinforced concrete and metal pipes.

2.2 FACTORS INFLUENCING SERVICE LIFE

Design service life is typically defined as the period of service without a need for major repairs. Highway drainage structures are usually designed with the goal of providing some pre-selected minimum number of years of service life. For corrugated metal pipes, this will normally be the period in years from installation until deterioration reaches the point of perforation of any point on the culvert. Reinforced concrete pipe service life is typically the period from installation until reinforcing steel is exposed , or a crack signifying severe distress develops. Plastic pipe service life may be considered at an end when excessive cracking, perforation or deflection has occurred. It is important to recognize that culverts are not assumed to be at or near the point of collapse at the end of their design service life. Rather, it is the period of little to no rehabilitative maintenance.

Some of the factors that affect service life are:

- Hydrogen-ion concentration (pH) of the surrounding soil and water;
- Soil resistivity, chloride and sulfate concentrations in the soil;
- Size, shape, hardness, and volume of bedload;
- Volume, velocity and frequency of streamflow in the culvert;
- Material characteristics of the culvert; and
- Anticipated changes in the watershed upstream of the culvert (such as development, industry, mining or logging).

2.2.1 Corrosion

Corrosion is the destruction of pipe material by chemical action. Most commonly, corrosion attacks metal culverts, or the reinforcement in concrete pipe, as the process of returning metals to their native state of oxides or salts. Similar processes can occur to the cement in concrete pipe if subjected to highly alkaline soils or to other pipe materials if subjected to extremely harsh environments. In order for corrosion to occur, an electrolytic corrosion cell must be formed. This requires the presence of water, or some other liquid to act as an electrolyte, as well as materials acting as an anode, cathode and conductor. As electrons move from the anode to the cathode, metal ions are released into solution, with characteristic pitting at the anode. The culvert will typically serve as both the anode and the cathode. Corrosion can affect either the inside or outside of a pipe, or both. The potential for corrosion to occur, and the rate at which it will progress, is variable and dependent upon a variety of factors.

Hydrogen Ion Concentration (pH)

The pH value is defined as the log of the reciprocal of the concentration of hydrogen ion in a solution. Values of pH in natural waters generally fall within the range of 4 to 10. A pH of less than 5.5 is usually considered to be strongly acidic, while values of 8.5 or greater are strongly alkaline.

Soil Resistivity

Resistivity of soil is a measure of the soils ability to conduct electrical current. It is affected primarily by the nature and concentration of dissolved salts, as well as the temperature, moisture content, compactness, and the presence of inert materials such as stones and gravel. The greater the resistivity of the soil, the less capable the soil is of conducting electricity and the lower the corrosive potential.

The unit of measurement for resistivity is ohm-centimeters, or more precisely, the electrical resistance between opposite faces of a one-centimeter cube. Resistivity values in excess of about 5000 ohm-cm are considered to present limited corrosion potential. Resistivities below the range of 1000 to 3000 ohm-cm will usually require some level of pipe protection, depending upon the corresponding pH level (e.g. if pH < 5.0, enhanced pipe protection may be needed for resistivities below 3000 ohm-cm; if pH > 6.5, enhanced pipe protection may not be needed unless resistivities are below 1500 ohm-cm). As a comparative measure, resistivity of seawater is in the range of 25 ohm-cm, clay soils range form approximately 750-2000 ohm-cm and loams from 3000-10,000 ohm-cm. Soils that are of a more granular nature exhibit even higher resistivities.

Chlorides

Dissolved salts containing chloride ions can be present in the soil or water surrounding a culvert. Dissolved salts can enhance culvert durability if their presence decreases oxygen solubility, but in most instances corrosive potential is increased as the negative chloride ion decreases the resistivity of the soil and/or water and destroys the protective film of anodic areas. Chlorides, as with most of the more common corrosive elements, primarily attack unprotected metal culverts and the reinforcing steel in concrete culverts if concrete cover is inadequate, cracked or highly permeable.

Sulfates

Sulfates can be naturally occurring or may be a result of man's activities, most notably mine wastes. Sulfates, in the form of hydrogen sulfide can also be created from biological activity, which is more common in wastewater or sanitary sewers and can combine with oxygen and water to form sulfuric acid. Although high concentrations can lower pH and be of concern to metal culverts, sulfates are typically more damaging to concrete. Typically, the sulfate in one or more various forms combines with the lime in cement to form calcium sulfate, which is structurally weak. Concrete pipe is normally sufficient to withstand sulfate concentrations of 1000 parts per million (ppm) or less. For higher concentrations of sulfates, higher strength concrete, concrete with lower amounts of calcium aluminate (under 5%) or special coatings may be necessary.

2.2.2 Abrasion

Abrasion is the gradual wearing away of the culvert wall due to the impingement of bedload and suspended material. Abrasion will almost always manifest itself first in the invert of the culvert. As with corrosion, abrasion potential is a function of several items, including culvert material, frequency and velocity of flow in the culvert and composition of bedload.

Bedload

By far, bedload is the leading cause of culvert abrasion. Critical factors in evaluation of the abrasive potential of bedload material are the size, shape and hardness of the bedload material, and the velocity and frequency of flow in the culvert. Generally, flow velocities less than 5 ft/sec are not considered to be abrasive, even if bedload material is present. Velocities in excess of 15 ft/sec which carry bedload, are considered to be very abrasive and some modifications to protect the culvert should be considered.

Tests performed on concrete pipe have generally shown excellent wear characteristics. Although high velocity flow will induce abrasion regardless of the size of bedload particles, tests performed on concrete pipe have shown that cobble and larger sizes will induce higher wear rates that sands and gravels.

Steel culverts are susceptible to the dual action of abrasion and corrosion. Once the thin protective coating on a steel pipe is worn away whether if is zinc or other substance, exposure to low resistivity and/or low pH environments can dramatically shorten the life of a steel culvert.

Plastic culvert materials (both polyvinyl chloride and high density polyethylene) exhibit good abrasion resistance. Since plastic is not subject to corrosion it will not experience the dual action of corrosion and abrasion.

2.3 PIPE DURABILITY

The Department is presently doing a statewide condition survey of centerline culverts. It is planned to correlate the results of the survey with information regarding soil and water properties including pH and resistivity measurements to develop a revised policy regarding the use of metal culverts. Until that study is completed the usage criteria for prefabricated corrugated galvanized steel culverts that was previously in the Drainage Manual will remain in effect and are given in Table 2.1 and Figure 2.1. Structural plate culverts although normally having heavier galvanizing and a greater metal thickness than prefabricated culverts are still vulnerable to corrosive attack in an aggressive environment. Therefore, usage criteria for structural plate culverts is included in Table 2.1.

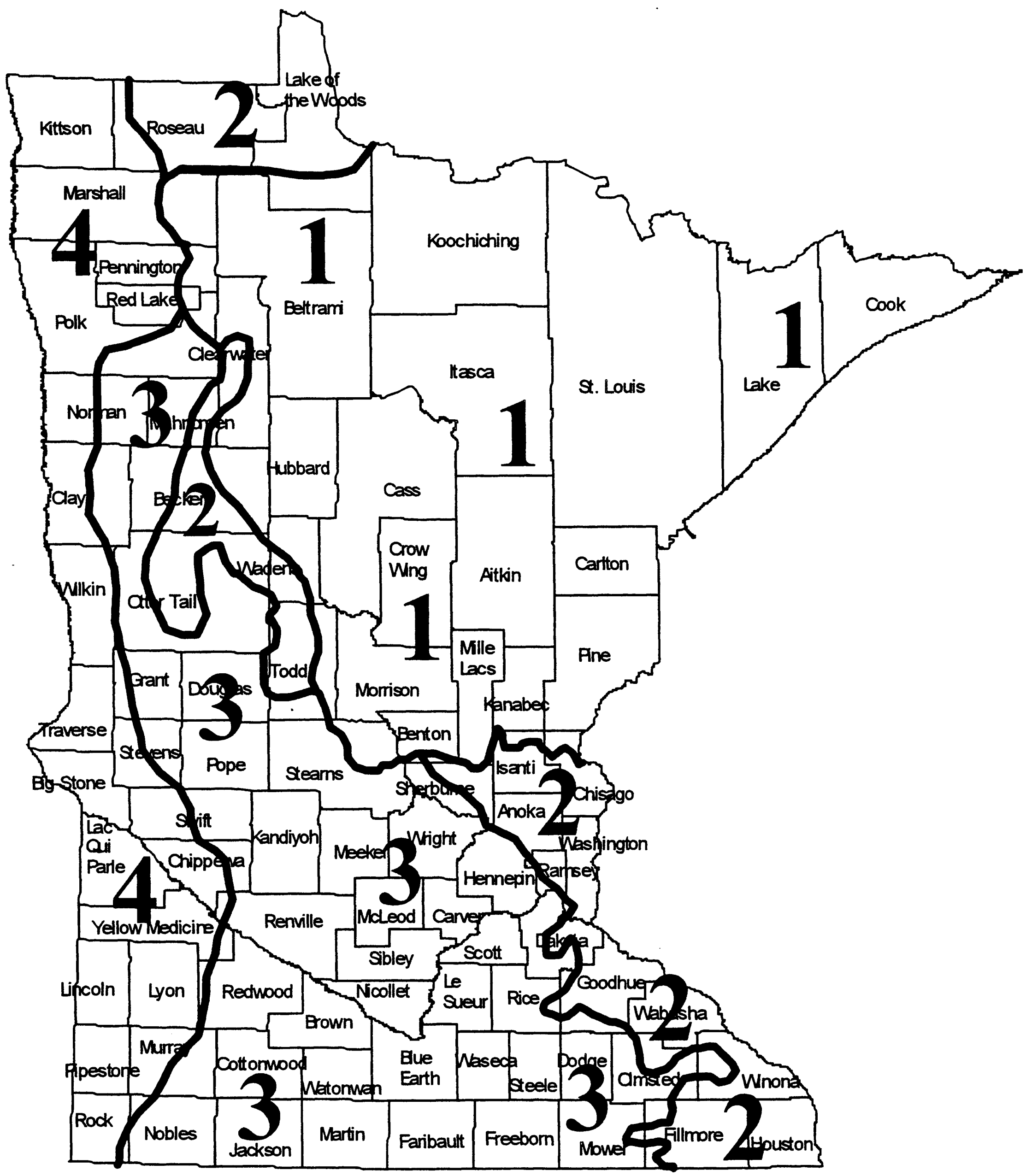


Figure 2.1 Four Soil Zones of Minnesota

Table 2.1 Drainage Condition at Culvert Location

Zone [4]	Water [1]	Prefabricated Corrugated Galvanized Steel Culvert	Structural Plate Culvert
1	Dry	Yes	Yes
	Wet	No	Yes [3]
2	Dry	Yes	Yes
	Wet	Yes , if not acid [2]	Yes [3]
3	Dry	Yes	Yes
	Wet	No	Yes [3]
4	Dry	Yes	Yes
	Wet	Yes	Yes

[1] Dry refers to structures that drain out after rainfall or snow melt and Wet is when there is standing or flowing water practically the entire year.

[2] District Soils Engineers should make pH determinations of samples from drainage area of the proposed culvert.

[3] Provided the location is not in a swamp or that the soil or water does not have a pH of 6.5 or less. The District Soil Engineer should take samples from the drainage area for pH determination.

[4] The Zones referred to in the Table 2.1 criteria for selecting prefabricated and structural plate culverts are shown in Figure 2.1.

For locations where corrugated steel (galvanized) pipe is not recommended or where its service life is limited by pH and/or resistivity, the following information is provided for guidance regarding service life and use of increased steel thickness or protective coatings.

Two nationally recognized methods for estimating corrugated steel pipe (AASHTO M36/M36M) service life are the empirical charts developed by the California Department of Transportation (Caltrans Test Method 642-C) and the American Iron and Steel Institute (AISI). Both of these charts require, as a minimum, site specific pH and resistivity data in order to estimate the pipe's service life.

These two methods are essentially identical in their form, but make different assumptions in the definition of service life. The California chart assumes that the end of the maintenance free service life occurs when the culvert first experiences perforation. This corresponds to approximately a 13% loss of metal thickness over the entire invert area. The AISI chart, which is based on the same data as the California chart, allows total loss to reach approximately 25% before indicating that the service life has been reached. Depending upon service conditions, such as continually submerged pipe, either test method can underestimate or overestimate the usable life that a culvert can be expected to provide. Because the Department has not yet completed its condition survey of centerline culverts, the California chart is provided as guidance in determining average life of galvanized pipe and the use of increased steel thickness or protective coatings.

There is little data available regarding abrasion. It is recommended that a paved invert be considered for metal pipes if abrasion is considered to be a concern. For structural plate pipe consideration should be given to increasing the thickness of the steel if abrasion is considered to be a concern. For further information on protecting metal pipe against corrosion and/or abrasion designers may seek additional information from the State Hydraulics Engineer.

Table 2.2 Average Life Adjustment for Gage and Material

Gage	18	16	14	12	10	8
Thickness (inches)	0.052	0.064	0.079	0.109	0.138	0.168
Thickness (mm)	1.32	1.63	2.01	2.77	3.51	4.27
GALV.	1	1.3	1.6	2.2	2.8	3.4
ALT2.	2.3	2.6	2.9	3.5	4.1	4.7

Base gage for Figure 2.2 is 18 gage. Adjustment factors are given in Table 2.2. To get the service life for 16 gage pipe (Mn/DOT minimum gage) multiple service life for 18 gage by 1.3.

Polymeric coated pipe may be considered as an equal to Aluminized Type 2 (ALT2.) Pipe.

Figure 2.2 is based on Equations 2.1 and 2.2.
For pH of environment normally greater than 7.3

$$YEARS = 1.47R^{0.41} \tag{2.1}$$

For pH of environment normally less than 7.3

$$YEARS = 13.79\left[Log_{10}R - Log_{10}\left(2160 - 2490Log_{10}pH\right)\right] \tag{2.2}$$

Where: R = minimum resistivity

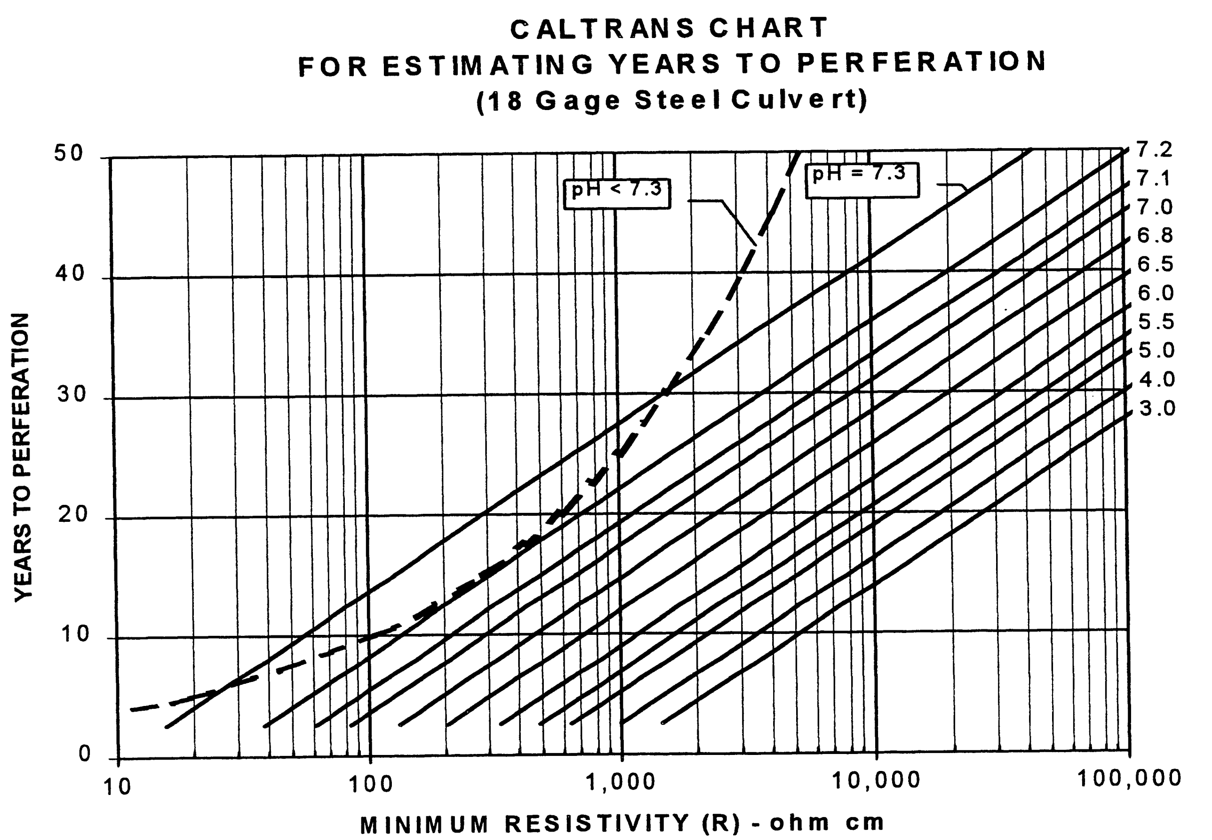


Figure 2.2 CALTRANS Service Life Chart
Source: Highway Drainage Guidelines (AASHTO, 1999)

2.4 MATERIAL TYPES FOR DRAINAGE FACILITIES

Following is the policy for selecting material types for culverts, storm drains and tile.

2.4.1 Culvert Materials

Pipe for culverts shall be selected on the basis of the type which best fulfills all of the engineering requirements for a specific installation. Factors to be considered in fulfilling the engineering requirements should be hydraulic performance, structural stability, serviceability, and economy. The culvert design sheet shall provide documentation for each pipe installation indicating the engineering considerations which dictate the selection of the specific type of pipe.

If, for engineering reasons, the use of corrugated metal pipe is necessary in areas that have been detrimental to this type of pipe, the designer must take proper precautions such as increasing the thickness of the base metal or providing a protective coating to assure required serviceability.

Pipes for centerline culverts shall be selected on the basis of engineering analysis which result in the most favorable combination of hydraulic performance, structural stability, serviceability, and economy.

Reinforced concrete pipe, plain galvanized corrugated steel pipe or corrugated polyethylene pipe will normally be considered acceptable for culverts installed under minor side road approaches and private entrances except where engineering considerations dictate otherwise. If site considerations dictate, corrugated metal pipe with a protective coating could be used. The designer may choose to allow alternate material types in bidding proposals.

2.4.2 Storm Drain Material

Reinforced concrete pipe will normally be required for all storm drains. Corrugated polyethylene pipe may be allowed as an alternate to reinforced concrete pipe for 12" - 36" diameter pipes.

2.4.3 Tile Materials

For agricultural tile line crossings, 12-inch reinforced concrete pipe will generally be required between points five feet outside the toe of embankment slopes for tile lines 12 inches or less in diameter. Equivalent size reinforced concrete pipe will be required for tile lines larger than 12 inches in diameter.

2.5 PIPE INSTALLATION

2.5.1 Pipe Bedding

Since bedding is an important element in determining the ability of pipe to carry load, the various types of bedding in general use are explained below. An additional consideration in bedding conditions occurs when bell end pipe is used. The bedding must be excavated to accept the bell end so that the pipe is supported along its full length and not just at the bell.

Class A Bedding (Concrete Cradle)
Class A bedding consists of a continuous monolithic, concrete cradle having a minimum thickness under the pipe of 1/4 of the nominal inside diameter or span and extending up the sides of the pipe for a height equal to 1/4 the outside diameter or rise. The width of the cradle must equal or exceed the outside diameter or span of the pipe plus eight inches.

Class B Bedding (First Class Bedding)
Class B bedding consists of bedding the pipe on a minimum six-inch thickness of granular bedding accurately shaped by means of a template to fit the lower part of the pipe exterior for a width of at least 60 percent of the diameter for round pipe and at least 80 percent of the span for pipe arches. The existing ground at the culvert site is first excavated to an elevation which is approximately 15 percent of the outside diameter or rise of the pipe above the established grade for the bottom of the pipe. Then the foundation for the bedding is prepared by carefully excavating to the required depth and shape of the bedding.

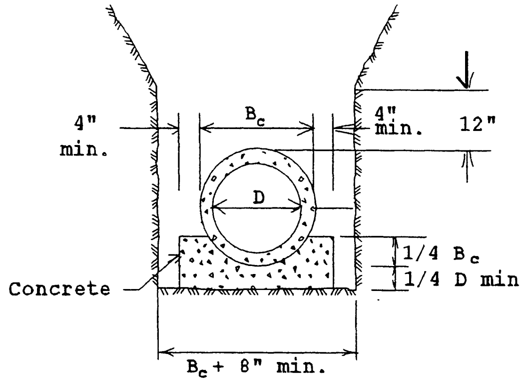
Class C Bedding (Ordinary Bedding)
Class C bedding consists of carefully shaping the foundation soil to fit the lower part of the pipe exterior to a depth of at least 15 percent of the outside diameter for circular pipes, and at least equal to one-half of the height of pipe-arch structures.

Class D Bedding (Impermissible Bedding)
As the name implies, Class D bedding is not ordinarily permitted for pipe installations, particularly if the pipe is to be subjected to even moderate dead or live loads. Under current specifications, no special shaping of the foundation is required for entrance culverts unless they are to be laid in a trench. This is the only condition under which Class D bedding is permitted in highway construction. When an entrance culvert is to be laid in a trench, Class C bedding is required unless a higher class of bedding is specified in the plan or special provisions.

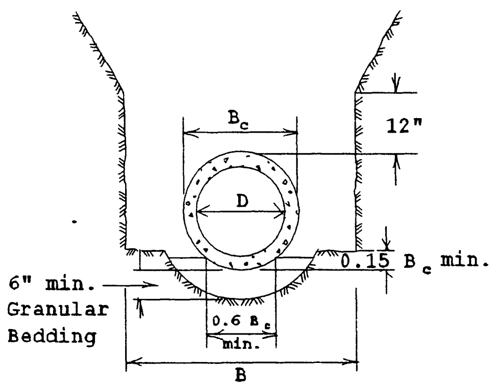
Rock or Unyielding Foundations
When rigid or flexible pipe is laid on rock or other unyielding material, a trench must be excavated in the underlying unyielding material or rock. The trench is excavated to a depth of one foot below the bottom of the structure for its full length, and to a width two feet wider than the outside extremities of the pipe. The excavated trench is backfilled with selected, compressible, mineral soil placed in layers not exceeding six inches in uncompacted thickness and each layer is lightly compacted to form a uniform but yielding foundation. This backfill is constructed to such elevation that the proper bedding can be constructed.

For further information regarding pipe installation see Chapter 8 of the Road Design Manual and/or the District Soils Engineer. Figures 2.3 and 2.4 show classes of bedding for trench and embankment conditions.

Where: B_c = outside diameter of the pipe
D = inside diameter of the pipe
B = trench width

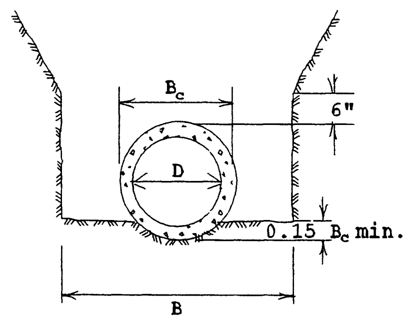


CLASS A



Pipe Dia.	B
36" or less	B_c+ 24"
42" to 54"	1.5 x B_c
60" or over	B_c+ 36"

CLASS B



Pipe Dia.	B
36" or less	B_c+ 24"
42" to 54"	1.5 x B_c
60" or over	B_c+ 36"

CLASS C

Figure 2.3 Classes of Bedding for Trench Conditions

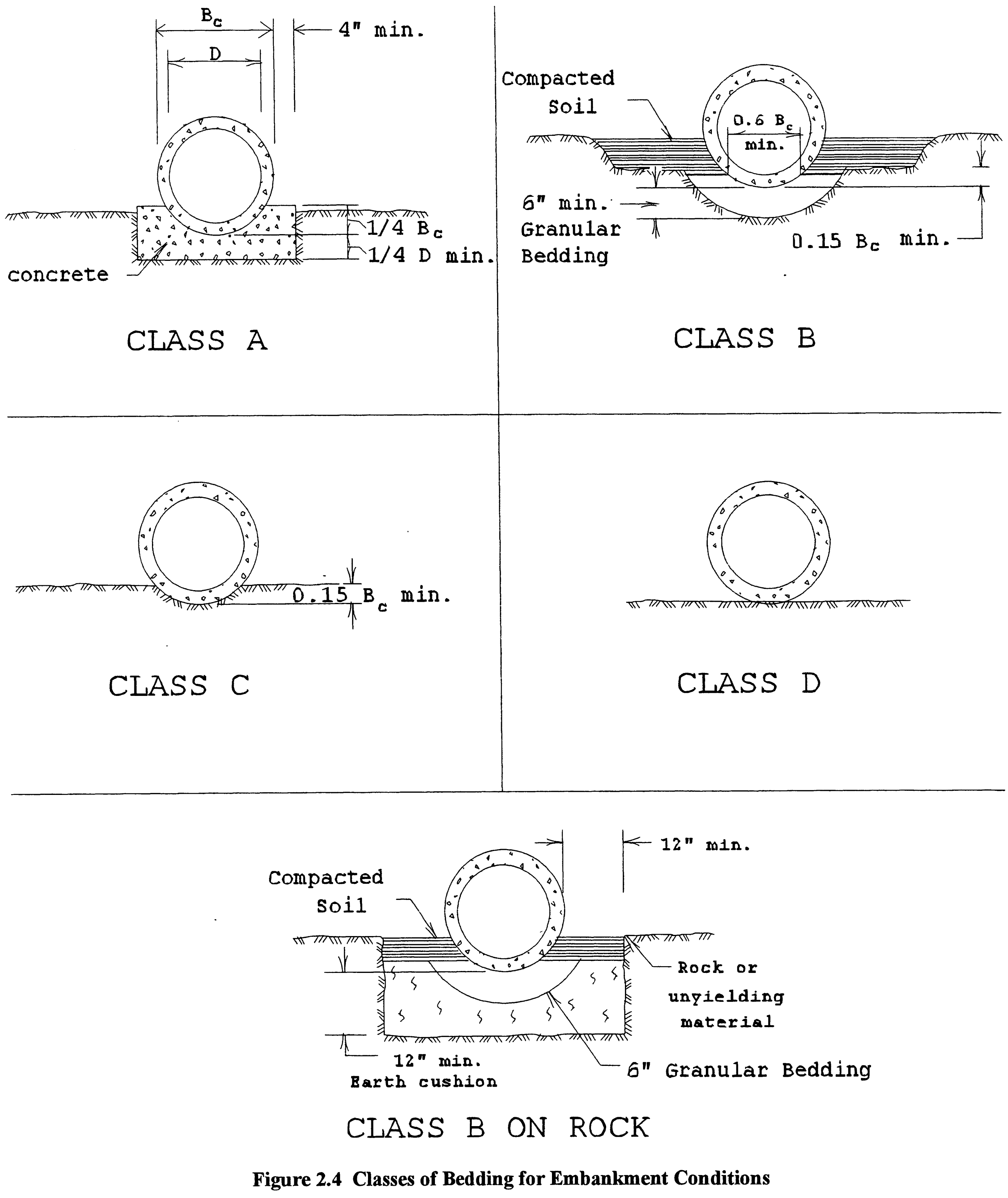


Figure 2.4 Classes of Bedding for Embankment Conditions

2.5.2 Concrete Pipe Load Tables

Trench width, backfill height and embankment fill are provided for circular concrete pipe in Tables 2.3, 2..4 2.5, 2.6 and 2.7. Loading and installation specifications for concrete arch pipe is included in the Standard Plate 3014.

Table 2.3 Trench Width for Concrete Pipe with Class B Bedding

Circular Concrete Pipe
Narrow and Wide Trench Widths in Feet
for Class B Bedding, Measured at Top of Pipe in Feet

Pipe Class	Class II		Class III		Class IV		Class V	
Pipe Diameter \ Trench Type	N	W	N	W	N	W	N	W
12	3.5	3.5	3.5	3.5	3.5	3.5	3.5	4.0
15	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.5
18	4.0	4.0	4.0	4.0	4.0	4.5	4.0	5.0
21	4.5	4.5	4.5	4.5	4.5	5.0	4.5	5.5
24	4.5	5.0	4.5	5.0	4.5	5.5	4.5	6.0
27	5.0	5.0	5.0	5.5	5.0	6.0	5.0	6.5
30•	5.5	5.5	5.5	6.0	5.5	6.5	5.5	7.0
33	5.5	6.0	5.5	6.5	5.5	7.0	5.5	7.5
36	6.5	6.5	6.5	7.0	6.5	7.5	6.5	8.0
42	7.0	7.5	7.0	7.5	7.0	8.5	7.0	9.0
48	7.5	8.0	7.5	8.5	7.5	9.0	7.5	10.0
54	8.5	9.0	8.5	9.5	8.5	10.0	8.5	11.0
60	9.0	9.5	9.0	10.0	9.0	11.0	9.0	12.0
66	9.5	10.0	9.5	11.0	9.5	12.0	9.5	12.5
72	10.0	11.0	10.0	11.5	10.0	13.0	10.0	13.5
78	10.5	11.5	10.5	12.0	10.5	13.5	10.5	14.5
84	11.0	12.0	11.0	13.0	11.0	14.5	11.0	15.5
90	12.0	12.5	12.0	13.5	12.0	15.0	12.0	16.5
96	12.5	13.0	12.5	14.0	12.5	16.0	12.5	17.0
102	13.0	13.5	13.0	15.0	13.0	17.0	13.0	18.0
108	13.5	14.5	13.5	15.5	13.5	17.5	13.5	19.0

N = narrow trench: minimum width
W = wide trench: transition width
Source: CRETEX Precast Concrete Products

Table 2.4 Trench Fill Height for Concrete Pipe with Class B Bedding

Circular Concrete Pipe
Height of Backfill in Feet for Class B Bedding
Measured at Top of Pipe in Feet, 120 PCF Soil Density

Pipe Class	Class II		Class III		Class IV		Class V	
Pipe Diameter \ Trench Type	N	W	N	W	N	W	N	W
12	8	8	11	11	16	16	*	24
15	8	8	11	11	16	16	*	24
18	8	8	11	11	23	17	*	25
21	8	8	11	11	23	17	*	25
24	9	8	11	11	23	17	*	25
27	9	9	13	12	23	17	*	26
30	9	9	13	12	23	17	*	26
33	9	9	13	12	23	17	*	26
36	9	9	13	12	23	17	*	26
42	10	9	14	12	24	18	*	26
48	10	9	14	12	24	18	*	26
54	10	9	14	12	24	18	*	27
60	11	10	15	12	24	18	*	27
66	11	10	15	12	25	18	*	27
72	11	10	16	13	26	18	*	27
78	11	10	16	13	27	18	*	27
84	12	10	17	13	27	18	*	27
90	12	11	17	13	27	19	*	27
96	12	11	17	14	27	19	*	27
102	12	11	17	14	27	19	*	27
108	12	11	17	14	27	19	*	27

N = narrow trench: minimum width
W = wide trench: transition width
* Fill height greater than 40 feet, D-load equation must be used.
Source: CRETEX Precast Concrete Products

Table 2.5 Trench Width for Concrete Pipe with Class C Bedding

Circular Concrete Pipe
Narrow and Wide Trench Widths in Feet for
Class C Bedding, Measured at Top of Pipe in Feet

Pipe Class	Class II		Class III		Class IV		Class V	
Pipe Diameter / Trench Type	N	W	N	W	N	W	N	W
12	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
15	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.5
18	4.0	4.0	4.0	4.0	4.0	4.5	4.0	5.0
21	4.5	4.5	4.5	4.5	4.5	5.0	4.5	5.5
24	4.5	4.5	4.5	5.0	4.5	5.0	4.5	6.0
27	5.0	5.0	5.0	5.5	5.0	5.5	5.0	6.5
30	5.5	5.5	5.5	5.5	5.5	6.0	5.5	7.0
33	5.5	6.0	5.5	6.0	5.5	6.5	5.5	7.0
36	6.5	6.5	6.5	6.5	6.5	7.0	6.5	7.5
42	7.0	7.5	7.0	7.5	7.0	8.0	7.0	8.5
48	7.5	7.5	7.5	8.0	7.5	9.0	7.5	9.5
54	8.5	8.5	8.5	9.0	8.5	9.5	8.5	10.5
60	9.0	9.0	9.0	9.5	9.0	10.5	9.0	11.5
66	9.5	9.5	9.5	10.0	9.5	11.5	9.5	12.0
72	10.0	10.0	10.0	11.0	10.0	12.0	10.0	13.0
78	10.5	10.5	10.5	11.5	10.5	13.0	10.5	14.0
84	11.0	11.5	11.0	12.0	11.0	13.5	11.0	15.0
90	12.0	12.0	12.0	13.0	12.0	15.0	12.0	15.5
96	12.5	12.5	12.5	13.5	12.5	15.5	12.5	16.5
102	13.0	13.0	13.0	14.0	13.0	16.0	13.0	17.5
108	13.5	13.5	13.5	14.5	13.5	16.5	13.5	18.0

N = narrow trench: minimum width
W = wide trench: transition width
Source: CRETEX Precast Concrete Products

Table 2.6 Trench Fill Height for Concrete Pipe with Class C Bedding

Circular Concrete Pipe
Height of Backfill in Feet for Class C Bedding
Measured at Top of Pipe in Feet, 120 PCF Soil Density

Pipe Class	Class II		Class III		Class IV		Class V	
Pipe Diameter \ Trench Type	N	W	N	W	N	W	N	W
12	6	6	9	9	13	13	21	19
15	6	6	9	9	13	13	22	19
18	7	7	9	9	14	13	34	20
21	7	7	9	9	14	14	34	20
24	7	7	9	9	14	14	34	20
27	7	7	9	9	16	14	34	20
30	7	7	10	9	16	14	34	20
33	7	7	10	9	16	14	34	20
36	7	7	10	9	17	14	34	21
42	7	7	10	9	17	14	34	21
48	8	8	11	10	18	14	34	21
54	8	8	11	10	18	14	34	21
60	8	8	11	10	18	14	34	21
66	8	8	11	10	18	14	34	21
72	8	8	12	11	19	14	34	21
78	9	8	12	11	20	15	34	21
84	9	9	12	11	20	15	34	21
90	9	9	12	11	20	15	34	22
96	9	9	13	11	20	15	34	22
102	9	9	13	11	20	16	34	22
108	9	9	13	12	20	16	34	22

N = narrow trench: minimum width
W = wide trench: transition width
Source: CRETEX Precast Concrete Products

Table 2.7 Embankment Fill Height for Concrete Pipe

Circular Concrete Pipe
Embankment Fill Height in Feet
Measured from Top of Pipe in Feet, 120 PCF Soil Density

Pipe Class	Class II				Class III				Class IV				Class V			
Bedding / Diameter	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
12	16	10	8	5	21	13	11	6	31	19	16	10	*	29	24	15
15	16	10	8	5	21	13	11	7	32	19	16	10	*	30	25	15
18	16	10	8	5	21	13	11	7	32	20	17	10	*	30	25	15
21	16	10	8	5	22	14	12	7	33	20	17	10	*	31	25	15
24	17	10	9	5	22	14	12	7	33	21	17	11	*	31	26	16
27	17	10	9	6	22	14	12	7	33	21	18	11	*	31	26	16
30	17	10	9	6	23	14	12	7	33	21	18	11	*	31	26	16
33	17	11	9	6	23	14	12	7	34	21	18	11	*	31	26	16
36	17	11	9	6	23	14	12	8	34	22	18	11	*	32	27	16
42	17	11	9	6	24	15	12	8	34	22	18	11	*	32	27	16
48	18	11	9	6	24	15	13	8	34	22	18	11	*	32	27	17
54	18	11	10	7	24	15	13	8	35	22	18	11	*	32	27	17
60	18	11	10	7	25	15	13	9	35	22	18	11	*	33	27	17
66	18	11	10	7	25	15	13	9	35	22	19	11	*	33	27	17
72	19	12	11	7	25	15	13	9	35	22	19	11	*	33	27	17
78	19	12	11	7	25	15	13	9	36	22	19	11	*	33	27	17
84	19	12	11	7	25	15	13	10	36	22	19	12	*	33	28	17
90	19	12	11	8	25	15	13	10	36	22	19	12	*	33	28	17
96	19	12	11	8	25	15	13	10	36	22	19	12	*	33	28	17
102	19	13	12	8	25	15	14	10	36	22	19	12	*	33	28	17
108	19	13	12	8	25	16	15	10	36	22	19	13	*	33	28	18

Fill heights are based on a 0.7 settlement ratio
Projection ratios are as follows: Class A = 0.7, Class B = 0.5, Class C = 0.7 and Class D = 0.9
* Fill height greater than 45 feet, D-load equation must be used.
Source: CRETEX Precast Concrete Products

2.5.3 Metal Pipe Load Tables

Metal load tables are provided for common metal types, shapes and sizes. The tables are based on the following criteria:

- Maximum heights of cover by AASHTO Design H25 Wheel Loading
- Minimum soil corner bearing pressure for pipe-arch = 4000 lbs/ft^2
- Maximum covers for a given gage fluctuate as span increases due to varying ratio of top radius to corner radius.
- Minimum cover height is measured from top of pipe to top of rigid pavement, or bottom of flexible pavement.
- Equivalent diameter in metal pipe-arch tables refers to the diameter of round corrugated steel pipe from which a pipe-arch or other shape is formed.

Table 2.8 2 ⅔" x ½" Corrugated Steel Round Pipe

Span (Inches)	Minimum Cover (ft)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
		0.064 16	0.079 14	0.109 12	0.138 10	0.168 8
12	1	213	266			
15	1	170	212			
18	1	142	177			
21	1	121	152			
24	1	106	133	186		
30	1	85	106	149		
36	1	71	88	124	159	
42	1	60	76	106	137	167
48	1	53	66	93	119	146
54	1		59	82	106	130
60	1			74	95	117
66	1				87	106
72	1				79	97
78	1					86
84	1					75

Limits for Checks:
Flexibility Factor = 0.0430
Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 2.9E+07 PSI
Minimum Yield Point = 33000 PSI
Min. Tensile Strength = 45000 PSI

Notes:
Helical Pipe (lock seam or weld seam)

Source: CONTECH Construction Products Inc.

Table 2.9 3" x 1" or 5" x 1" Corrugated Steel Round Pipe

Span (Inches)	Minimum Cover (ft)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
		0.064 16	0.079 14	0.109 12	0.138 10	0.168 8
54	1	48	60	84	109	133
60	1	43	54	76	98	120
66	1	39	49	69	89	109
72	1	36	45	63	81	100
78	1	33	41	58	75	92
84	1	31	38	54	70	85
90	1	29	36	50	65	80
96	1		34	47	61	75
102	1.5		32	44	57	70
108	1.5			42	54	66
114	1.5			40	51	63
120	1.5			38	49	60
126	1.5				46	57
132	1.5				44	54
138	1.5				42	52
144	1.5					50

Limits for Checks:
Flexibility Factor = 0.0330
Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 2.9E+07 PSI
Minimum Yield Point = 33000 PSI
Min. Tensile Strength = 45000 PSI

Notes:
- Helical Pipe (lock seam or weld seam)
- Maximum cover for 5" x 1" pipe are shown.
- For 3" x 1" pipe increase values by 13%

Source: CONTECH Construction Products Inc.

Table 2.10 3/4" x 3/4" x 7-1/2" Steel Spiral Rib Round Pipe

Span (Inches)	Minimum Cover (feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage			
		0.064 16	0.079 14	0.109 12	0.138 10
18	1	93	131		
21	1	80	112		
24	1	70	98		
27	1	62	88		
30	1	56	78		
36	1	46	65	109	
42	1	40	56	93	138
48	1	35	49	81	121
54	1.5	**31**[1]	43	72	107
60	1.5		39	65	96
66	1.5		**35**[1]	59	88
72	1.5			54	80
78	2			50	73
84	2			**46**[1]	69
90	2			**43**[1]	64
96	2				59
102	2.5				**53**[1]

Limits for Checks:

Wall Area FS = 2

Buckling FS = 2

Constants:

Soil Density = 120 # / CU FT

Modulus of Elasticity = 2.9E+07 PSI

Minimum Yield Point = 33000 PSI

Min. Tensile Strength = 45000 PSI

Notes:

- Helical Pipe (lock seam or weld seam)
- Embankment Installation

[1] Bold values are for Trench Installation only

Source: CONTECH Construction Products Inc.

Table 2.11 2 ⅔" x ½" Corrugated Steel Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minumum Cover (feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
				0.064 16	0.079 14	0.109 12	0.138 10	0.168 8
15	17	13	1	13	13			
18	21	15	1	13	13			
21	24	18	1	13	13			
24	28	20	1	13	13	13		
30	35	24	1	13	13	13		
36	42	29	1	13	13	13	13	
42	49	33	1		13	13	13	13
48	57	38	1			12	12	12
54	64	43	1			12	12	12
60	71	47	1				12	12
66	77	52	1					12
72	83	57	1					12

Notes: - Helical pipe (lock seam or weld seam)

Limits for Checks:

Flexibility Factor = 0.0430

Wall Area FS = 2

Buckling FS = 2

Constants:

Soil Density = 120 # / CU FT

Modulus of Elasticity = 2.9E+07 PSI

Minimum Yield Point = 33000 PSI

Min. Tensile Strength = 45000 PSI

Source: CONTECH Construction Products Inc.

Table 2.12 3" x 1" or 5" x 1" Corrugated Steel Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minimum Cover (feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage		
				0.109 12	0.138 10	0.168 8
54	60	46	1.25	20	20	20
60	66	51	1.25	20	20	20
66	73	55	1.5	20	20	20
72	81	59	1.5	17	17	17
78	87	63	1.5	17	17	17
84	95	67	1.5	17	17	17
90	103	71	1.5	16	16	16
96	112	75	1.75	16	16	16
102	117	79	1.75	16	16	16
108	128	83	2		16	16
114	137	87	2		16	16
120	142	91	2			16

Notes: - Helical pipe (lock seam or weld seam)

Limits for Checks:

Flexibility Factor = 0.0330

Wall Area FS = 2

Buckling FS = 2

Constants:

Soil Density = 120 # / CU FT

Modulus of Elasticity = 2.9E+07 PSI

Minimum Yield Point = 33000 PSI

Min. Tensile Strength = 45000 PSI

Source: CONTECH Construction Products Inc.

Table 2.13 3/4" x 3/4" x 7 ½" Steel Spiral Rib Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minimum Cover (feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage		
				0.064 16	0.079 14	0.109 12
18	20	16	1	16	16	
21	23	19	1	15	15	
24	27	21	1	14	14	
30	33	26	1	14	14	
36	40	31	1	13	13	13
42	46	36	1	13	13	13
48	53	41	1.25	13	13	13
54	60	46	1.5		13	13
60	66	51	1.5			13
66	73	55	1.75			13
72	81	59	1.75			13

Limits for Checks:

Wall Area FS = 2

Buckling FS = 2

Constants:

Soil Density = 120 # / CU FT

Modulus of Elasticity = 2.9E+07 PSI

Minimum Yield Point = 33000 PSI

Min. Tensile Strength = 45000 PSI

Notes:

- Helical pipe (lock seam or weld seam)
- Spiral Rib Pipe; Type I Installation (Embankment Condition)

Source: CONTECH Construction Products Inc.

Table 2.14 6" x 2" Steel Structural Plate Round

Span (Inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage						
		0.109 12	0.138 10	0.168 8	0.188 7	0.218 5	0.249 3	0.280 1
60	1	46	68	90	103	124	146	160
66	1	42	62	81	93	113	133	145
72	1	38	57	75	86	103	122	133
78	1	35	52	69	79	95	112	123
84	1	33	49	64	73	88	104	114
90	1	31	45	60	68	82	97	106
96	1	29	43	56	64	77	91	100
102	1.5	27	40	52	60	73	86	94
108	1.5	25	38	50	57	69	81	88
114	1.5	24	36	47	54	65	77	84
120	1.5	23	34	45	51	62	73	80
126	1.5	22	32	42	49	59	69	76
132	1.5	21	31	40	46	56	66	72
138	1.5	20	29	39	44	54	63	69
144	1.5	19	28	37	43	51	61	66
150	2	18	27	36	41	49	58	64
156	2	17	26	34	39	47	56	61
162	2	17	25	33	38	46	54	59
168	2	16	24	32	36	44	52	57
174	2	16	23	31	35	42	50	55
180	2	15	22	30	34	41	48	53
186	2	15	22	29	33	40	47	51
192	2		21	28	32	38	45	50
198	2.5		20	27	31	37	44	48
204	2.5		20	26	30	36	43	47
210	2.5			25	29	35	41	45
216	2.5			25	28	34	40	44
222	2.5			24	27	33	39	43
228	2.5			23	27	32	38	42
234	2.5			23	26	31	37	41
240	2.5				25	31	36	40
246	3				25	30	35	39
252	3					29	34	38
258	3					28	34	37
264	3					28	33	36
270	3						32	35
276	3						31	34
282	3						31	34
288	3.5						30	33
294	3.5							32
300	3.5							32
306	3.5							31

Limits for Checks:

Flexibility Factor = 0.0200

Wall Area FS = 2

Seam Strength = 3

Buckling FS = 2

Constants:

Soil Density = 120 # / CU FT

Modulus of Elasticity = 2.9E+07 PSI

Minimum Yield Point = 33000 PSI

Min. Tensile Strength = 45000 PSI

Notes:

- Seam strength values are based on four 3/4 inch diameter A449 bolts/foot.

Source:

CONTECH Construction Products Inc.

Table 2.15 6" x 2" Steel Structural Plate Pipe-Arch

Span (inches)	Rise (inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage						
			0.109 12	0.138 10	0.168 8	0.188 7	0.218 5	0.249 3	0.280 1
73	55	1	16	16	16	16	16	16	16
76	57	1	15	15	15	15	15	15	15
81	59	1	14	14	14	14	14	14	14
84	64	1	14	14	14	14	14	14	14
87	63	1	13	13	13	13	13	13	13
92	65	1	13	13	13	13	13	13	13
95	67	1	12	12	12	12	12	12	12
98	69	1.5	12	12	12	12	12	12	12
103	71	1.5	11	11	11	11	11	11	11
106	73	1.5	11	11	11	11	11	11	11
112	75	1.5	10	10	10	10	10	10	10
114	77	1.5	10	10	10	10	10	10	10
117	79	1.5	10	10	10	10	10	10	10
123	81	1.5	9	9	9	9	9	9	9
128	83	1.5	9	9	9	9	9	9	9
131	85	1.5	9	9	9	9	9	9	9
137	87	1.5	8	8	8	8	8	8	8
139	89	1.5	8	8	8	8	8	8	8
142	91	1.5	8	8	8	8	8	8	8
148	93	2	8	8	8	8	8	8	8
150	95	2	8	8	8	8	8	8	8
152	97	2	7	7	7	7	7	7	7
154	100	2	7	7	7	7	7	7	7
159	112	2	12	12	12	12	12	12	12
162	114	2	12	12	12	12	12	12	12
168	116	2	12	12	12	12	12	12	12
170	118	2	12	12	12	12	12	12	12
173	120	2	11	11	11	11	11	11	11
179	122	2	11	11	11	11	11	11	11
184	124	2	11	11	11	11	11	11	11
187	126	2	11	11	11	11	11	11	11
190	128	2	10	10	10	10	10	10	10
195	130	2.5	10	10	10	10	10	10	10
198	132	2.5	10	10	10	10	10	10	10
204	134	2.5	10	10	10	10	10	10	10
206	136	2.5	10	10	10	10	10	10	10
209	138	2.5	9	9	9	9	9	9	9
215	140	2.5	9	9	9	9	9	9	9
217	142	2.5	9	9	9	9	9	9	9
223	144	2.5	9	9	9	9	9	9	9
225	146	2.5	9	9	9	9	9	9	9
231	148	2.5		8	8	8	8	8	8
234	150	2.5		8	8	8	8	8	8
236	152	2.5		8	8	8	8	8	8
239	154	2.5		8	8	8	8	8	8
245	156	3		8	8	8	8	8	8
247	158	3		8	8	8	8	8	8

Limits for Checks:

Flexibility Factor	= 0.030
Wall Area FS	= 2
Buckling FS	= 2
Seam Strength	= 3

Constants:

Soil Density	= 120 # / CU FT
Modulus of Elasticity	= 2.9E+07 PSI
Minimum Yield Point	= 33000 PSI
Min. Tensile Strength	= 45000 PSI

Notes:

- 159" to 247" spans have 31" corner radius
- 73" to 154" spans have 18" corner radius
- Seam strength values are based on four 3/4 inch diameter A449 bolts/foot.

Source:
CONTECH Construction Products Inc.

Table 2.16 2 ⅔" x ½" Corrugated Aluminum Round Pipe

Span (Inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
		0.060 16	0.075 14	0.105 12	0.135 10	0.164 8
12	1	155	193			
15	1	124	154			
18	1	103	129			
21	1	88	110			
24	1	77	96	135		
27	1		86	120		
30	1		77	108		
36	1		64	90	116	
42	1			77	99	
48	1.25			66	86	106
54	1.25			54	70	87
60	1.5				57	71
66	1.5					57
72	1.5					45

Limits for Checks:
Flexibility Factor

For 16 gage	= 0.031
For 14 gage	= 0.061
For 12 gage	= 0.092
Wall Area FS	= 2
Buckling FS	= 2

Constants:

Soil Density	= 120 # / CU FT
Modulus of Elasticity	= 1E+07 PSI
Minimum Yield Point	= 24000 PSI
Min. Tensile Strength	= 31000 PSI

Notes:
Helical pipe lock seam

Source:
CONTECH Construction Products Inc.

Table 2.17 3" x 1" Corrugated Aluminum Round Pipe

Span (Inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
		0.060 16	0.075 14	0.105 12	0.135 10	0.164 8
30	1	71	89			
36	1	59	74	104		
42	1	50	63	89		
48	1	44	55	78	104	
54	1.25	39	49	69	92	109
60	1.25	35	44	62	83	98
66	1.5	32	40	56	75	89
72	1.5	29	37	52	69	81
78	1.75		34	48	64	75
84	1.75			44	59	70
90	2			41	55	65
96	2			38	51	60
102	2				46	54
108	2				41	49
114	2					44
120	2					40

Limits for Checks:

Flexibility Factor	= 0.0600
Wall Area FS	= 2
Buckling FS	= 2

Constants:

Soil Density	= 120 # / CU FT
Modulus of Elasticity	= 1E+07 PSI
Minimum Yield Point	= 24000 PSI
Min. Tensile Strength	= 31000 PSI

Notes:
Helical pipe lock seam

Source: CONTECH Construction Products Inc.

Table 2.18 3/4" x 3/4" x 7 ½" Aluminum Spiral Rib Round Pipe

Span (inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage			
		0.060 16	0.075 14	0.105 12	0.135 10
18	1	55			
21	1	47			
24	1	41	57		
30	1.25	33	45	73	
36	1.5	**27** [1]	38	61	86
42	1.5		**32** [1]	52	74
48	1.5			46	65
54	1.75			40	57
60	2			**36** [1]	52
66	2				47
72	2.25				**43** [1]

Limits for Checks:

Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 1E+07 PSI
Minimum Yield Point = 24000 PSI
Min. Tensile Strength = 31000 PSI

Notes:
- Helical pipe lock seam
- Type I Embankment Installation

[1] Bold values are for Type I Trench Installation

Source:
CONTECH Construction Products Inc.

Table 2.19 2 ⅔" x ½" Corrugated Aluminum Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage				
				0.060 16	0.075 14	0.105 12	0.135 10	0.164 8
15	17	13	1	13	13			
18	21	15	1	13	13			
21	24	18	1	13	13			
24	28	20	1	13	13	13		
30	35	24	1	13	13	13		
36	42	29	1	13	13	13	13	
42	49	33	1.25			13	13	
48	57	38	1.25			12	12	12
54	64	43	1.5				12	12
60	71	47	1.5					12

Limits for Checks:
Flexibility Factor
For 16 gage = 0.031
For 14 gage = 0.061
For 12 gage and heavier = 0.092
Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 1E+07 PSI
Minimum Yield Point = 24000 PSI
Min. Tensile Strength = 31000 PSI

Notes:
- Helical pipe lock seam

Source:
CONTECH Construction Products Inc.

Table 2.20 3" x 1" Corrugated Aluminum Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage			
				0.075 14	0.105 12	0.135 10	0.164 8
54	60	46	1.25	20	20	20	20
60	66	51	1.5	20	20	20	20
66	73	55	1.75	20	20	20	20
72	81	59	1.75		17	17	17
78	87	63	2		17	17	17
84	95	67	2		17	17	17
90	103	71	2			16	16
96	112	75	2				16

Notes: - Helical pipe lock seam

Limits for Checks:
Flexibility Factor = 0.0600

Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 1E+07 PSI
Minimum Yield Point = 24000 PSI
Min. Tensile Strength = 31000 PSI
Source: CONTECH Construction Products Inc.

Table 2.21 3/4" x 3/4" x 7-1/2" Aluminum Spiral Rib Pipe-Arch

Equivalent Pipe Diameter (inches)	Span (inches)	Rise (inches)	Minimum Cover (Feet)	Maximum Cover (Feet) by Thickness (Inches) and Gage			
				0.064 16	0.079 14	0.109 12	0.135 10
18	20	16	1	16			
21	23	19	1	15			
24	27	21	1.25	14	14		
30	33	26	1.5	14	14	14	
36	40	31	1.5		13	13	13
42	46	36	1.5			13	13
48	53	41	1.75			13	13
54	60	46	1.75				13
60	66	51	1.75				13

Limits for Checks:
Wall Area FS = 2
Buckling FS = 2

Constants:
Soil Density = 120 # / CU FT
Modulus of Elasticity = 1E+07 PSI
Minimum Yield Point = 24000 PSI
Min. Tensile Strength = 31000 PSI

Notes:
- Helical pipe lock seam
- Spiral Rib Pipe; Type I Installation (Embankment Condition)

Source:
CONTECH Construction Products Inc.

2.6 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 1999. *Highway Drainage Guidelines, Volume XIV, Highway Drainage Guidelines for Culvert and Inspection and Rehabilitation.* Task Force on Hydrology and Hydraulics, AASHTO Highway Subcommittee on Design. ISBN: 1-56051-128-1.

Chapter 3 HYDROLOGY

3.1 INTRODUCTION

Analysis of peak rate of runoff, volume of runoff, and time distribution of flow is fundamental for design of highway drainage facilities. Most drainage facility designs require determination of a peak flow rate while others require a runoff hydrograph that provides an estimate of runoff volume. Peak flow rates are most often used for design of bridges, culverts, roadside ditches, and small storm sewer systems. Drainage systems involving detention storage, pumping stations and large or complex storm sewer systems require the development of a runoff hydrograph to estimate volume of runoff.

The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex, and too little data is available on the factors influencing the rainfall-runoff relationship to expect exact solutions. Any hydrologic analysis is only an approximation, though an error in the estimate of peak runoff effects the design. Under prediction of the peak runoff can result in a structure that is undersized and may contribute to flooding while over predicting the peak runoff may lead to an oversized drainage facility that costs more than necessary.

This chapter provides design procedures for hydrologic analysis. For a more detailed discussion refer to the publications, *Highway Drainage Guidelines - Volume II*, (AASHTO, 1999) or *Highway Hydrology - HDS-2*, (FHWA, 1996).

3.1.1 Definition

Hydrology is a science dealing with the interrelationship and movement of various forms of water on and under the earth surface and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are measured in terms of peak runoff or hydrograph having a discharge in cubic feet per second (cfs). Structures designed to control volume of runoff, like detention storage facilities, or situations where flood routing is used, the entire discharge hydrograph will be of interest.

3.1.2 Concept Definitions

The following terms will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies.

Antecedent Moisture Conditions	Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes. Antecedent moisture has a rapidly decreasing influence on runoff as the flood recurrence interval becomes longer.
Depression Storage	Depression storage is the natural depressions within a watershed which store runoff. Generally after the depression storage is filled runoff will commence.
Drainage Area (A)	The area draining into a stream at a given point along the stream.
Frequency	Frequency is the number of times a flood of a given magnitude or greater can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.
Hydraulic Roughness	Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel as well as the channel storage characteristics.
Hydrograph	The hydrograph is a graph of the time distribution of runoff from a watershed.
Hydrologic Soil Group	A group of soils having the same runoff potential under similar storm and cover conditions.
Hyetographs	The hyetograph is a graph of the time distribution of rainfall over a watershed.

Infiltration	Infiltration is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.
Interception	Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.
Lag Time	The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.
Peak Discharge	The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.
Rainfall Excess	The rainfall excess is the water available to runoff after interception, depression storage and infiltration have been satisfied.
Rainfall Intensity (I)	Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour.
Recurrence Interval	The average number of years between occurrences of a discharge or rainfall that equals or exceeds the given magnitude.
Runoff (Q)	The part of the precipitation which runs off the surface of a drainage area after all abstractions are accounted for.
Runoff Coefficient	A factor representing the portion of runoff resulting from a unit rainfall. Principally dependent on terrain, topography, slope, land use and soil type.
Time of Concentration	The time of concentration is the time it takes a drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the point under investigation.
Ungaged Stream Sites	Locations at which no systematic records are available regarding actual stream flows.
Unit Hydrograph	A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution and which lasts for a unit duration of time. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area.

3.1.3 Factors Affecting Flood Runoff

In the hydrologic analysis for a drainage structure, there are many variable factors that affect floods. Some of the factors which need to be recognized and considered on an individual site by site basis are such things as:

- Drainage basin characteristics
  - drainage area size,
  - drainage area shape,
  - drainage area orientation,
  - slope of terrain,
  - land use; consider watershed development potential,
  - geology,
  - soil type,
  - surface infiltration,
  - storage,
  - antecedent moisture condition, and
  - storage potential: overbank, ponds, wetlands, reservoirs, channel, etc.

- Stream channel characteristics
  - channel slope,
  - channel geometry,
  - channel configuration,
  - natural and artificial controls,
  - channel modification,
  - aggradation/degradation, and
  - ice and debris.
- Flood plain characteristics
  - type of soil, and
  - ground cover.
- Meteorological characteristics
  - precipitation amount,
  - time rate of precipitation, hyetograph,
  - storm cell size,
  - storm cell distribution,
  - storm direction, and
  - type of precipitation: rain, snow, hail, or combinations thereof.

3.1.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site. It is the responsibility of the designer to determine what information is available and applicable to a particular analysis.

All hydrologic analysis shall consider the flood history of the area and the effect of these historical floods on existing and proposed structures. The flood history shall include the historical floods and the flood history of any existing structures. Files should be reviewed for documentation of relevant communications, studies and investigations. Surveys should be conducted to provide enough field data for analysis. Typical data that is obtained in such surveys or studies are: topographic maps, aerial photographs, streamflow records, historical highwater elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, and designated or regulatory flood plain areas.

Interagency coordination is necessary since many levels of government plan, design, and construct highway and water resource projects which might have an affect on each other. Agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analysis. Agencies include: Department of Natural Resources (DNR), U. S. Fish and Wildlife Service, U. S. Army Corps of Engineers (USACE), Watershed District & Management Organizations, Counties, Cities, Pollution Control Agency (PCA), Natural Resources Conservation Service (NRCS) previously Soil Conservation Service (SCS) and U. S. Geologic Survey (USGS).

3.2 DESIGN FREQUENCY

Design frequency should be selected commensurate with cost of the facility, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. When long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. Consideration should be given to what frequency flood was used to design other structures along a highway corridor. In selecting a design frequency, Storm Water Management Plans should be considered where watershed changes could reasonably occur over the anticipated life of the drainage facility.

It is not economically feasible to design highway structures for the maximum runoff that a watershed is capable of producing. Therefore, a design frequency must be established. The design frequency for a given flood is defined as the reciprocal of the probability that a flood flow will be exceeded in a given year. The frequency is analogous to the recurrence interval. A 50-year recurrence interval means that every year there is a 2% chance that a 50-year flood will occur at a given point, and it could conceivably occur in several consecutive years. Over a long period of time, the 50-year flood will be equaled or exceeded on the average of once every fifty years.

While drainage structures are designed to operate for a given design frequency, performance should be checked for the review frequency. After sizing a drainage facility to pass a peak flood or the hydrograph corresponding to the design frequency, it may be necessary to review this proposed facility considering a larger discharge to insure that there are no unexpected flood hazards inherent in the proposed facility(ies). Potential impacts to consider include possible flood damage due to high embankments where overtopping is not practical, backup due to the presence of noise walls, and flood damage where a storm drain might back up. The flood damage potential due to bridges and major culverts (greater than 48") should be reviewed for the 100 year frequency. The scour potential for bridge substructures should be reviewed for the 500 year frequency or overtopping event.

3.2.1 Design Frequency Policy

The design frequency used to design a hydraulic structure is determined by the type, size, and location of the structure. Design frequency for inlets and storm sewers is based on the allowable spread on the roadway. Minor culverts (48" or less in diameter) shall be designed using a 50 year frequency. Major culverts (larger than 48" in diameter) and bridges require completion of a Risk Assessment Form (Appendix A) to determine the appropriate design frequency.

Bridges and Centerline Culverts

For all bridges over waterways, and for major centerline culverts (larger than 48"), a risk assessment shall be completed. Instead of arbitrary design frequencies, the risk assessment procedure takes into consideration capital costs and risks, and other economic, engineering, social, and environmental concerns. The risk assessment is based on the:

- the overtopping flood or the base flood, whichever is greater , or
- the greatest flood which must flow through the highway drainage structure where overtopping is not practicable. This is considered to be a 500-year frequency flood. If flood frequency data is not available, use 1.7 x 100 year flood.

Table 3.1 gives the guidelines for the recommended minimum overtopping flood criteria which should be used for a risk assessment. The risk assessment procedure is difficult to apply to small culverts. Consequently, a formal risk assessment or analysis will ordinarily not be required for minor culverts (48 in. diameter or less) unless there is significant flood damage potential. The design frequency for minor centerline culverts shall be a minimum of 50 year frequency. A copy of the risk assessment form and other information is provided in Appendix A.

Table 3.1 Minimum Overtopping Flood Frequency for Risk Assessment

PROJECTED ADT	MINIMUM OVERTOPPING FLOOD FREQUENCY
0 - 10	2 year
11 - 49	5 year
50 - 399	10 year
400 - 1499	25 year
1500 and up	50 year

<u>Entrance Culverts</u>
Entrance culverts shall be a minimum of 15" in diameter. They should be designed for a 10 year frequency and an overtopping area should be provided.

<u>Storm Drains</u>
Storm drains shall be designed to accommodate a discharge with a given return period(s) or frequency. To select the return period, assume the storm runoff should not increase the flood hazard significantly for the property. Also, the runoff should not encroach on to the street or highway so as to cause a significant traffic hazard, or limit traffic, and pedestrian movement to an unreasonable extent.

The design storm frequency for pavement drainage and the other components of the drainage system should be consistent. In order for it to be meaningful criteria, the design frequency must be tied to a design water spread. Table 3.2 gives Mn/DOT's established criteria for design frequency and allowable water spread.

Ramps, Loops, Turn Lanes, Acceleration and Deceleration Lanes should generally be designed to the same frequency as the mainline. Where the speed limit is 35 mph or less and there are no shoulders or parking lanes, the allowable spread can encroach up to ½ D for short periods of time. High capacity inlets such as slotted vane drains should be considered at locations where excessive spread may cause inconvenience or safety hazards to motorists.

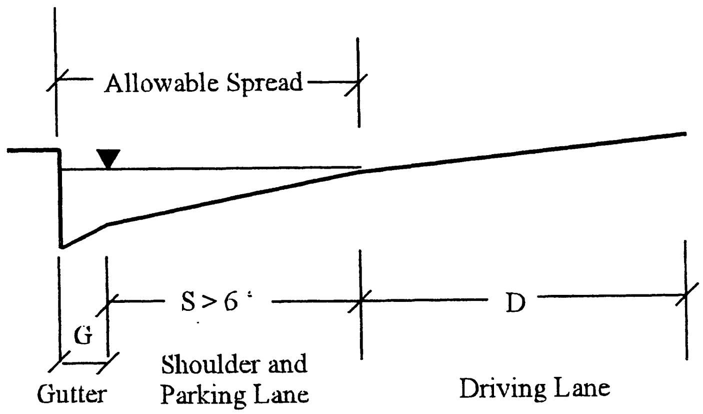


Figure 3.1 Allowable Spread for Shoulders of 6 Feet or more

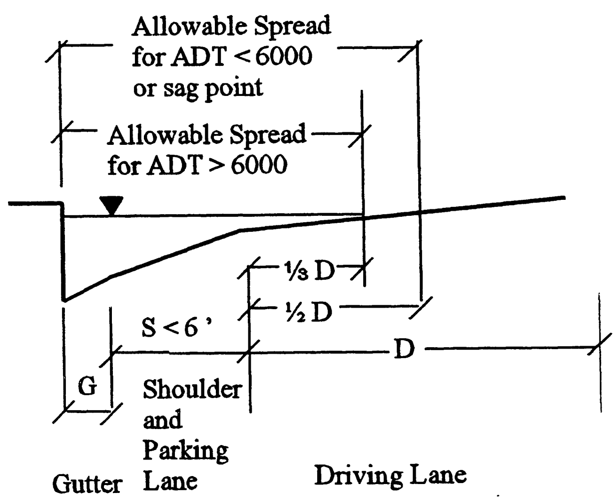


Figure 3.2 Allowable Spread for Shoulders Less than 6 Feet

Table 3.2 Design Frequency for Storm Drains

PROJECTED 2-WAY ADT (vehicles per day, VPD)	DESIGN FREQUENCY (Year)	ALLOWABLE SPREAD[1]
> 6000	10	P, S, or a D
	50 Year at sag point[2]	P, S, or ½ D
2000-6000	10	P, S, or ½ D
1000-1999	5	P, S, or ½ D
< 1000	3	P, S, or ½ D

[1] P = Parking lane, S = Shoulder of 6′ or more, D = Driving lane if there is no shoulder.
[2] Sag Point refers to a true sag where flooding of 2' or more can occur.

3.2.2 Rainfall vs. Flood Frequency

Drainage structures are designed for a designated flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed in most cases that the 10-year rainfall will produce the 10-year flood. In general, rainfall is used to design storm drains and culverts with small drainage areas. The rational method and SCS methods use the assumption that a rainfall of a certain frequency will produce a flood of the same frequency.

Rainfall intensity-duration curves have been developed for the common design frequencies. Rainfall intensity data for Minnesota generated from HYDRO-35 is provided in Section 3.5.4. Appendix B contains 24 hour rainfall amounts for various recurrence intervals in Minnesota based on the U.S. Weather Bureau's *Technical Publication No. 40* (Hershfield, 1961). Additional data on rainfall can be found in other sources. Data on historical storms may be available from the State Climatologist.

3.3 HYDROLOGIC PROCEDURE SELECTION

Streamflow measurements are usually unavailable for determining flood frequency relationships at a site. In such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. The design discharge for other structures on the stream and historical data should be reviewed, and consideration given to previous studies and Flood Insurance Studies. Mn/DOT's practice shall be to use the discharge that best reflects local project conditions with the reasons documented.

3.3.1 Peak Flow Rates or Hydrographs

Determination of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design includes storage basins or complex conveyance networks, a flood hydrograph is usually required. The development of runoff hydrographs is usually accomplished using computer programs.

3.3.2 Hydrologic Procedures Options

The recommended hierarchy for selecting a method of computing discharge for design of highway structures in Minnesota and the circumstances for their use are listed below and shown in Table 3.3. Where feasible, the hydrologic model should be calibrated to local conditions and tested for accuracy and reliability. Consider design discharges for other structures in the area and historical data for the area. In general the results from different hydrologic models should not be averaged.

- Rational method shall be used only for drainage areas less than 200 acres, and preferably only for developed areas.
- Both the graphical peak discharge and tabular hydrograph method available in TR-55 are simplified procedures are derived from TR-20.
- The TR-55 graphical method may be used for small rural watersheds (1 - 2000 acres).
- TR-55's tabular hydrograph method can be applied to larger watershed areas by splitting a non-homogenous watershed into homogenous subareas.
- Minnesota USGS regression equations should be used for routine designs of bridges and culverts with 54" or larger widths and in accordance with the other limitations of the regression equations, unless there is gaging station data or historical evidence suggesting other alternatives.
- Log Pearson III analyses of stream gaging station data should be used for all routine designs with gaging station data, provided there is at least 10 years of continuous record for 10-year discharge estimates and 25 years for 100-year discharge estimates.
- The SCS unit hydrograph method contained in TR-20 should be used for storage routing or storage design.
- Suitable computer programs such as HEC 1, TR-20, TR-55, and HYDRAIN may be used for hydrologic calculations. Other computer programs that incorporate the recommended methodology may also be used. Output from all programs must be reviewed to see that the answers are reasonable.
- The 100-year discharges specified in the FEMA flood insurance study shall be used to analyze impacts of a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, discharges based on current methods may be used subject to receipt of necessary regulatory approvals.

Table 3.3 Selection of Discharge Computation Method

FACILITY DESCRIPTION	METHOD				
	Frequency Analysis	Regression Equations	SCS Hydrograph	SCS Peak Discharge	Rational
Stream Flow, Channels, Bridges, & Culverts Greater then 48"	customary method	alternate method	complex facilities or hydrograph needed	alternate method	
Culverts 48" and Smaller		alternate method	complex facilities or hydrograph needed	customary method	alternative method
Storm Drains, Roadside Ditches, & Side Culverts			complex facilities or hydrograph needed		customary method
Detention Basins			customary method		preliminary evaluation, or permit review

3.4 TIME OF CONCENTRATION

The time of concentration (t_c) is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Water moves through a watershed as a combination of overland and channelized flow. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection. t_c is an important variable in many hydrologic methods, including the Rational and Soil Conservation Service (SCS) procedures. For the same watershed, the shorter the t_c, the larger the peak discharge for similar land use. Many of the computation methods for determining time of concentration are available within most commonly used computer applications.

3.4.1 Total Time of Concentration

To obtain the total time of concentration, the channel flow time, or travel time is calculated and added to the overland flow time. The units must be consistent for all values in Equation 3.1. The Rational Equation normally uses t_c in minutes, SCS procedures generally use t_c in hours. The total time of concentration is:

$$t_c = t_o + t_t \qquad \textbf{(3.1)}$$

Where: t_c = total time of concentration
t_o = overland flow time
t_t = travel time

In some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to influence the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application.

When designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to streets which reduces the time of concentration. Generally the overland flow path is less than 200 feet in urban areas and 400 feet in rural areas.

For storm drainage systems, the designer is often concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet, which is known as the inlet time. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of the gutter to the inlet. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. Travel time between inlets is not considered. When the total time of concentration for pavement drainage inlets is less than seven minutes, a minimum of seven minutes should be used to estimate the duration of rainfall.

When determining pipe size, the time of concentration (t_c) for any point along a storm drain is the time to reach the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in a storm drainage system, the longest t_c is used to estimate the intensity (I). Exceptions to this generality, include a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be aware of this possibility when joining drainage areas and determine which drainage area governs.

3.4.2 Travel Time

Travel time is the length of time it takes to travel as channelized flow. Manning's Equation and the Continuity Equation calculate velocity not time. After determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing channel length by average pipe velocity.

$$t_t = \frac{L}{60V} \tag{3.2}$$

Where: t_t = travel time (min)
L = length which runoff must travel (ft)
V = estimated or calculated velocity (ft/s)

3.4.3 Selection of Method

When selecting a method for determining the time of concentration, consider the conditions for which the equation was developed and how they compare to the drainage area being evaluated; what hydrologic computation procedures will be applied; and what type of facility is being designed. The methods for determining time of concentration included in this section are applicable for many different hydrologic computation procedures, though when SCS methods are used to compute discharge, time of concentration should be determined using the methods recommended by the Natural Resource Conservation Service (NRCS).

Table 3.4 Methods for Calculating Time of Concentration

METHOD	COMMENTS
Kinematic Wave Equation	Overland flow time. Requires iterative solution.
Manning's Kinematic Solution	Overland flow time. The maximum flow length is 300'.
Manning's Equation • Overland Flow • Triangular Gutter Flow • Pipe Flow	Flow velocity for non-pressure flow in pipes, open channels, gutter flow or overland flow.
Continuity Equation	Flow velocity for pressure flow in pipes.

3.4.4 Kinematic Wave Equation

HEC-12 (FHWA, 1984) recommends the kinematic wave equation as the most realistic method for estimating overland flow time of concentration.

$$t_o = \frac{56L^{0.6}n^{0.6}}{i^{0.4}S^{0.3}} \tag{3.3}$$

Where: t_o = time of overland flow (sec)
L = overland flow length (ft)
n = Manning's roughness coefficient
i = rainfall rate (in/hr)
S = average slope of the overland area

When using this equation, both the time of concentration and rainfall intensity are unknowns and trial and error iterations are required. A value for rainfall intensity (i) is assumed and the related time of concentration found. The assumed rainfall intensity must then be checked against the rainfall Intensity-Duration-Frequency curve for the frequency of the event chosen for the particular design problem, and the procedure repeated until the assumed rainfall intensity is in agreement with the intensity associated with the time of concentration.

3.4.5 Manning's Kinematic Solution

For sheet (overland) flow for a distance of less than 300 feet, TR-55 (SCS, 1986) recommends using Manning's kinematic solution to compute t_o. This simplified form of the Manning's kinematic solution is based on the following assumptions: shallow steady uniform flow, constant intensity of rainfall excess (rain available for runoff), rainfall duration of 24 hours, and minor infiltration effect on travel time.

$$t_o = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}} \qquad (3.4)$$

Where: t_o = overland flow time (hr)
n = Manning's roughness coefficient, Table 3.5
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (ft/ft), assumed equivalent to land slope

Table 3.5 Roughness Coefficients (Manning's n) For Sheet Flow

SURFACE DESCRIPTION		n[1]
Smooth Surfaces (concrete, asphalt, gravel, or bare soil)		0.011
Fallow (no residue)		0.05
Cultivated Soils	Residue cover ≤ 20%	0.06
	Residue cover > 20%	0.17
Grass	Short grass prairie	0.15
	Dense grasses[2]	0.24
	Bermuda grass	0.41
Range (natural)		0.13
Woods[3]	Light underbrush	0.40
	Dense underbrush	0.80

[1] The n values are a composite of information complied by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft, the only part of the plant cover that will obstruct sheet flow.

3.4.6 Manning's Equation

For watersheds with storm drains or channels, the travel time must be added into the total time of concentration. Manning's Equation is used to determine the average velocity, usually determined for bank-full flow conditions. The velocity calculated from Mannings equation is plugged into Equation 3.2 to calculate the travel time. Open channels are assumed to begin where the surveyed cross sections have been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (3.5)$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius (ft) = Area (ft^2) / Wetted Perimeter (ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.7 Overland Flow

Manning's Equation is often used to calculate the flow velocity in order to estimate time of concentration for shallow concentrated flow overland. The SCS Upland Method is a graphical version of this equation, and a t_c calculated with the following assumption can be used with the SCS Peak Flow method. Determine velocity from Equation 3.5 with the assumption hydraulic radius (R) is equal to the flow depth (d). Table 3.6 gives approximate values for "d" and "n" based on type of cover. Apply Equation 3.2 to calculate t_c for a known length and velocity.

Table 3.6 Roughness Coefficients (Manning's n) For Overland flow

SURFACE DESCRIPTION	MANNING'S N	
	FLOW DEPTH d = 0.1 feet	FLOW DEPTH d = 0.2 feet
Forest with heavy litter, hay meadow & brush	0.13	0.20
Cultivated: minimum tillage, contoured or strip cropped & woodland	0.065	0.10
Pasture or cultivated straight row crop	0.045	0.07
Nearly bare and untilled	0.03	0.05
Paved area	0.015	0.025

Source: SCS Hydrology Guide for Minnesota Figure 4-1

Simplified Manning's Equations are available (SCS, 1986) for grassed waterway (unpaved areas) and paved areas to determine travel time for shallow concentrated flow.

Unpaved $$V = 16.1345(S)^{0.5} \tag{3.6}$$

Paved $$V = 20.3285(S)^{0.5} \tag{3.7}$$

Where: V = average velocity (ft/s)
S = slope of hydraulic grade line (ft/ft), watercourse slope

3.4.8 Triangular Gutter Flow

The travel time for gutter flow is estimated using an average velocity of the flow. HEC-12 (FHWA, 1984) contains a nomograph for determining the velocity in a triangular gutter section given the watercourse slope, gutter cross slope and water spread. The equation can be modified for composite cross sections. For a triangular channel with uniform inflow per length and zero flow at the upstream end, the average velocity will occur where the spread is 65% of the maximum. For a triangular channel Manning's equation becomes:

$$V = \frac{1.12}{n} T^{2/3} S_x^{2/3} S^{1/2} \tag{3.8}$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
T = spread across cross section (ft)
S_x = cross slope (ft/ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.9 Pipe Flow

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full for the design discharge. For non-pressure flow, the velocity can be determined using Manning's equation. For circular pipes flowing full, the equation becomes:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \tag{3.9}$$

Where: V = mean velocity of flow (ft/s)
n = Manning's roughness coefficient
D = diameter of circular pipe (ft)
S = slope of the hydraulic grade line (ft/ft)

3.4.10 Continuity Equation

If the pipes of a storm drainage system will operate under pressure flow, the continuity equation should be used to determine the velocity.

$$V = \frac{Q}{A} \tag{3.10}$$

Where: V = mean velocity of flow (ft/s)
Q = discharge in pipe (cfs)
A = area of pipe (ft^2)

3.5 RATIONAL METHOD

The rational method is commonly used to calculate the peak flow from small drainage areas. It is recommended for estimating the design storm peak runoff for areas up to 200 acres.

3.5.1 Application

The rational formula applies best to developed areas with significant amount of pavement or gutters and is typically used for designing storm drain systems, and small or sideline culverts. The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration. The equation is:

$$Q = CIA = \left(\sum CA\right) I \tag{3.11}$$

Where: Q = discharge (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = rainfall intensity (in/hr)
A = drainage area (acres)

The peak discharge calculated by the rational formula is very sensitive to the values selected for the parameters. The designer must use good engineering judgment in estimating values that are used in the method. The first step in applying the rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection should be conducted to determine if natural drainage divides have been altered. Restrictions to the natural flow such as highway crossings and dams that exist in the drainage area should be investigated to determine how they affect the design flows.

3.5.2 Limitations

The assumptions of the rational method which limit its use to small drainage areas of up to 200 acres include:

- The rate of runoff resulting from any rainfall intensity is maximum when the rainfall intensity lasts as long or longer than the time of concentration of the drainage area. This assumption limits the size of the drainage basin that can be evaluated by the rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows.
- The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration. Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control.
- The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume. This assumption is reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. The selected runoff coefficient must be appropriate for the storm, soil, and land use conditions.
- The peak rate of runoff is sufficient information for the design.

3.5.3 Runoff Coefficient

The runoff coefficient (C) requires engineering judgment and understanding on the part of the designer to apply correctly. Typical coefficients represent the integrated effects of many drainage basin parameters. The selected value should incorperate storm, soil and land use conditions. The runoff coefficients for various types of surfaces are shown in Table 3.7. The total CA value is to be based on a ratio of the drainage areas associated with each C value as follows:

$$Total(CA) = \left(\sum CA\right) = A_1C_1 + A_2C_2 + \ldots + A_nC_n \tag{3.12}$$

Selection of the runoff coefficients (C) requires good engineering judgement. The designer should document justification for the C values used at a given site. It is not appropriate to use the lowest value in a range, unless warranted by local site conditions. Considerations for selecting C include rainfall intensity, slope, soil type, direct connection of impervious area, future land use, detention effects.

- The coefficients provided in Table 3.7 are applicable for storms of 5 yr to 10 yr frequencies. Less frequent, higher intensity storms will require a higher coefficient because infiltration and other losses have a proportionally smaller effect on runoff.
- As the slope of the drainage basin increases, the selected C value should also increase. This is caused by the fact that as the slope of the drainage area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area.
- Consider soil type and infiltration rates when selecting C. Sandy soils infiltrate more resulting in less runoff, while clay soils produce more runoff.
- In determining the runoff coefficient (C) values for the drainage area, consider future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system.
- The effects of upstream detention facilities and local stormwater management plans should be taken into account.

Table 3.7 Runoff Coefficients for Rational Formula

TYPE OF DRAINAGE AREA			RUNOFF COEFFICIENT
Business	Downtown areas		0.70 - 0.95
	Neighborhood areas		0.50 - 0.70
Residential	Single-family areas		0.30 - 0.50
	Multi-units	Detached	0.40 - 0.60
		Attached	0.60 - 0.75
	Suburban		0.25 - 0.40
	Apartment dwelling areas		0.50 - 0.70
Industrial	Light areas		0.50 - 0.80
	Heavy areas		0.60 - 0.90
Parks, cemeteries			0.10 - 0.25
Playgrounds			0.20 - 0.35
Railroad yard areas			0.20 - 0.40
Unimproved Urban Areas			0.10 - 0.30
Lawns	Sandy soil	flat, 2%	0.05 - 0.10
		average, 2 - 7%	0.10 - 0.15
		steep, 7%	0.15 - 0.20
	Heavy soil	flat, 2%	0.13 - 0.17
		average, 2 - 7%	0.18 - 0.22
		steep, 7%	0.25 - 0.35
Streets	Asphaltic		0.70 - 0.95
	Concrete		0.80 - 0.95
	Brick		0.70 - 0.85
Drives and walks			0.75 - 0.85
Roofs			0.75 - 0.95
Rural	Average infiltration rates sandy & gravel soils	Cultivated	0.20
		Pasture	0.15
		Woodlands	0.10
	Average infiltration rates; Loams and similar soils with no clay pans	Cultivated	0.40
		Pasture	0.35
		Woodlands	0.30
	Below average infiltration rates; heavy clay soils; soils with a clay pans near the surface; shallow soil above impervious rock	Cultivated	0.50
		Pasture	0.45
		Woodlands	0.40

Source: data for urban type drainage areas ASCE, 1960

3.5.4 Rainfall Intensity

The average rainfall intensity (I) is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. The time of concentration is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Methods for computing time of concentration are given in Section 3.4.

Intensity is the rate of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain. For example, an intensity of 5 inches per hour for a duration of 5 minutes indicates a total rainfall amount of 5 X 5/60 = 0.42 inches. Rainfall frequency relations will vary from one area to another. An approach to selecting the values is to review the sources for a particular area and then select the sources that seems most appropriate.

The value for Intensities (I) for storm durations of up to 60 minutes and for typical recurrence frequencies are given in Table 3.8. This table correspond to the 3 zones of the state shown on Figure 3.3. The intensity-duration-frequency (IDF) curves for each zone are provided in Figures 3.4, 3.5 and 3.6 The curves were developed using the procedure from NWS technical memorandum *HY-DRO-35* (Frederick, et al. 1977).

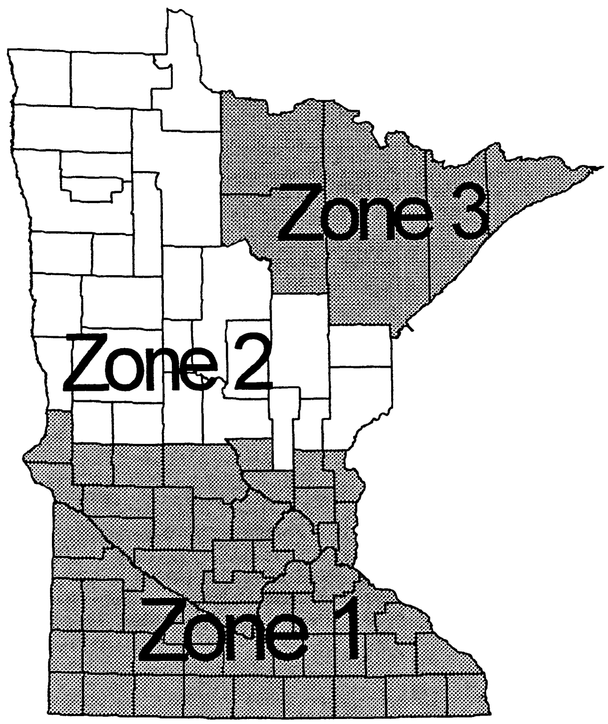


Figure 3.3 Minnesota Zones for Intensity-Duration-Frequency (IDF) Curves

Table 3.8 Rainfall Intensity - Duration - Frequency Tables for Minnesota

RAINFALL INTENSITY (in/hr)

ZONE 1 - SOUTHERN					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	5.2	4.1	3.4	2.3	1.4
3	5.6	4.5	3.8	2.6	1.6
5	6.3	5.1	4.3	2.9	1.9
10	7.1	5.8	4.9	3.4	2.2
25	8.3	6.8	5.8	4.0	2.6
50	9.3	7.7	6.5	4.5	2.9

ZONE 2 - NORTHWEST AND CENTRAL					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	5.0	3.9	3.2	2.1	1.3
3	5.5	4.3	3.6	2.4	1.5
5	6.2	4.9	4.1	2.8	1.8
10	7.0	5.6	4.7	3.2	2.1
25	8.2	6.6	5.5	3.8	2.5
50	9.2	7.3	6.2	4.3	2.8

ZONE 3 - NORTHEAST					
Event Frequency (Years)	Rain Duration or Time of Concentration (Minutes)				
	5	10	15	30	60
2	4.9	3.6	3.0	1.9	1.2
3	5.3	4.0	3.2	2.2	1.4
5	5.9	4.5	3.7	2.5	1.6
10	6.6	5.0	4.2	2.8	1.8
25	7.7	5.9	4.9	3.4	2.2
50	8.5	6.6	5.4	3.8	2.5

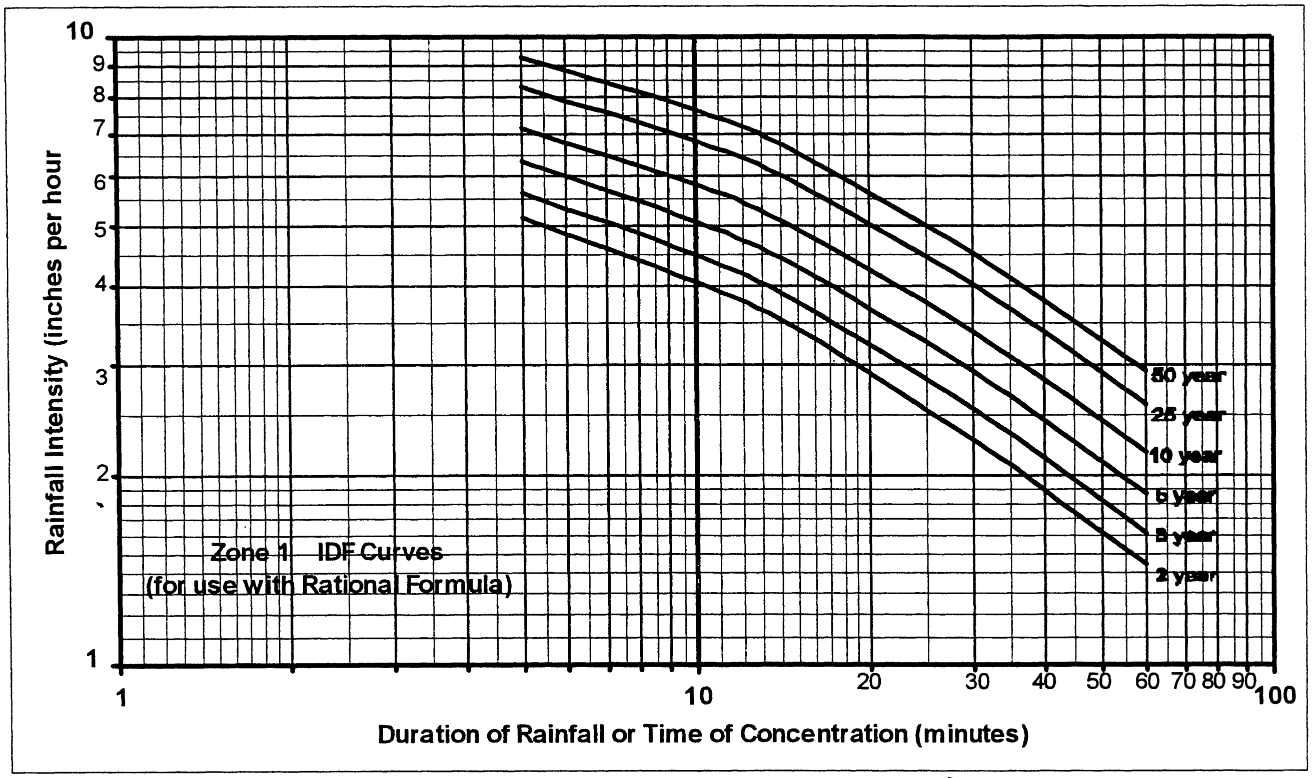


Figure 3.4 Zone 1 Southern Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

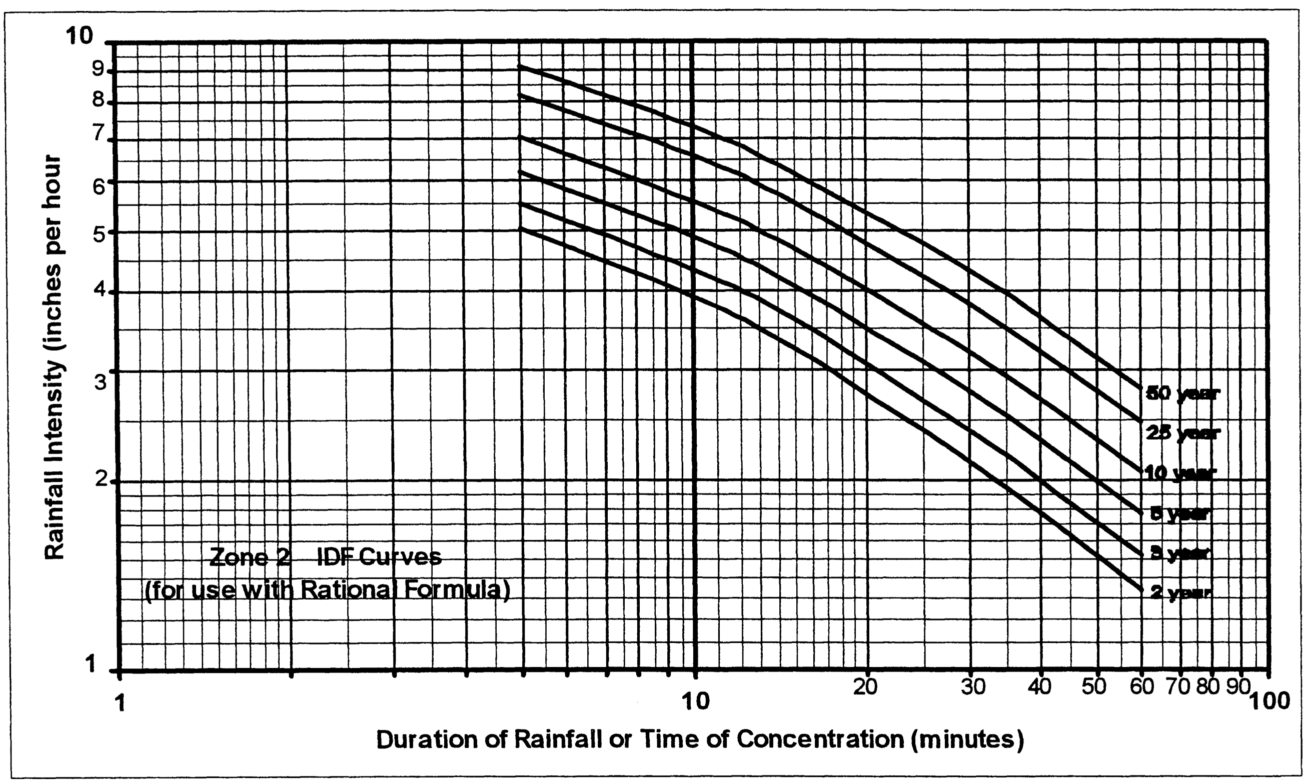


Figure 3.5 Zone 2 Northwest and Central Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

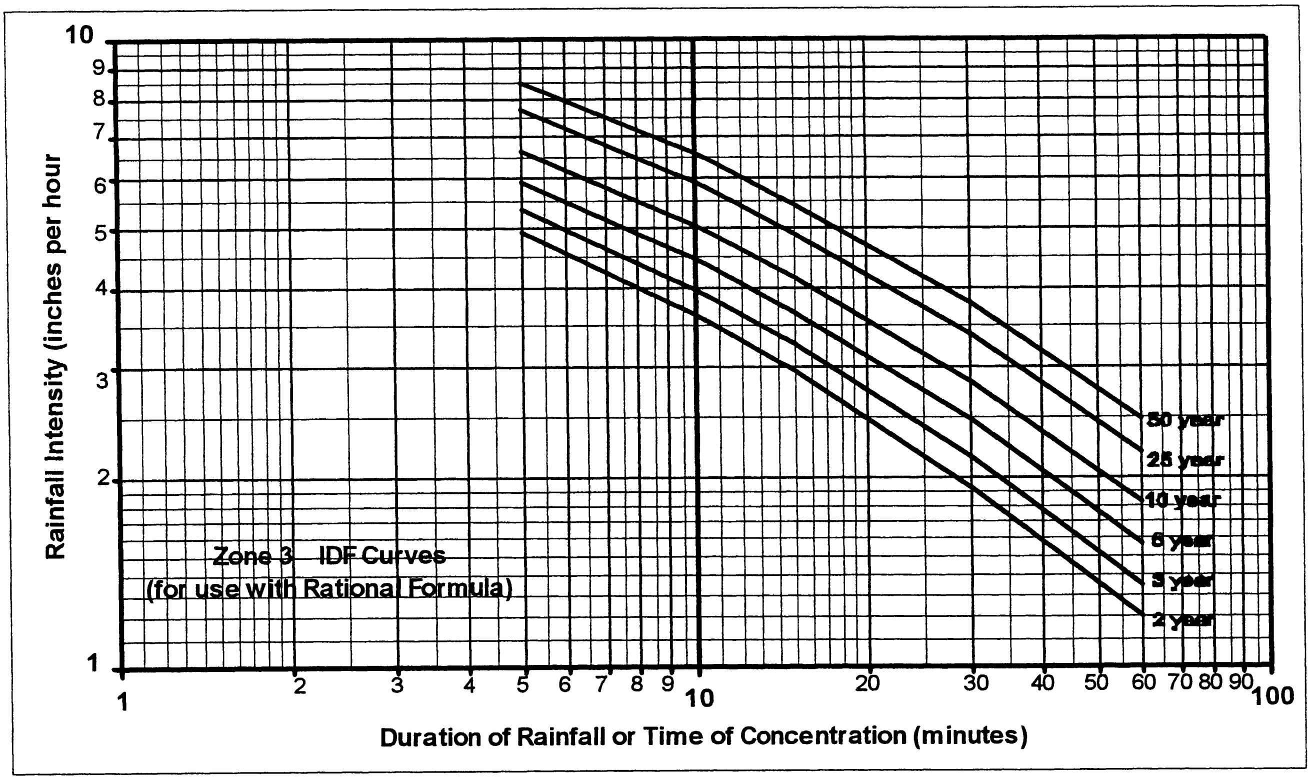


Figure 3.6 Zone 3 Northeast Minnesota Rainfall Intensity - Duration - Frequency (IDF) Curves

3.6 SCS METHOD

The SCS Method was developed by the U. S. Soil Conservation Service (SCS) before the name of the agency changed to the Natural Resources Conservation Service (NCRS). The technique estimates the rate of runoff from the following parameters: drainage area, runoff factor (Curve Number), time of concentration, and rainfall. The SCS approach considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. A peak flow can be determined graphically, or a hydrograph can be generated. The hydrograph procedure is used primarily for the design and analysis of storage areas. The peak flow procedure is generally used for designing culverts. The SCS procedures are widely used to determine runoff, peak flow, and hydrographs for ungaged watersheds.

The runoff and hydrograph procedures are explained in the SCS *National Engineering Handbook* (SCS, 1985), Section 4 (NEH-4). Additional information is in the *Hydrology Guide for Minnesota* (SCS, ----), *SCS Technical Release No. 55* (SCS, 1986), and *SCS Engineering Field Manual, Chapter 2* (SCS, 1989).

3.6.1 Application

In synthetic hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall. This is defined as the amount of rainfall that exceeds the capability of the land to infiltrate or otherwise retain the rain water. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads and roofs are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The SCS method uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher the runoff potential. A relationship between rainfall and runoff was derived by NCRS from experimental plots for numerous soils and vegetative cover conditions. The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour storm rainfall events.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \textbf{(3.13a)}$$

Initial abstraction (I_a) is all losses that occur before runoff begins, including surface storage, interception, and infiltration. Based on studies of experimental watershed data a relationship between I_a and S was developed, where initial abstraction (I_a) is 20% of the maximum retention. Substituting 0.2S for I_a, the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \textbf{(3.13b)}$$

S is related to the soil and cover conditions of the watershed through the Curve Number (CN):

$$S = \frac{1000}{CN} - 10 \qquad \textbf{(3.14)}$$

Where: Q = accumulated direct runoff (inches)
P = accumulated rainfall (inches)
I_a = initial abstraction (inches)
S = potential maximum retention after runoff begins (inches)
CN = SCS Curve Number

3.6.2 Limitations

The following limitations apply to SCS runoff procedures:

- Runoff from snowmelt or rain on frozen ground cannot be estimated using the procedures given in this section.
- The Curve Number procedure is less accurate when runoff is less than 0.5 inch.
- The SCS runoff procedures apply only to direct surface runoff. In some cases, large sources of subsurface flow can contribute to the runoff.
- The TR-55 graphical method may be used only for hydrologically homogeneous watersheds because the procedure is limited to a single watershed subarea where the drainage area is greater than 1.0 acre and less than 2,000 acres.
- The TR-55 tabular hydrograph method can be used for a heterogeneous watershed that is divided into homogeneous subareas. The hydrographs from each subarea can be routed and combined.
- The runoff equation assumes the initial abstraction (I_a) to be 20% of the maximum retention based on studies on small watersheds. I_a consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors. This approximation can be significant in urban areas because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect can occur if the impervious areas have surface depressions that store some runoff.
- The watershed should have one main stream. If more than one exists, the branches must have nearly equal T_c values.
- The watershed must be hydrologically similar; i.e., able to be represented by a weighted CN. Land use, soils, and cover are distributed uniformly throughout the watershed or subwatershed.
- If the computed T_c is less than 0.1 hour, use 0.1 hour. If the computed T_c is greater than 10 hours, peak discharge should be estimated by using the NEH-4 procedures (SCS, 1985), which are automated in the TR-20 computer program.
- Use the same procedure to estimate time of concentration (T_c) when calculating the peak discharge for both the present and developed conditions of a watershed.
- If depression storage constitutes more than one-third of the total drainage area or if it intercepts the drainage, the procedures in NEH-4 should be used.
- When the weighted CN is less than 40 or more than 98, use another procedure to estimate peak discharge.
- If the drainage area is greater than 10 square miles, the point rainfall should be adjusted using the procedure from Chapter 4, NEH-4 (SCS, 1985) or TP-40 (Hershfield, 1961).

3.6.3 Rainfall

The SCS method is based on a 24-hour storm event with a specified time distribution. For design purposes, a Type I or Type II time distribution may be used in Minnesota. The storm distributions (Figure 3.7) are "typical" time distributions which the NCRS has developed using rainfall records from around the country. To use a distribution it is necessary for the user to obtain the 24-hour rainfall value. Appendix B contains 24 hour rainfall amounts for various return periods. Historical storm events can also be used as input.

For Minnesota, the local office of the NCRS recommends the Type I storm distribution for rural watersheds greater than 30 acres. The Type II storm distribution is recommended for urban watersheds, for small watersheds (less than 30 acres), and steep drainage areas.

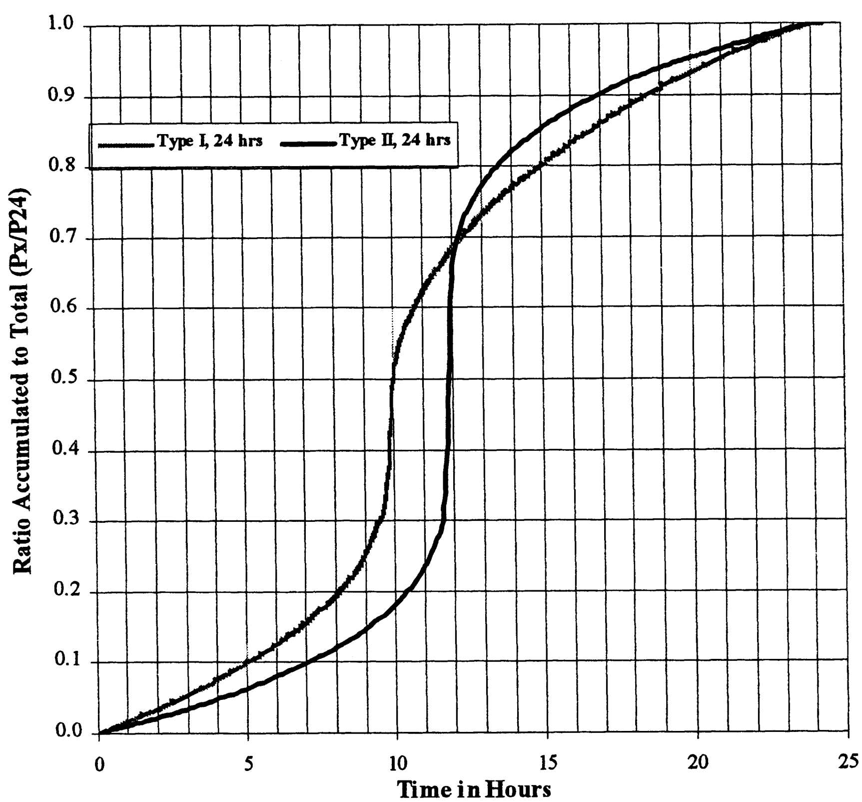


Figure 3.7 Type I and Type II Design Storm Curves
Source: Developed from SCS, 1986 Exhibit 5

3.6.4 Hydrologic Soil Group

Soil properties influence the relationship between runoff and rainfall since different soil groups have different rates of infiltration. Based on these infiltration rates, the Natural Resources Conservation Service (NRCS) has divided soils into four hydrologic soil groups as follows:

Group A Low runoff potential - Soils having high infiltration rates even when thoroughly wetted, consisting chiefly of deep, well to excessively drained sands and/or gravel. These soils have a high rate of water transmission and would result a low runoff potential. Minimum infiltration rate: 0.30 to 0.45 inch per hour.

Group B Soils having moderate infiltration rates when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission. Minimum infiltration rate: 0.15 to 0.30 inch per hour.

Group C Soils having slow infiltration rates when thoroughly wetted, consisting chiefly of soils with a layer that impedes the downward movement of water, or soils with moderately fine to fine texture and a slow infiltration rate. These soils have a slow rate of water transmission. Minimum infiltration rate: 0.05 to 0.15 inch per hour.

Group D High runoff potential - Soils having very slow infiltration rates when thoroughly wetted, consisting chiefly of clay soils with a high swelling potential; soils with a high permanent water table; soils with claypan or clay layer at or near the surface; and shallow soils over nearly impervious materials. These soils have a very slow rate of water transmission. Minimum infiltration rate: 0 to 0.05 inch per hour.

Soil classifications for Minnesota soils are given in Table 3.9. Where the Soil Class is given as B/D the first letter represents the drained condition and the second letter is the undrained condition. Detailed information on the soil type may be found in the county soil survey report or can be requested from the soils engineer. Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

3.6.5 Curve Number

The SCS runoff procedure uses a Curve Number (CN) based on the watershed's soil and cover conditions. The CN describes average conditions that are useful for design purposes. Tables 3.10 and 3.11 provide curve numbers (CN) for urban and agricultural land uses. In selecting the curve number, the Minnesota NCRS recommends that ratings as to "Poor" or "Good" are based largely on the proportion of dense vegetation in the rotation. "Good" will generally be used for cultivated land in Minnesota except where land is very droughty or severely abused.

Curve numbers are determined based upon soil hydrologic groups present in the drainage area, and are also influenced by land use type (e.g., cultivated versus forested). These soil hydrologic groups are discussed in detail in Section 3.6.4. The soil hydrologic groups (A-D) are primarily a function of soil texture and structure as well as moisture content. Group A soil is most permeable, while groups B-D are progressively less permeable, with group D exhibiting the least infiltration and most runoff per unit land area. The influence of the soil hydrologic groups upon the curve numbers for various land use types is shown in Table 3.11. The soil hydrologic group, condition of the land, and its use are all important for determining the curve number and estimating runoff.

The most accurate way to determine the curve number is to identify the soil group or groups in the drainage area in question using a county soil survey from Natural Resources Conservation Service (NRCS). This should be available in a local or county NRCS office and/or it may be stored in the Mn/DOT District Office. As of the year 2000, approximately 75% of the state of Minnesota was mapped by NRCS into soil classifications with the respective soil groups referenced accordingly. Table 3.9 is similar to the soil classification and soil hydrologic group reference data that can normally be found in the NRCS county soil survey in a table referred to as "Water Features" near the end of the document. Note that each soil classification is assigned a soil hydrologic group in Table 3.9.

When the SCS method of predicting runoff is used, the NRCS soil survey data should be employed whenever possible as it is the best soil information available in most cases. If soil hydrologic group data is unavailable through the county surveys, another good data source is the NRCS "STATSGO" GIS database. The scale of this data base map is 1:250,000 and views of the drainage area being investigated can be manipulated using Arc View. Hydrologic soil groups are presented as a GIS theme in Arc View using the STATSGO database.

Curve numbers are often finalized after field inspection of the watershed and review of aerial photographs, zoning maps, and soil type maps. Future changes in land use that might occur during the service life of the proposed facility should be evaluated as such changes could lead to a larger volume of runoff and higher peak discharges. Also, consider the effects of upstream detention facilities and local stormwater management plans may be taken into account. Compute a weighted curve number for the drainage area. Sum up the products of area and CN for each surface/soil type, and divide by the total drainage area.

$$\text{Weighted } CN = \frac{CN_1 A_1 + CN_2 A_2 + \ldots + CN_n A_n}{TotalA} \tag{3.15}$$

Where: CN = Curve Number
A = Area

Urban development has its own unique charactersitics. Impervious surfaces such as roads, parking lots, and buildings cover a significant amount of an urban watershed. The urban CN's given in Table 3.11 apply for typical land use, assuming percentages of impervious area specified in the table. Curve numbers with other percent imperviousness can be calculated using a weighted CN approach. The impervious areas are assumed to have a CN of 98 and pervious areas are assumed to be equivalent to open space in good condition.

$$CN_w = CN_p(1-f) + f(98) \tag{3.16}$$

Where: CN_w = weighted Curve Number
CN_p = pervious Curve Number from Table 3.11
f = fraction of imperviousness (decimal value)

The urban CN's listed in Table 3.11 were developed with the assumption that impervious areas are directly connected to the drainage system. The impervious area is considered connected if the runoff flows directly into the drainage system, or flows as concentrated shallow flow over a pervious area and then into a drainage system. Runoff from an unconnected impervious area is spread over a pervious area as sheet flow and may result in reduced peak runoff rates and volumes of direct flood runoff. An adjusted CN can be calculated with Equation 3.17 if the total impervious area is less than 30%. TR-55 (SCS, 1986) includes more information on the procedure to modify the CN for impervious area.

$$CN_c = CN_p + \left(\frac{P_i}{100}\right)(98 - CN_p)(1 - 0.5R) \qquad \text{For } P_i \leq 30\% \tag{3.17}$$

Where: CN_c = composite Curve Number
CN_p = pervious Curve Number from Table 3.11
P_i = percent impervious (%)
R = ratio of unconnected impervious area to the total impervious area

Table 3.9 Hydrologic Soil Classifications for Minnesota

Aastad	B	Blue Earth	B/D	Dalbo	B	Flak	C	Hidewood	C	Lohnes	A
Aazdahl	B	Bluffton	C/D	Darfur	B/D	Flaming	A	Hillet	C/D	Lomax	B
Adolph	B/D	Bold	B	Darnen	B	Flandreau	B	Hiwood	A	Loxley	A/D
Adrian	A/D	Boone	A	Dassel	B/D	Flom	B/D	Hixton	B	Lupton	A/D
Afton	C/D	Boots	A/D	Dawson	A/D	Floyd	B	Holdingford	C	Lura	C/D
Ahmeek	C	Borup	B/D	Deerwood	B/D	Foldahl	B	Houghton	A/D	Maddock	A
Alcester	B	Braham	B	Derinda	C	Forada	B/D	Hubbard	A	Madelia	B/D
Allendale	B	Brainerd	C	Dickey	A	Fordville	B	Huntsville	B	Mahtowa	C/D
Almena	C	Bremer	C	Dickinson	B	Forman	B	Ihlen	B	Malachy	B
Alstad	B	Brickton	C	Dickman	A	Formdale	B	Indus	D	Marcus	B/D
Alvin	B	Brill	B	Dinsdale	B	Fossum	A/D	Insula	D	Markey	A/D
Amery	B	Brodale	C	Divide	B	Foxhome	B	Isan	A/D	Marlean	B
Ames	C/D	Brookings	B	Dodgeville	B	Fram	B	Isanti	A/D	Marna	D
Ankeny	B	Brophy	A/D	Doland	B	Freeon	B	Jackson	B	Marquette	A
Anoka	B	Brownton	C/D	Donaldson	B	Freer	C	Joliet	D	Marshan	B/D
Antigo	B	Burkhardt	B	Donnan	C	Frontenac	B	Joy	B	Marysland	B/D
Arcola	C	Burnsville	B	Doran	C	Fulda	C/D	Judson	B	Mavie	B/D
Aredale	B	Buse	B	Dorchester	B	Gale	B	Kamrar	B	Maxcreek	B/D
Arenzville	B	Calamine	C/D	Dorset	B	Galva	B	Kanaranzi	B	Maxfield	B/D
Arland	B	Calco	C/D	Dovray	C/D	Garnes	B	Karlstad	A	Mayer	B/D
Arveson	A/D	Campia	B	Downs	B	Garwin	C/D	Kasota	C	Mazaska	C/D
Arvilla	B	Canisteo	C/D	Dubuque	B	Glencoe	B/D	Kasson	C	McDonaldsville	C/D
Athelwold	B	Carlos	A/D	Duelm	A	Glyndon	B	Kato	C	McIntosh	B
Atkinson	B	Caron	A/D	Duluth	C	Gonvick	B	Kegonsa	B	McPaul	B
Auburndale	C/D	Cashel	C	Dunbarton	D	Gotham	A	Kennebec	B	Medary	C
Augsburg	B/D	Cathro	A/D	Dundas	B/D	Granby	A/D	Kenyon	B	Meehan	A/D
Automba	B	Cannahon	D	Dunnville	B	Grays	B	Kilkenny	B	Menagha	A
Badger	C/D	Chaseburg	B	Dusler	C	Greenwood	A/D	Kingsley	B	Meridian	B
Barbert	D	Chaska	B/D	Eckman	B	Grimstad	B	Kingston	B	Merton	B
Barnes	B	Chelsea	A	Edison	B	Grogan	B	Kittson	C	Merwin	A/D
Baroda	D	Chetek	B	Edwards	B/D	Growton	B	Klinger	B	Mesaba	C
Barrington	B	Chilgren	C	Egeland	B	Grygla	B/D	Kranzburg	B	Metogga	A/D
Barronett	B/D	Clarion	B	Elderon	B	Guckeen	C	Kratka	B/D	Milaca	C
Barrows	B/D	Clontarf	B	Eleva	B	Halder	C	LaPrairie	B	Millerville	A/D
Barto	B/D	Cloquet	B	Ely	B	Hamar	A/D	Lamont	B	Millington	B
Baudette	B	Clyde	B/D	Embden	B	Hamel	C	Lamoure	C	Minneiska	C
Bearden	C	Collinwood	C	Emmert	A	Hamerly	C	Langhei	B	Minneopa	B
Beauford	D	Colo	B/D	Enloe	D	Hangaard	A/D	Langola	B	Minnetonka	D
Becker	B	Colvin	C/D	Enstrom	B	Hanska	C	Lasa	A	Moland	B
Bellechester	A	Comfrey	B/D	Erin	B	Hantho	B	Lawler	B	Moody	B
Beltrami	B	Conic	C	Estelline	B	Harps	B/D	Lawson	B	Moose Lake	A/D
Bena	A	Capaston	D	Estherville	B	Harpster	B/D	LeSueur	B	Mora	C
Benoit	B/D	Cordova	C/D	Etter	B	Hatfield	B/D	Lemond	B/D	Mosomo	A
Beotia	B	Cormant	A/D	Everly	B	Hattie	C	Lerdal	C	Mt. Carroll	B
Bergland	D	Crippin	B	Eyota	A	Haug	B/D	Lester	B	Muscatine	B
Bertrand	B	Crocker	A	Fairhaven	B	Havana	B	Letri	B/D	Muskego	A/D
Beseman	A/D	Corfton	B	Fargo	C	Hayden	B	Lilah	A	Nebish	B
Billett	B	Cromwell	A	Farrar	B	Hayfield	B	Linder	B	Nemadji	B
Biscay	B/D	Curran	C	Faxon	B/D	Hecla	A	Lindstrom	B	Nereson	B
Bixby	B	Cushing	B	Fayette	B	Hegne	C/D	Lino	B	Nessel	B
Blackhoof	C/D	Cutfoot	A	Fedji	A	Hesch	B	Lismore	B	Newfound	C
Blomford	B/D	Cylinder	B	Fieldon	B/D	Heyder	B	Litchfield	A	Newglarus	B
Blooming	B	Dakota	B	Finchford	A	Hibbing	C	Lobo	D	Newry	B

Source: SCS Hydrology Guide for Minnesota

Table 3.9 (continued) Hydrologic Soil Classifications for Minnesota

Newson	A/D	Palms	A/D	Rockton	B	Shields	C	Tama	B	Vlasaty	C
Nicollet	B	Palsgrove	B	Rockwell	B/D	Shooker	C	Taopi	C	Wacousta	B/D
Nokasippi	D	Parent	B/D	Rockwood	C	Shorewood	C	Tara	B	Wadena	B
Nokay	C	Parnell	C/D	Rolfe	C/D	Shullsburg	C	Tawas	A/D	Wahpeton	C
Nordness	B	Pelan	B	Roliss	B/D	Sinai	C	Taylor	C	Waldorf	C/D
Normania	B	Percy	B/D	Rondeau	A/D	Singsaas	B	Tell	B	Warba	B
Northcote	C/D	Perella	B/D	Ronneby	C	Sioux	A	Terril	B	Warman	B/D
Nowen	B/D	Plainfield	A	Rosemount	B	Skyberg	C	Tilfer	B/D	Waskish	A/D
Noyes	C/D	Poinsett	B	Rosendale	B	Sletten	B/D	Timula	B	Watab	C
Nutley	C	Pomroy	B	Roseville	B	Soderville	A	Toddville	B	Watseka	A
Nymore	A	Poppleton	A	Rosholt	B	Sogn	D	Toivola	A	Waubay	B
Oak Lake	B	Port Byron	B	Rothsay	B	Sparta	A	Tonka	C/D	Waubeek	B
Ocheyedan	B	Prebish	C/D	Rushmore	B/D	Spencer	C	Torning	B	Waucoma	B
Ogilvie	B/D	Primghar	B	Ryan	D	Spicer	B/D	Towner	B	Waukee	B
Okoboji	B/D	Protvin	C	Sac	B	Spillville	B	Trent	B	Waukegan	B
Oldham	C/D	Quam	B/D	Salinda	A	Spooner	C/D	Tripoli	B/D	Waukon	B
Omega	A	Quetico	D	Santiago	B	Spottswood	B	Trosky	B/D	Webster	B/D
Onamia	B	Racine	B	Sargeant	D	Storden	B	Truman	B	Whalan	B
Ontonagon	D	Radford	B	Sartell	A	Strandquist	B/D	Twig	A/D	Wheatville	B
Opole	B	Ransom	B	Sattre	B	Stronghurst	B	Udolpho	B/D	Whitewood	C/D
Orion	B	Rasset	B	Sawmill	B/D	Stuntz	C	Ulen	B	Wildwood	C/D
Oronoco	B	Rauville	C/D	Schapville	C	Suamico	A/D	Upsala	C	Wilmonton	B
Osakis	B	Readlyn	B	Schley	B	Svea	B	Urness	B/D	Winger	B/D
Oshawa	C/D	Redby	B	Seaforth	B	Sverdrup	B	Vallers	C	Wyndmere	B
Ossian	B/D	Renova	B	Seaton	B	Swenoda	B	Vasa	B	Zell	B
Ostrander	B	Renshaw	B	Seelyeville	A/D	Syrene	B/D	Ves	B	Zimmerman	A
Otter	B/D	Rib	C	Shakopee	C/D	Talcot	B/D	Vienna	B	Zumbro	A
Otterholt	B	Richwood	B	Shawano	A	Tallula	B	Viking	D	Zwingle	D
Paget	C	Rifle	A/D	Shible	B						

Source: SCS Hydrology Guide for Minnesota

Table 3.10 SCS Curve Numbers for Rural Land Uses

Cover		Condition or Rotation	Acres Per Practice	Curve Numbers for Moisture Condition II				Product
				A Soils	B Soils	C Soils	D Soils	
Fallow	Straight Row			77	86	91	94	
Row Crops	Straight Row	Poor		72	81	88	91	
		Good		67	78	85	89	
		Mulch till		61	76	84	87	
	Contoured[2]	Poor		70	79	84	88	
		Good		65	75	82	86	
		Mulch till		62	73	80	85	
	C and T[1]	Poor		66	74	80	82	
		Good		62	71	78	81	
		Mulch till		61	70	77	80	
Small Grain	Straight Row	Poor		65	76	84	88	
		Good		63	75	83	87	
		Mulch till		58	74	82	86	
	Contoured[2]	Poor		63	74	82	85	
		Good		61	73	81	84	
		Mulch till		59	72	80	83	
	C and T[1]	Poor		61	72	79	82	
		Good		59	70	78	81	
		Mulch till		58	69	77	80	
Legumes or Rotation Meadow	Straight Row	Poor		66	77	85	89	
		Good		58	72	81	85	
	Contoured[2]	Poor		64	75	83	85	
		Good		55	69	78	83	
	C and T[1]	Poor		63	73	80	83	
		Good		51	67	76	80	
Pasture		Poor		68	79	86	89	
		Fair		49	69	79	84	
		Good		39	61	74	80	
Meadow (Permanent)		Good		30	58	71	78	
Wood or Forest Land		Poor		45	66	77	83	
		Fair		36	60	73	79	
		Good		25	55	70	77	
Farmsteads		- - -		59	74	82	86	
Roads (including R/W)	Dirt Surface	- - -		72	82	87	89	
	Hard Surface	- - -		74	84	90	92	
Impervious Surface		- - -		100	100	100	100	
Water Surface (lakes, ponds)		- - -		100	100	100	100	
Swamp	Open water[3]	- - -		85	85	85	85	
	Vegetated[4]	- - -		78	78	78	78	
Residential	Low Density	- - -		47	65	76	82	
	Medium Density	- - -		54	70	79	84	
	High Density	- - -		70	81	87	90	
Commercial and Industrial		- - -		86	91	93	94	

Total Acres = ____________ Product Total = ____________

$$\text{Weighted Runoff Curve Number} = \frac{\text{Product Total}}{\text{Total Acres}} = \frac{\_\_\_\_\_\_\_\_}{} = \_\_\_\_\_\_\_\_$$

Source: SCS Hydrology Guide for Minnesota Figure 3-1

[1] Contoured and graded terraces or land with less tan 2% slope

[2] Includes level terraced areas (runoff corrected by volume)

[3] 1/3 of swam surface is open water

[4] Swamp has no open water and the design is a 25-year frequency or less.

Table 3.11 SCS Curve Numbers for Urban Land Uses

Cover	Condition or Rotation	Acres Per Practice	Curve Numbers for Moisture Condition II				Product
			A Soils	B Soils	C Soils	D Soils	
Cultivated Land	Without conservation treatment		72	81	88	91	
	With conservation treatment		62	71	78	81	
Pasture or Range Land	Poor		68	79	86	89	
	Good		39	61	74	80	
Meadow	Good		30	58	71	78	
Wood or Forest Land	Poor: thin stand, no mulch		45	66	77	83	
	Good		25	55	70	77	
Open Spaces (lawns, parks, golf courses, cemeteries)	Good: 75% or more grass cover		39	61	74	80	
	Fair: 50% to 75% grass cover		49	69	79	84	
Commercial and Business Areas (85% impervious)			89	92	94	95	
Industrial Districts (72% impervious)			81	88	91	93	
Residential[1]	≤ 1/8 acre lot 65% impervious		77	85	90	92	
	1/4 acre lot 38% impervious		61	75	83	87	
	1/3 acre lot 30% impervious		57	72	81	86	
	½ acre lot 25% impervious		54	70	80	85	
	1 acre lot 20% impervious		51	68	79	84	
Paved Parking Lots, Roofs, Driveways			98	98	98	98	
Streets and Roads	paved with curb and storm drain		98	98	98	98	
	gravel		76	85	89	91	
	dirt		72	82	87	89	
Marsh			85	85	85	85	
Other							

Total Acres = ________ Product Total = ________

$$\text{Weighted Runoff Curve Number} = \frac{\text{Product Total}}{\text{Total Acres}} = \underline{\qquad\qquad} = \underline{\qquad\qquad}$$

Source: SCS Hydrology Guide for Minnesota Figure 3-2

[1] For land uses with impervious areas, curve numbers are computed assuming that 100% of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98

[2] Includes paved streets

Table 3.12 Rainfall Groups For Antecedent Runoff Conditions

Antecedent Conditions	Conditions Descriptions	Five-day Antecedent Rainfall	
		Growing Season	Dormant Season
Dry I	An optimum condition of watershed soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place.	< 1.4 inches	< 0.5 inches
Average II	The average case for annual floods	1.4 to 2.1 inches	0.5 to 1.1 inches
Wet III	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm.	> 2.1 inches	> 1.1 inches

Source: Soil Conservation Service

Table 3.13 ARC Conversion

CN for ARC Average II	Corresponding CN's	
	Dry I	Wet III
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: Soil Conservation Service

Antecedent Runoff Condition (ARC) is an index of runoff potential before a storm event, and attempts to account for variation in CN at a site from storm to storm. (ARC may also be called the Antecedent Moisture Condition, AMC). Table 3.12 describes the three conditions

The average antecedent runoff condition is generally recommended for design purposes. The wet and dry conditions are used for simulating historic conditions. The wet antecedent runoff condition may also be used for a conservative design. Table 3.13 shows the converted curve numbers for other antecedent moisture conditions.

3.6.6 Peak Discharge Procedure

The NCRS developed a graphical procedure to estimate the peak discharge from a small watershed. Rainfall, curve number, time of concentration, and drainage area, are required. Rainfall is estimated from the type of storm distribution, and 24-hour rainfall for the watershed. The watershed drainage area must be greater than 1.0 acre and less than 2,000 acres. If the drainage area is outside these limits, use TR-20 procedures. The peak discharge equation is:

$$q_p = q_u A Q F_p \tag{3.18}$$

Where: q_p = peak discharge (cfs)
q_u = unit peak discharge (cms/in)
A = drainage area (mi^2)
Q = direct runoff (in)
F_p = pond and swamp adjustment factor
Only use adjustment factor for ponds or swamps that are not in the t_c flow path.
(See Table 3.14)

The peak discharge can be calculated using the following procedure:

Step 1 Determine drainage area (A). The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas, it might be necessary to divide the area into subareas to account for major land use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on flood flow. When subareas are used, a hydrograph procedure is required to determine the combined discharge. Also a field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub-drainage areas.

Table 3.14 Adjustment Factor (F_p) for pond and swamp areas

Percent of pond and swamp area (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Source: SCS, 1986

Step 2 Determine curve number (CN).

A. Determine soil group from Table 3.9.
B. Use Table 3.10 or 3.11 to find CN
C. If area is urbanized, determine if default assumptions are valid. If the % impervious is not the same as the default values listed in Table 3.11, or impervious areas are not directly connected to the drainage system then adjust the CN using Equations 3.16 and 3.17.
D. Calculate Weighted CN, Equation 3.15.
E. Consider Antecedent Moisture Condition. If Average (II) is not valid, use Table 3.13 to adjust the CN value.

Step 3 Determine the design frequency according to guidance in Section 3.2.

Step 4 Determine Time of Concentration t_c. Time of concentration (t_c) influences the shape and the peak of the runoff hydrograph. The SCS defines t_c as the time required for water to travel from the most hydraulically distant point in a watershed to the outlet. SCS methods generally require t_c in hours. Procedures to calculate Time of Concentration are in Section 3.4.

Step 5 Determine P, the 24 hour rainfall (inches), for the design frequency from Appendix B.

Step 6 Find the initial abstraction (I_a) from Table 3.15 using the watershed CN.

Step 7 Calculate I_a/P, this ratio is a parameter that indicates how much of the total rainfall is needed to satisfy the initial abstraction. The larger the I_a/P ratio, the lower the unit peak discharge (q_u) for a given t_c. If the computed I_a/P ratio is outside the range of 0.1 to 0.50, then the limiting values should be used; i.e., use 0.1 if less than 0.1 and use 0.5 if greater than 0.5.

Step 8 Determine q_u, unit peak discharge, from Figure 3.8 or 3.9 depending on the rainfall distribution type. If the ratio falls between the limiting values, use linear interpolation

Step 9 Find Q, runoff (inches), using Equations 3.13b and 3.14.

Step 10 Calculate q_p, peak discharge (cfs), from Equation 3.18. Apply F_p (Table 3.14) if applicable.

Table 3.15 I_a Values for Runoff Curve Numbers

Curve Number	I_a	Curve Number	I_a	Curve Number	I_a	Curve Number	I_a
40	3.000	54	1.704	68	0.941	82	0.439
41	2.878	55	1.636	69	0.899	83	0.410
42	2.762	56	1.571	70	0.857	84	0.381
43	2.651	57	1.509	71	0.817	85	0.353
44	2.545	58	1.448	72	0.778	86	0.326
45	2.444	59	1.390	73	0.740	87	0.299
46	2.348	60	1.333	74	0.703	88	0.273
47	2.255	61	1.279	75	0.667	89	0.247
48	2.167	62	1.226	76	0.632	90	0.222
49	2.082	63	1.175	77	0.597	91	0.198
50	2.000	64	1.125	78	0.564	92	0.174
51	1.922	65	1.077	79	0.532	93	0.151
52	1.846	66	1.030	80	0.500	94	0.128
53	1.774	67	0.985	81	0.469	95	0.105

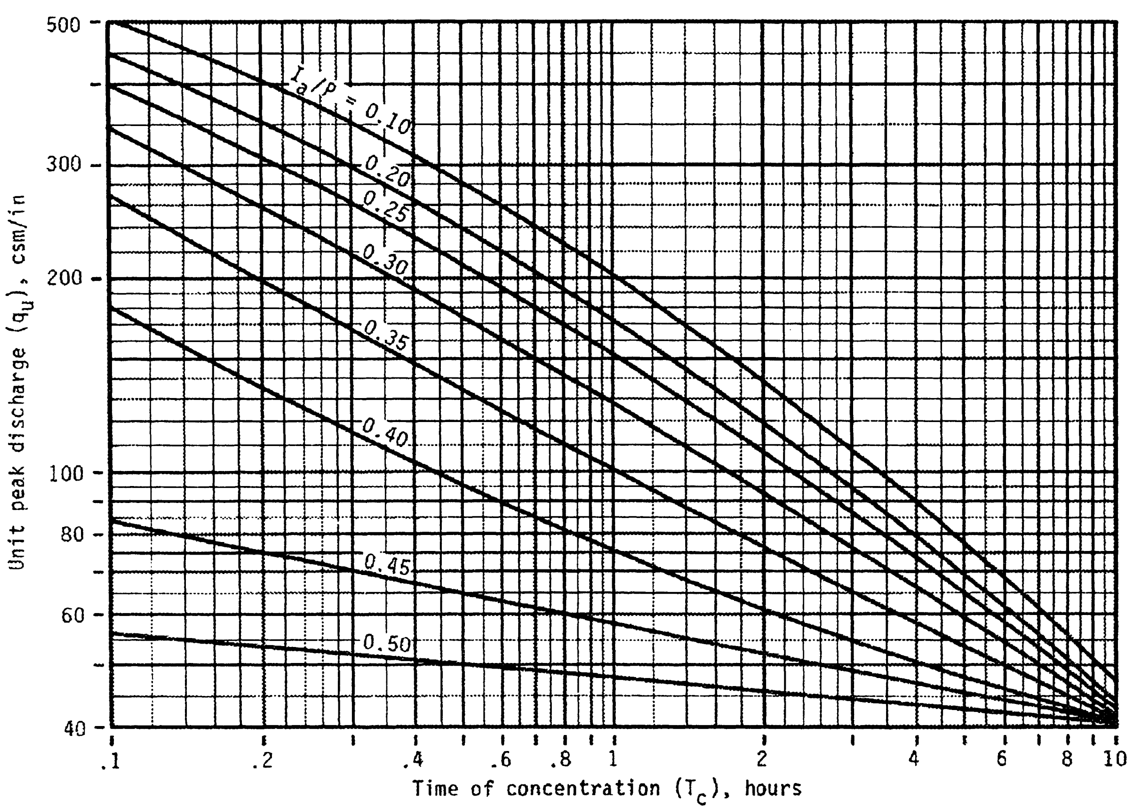


Figure 3.8 Unit Peak Discharge (q_u) for SCS Type I Rainfall Distribution
Source: Engineering Field Manual, Chapter 2 (SCS, 1989)

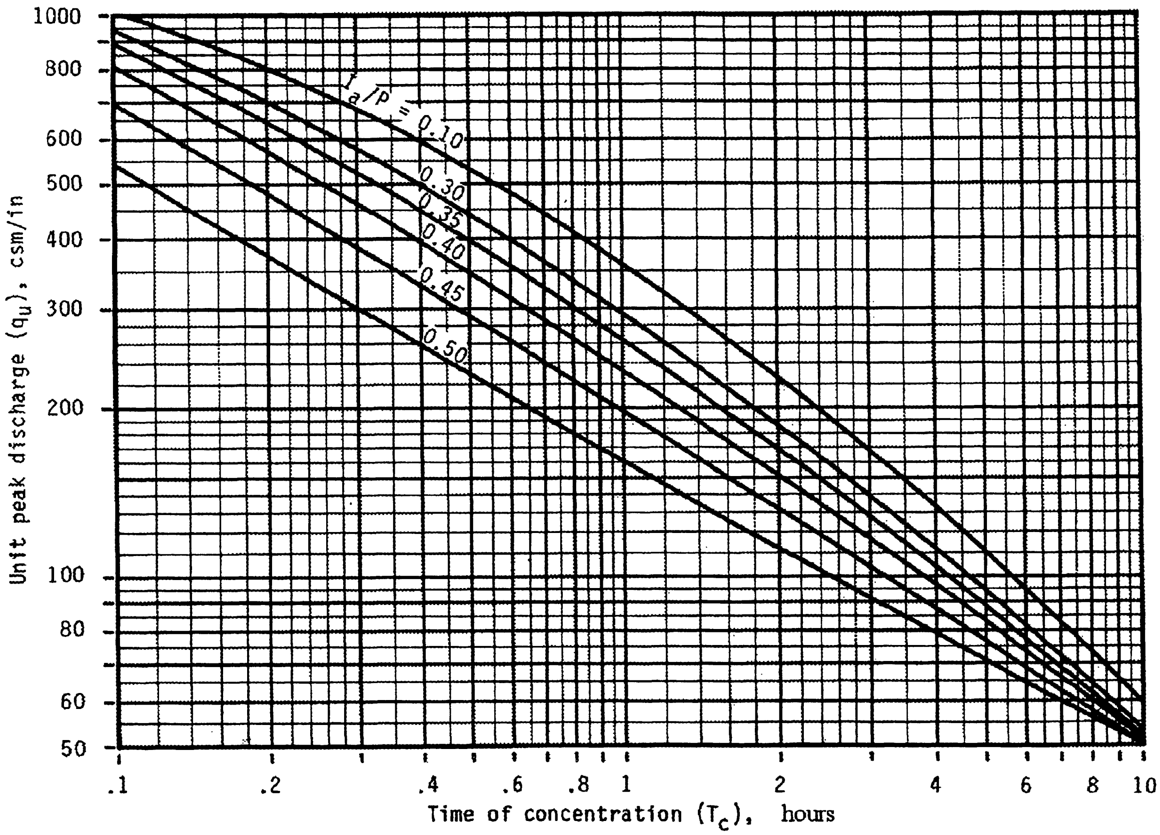


Figure 3.9 Unit Peak Discharge (q_u) for SCS Type II Rainfall Distribution
Source: Engineering Field Manual, Chapter 2 (SCS, 1989)

3.6.7 SCS Hydrograph Procedure

The SCS hydrograph is a synthetic unit hydrograph developed by analysis of many natural unit hydrographs. A unit hydrograph represents the time distribution of flow resulting from one inch of direct runoff occurring over the watershed in a specified time. The hydrograph shape is dependant on the peak discharge for a given rainfall and the basin lag. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. The hydrograph can be used to model drainage basins of various sizes and shapes. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

Hydrograph development and routing is generally done with a computer. TR-20 (SCS, 1982) is a computer program developed by NCRS that develops hydrographs, routes the hydrographs through storage, and combines hydrographs. TR-20, or another computer program that utilizes TR-20 or TR-55 methodology, is generally recommended when a hydrograph is needed for design. NCRS used results from running TR-20 to develop a simplified graphical procedure to obtain the peak discharge for small watersheds. This procedure is outlined in *Hydrology Guide for Minnesota* (SCS), *SCS Engineering Field Manual, Chapter 2* (SCS, 1989), and *SCS Technical Release No. 55* (SCS, 1986).

A brief explanation of some aspects of the SCS procedure and the TR-20 computer model are given below. Further information can be found in the TR-20 manual and NEH-4. Consult the User's Manual to use TR-20 or another computer program to develop a hydrograph using the SCS procedures. The SCS procedure to estimate the peak rate of discharge is:

$$q_p = \frac{484AQ}{T_p} \tag{3.19}$$

Where: q_p = peak rate of discharge (cfs)
A = area (mi^2)
Q = storm runoff (inches)
T_p = time to peak (hrs)

$$T_p = \frac{D}{2} + L \tag{3.20}$$

Where: D = storm duration, hrs
L = watershed lag, hrs (on average L = 0.6 t_c)

Once q_p and T_p have been determined, a synthetic unit hydrograph can be constructed. The constant 484, or peak rate factor, is valid for the SCS dimensionless unit hydrograph and is the basis for the graphical procedures and the default hydrograph in TR-20. This constant affects the shape of the hydrograph and has been known to vary from about 600 in steep terrain to 300 in very flat swampy area. NEH-4 contains further information on the hydrograph procedures.

The TR-20 computer program is a single event model which computes direct runoff resulting from a synthetic or natural rainstorm. It develops hydrographs from runoff and routes the flow through stream channels and reservoirs. It combines the routed hydrograph with those from tributaries, and computes the peak discharges, their times of occurrence and the water surface elevations at any desired cross-section or structure. The watershed should be divided into as many subareas as necessary to define hydrologic and structural effects. Tributaries, watershed shape, valley slope, runoff curve number homogeneity, and water storage areas can all determine where subareas are required. Each sub-watershed is assumed to be hydrologically homogeneous. For each sub-watershed, the user will be required to enter the drainage area in square miles, the runoff curve number, and the time of concentration in hours. For best results in using the TR-20 computer program, the time of concentration for each subarea should be of the same magnitude, with the longest t_c less than 3 times the shortest t_c. The largest sub-watershed area should be less than 5 times the smallest sub-watershed area. The output from the program can be affected by the main time increment which is user supplied. In general, use the time increment equal to 0.1 to 0.2 times the shortest t_c, but not less than 0.1 hour. The user may need to try several different time increments.

3.7 USGS REGRESSION EQUATIONS

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Regression studies are statistical practices used to develop runoff equations. These equations are used to relate peak flow at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics The U.S. Geological Survey (USGS) developed the regression equations for Minnesota and published them in *Techniques for Estimating Magnitude and Frequency of Floods in Minnesota* (Guetzkow, 1977). Since then regression analysis was redone and new equations developed and published twice. Once with the same title *Techniques for Estimating the Magnitude and Frequency of Floods in Minnesota* (Jacques and Lorenz, 1988) developed in 1987 and published in 1988, and also as *Techniques for Estimating Peak Flow on Small Streams in Minnesota* (Lorenz, Carlson and Sanocki, 1997).

Regression analyses use stream gage data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar characteristics. Separate regression analysis were performed and equations developed at 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals for each hydrologically similar region. The most significant characteristic of the equations is the drainage area size above the point of interest. Additional basin characteristics such as main channel slope, lake storage, total storage and mean annual runoff are included as independent variables when they were determined to be statistically significant.

3.7.1 Application

The USGS Regression Equations may be used to design culverts and bridges in Minnesota. The USGS Regression Equations, obtained from multiple-regression analyses of gaging-station data in each hydrologic region, can be used to obtain flood-frequency estimates for ungaged sites on unregulated streams. Peak discharges for selected recurrence intervals can be computed from the empirical equations that relate flood magnitude to basin characteristics. Historical observations and other methods, should be used to evaluate the results. Where there is stream gage data, the findings from a Log Pearson III method should govern, provided there is at least 10 years of stream gage record. Reasonable and prudent judgment along with consideration of the standard regression error shall be used in reaching a design decision. Typical regression equation format:

$$Q_i = aA^b S^c \left(St + 1\right)^d \tag{3.21}$$

Where: Q_i = flood frequency estimate for recurrence interval i
A = drainage area in square miles
S = main channel slope in feet per mile
St = percent of drainage area occupied by lakes, ponds or wetlands
a, b, c, d = variables

The regression equations, hydrologic region boundaries, variable definitions, procedures for use and discussion on accuracy and limitations are provided in the USGS reports listed in Section 3.7 and will not be duplicated in this manual.

3.7.2 Limitations

The basin characteristics of the stream being analyzed should be within the limits of those used to develop the equations. Each USGS report has a table showing the upper and lower limits for basins characteristics used in the regression analysis; care should be exercised using equation outside of the specified limits. The regression equations are intended to be used on streams which are not significantly affected by manmade regulation, urbanization, or diversion. The equations should not be used immediately downstream of a lake or ponding area, but may be used to determine the peak inflow to be routed through an impoundment.

3.7.3 Procedure

There are three sets of equations that can be applied to a particular site. These include the 1997 regression equations developed by Lorenz, Carlson and Sanocki; 1987 regression equations developed by Jacques and Lorenz; and the 1977 regression equations developed by Guetzkow. Although the later equations were intended to supersede earlier equations, it is advisable to compute the discharge using multiple methods, and compare to other historical data or methods. The 1987 procedure divides the state into 4 hydrologic regions, where the 1977 procedure used 8 different hydrologic regions in the state and the 1997 procedures use 6 regions. Although the later reports used gaging stations with longer periods of records and more stations, it is valid to use the earlier 1977 and 1987 equations as a check.

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. Because of the distance between stream gages, the regional boundaries cannot be considered as precise. The watershed of interest may lie partly within two or more hydrologic regions or it may lie totally within a hydrologic region, but close to a hydrologic region boundary. In these instances care must be exercised in using regression equations. A field visit is recommended to first collect all available historical flood data as well as to compare the project's watershed characteristics with those of the abutting hydrologic regions. For drainage areas near regional divides, the equations for both regions should be evaluated and the results compared. The coefficients in the equations can also be used to transfer flood flows from a gaged site to an ungaged site on the same stream. The regional boundaries generally follow watershed basin divides.

3.8 ANALYSIS OF STREAM GAGE DATA

Many gaging stations exist throughout Minnesota where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length of time, a frequency analysis may be made. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways.

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.
- If the facility site is nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.
- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations are usually furnished by established hydrologic agencies and the designer will not be involved in their development.

3.8.1 Application

Mn/DOT may use a frequency analysis of stream gage data at appropriate locations when there is sufficient years of measured stream gage record. The preferred method of estimating flood frequency curves from stream gage data is a statistical method, which makes use of the Log Pearson Type III frequency distribution.

The analysis of gaged data permits an estimate of the peak discharge for the desired return period at a particular site. Experience has shown that statistical frequency distributions may be more representative of naturally occurring floods and can be reliable when used for prediction. Although several different distributions are used for frequency analysis, experience has shown the Log-Person Type III distribution to be one of the most useful. The Log-Pearson III distribution and the process of fitting it to a particular data sample are described in detail in Water Resources Council Bulletin 17B, *Guidelines for Determining Flood Flow Frequency*, (WRC, 1981). Special handling of outliers, historical data, incomplete data, and zero flow years is covered in detail in Bulletin 17B.

There are two alternative methods for determining the value of the skew coefficient to be used in calculating the Log-Pearson Type III curve fit. The value of skew that is calculated directly from the gage data using the above formula is called the station skew. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record. Often, the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data.

3.8.2 Transferring Gaged Data

Gaged data may be transferred to an ungaged site on the same river as the gaged site provided such data are nearby, within the same hydrologic region, and there are no major tributaries or diversions between the gage and the site of interest. These procedures make use of the constants obtained in developing the regression equations. To transfer discharge data from a gaged site to an ungaged site on the same stream:

$$Q_u = Q_g \left(\frac{A_u}{A_g} \right)^B \qquad \textbf{(3.22)}$$

Where: Q_u = flood frequency estimate to ungaged site
Q_g = flood frequency estimate for gaged site
A_u = drainage area for ungaged site
A_g = drainage area for gaged site
B = exponent for drainage area from the appropriate regression equation

This transfer relation can be used where drainage area size differs by no more than 50%. If other basin characteristics differ significantly, they should also be included by taking the ratio of the parameters raised to the power given in the regression equation. If the period of record at the gaged site is short, a weighted average of the results of the transfer equation and regression equation should be used.

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the drainage area exponent given in the appropriate regression equation. Thus on streams where no gaging station is in existence, records of gaging stations in nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. This procedure is repeated for each available nearby watershed and the results are averaged to obtain a value for the desired flood frequency relationships in the ungaged watershed.

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Chapter 4 CHANNELS

4.1 INTRODUCTION

Hydraulic design associated with natural channels and roadway ditches is a process which selects and evaluates alternatives according to established criteria. These criteria are the standards established by Mn/DOT to insure that a highway facility meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare. The purpose of this chapter is to establish Mn/DOT policy, specify design criteria, review design philosophy, and outline channel design procedures.

Channel analysis is necessary for the design of transportation drainage systems in order to assess:
- potential flooding caused by changes in water surface profiles,
- disturbance of the river system upstream or downstream of the highway right-of-way,
- changes in lateral flow distributions,
- changes in velocity or direction of flow,
- need for conveyance and disposal of excess runoff, and
- need for channel lining to prevent erosion.

4.1.1 Definition

Open channels are a natural or man-made conveyance for water in which:
- the water surface is exposed to the atmosphere, and
- the gravity force component in the direction of motion is the driving force.

There are various types of open channels encountered by the designer of transportation facilities: stream channel, roadside channel or ditch, irrigation channel, and drainage ditch. The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts and storm drains. While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) will be treated separately in this chapter as needed.

Stream channels are:
- usually natural channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped geomorphologically by the long term history of sediment load and water discharge.

Artificial channels include roadside channels, irrigation channels and drainage ditches which are:
- constructed channels with regular geometric cross sections, and
- unlined, or lined with artificial or natural material to protect against erosion.

4.1.2 Concept Definitions

Critical Flow — Critical flow occurs when the specific energy is a minimum. Plotting depth against specific energy at a constant discharge results in a curve where the minimum specific energy occurs at a depth called critical depth, where the Froude number has a value of one.

The general expression for flow at critical depth is:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{4.1}$$

Where:
- Q = discharge (cfs)
- g = acceleration due to gravity (32.2 ft/s^2)
- A = flow area (ft^2)
- T = channel top width at the water surface (ft)

Critical Flow (continued)

Critical depth is also the depth of maximum discharge when the specific energy is held constant. These relationships are illustrated in Figure 4.1. During critical flow the velocity head is equal to half the hydraulic depth. When flow is at critical depth, Equation 4.1 must be satisfied, no matter what the shape of the channel.

Froude Number

The Froude number, F_r, is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces. For rectangular channels the hydraulic depth is equal to the flow depth. This expression for Froude number applies to any single section channel and is defined by:

$$F_r = \frac{V}{\left(\frac{gd}{\alpha}\right)^{0.5}} \qquad (4.2)$$

Where:
- V = mean velocity (ft/s); V = Q/A,
- g = acceleration due to gravity (32.2 ft/s^2)
- d = hydraulic depth (ft); d = A/T
- A = cross-sectional area of flow(ft^2)
- Q = total discharge (cfs)
- T = channel top width at the water surface (ft)
- α = velocity distribution coefficient

Gradually Varied Flow

A nonuniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, is referred to as a gradually-varied flow; otherwise, it is considered to be rapidly-varied flow.

Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures.

Specific Energy

Specific energy, E, is defined as the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head:

$$E = y + \alpha\frac{V^2}{2g} \qquad (4.3)$$

Where:
- y = depth (ft)
- α = kinetic energy correction coefficient
  The velocity distribution coefficient is taken to have a value of one for turbulent flow in prismatic channels, but may be significantly different that one in natural channels.
- V = mean velocity (ft/s)
- g = acceleration due to gravity (32.2 ft/s^2)

Steady and Unsteady Flow

A steady flow is one in which the discharge passing a given cross-section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

Subcritical Flow

Depths greater than critical depth occur in subcritical flow where the Froude number is less than one. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

Supercritical Flow — Depths less than critical depth occur in supercritical flow where the Froude number is greater than one. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

Thalweg Profile — The line extending along a channel profile that follows the lowest elevation of the channel bed.

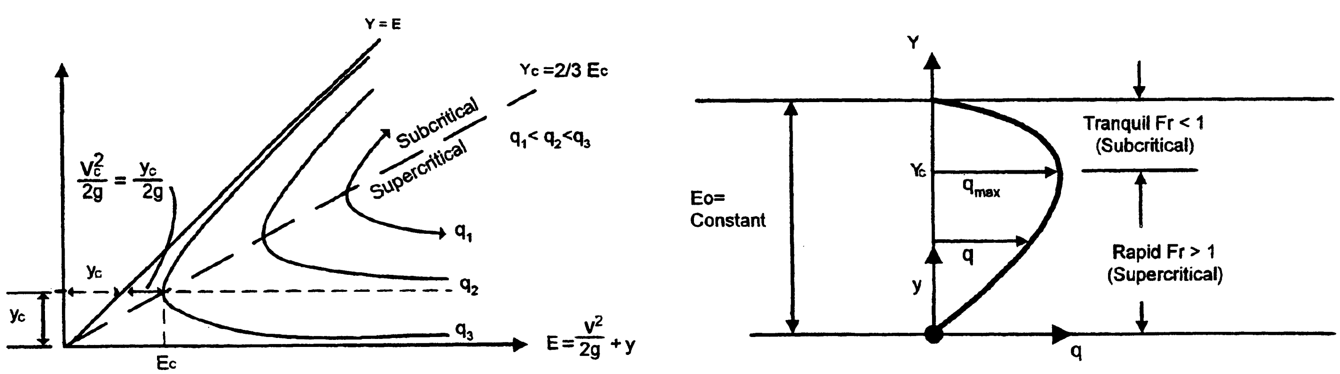
Total Energy Head — The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 4.1 for a plot of the specific energy diagram.

Uniform and Nonuniform Flow — A nonuniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction; however, nonuniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Velocity Distribution Coefficient — Due to the presence of a free surface and also due to friction along the channel boundary, the velocities in a channel are not uniformly distributed in the channel section. As a result of nonuniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, α, defined as:

$$\alpha = \frac{\sum_{i=1}^{n} (K_i^3/A_i^2)}{(K_t^3/A_t^2)} \tag{4.4}$$

Where: K_i = conveyance in subsection (see Equation 4.8)
K_t = total conveyance in section (see Equation 4.8)
A_i = cross-sectional area of subsection (ft^2)
A_t = total cross-sectional area of section (ft^2)
n = number of subsections



(a) Specific Energy Diagram **(b) Discharge Diagram**

Figure 4.1 Specific Energy And Discharge Diagram For Rectangular Channels
Source: adopted From Highways In The River Environment (FHWA, 1990)

4.2 DESIGN CRITERIA

Design criteria establishes the standards by which a policy is placed into action. They form the basis for the selection of the final design configuration. Listed below are the design criteria which shall be considered for channel design.

4.2.1 Policy

Policy is a set of goals that establish a definite course or method of action and are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. The following policies are specific to channels:

- Channel designs and/or designs of highway facilities that impact channels shall satisfy the policies of the Federal Highway Administration applicable to floodplain management if Federal funding is involved.
- Federal Emergency Management Agency floodway regulations and U.S. Army Corps of Engineers wetland restrictions for permits shall be satisfied.
- Coordination with other Federal, State, and local agencies concerned with water resources planning shall have high priority in the planning of highway facilities.
- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of artificial drainage channels.
- The design of artificial drainage channels or other facilities shall consider the frequency and type of maintenance expected and make allowance for access of maintenance equipment.
- A stable channel is the goal for all channels that are located on highway right-of-way or that impact highway facilities.
- Environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and channel stability shall be assessed.
- The range of design channel discharges shall be selected and approved by the designer based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions.

4.2.2 Stream Channels

The following criteria applies to natural channels:

- The hydraulic effects of flood plain encroachments shall normally be evaluated over a full range of frequency-based peak discharges from the mean annual or bank full flood through the 500-year flood on any major highway facility as deemed necessary by the designer.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander pattern, roughness, sediment transport, and slope shall conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization shall be provided, when appropriate, to prevent bank erosion resulting from stream disturbances caused by highway projects and shall include both upstream and downstream banks as well as the local site.
- Provision should be incorporated into the design and construction for access by maintenance personnel and equipment to maintain features such as dikes and levees.

4.2.3 Roadside Channels

The following criteria applies to roadside channels:

- The design discharge for permanent roadside ditch linings shall have a 10-year frequency while temporary linings shall be designed for the 2-year frequency flow. Engineering judgement shall be utilized in selecting a design frequency for evaluating flood damage potential; however, a 100 year frequency is generally recommended.
- Channel side slopes shall not exceed the angle of repose of the soil and/or lining and shall be 1V:3H or flatter.
- Channel side slopes should meet requirements for clear zones as specified in the Mn/DOT Road Design Manual.
- Ditch depths are typically 4′ to 6′ in order to provide adequate drainage for the base of the road.
- Flexible linings shall be designed according to the method of allowable tractive force.
- Channel freeboard shall be the larger of one foot or two velocity heads.

4.3 OPEN CHANNEL FLOW

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method is a simple application of Manning's equation to determine tailwater rating curves for culverts, or to analyze other situations in which uniform or nearly uniform flow conditions exist. Manning's equation can be used to estimate highwater elevations for bridges that do not constrict the flow. The step-backwater method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water surface elevations for bridge hydraulic design, or to analyze other gradually-varied flow problems in streams.

The single-section method will generally yield less reliable results because it requires more judgment and assumptions than the step-backwater method. In many situations, however, the single-section method is all that is justified for instance when analyzing a standard roadway ditch, culvert, or storm drain outfall.

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1970; Henderson, 1966). The basic principles of fluid mechanics (continuity, momentum, and energy) can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels.

4.3.1 Flow Classification

The classification of open channel flow can be summarized as follows:

<u>Steady Flow</u>

- Uniform Flow
- Nonuniform Flow
  - Gradually Varied Flow
  - Rapidly Varied Flow

<u>Unsteady Flow</u>

- Unsteady Uniform Flow (rare)
- Unsteady Nonuniform Flow
  - Gradually Varied Unsteady Flow
  - Rapidly Varied Unsteady Flow

The steady uniform flow case and the steady nonuniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

4.3.2 Equations

The following equations are those most commonly used to analyze open channel flow.

<u>Continuity Equation</u>

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of one dimensional, steady flow of an incompressible fluid, it assumes the simple form where the subscripts 1 and 2 refer to successive cross sections along the flow path:

$$Q = A_1 V_1 = A_2 V_2 \tag{4.5}$$

Where: Q = discharge (cfs)
A = cross-sectional area of flow (ft^2)
V = mean cross-sectional velocity (ft/s); Where V is perpendicular to cross-section

Manning's Equation

For a given depth of flow in an open channel with steady, uniform flow, the mean velocity, V, can be computed with Manning's equation:

$$V = 1.49\frac{R^{2/3}S^{1/2}}{n} \tag{4.6}$$

Where: V = mean velocity (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius (ft); R = area/wetted perimeter
A = cross-sectional area of flow (ft^2)
P = wetted perimeter (ft)
S = slope of the energy gradeline (ft/ft);
For steady uniform flow, S ≈ channel slope (ft/ft)

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The range of n values for various types of channels and floodplains is given in Table 4.1. The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = VA = 1.49\frac{AR^{2/3}S^{1/2}}{n} \tag{4.7}$$

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from equation 4.7 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial and error solution. If the normal depth is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

Conveyance

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance, K. The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, α (see Equation 4.4).

$$K = 1.49\frac{AR^{2/3}}{n} \tag{4.8}$$

and then Equation 4.7 can be written as:

$$Q = KS^{1/2} \tag{4.9}$$

Energy Equation

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities which give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$h_1 + \alpha_1 \frac{V_1^2}{2g} = h_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \tag{4.10}$$

Where: h_1 = upstream stage(ft)
h_2 = downstream stage (ft)
α = velocity distribution coefficient
V = mean velocity (ft/s)
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance (ft)

The stage, h is the sum of the elevation head, z at the channel bottom and the pressure head, or depth of flow, y (h=z+y). The terms in the energy equation are illustrated graphically in Figure 4.2. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

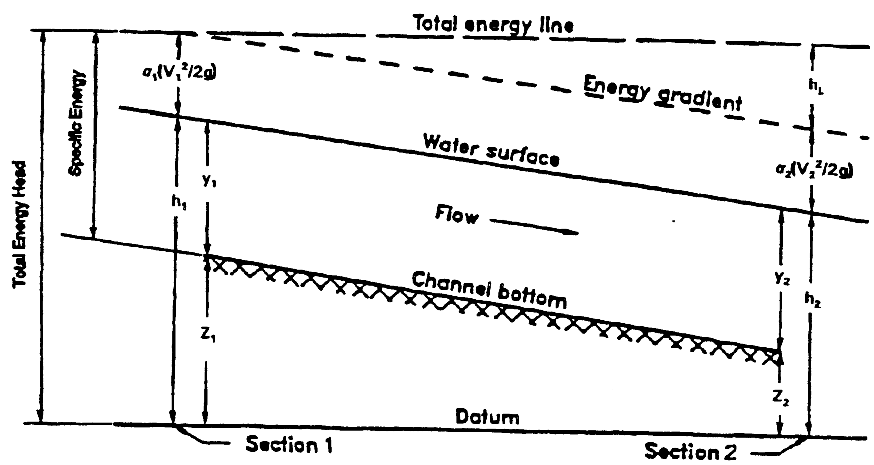


Figure 4.2 Terms In The Energy Equation
Source: FHWA, 1990

Shear Stress
The shear stress is the hydrodynamic force of water flowing in a channel. Stable channel design is based on the rule that the flow induced shear stress should not exceed the permissible shear stress for the lining material. Shear stress is calculated:

$$\tau = \gamma R S \tag{4.11}$$

If the width is very large in relation to the depth substitute d = R, to compute the maximum shear stress:

$$\tau_d = \gamma d S \tag{4.12}$$

Where: τ = shear stress (lb/ft^2)
τ_d = maximum shear stress at normal depth (lb/ft^2)
τ_p = permissible shear stress (lb/ft^2)
γ = unit weight of water (62.4 lb/ft^3)
R = hydraulic radius (feet)
d = maximum depth of flow (feet)
S = average bed slope or energy slope (ft/ft)

Determine the permissible shear stress, τ_p in lbs/ft^2 for the lining material. If $\tau_d < \tau_p$ then lining is acceptable.

Table 4.1 Values of Roughness Coefficient for Uniform Flow

MANNINGS 'N' FOR UNIFORM FLOW				
Channel Type	Channel Description	Minimum	Normal	Maximum
Excavated or Dredged	Earth, straight and uniform	0.016	0.018	0.020
	1. Clean, recently completed	0.018	0.022	0.025
	2. Clean, after weathering	0.022	0.025	0.030
	3. Gravel, uniform section, clean	0.022	0.027	0.033
	Earth, winding and sluggish			
	1. No vegetation	0.023	0.025	0.030
	2. Grass, some weeds	0.025	0.030	0.033
	3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
	4. Earth bottom and rubble sides	0.025	0.030	0.035
	5. Stony bottom and weedy sides	0.025	0.035	0.045
	6. Cobble bottom and clean sides	0.030	0.040	0.050
	Dragline-excavated or dredged			
	1. No vegetation	0.025	0.028	0.033
	2. Light brush on banks	0.035	0.050	0.060
	Rock cuts			
	1. Smooth and uniform	0.025	0.035	0.040
	2. Jagged and irregular	0.035	0.040	0.050
	Channels not maintained, weeds and brush uncut			
	1. Dense weeds, high as flow depth	0.050	0.080	0.120
	2. Clean bottom, brush on sides	0.040	0.050	0.080
	3. Same, highest stage of flow	0.045	0.070	0.110
	4. Dense brush, high stage	0.080	0.100	0.140
Natural Streams	Streams on Plain			
	1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
Minor streams (top width	2. Same as 1, but more stones and weeds	0.030	0.035	0.040
at flood stage < 100 ft)	3. Clean, winding, some pools and shoals	0.033	0.040	0.045
	4. Same as 3, but some weeds and some stones	0.035	0.045	0.050
	5. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
	6. Same as 4, but more stones			
	7. Sluggish reaches, weedy, deep pools	0.045	0.050	0.060
	8. Very weedy reaches, deep pools, or floodways with heavy	0.050	0.070	0.080
	stand of timber and underbrush	0.075	0.100	0.150
	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
	1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
	2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Natural Streams	Pasture, no brush			
	1. Short grass	0.025	0.030	0.035
Flood Plains	2. High grass	0.030	0.035	0.050
	Cultivated area			
	1. No crop	0.020	0.030	0.040
	2. Mature row crops	0.025	0.035	0.045
	3. Mature field crops	0.030	0.040	0.050
	Brush			
	1. Scattered brush, heavy weeds	0.035	0.050	0.070
	2. Light brush and trees in winter	0.035	0.050	0.060
	3. Light brush and trees, in summer	0.040	0.060	0.080
	4. Medium to dense brush, in winter	0.045	0.070	0.110
	5. Medium to dense brush, in summer	0.070	0.100	0.116
	Trees			
	1. Dense Willows, summer, straight	0.110	0.150	0.200
	2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
	3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
	4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
	5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Natural Major Streams (top width at flood stage > 100 ft). [1]	Regular section with no boulders or brush	0.025	----	0.060
	Irregular and rough section	0.035	----	0.100

[1] The n value is less than that for minor streams of similar description, because banks offer less effective resistance. Source: Chow, 1970.

4.3.3 Cross Sections

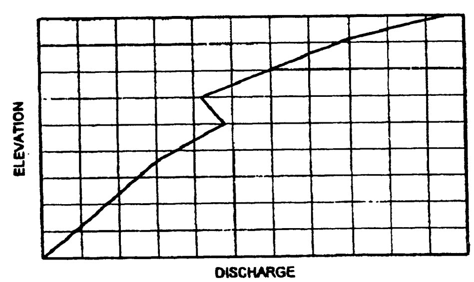
Cross sectional geometry of streams is defined by coordinates of lateral distance and ground elevation which locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible, but in wide floodplains or bends it may be necessary to use a section along intersecting straight lines, i.e. a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals in order to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and α, and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Davidian, 1984).

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and flood plains for which the discharge has been measured and Manning's n has been calculated are very useful (see Arcement and Schneider, 1984; Barnes, 1978). For situations lying outside the engineer's experience, a more regimented approach is presented in *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (Arcement and Schneider, 1984). Once the Manning's n values have been selected, it is highly recommended that they be verified or calibrated with historical high water marks and/or gaged streamflow data. Manning's n values for artificial channels are more easily defined than for natural stream channels. See Table 4.1 for typical n values of both artificial channels and natural stream channels.

The equations should be calibrated with historical high water marks and/or gaged streamflow data to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross section. Proper calibration is essential if accurate results are to be obtained.



SWITCHBACK

If the cross section is improperly subdivided, the mathematics of the Manning's equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback. This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

4.3.4 Single-Section Analysis

The single section analysis method (slope-area method) is simply a solution of Manning's equation for the normal depth of flow given the discharge and cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood. The stage-discharge curve can be determined as follows:

- Select the typical cross section at or near the location where the stage-discharge curve is needed.

- Subdivide cross section and assign n-values to subsections as described in Section 4.3.3
- Estimate water surface slope. Since uniform flow is assumed, the average slope of the streambed can usually be used.
- Apply a range of incremental water surface elevations to the cross section.
- Calculate the discharge using Manning's equation for each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross section and not along the vertical water interface between subsections.
- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should be made. This plot is the stage-discharge curve and it can be used to determine the water surface elevation corresponding to the design discharge or other discharge of interest.

In stream channels the transverse variation of velocity in any cross section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing erosion control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross section so that the total conveyance K_T of the cross section is the sum of the subsection conveyances. The total discharge is then $K_T S^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in (Vanoni, 1977) have been developed for this case (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

There may be locations where a stage-discharge relationship has already been measured in a channel. These usually exist at gaging stations on streams monitored by the USGS. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

4.3.5 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the FHWA/USGS program WSPRO, Corps of Engineers HEC-2 or HEC-RAS be used.

The WSPRO program has been designed to provide a water surface profile for six major types of open channel flow situations:

- unconstricted flow,
- single opening bridge,
- bridge opening(s) with spur dikes,
- single opening embankment overflow,
- multiple alternatives for a single site, and
- multiple openings.

The HEC-2 or HEC-RAS program developed by the Corps of Engineers is widely used for calculating water surface profiles for steady gradually varied flow in a natural or constructed channels. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs, and structures in the floodplain may also be considered in the computations. This program is designed for application in floodplain management and flood insurance studies.

The computation of water surface profiles by WSPRO, HEC-2 or HEC-RAS is based on the standard step method in which the stream reach is divided into a number of subreaches by cross sections spaced such that the flow is gradually-varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross-section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 4.3).

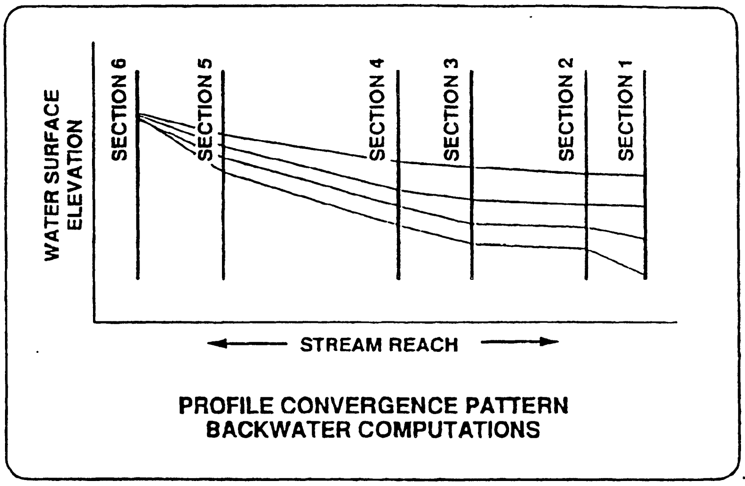


Figure 4.3 Profile Convergence Pattern Backwater Computation

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 4.4). The Corps of Engineers (USACE, 1986) developed equations for determining upstream and downstream reach lengths as follows:

$$L_{dn} = 8{,}000\frac{HD^{0.8}}{S} \tag{4.13}$$

$$L_{u} = 10{,}000\frac{HD^{0.6}\ HL^{0.5}}{S} \tag{4.14}$$

Where: L_{dn} = downstream study length (along main channel), (ft) for normal depth starting conditions

L_u = estimated upstream study length (along main channel), (ft) required for convergence of the modified profile to within 0.1 feet of the base profile

HD = average hydraulic depth (1-percent chance event flow area divided by the top width), (ft)

S = average reach slope (ft/mile)

HL = headloss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 1-percent chance flood (ft)

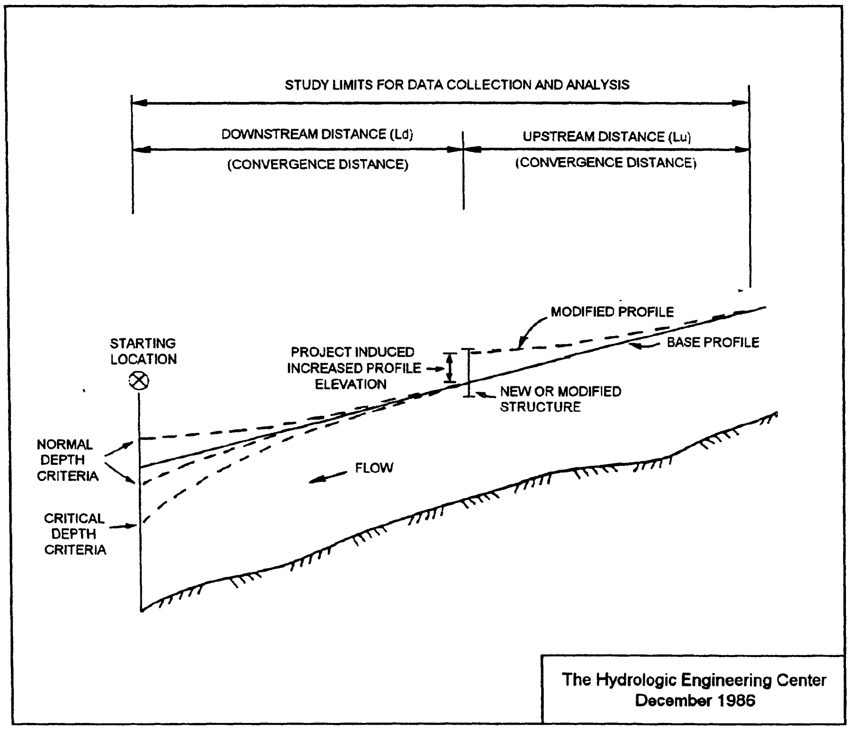


Figure 4.4 Profile Study Limits
Source: USACE, 1986

References (Davidian, 1984; USACE, 1986) are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. These references contain more specific guidance on cross-section determination, location, spacing and stream reach determination. Reference (USACE, 1986) investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross-section coordinate geometry. A sample procedure is available in *Hydrologic Engineering Methods For Water Resources Development - Volume 6, Water Surface Profiles,* The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, California.

4.3.6 Water and Sediment Routing

The BRI-STARS (Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation) Model was developed by the National Cooperative Highway Research Program and FHWA (Molinas, 2000). It is based on utilizing the stream tube method of calculation which allows the lateral and longitudinal variation of hydraulic conditions as well as sediment activity at various cross sections along the study reach. Both energy and momentum functions are used in the BRI-STARS model so the water surface profile computation can be carried out through combinations of subcritical and supercritical flows without interruption. The stream tube concept is used for hydraulic computations in a semi-two-dimensional way. Once the hydraulic parameters in each stream tube are computed, the scour or deposition in each stream tube determined by sediment routing will give the variation of channel geometry in the vertical direction.

The BRI-STARS model contains a rule-based expert system program for classifying streams by size, bed and bank material stability, planform geometry, and other hydrologic and morphological features. Due to the complexities of a single classification system that utilizes all parameters, no universally acceptable stream classification method presently exists. Consequently this model does not contain a single methodology for classifying all streams. Instead, methodologies were first classified according to the channel sediment sizes they were derived for, then within each size group, one or more classification schemes have been included to cover a wider range of environments. The stream classification information can be used to assist in the selection of model parameters and algorithms. Applications of the BRI-STARS can be summarized as follows:

- Fixed bed model to compute water surface profiles for subcritical, supercritical, or both flow conditions involving hydraulic jumps.
- Movable bed model to route water and sediment through alluvial channels.
- Use of stream tubes to allow the model to compute the variation of hydraulic conditions and sediment activity in the longitudinal as well as the lateral direction.
- The armoring option allows simulation of longer term riverbed changes.
- The minimization procedure option allows the model to simulate channel widening and narrowing processes.
- The local bridge scour option allows the computation of pier and abutment scour.
- The bridge routines for fixed geometry mode from WSPRO are available as an option in the program.

4.4 DESIGN PROCEDURE

The design procedures for all types of channels have some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

4.4.1 Stream Channels

The analysis of a stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The extent of the study should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain. Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a generalized process which will usually apply.

Step 1 **Assemble site data and project file.**

A. Data collection
- Topographic, site, and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gaging records
- Historic flood data and local knowledge

B. Studies by other agencies
- Flood insurance studies
- Floodplain studies
- Watershed studies

C. Environmental constraints
- Floodplain encroachment
- Floodway designation
- Fish and wildlife habitat
- Commitments in review documents

D. Design criteria (See Section 4.2)

Step 2 **Determine the project scope.**

A. Determine level of assessment
- Stability of existing channel
- Potential for damage
- Sensitivity of the stream

B. Determine type of hydraulic analysis
- Qualitative assessment
- Single-section analysis
- Step-backwater analysis
- 2-D modeling

C. Determine additional survey information
- Extent of streambed profiles
- Elevations of flood-prone property
- Details of existing structures
- Properties of bed and bank materials

Step 3 **Evaluate hydrologic variables.**

Compute discharges for selected frequencies.
Consult Hydrology Chapter

Step 4 **Perform hydraulic analysis.**

A. Single-section analysis
- Select representative cross section
- Select appropriate n values
- Compute stage-discharge relationship

B. Step-backwater analysis

C. Calibrate with known high water

Step 5 **Perform stability analysis.**

A. Geomorphic factors

B. Hydraulic factors

C. Stream response to change

D. For additional information
- HEC-20 Stream Stability
- Highways in the River Environment (HIRE)

Step 6 **Design countermeasures.**

A. Criteria for selection
- Erosion mechanism
- Stream characteristics
- Construction and maintenance requirements
- Vandalism considerations
- Cost

B. Types of countermeasures
- Meander migration countermeasures
- Bank stabilization
- Bend control countermeasures
- Channel braiding countermeasures
- Degradation countermeasures
- Aggradation countermeasures

C. For additional information
- HEC-23 Bridge Scour and Stream Instability Countermeasures
- Review References, Section 4.6

Step 7 **Documentation.**

Prepare report and file with background information

4.4.2 Roadside Channels

A roadside channel is defined as an open channel, usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal in cross section and lined with grass or a special protective lining. The primary function of roadside channels is to collect surface runoff from the highway and areas which drain to the right-of-way and convey the accumulated runoff to acceptable outlet points. A secondary function of a roadside channel is to drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

The alignment, cross section, and grade of roadside channels is usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner which assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects. Proper shaping such as slope and ditch rounding, cut to fill blending, built in gully elimination, and feathering cuts are effective means of erosion control. Each project is unique, but the following six basic design steps are normally applicable:

Step 1 **Establish a roadside plan.**

A. Collect available site data.
B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, etc.
C. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets.
D. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

Step 2 **Obtain or establish cross section data.**

A. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.
B. Choose channel side slopes based on geometric design criteria including safety, clear zone requirements, economics, soil, aesthetics, and access.
C. Establish bottom width of trapezoidal channel. A desirable width is 8′.
D. Identify features which may restrict cross section design:
   - right-of-way limits,
   - trees or environmentally-sensitive areas,
   - utilities, and
   - existing drainage facilities.

Step 3 **Determine initial channel grades.**

A. Plot initial grades on plan-profile layout. (Slopes in roadside ditch in cuts are usually controlled by highway grades.)
B. Provide minimum grade of 0.3%, if possible, to minimize ponding and sediment accumulation.
C. Consider influence of type of lining on grade.
D. Where possible, avoid features which may influence or restrict grade, such as utility locations.

Step 4 **Check flow capacities and adjust as necessary.**

A. Compute the design discharge at the downstream end of a channel segment (see Hydrology chapter).
B. Set preliminary values of channel size, roughness coefficient, and slope.
C. Determine maximum allowable depth of channel including freeboard.
D. Check flow capacity using Manning's equation and single-section analysis.
E. If capacity is inadequate, possible adjustments are as follows:
   - increase bottom width,
   - make channel side slopes flatter,
   - make channel slope steeper,
   - provide smoother channel lining,
   - install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity,
   - provide smooth transitions at changes in channel cross sections, or
   - provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5 **Determine channel lining/protection needed.**

A. Select a lining and determine the permissible shear stress τ_p in lbs/ft^2 from Table 4.3.
B. Estimate the flow depth and choose an initial Manning's n value from Table 4.4 or from Figures 4.5 through 4.10. To use Figures 4.5 through 4.10 determine the vegetation classification using Table 4.2. For a know channel geometry and therefore known hydraulic radius (R) and slope (S) use the figure to find Manning's n on the Y axis.
C. Calculate normal flow depth y_o (ft) at design discharge using Manning's equation and compare with the estimated depth. If they do not agree, repeat steps 5B and 5C.
D. Compute maximum shear stress at normal depth using Equation 4.11 or 4.12.
   If $\tau_d < \tau_p$ then lining is acceptable. Otherwise consider the following options:
   - choose a more resistant lining,
   - decrease channel slope,
   - decrease slope in combination with drop structures, and/or
   - increase channel width and/or flatten side slopes

Step 6 **Analyze outlet points and downstream effects.**

A. Identify any adverse impacts such as increased flooding or erosion to downstream properties which may result from one of the following at the channel outlet:
- increase or decrease in discharge,
- increase in velocity of flow,
- concentration of sheet flow,
- change in outlet water quality, or
- diversion of flow from another watershed.

B. Mitigate any adverse impacts identified in 6A. Possibilities include:
- enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
- install velocity control structures (energy dissipators),
- increase capacity and/or improve lining of downstream channel,
- install sedimentation/infiltration basins,
- install sophisticated weirs or other outlet devices to redistribute concentrated channel flow,
- eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion.

In order to obtain the optimum roadside channel system design, it may be necessary to make several trials of the previous procedure before a final design is achieved. More details on channel lining design may be found in HEC-15 (FHWA, 1988) including consideration of channel bends, steep slopes, and composite linings. The riprap design procedures covered in HEC-15 are for channels having a design discharge of 50 cfs or less. When the design discharge exceeds 50 cfs, the designer should refer to other sources such as Design of Riprap Revetment, HEC-11, (FHWA, 1989).

Table 4.2 Classification of Vegetal Covers as to Degrees of Retardancy

Retardance	Cover	Condition
Class A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem, Ischaemum	Excellent stand, tall (average 36")
Class B	Kudzu	Dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture: little bluestem, bluestem, blue gamma, other short and long stem midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good Stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
Class C	Crabgrass	Fair stand, uncut (10 - 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture: summer orchard grass redtop, Italian ryegrass, and common lespedeza	Good stand, uncut (6-8")
	Centipedegrass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 - 12")
Class D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3-6")
	Grass-legume mixture: fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza	Good Stand, uncut (4-5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
Class E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Table is used to determine the class of retardance used in Table 4.3 and Figures 4.6 - 4.10.
Covers classified have been tested in experimental channel. Covers were green and generally uniform. Source of table is HEC-15 (FHWA, 1988).

Table 4.3 Summary Of Permissible Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil Type	τ_p (lb/ft^2)	Protective Cover	Underlying Soil Type	τ_p (lb/ft^2)
Vegetation Class	Erosion Resistant or Erodible		Structural	Type I	
A		3.70	6 in Gabions		35
B		2.10	4 in Geoweb		10
C		1.00	Soil Cement (8% Cement)		>45
D		0.60			
E		0.35			
Temporary			Dycel w/o Grass		>7.0
Woven Paper		0.15	Petraflex w/o Grass		>32
Jute Net		0.45	Armorflex w/o Grass		12-20
Single Fiberglass		0.60	Enkamat w/3 in Asphalt		13-16
Double Fiberglass		0.85	Enkamat w/1 in Asphalt		<5
Straw w/Net		1.45			
Curled Wood Mat		1.55			
Synthetic Mat		2.00			
Gravel Riprap			Armorflex Class 30 with longitudinal and lateral cables, no grass		>34
D_{50} = 1 in		0.40			
D_{50} = 2 in		0.80			
Rock Riprap			Dycell 100, longitudinal cables, cells filled with mortar		<12
D_{50} = 6 in		2.50			
D_{50} = 12 in		5.00			
Concrete construction Blocks, granular filter underlayer	Type I	>20	Wedge-shaped blocks with drainage slot		>25

Type I soil is a silty clay to silty sand (SC-SM) with AASHTO classification A-4(0).
Source: FHWA-RD-89-199 (Clopper, 1989)

Table 4.4 Manning's Roughness Coefficients - Channel Lining Roughness Element Height, k_s

Lining Category	Lining Type	k_s (ft)	n - value for Depth Ranges 0 - 0.5 ft	0.5 - 2.0 ft	> 2.0 ft
Rigid	Concrete		0.015	0.013	0.013
	Grouted Riprap		0.040	0.030	0.028
	Stone Masonry		0.042	0.032	0.030
	Soil Cement		0.025	0.022	0.020
	Asphalt		0.018	0.016	0.016
Unlined	Bare Soil		0.023	0.020	0.020
	Rock Cut		0.045	0.035	0.025
Temporary*	Woven Paper Net	0.004	0.016	0.015	0.015
	Jute Net	0.038	0.028	0.022	0.019
	Fiberglass Roving	0.035	0.028	0.022	0.019
	Straw with Net	0.120	0.065	0.033	0.025
	Curled Wood Mat	0.110	0.066	0.035	0.028
	Synthetic Mat	0.065	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.083	0.044	0.033	0.030
	2-inch D_{50}	0.167	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.500	0.104	0.069	0.035
	12-inch D_{50}	1.000	–	0.078	0.040

Values listed are representative values for the respective depth ranges.
Manning's roughness coefficients, n, vary with the flow depth.
Source: HEC-15 (FHWA, 1988)

For riprap $k_s = D_{50}$.
* Some "temporary" linings become permanent when buried.

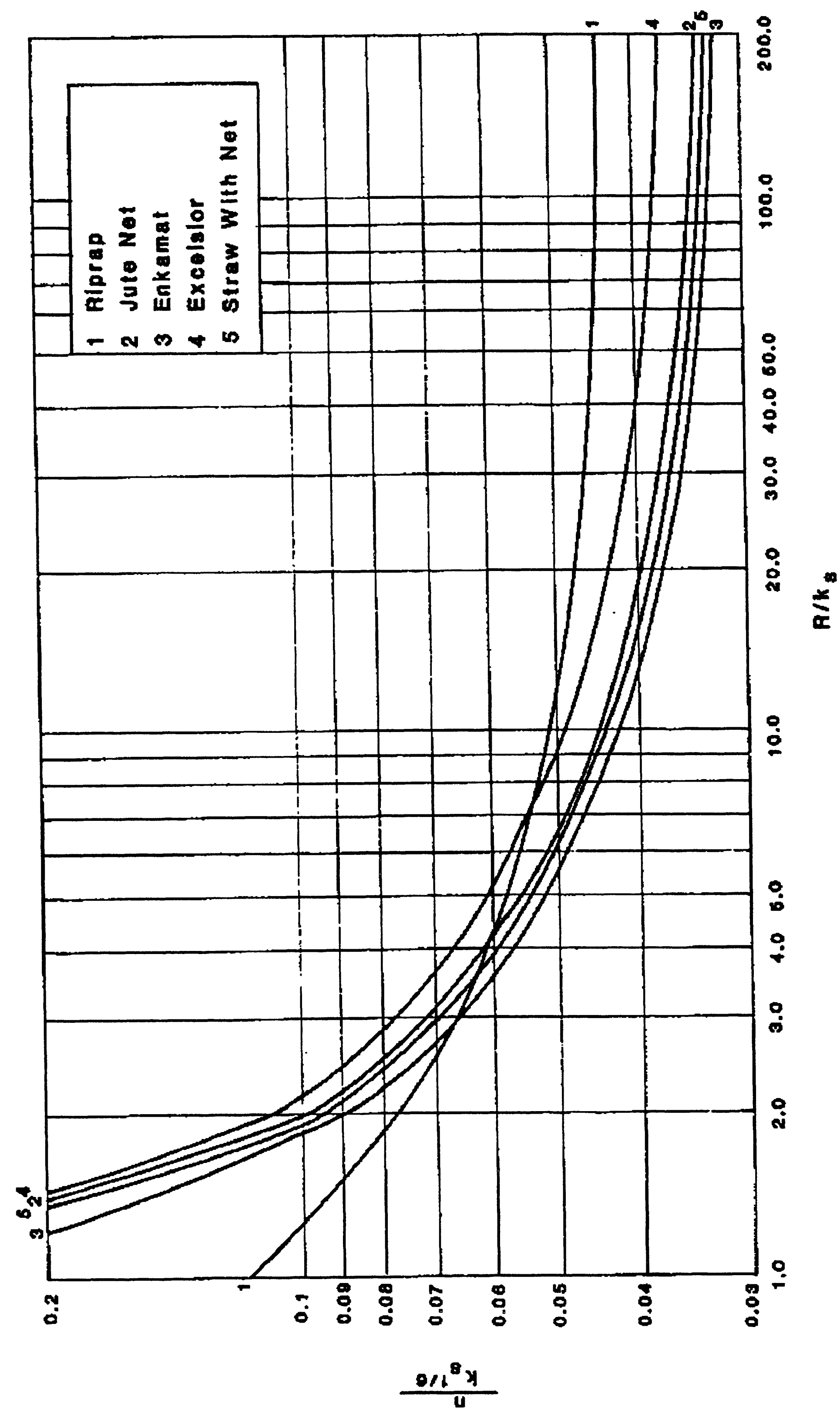


Figure 4.5 Manning's n Versus Relative Roughness for Selected Lining Types
Source: HEC-15 (FHWA, 1988)

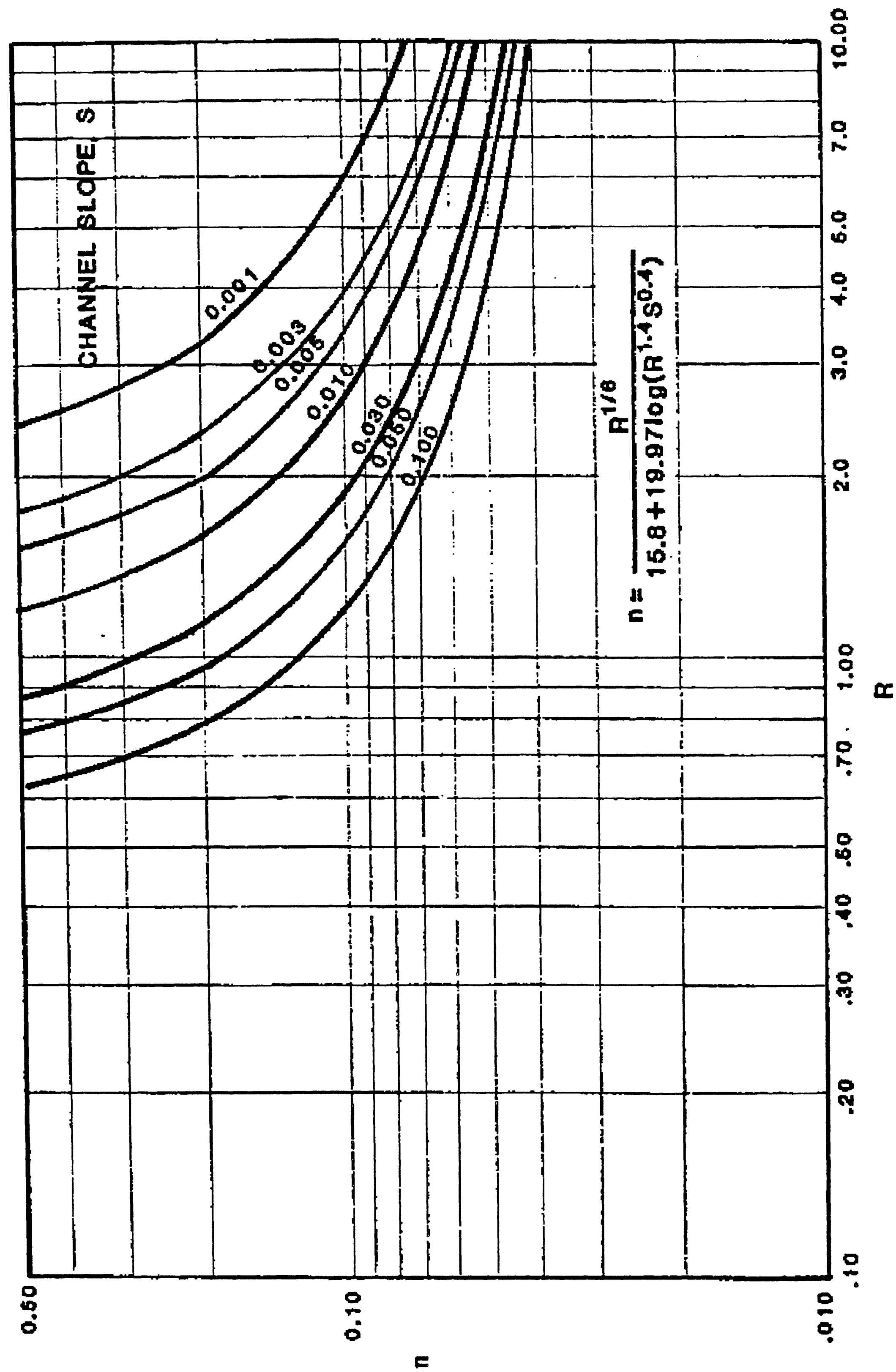


Figure 4.6 Manning's n Versus Hydraulic Radius, R, For Class A Vegetation
Source: HEC-15 (FHWA, 1988)

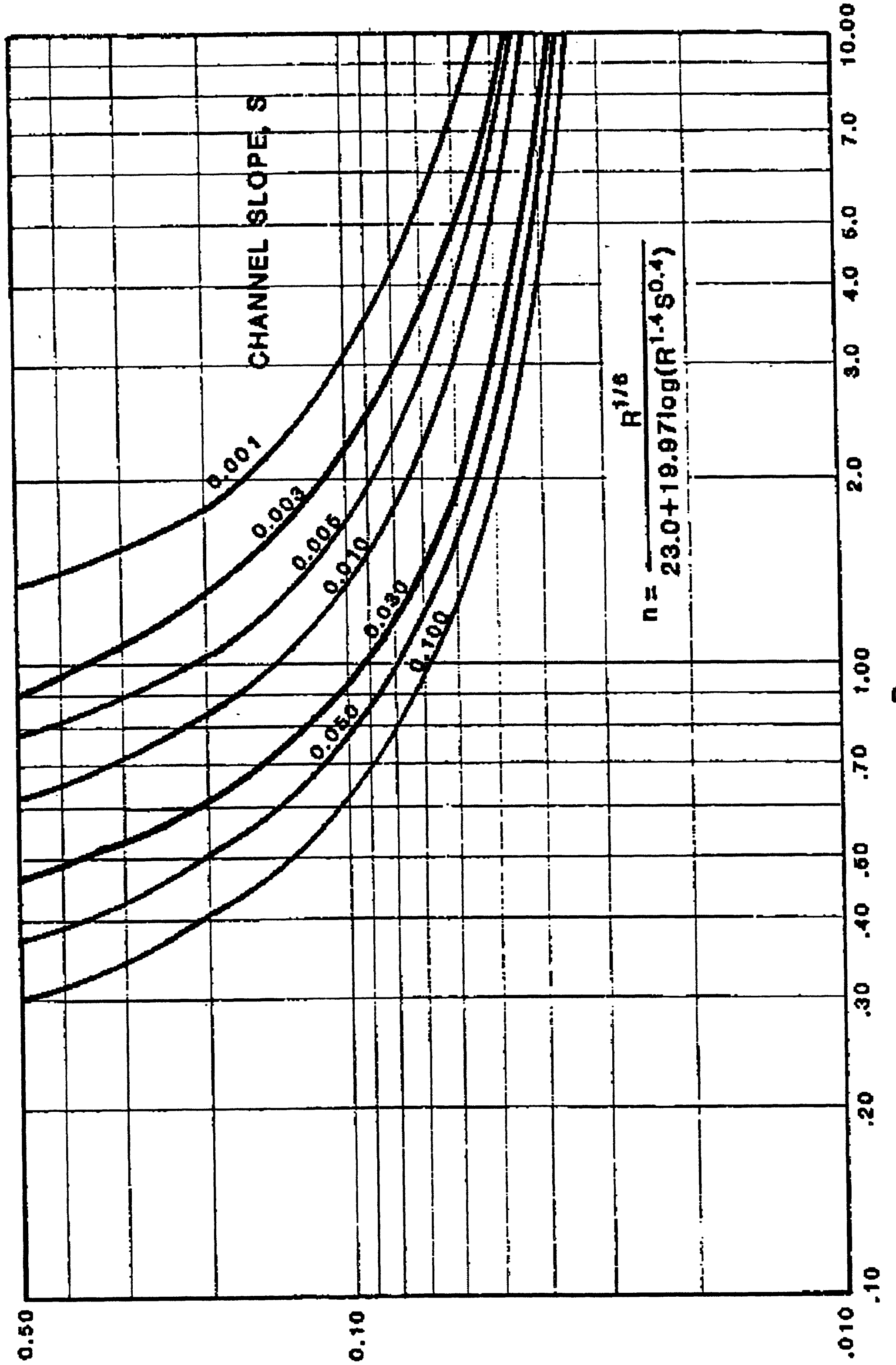


Figure 4.7 Manning's n Versus Hydraulic Radius, R for Class B Vegetation
Source: HEC-15 (FHWA, 1988)

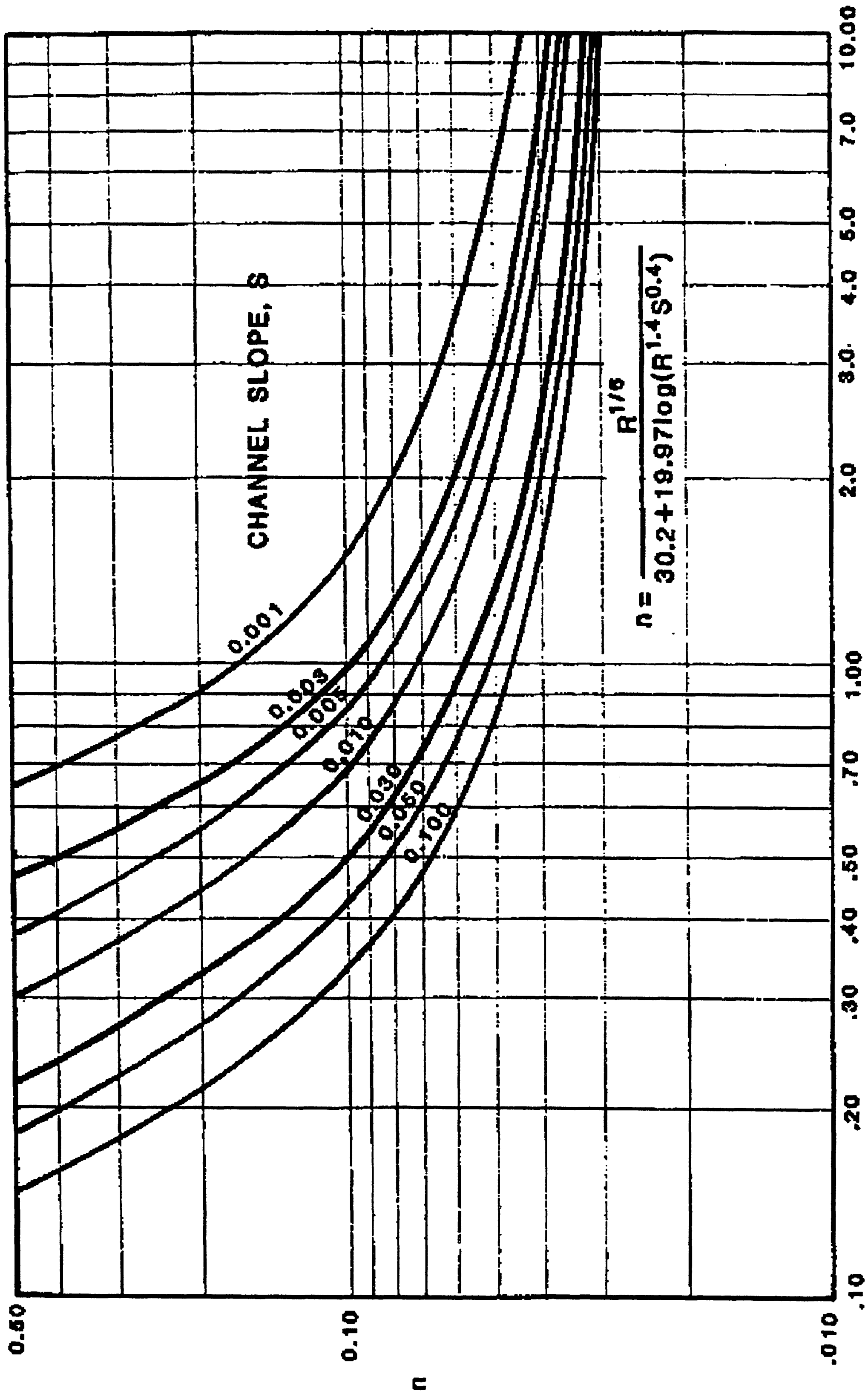


Figure 4.8 Manning's n Versus Radius, R, For Class C Vegetation
Sources: HEC-15 (FHWA, 1998)

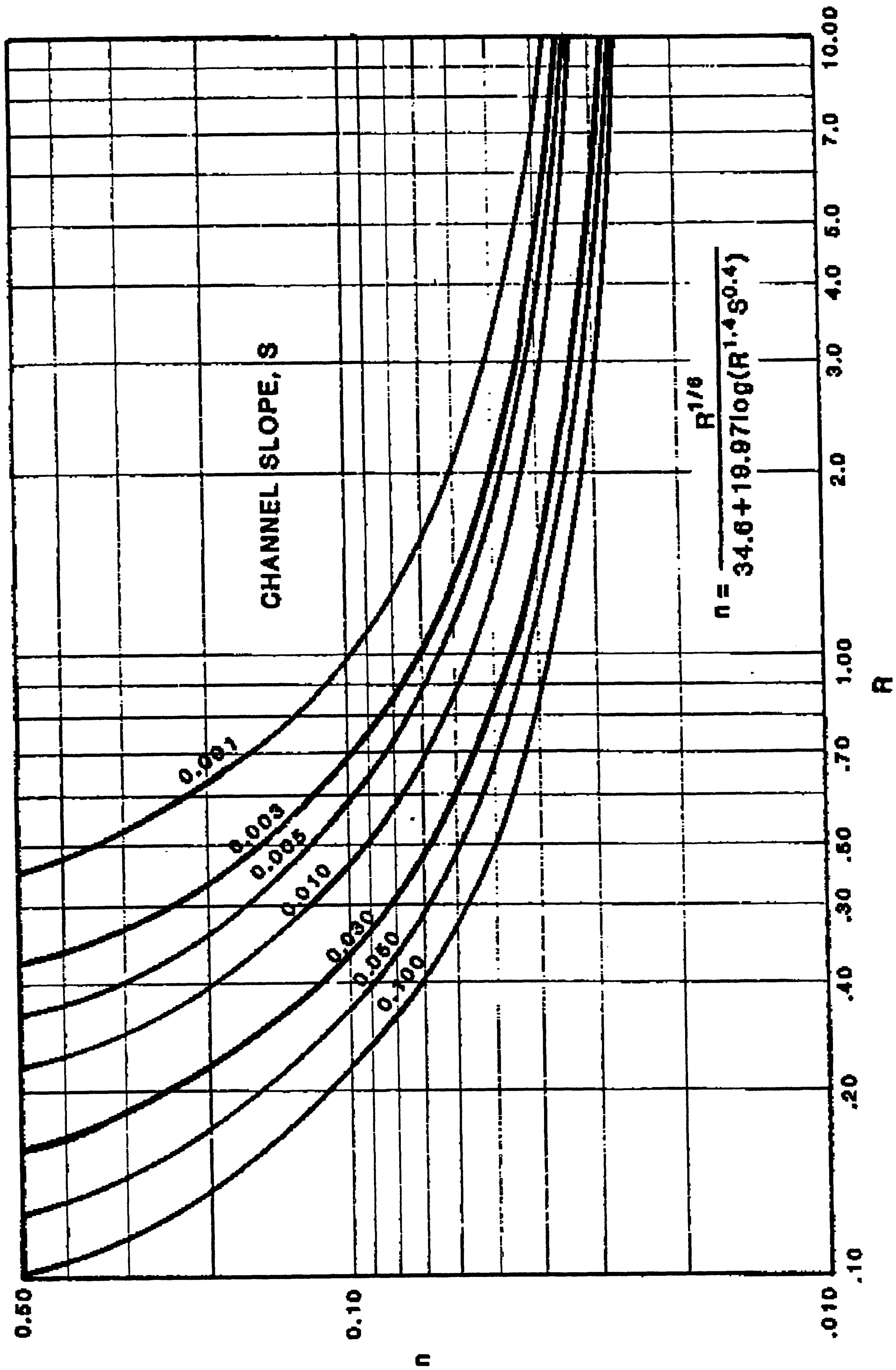


Figure 4.9 Manning's n Versus Hydraulic Radius, R, For Class D Vegetation
Source: HEC-15 (FHWA, 1988)

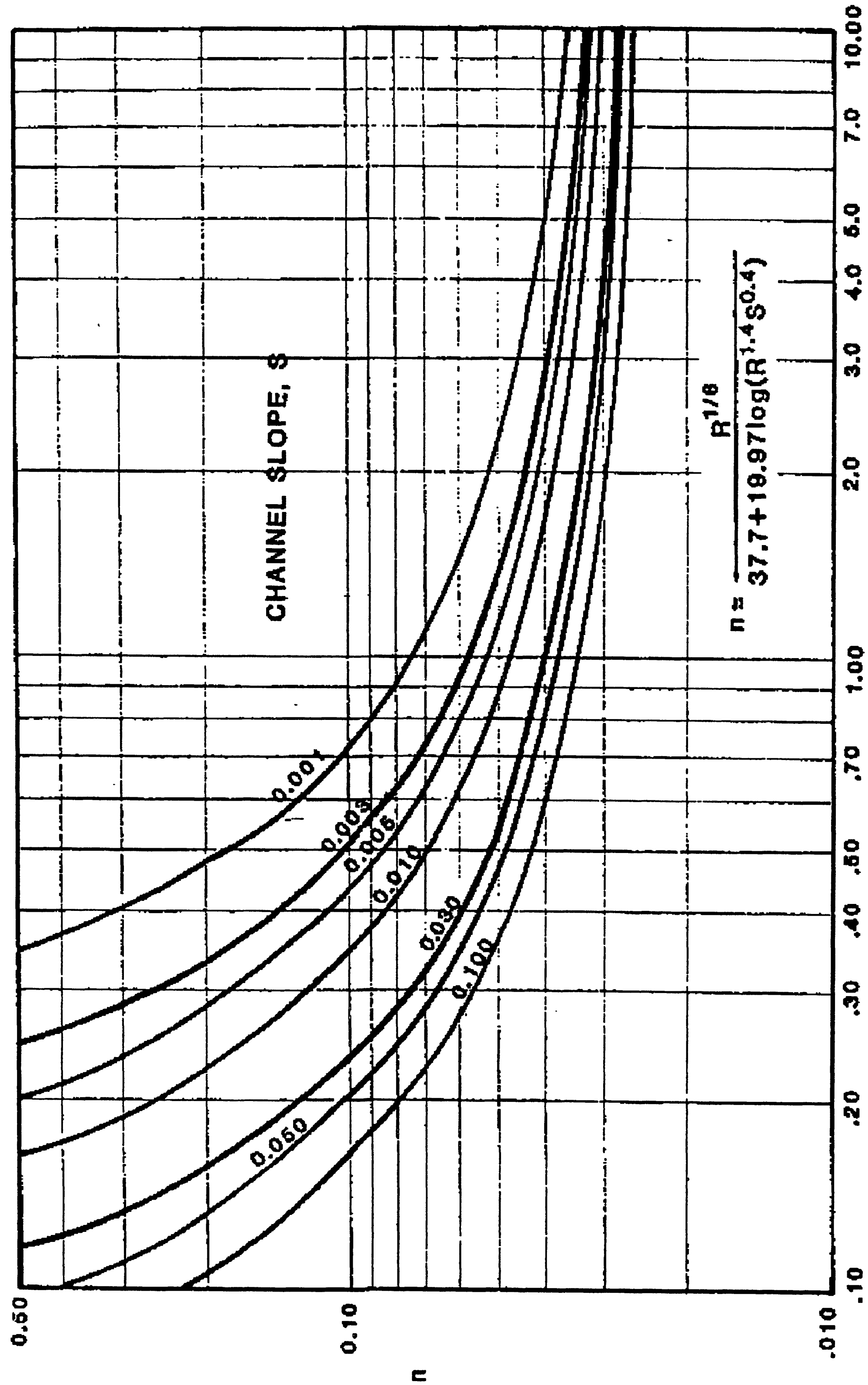


Figure 4.10 Manning's n Versus Hydraulic Radius, R, For Class E Vegetation
Source: HEC-15 (FHWA, 1988)

4.5 STREAM MORPHOLOGY

Most streams in Minnesota that highways cross or encroach upon are alluvial; that is, the streams are formed in materials that have been and can be transported by the stream. In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands and side channels will undergo modification with time. Alluvial channels continually change position and shape as a consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or human activities.

A study of the plan and profile of a stream is very useful in understanding stream morphology. Plan view appearance of streams are varied and result from many interacting variables. The planform of the stream may be straight, braided, or meandering. These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form, and bank erosion with time. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change a variable which can adversely affect the stream.

The natural stream channel will assume a geomorphological form which will be compatible with the sediment load and discharge history which it has experienced over time. To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion, and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

4.5.1 Levels of Assessment

The analysis and design of a stream channel will usually require an assessment of the existing channel and the potential for problems as a result of the proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation is the best means of identifying potential locations for channel bank erosion and subsequent channel stabilization. Analytical methods for the evaluation of channel stability can be classified as either hydraulic or geomorphic, and it is important to recognize that these analytical tools should only be used to substantiate the erosion potential indicated through observation. A brief description of the three levels of assessment are as follows:

Level 1 Qualitative assessment involving the application of geomorphic concepts to identify potential problems and alternative solutions. Data needed may include historic information, current site conditions, aerial photographs, old maps and survey notes, bridge design files, maintenance records, and interviews with long-time residents.

Level 2 Quantitative analysis combined with a more detailed qualitative assessment of geomorphic factors. Generally includes water surface profile and scour calculations. This level of analysis will be adequate for most locations if the problems are resolved and relationships between different factors affecting stability are adequately explained. Data needed will include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

Level 3 Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. Necessary only for high risk locations, extraordinarily complex problems, and possibly after the fact analysis where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing and/or physical modeling. Data needed will require Level 1 and 2 data as well as field data on bed load and suspended load transport rates and properties of bed and bank materials such as size, shape, gradation, fall velocity, cohesion, density, and angle of repose.

4.5.2 Factors That Affect Stream Stability

Factors that affect stream stability and, potentially, bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

Hydraulic Factors

- magnitude, frequency and duration of floods
- bed configuration
- resistance to flow
- water surface profiles

Geomorphic Factors

Figure 4.12 depicts examples of the various geomorphic factors.

- stream size
- valley setting
- natural levees
- sinuosity
- width variability
- bar development
- flow variability
- floodplains
- apparent incision
- channel boundaries
- degree of braiding
- degree of anabranching

Figure 4.11 depicts the changes in channel classification and relative stability to hydraulic factors. Rapid and unexpected changes may occur in streams in response to human activities in the watershed such as alteration of vegetative cover. Changes in perviousness can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts, reservoirs and changes in land use can have major effects on stream flow, sediment transport and channel geometry and location. Knowing that these activities can influence stream stability can help the designer anticipate some of the problems that can occur.

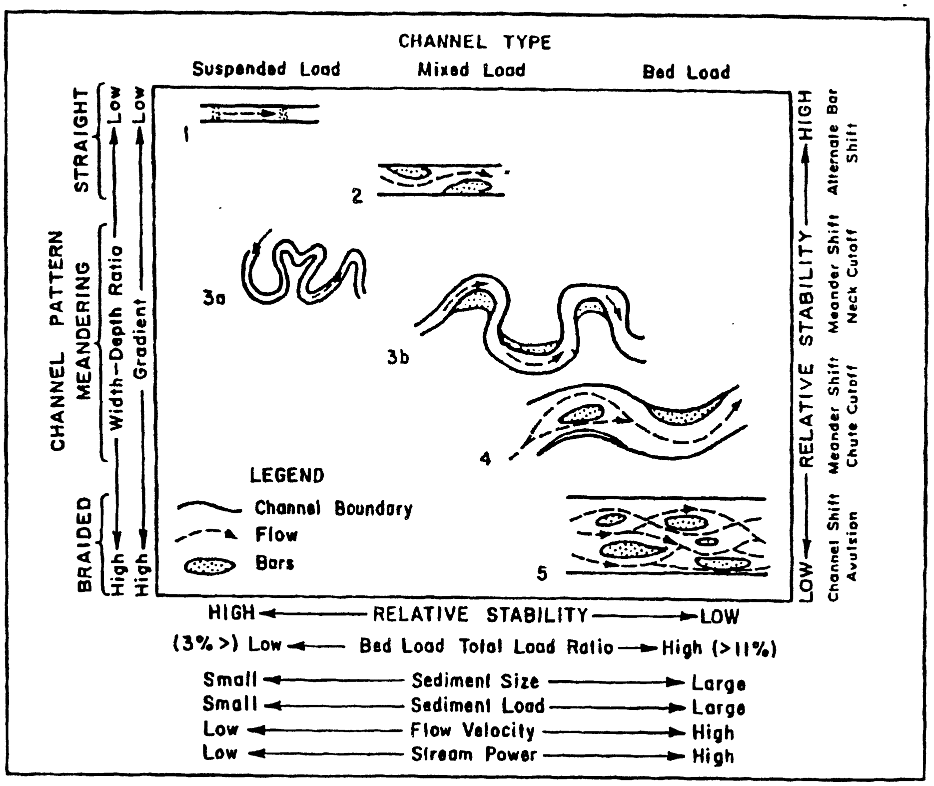


Figure 4.11 Channel Classification And Relative Stability As Hydraulic Factors Are Varied
Source: after Shen et. al., 1981

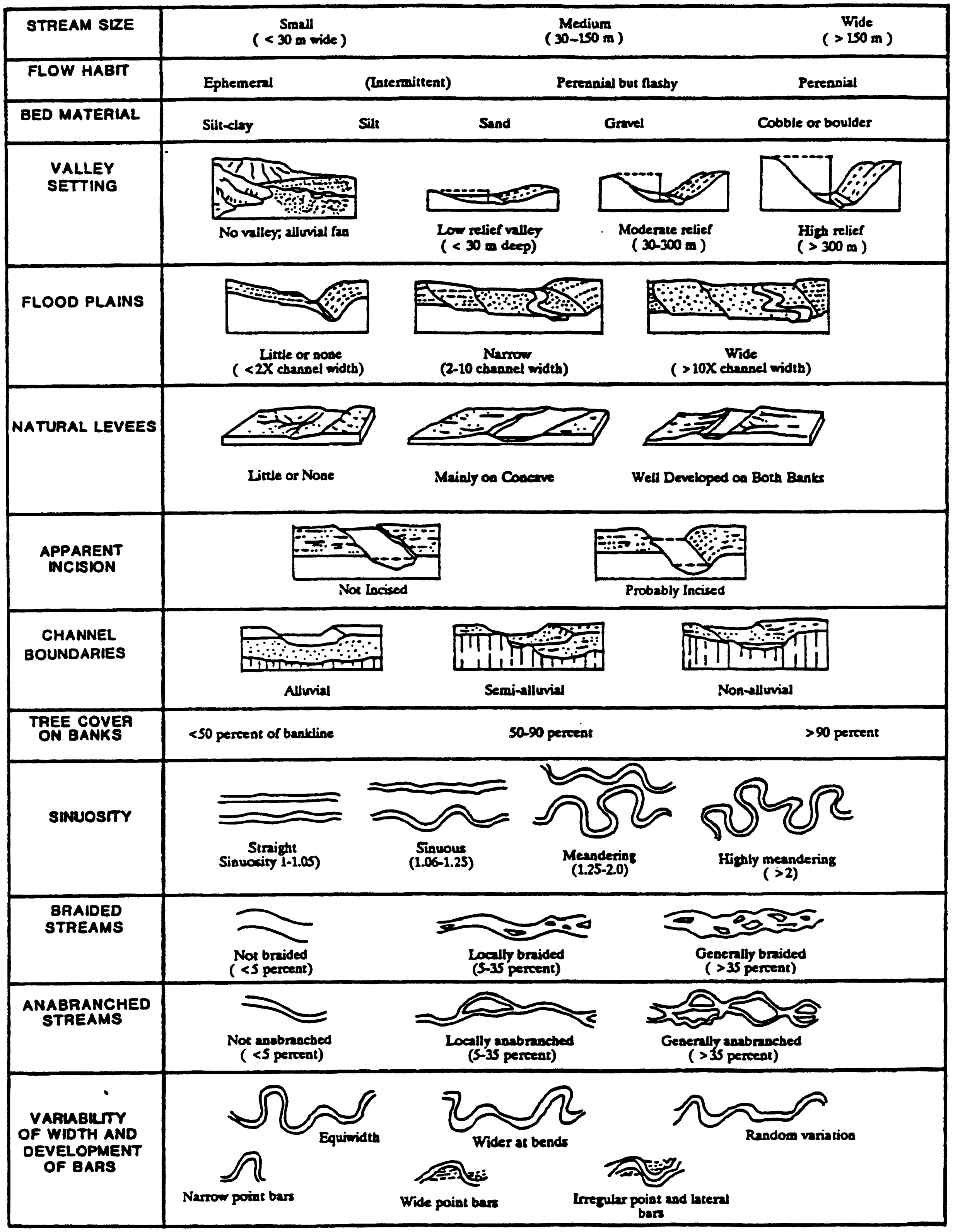


Figure 4.12 Geomorphic Factors That Affect Stream Stability
Source: adapted from Brice and Blodgett, 1978

Natural disturbances such as floods, drought, earthquakes, landslides, volcanoes and forest fires can also cause large changes in sediment load and major changes in the stream channel. Although difficult to plan for such disturbances, it is important to recognize that when natural disturbances do occur, it is likely that changes will also occur to the stream channel.

4.5.3 Stream Response to Change

The major complicating factors in river mechanics are: the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. In order to better understand the responses of a stream to the actions of human's and nature, a few simple hydraulic and geomorphic concepts are presented herein.
The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 4.13. Any natural or artificial change which alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Referring to

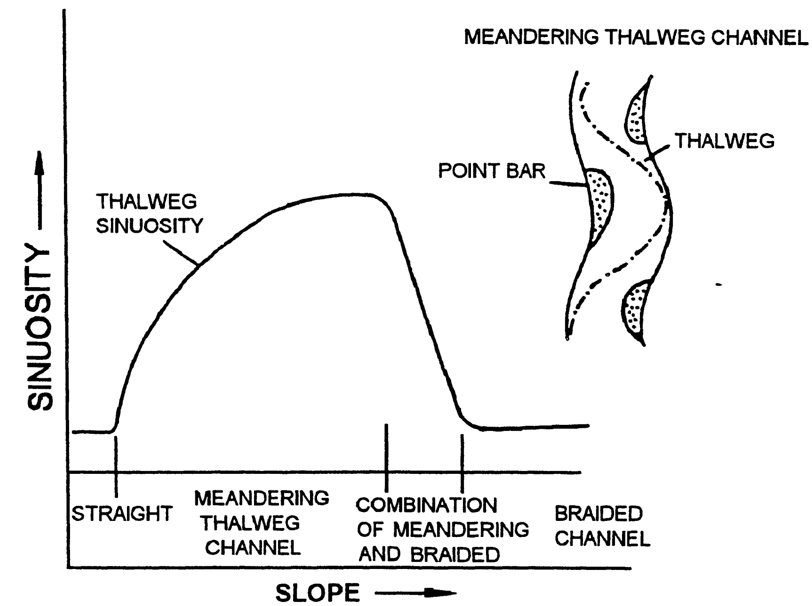


Figure 4.13 Sinuosity Vs Slope With Constant Discharge
Source: after Richardson et. al., 1990

Figure 4.13, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

The different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wave length, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change.

Figure 4.14 illustrates the dependence of river form on channel slope and discharge. It shows that when $SQ^{1/4} \leq .0017$ in a sandbed channel, the stream will meander. Similarly, when $SQ^{1/4} \geq .010$, the stream is braided. In these equations, S is the channel slope in feet per foot and Q is the mean discharge in cfs. The zone between the lines defining braided streams and meandering streams in Figure 4.14 is the transitional range, i.e., the range in which a stream can change readily from one stream form to the other.

Many United States rivers plot in this zone between the limiting curves defining meandering and braided streams. If a stream is meandering but its discharge and slope border on a boundary of the transitional zone, a relatively small increase in channel slope may cause it to change, in time, to a transitional or braided stream.

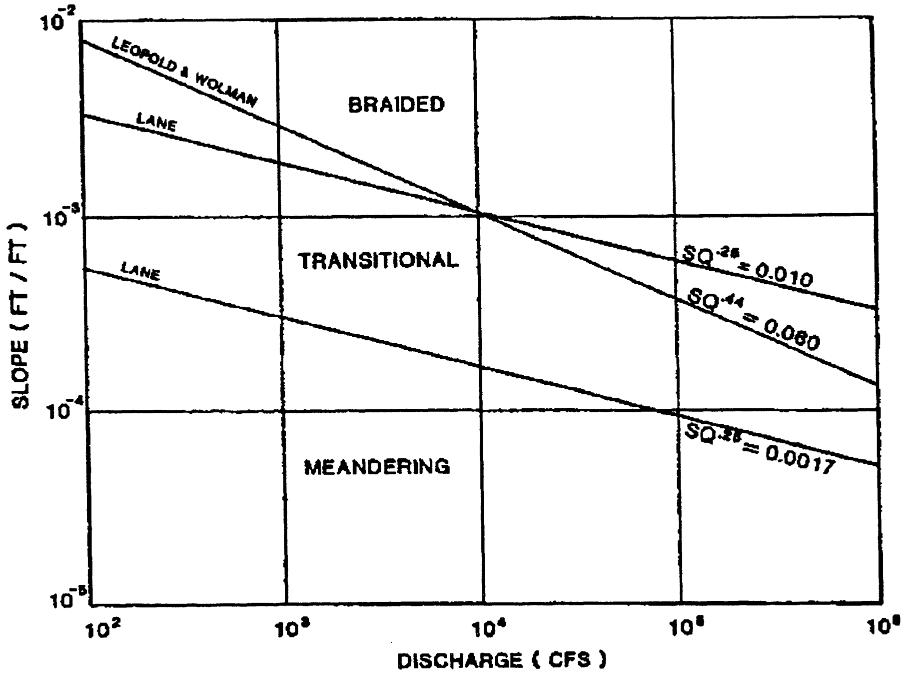


Figure 4.14 Slope-Discharge For Braiding or Meandering Bed Streams
Source: After, lane, 1957

4.5.4 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop. The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. Below is a brief discussion of possible countermeasures for some common river stability problems. The reader is encouraged to consult with the references listed at the end of this chapter for detailed information on the design and construction of the countermeasures.

Meander Migration

The best countermeasure against meander migration is a crossing location on a relatively straight reach of stream between bends. Other countermeasures include the protection of an existing bank line, the establishment of a new flowline or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Countermeasures may be used individually or a combination of two or more countermeasures may be used to combat meander migration at a site. (Highways in the River Environment, FHWA, 1990; and HEC-20, FHWA, 1991).

Channel Braiding

Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels. The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

Degradation

Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams. Longitudinal stone dikes placed at the toe of channel banks can be effective counter measures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (HEC-20, FHWA, 1991).

Aggradation

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these. Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation. Another technique which shows promise is the submerged vane technique developed by the University of Iowa. The studies suggest that the submerged vane structure may be an effective, economic, low-maintenance, and environmentally acceptable sediment-control structure with a wide range of applications (HEC-20, FHWA, 1991; Odgaard, et. al, 1986).

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Chapter 5 CULVERT

5.1 INTRODUCTION
This chapter provides design procedures for the hydraulic design of highway culverts which are based on the Federal Highway Administration (FHWA) Hydraulic Design Series No. 5 (HDS-5), *Hydraulic Design of Highway Culverts* (FHWA, 1985).

5.1.1 Definition
A culvert is defined as a structure sized hydraulically to convey surface water runoff under a highway, railroad, or other embankment. Culverts are:
- structures distinguished from bridges by being covered with an embankment and generally composed of a structural material around the entire perimeter with some exceptions such as a MN/DOT Arch which may utilize the natural streambed and appropriate erosion protection as the bottom.
- classified as a bridge when horizontal opening width is 10 feet or greater measured perpendicular to the roadway centerline, however, the structure is analyzed using procedures defined in this chapter.

5.1.2 Concept Definitions
The following are discussions of concepts which are important in culvert design.

Barrel Area	Barrel area is measured perpendicular to the flow and refers to the water area in the barrel.
Barrel Length	Barrel length is the total culvert length from the entrance to the exit of the culvert. Because the height of the barrel, barrel slope, and barrel skew influence the actual length, an approximation of barrel length is usually necessary to begin the design process.
Barrel Roughness	Barrel Roughness is a function of the material used to fabricate the barrel. Typical materials include concrete, corrugated metal and plastic. The roughness is represented by a hydraulic resistance coefficient such as the Mannings "n" value.
Barrel Slope	Barrel slope is the actual slope of the culvert barrel, and is often the same as the natural stream slope.
Critical Depth	Critical depth is the depth at which the specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry there is only one critical depth.
Crown	The crown is the inside top of the culvert.
Flowline	The flowline is the bottom invert of a conduit. An exception is when the invert is buried and riprap or other fill material is placed in the culvert, the flowline is then the top of the fill material.
Free Outlet	A free outlet has a tailwater equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the backwater profile upstream of the tailwater.
Headwater, HW	That depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.
Improved Inlet	An improved inlet has an entrance geometry which decreases the flow constriction at the inlet and thus increases the capacity of culverts. These inlets are referred to as either side- or slope-tapered (walls or bottom tapered).
Invert	The invert is the inside bottom of the culvert.
Normal Flow	Normal flow occurs in a channel reach when the discharge, velocity and depth of flow do not change throughout the reach. The water surface profile and channel bottom slope will be parallel. This type of flow will exist in a culvert operating on a uniform slope provided the culvert is sufficiently long.
Slope	There are two classifications of slope, steep and mild. • Steep slope occurs where the critical depth is greater than the normal depth. • Mild slope occurs where critical depth is less than normal depth.

Stage Increase	The difference between headwater and the unconstricted water surface just upstream of the culvert.
Submerged	Submergence can occur at either the inlet and/or the outlet. • A submerged outlet occurs where the tailwater elevation is higher than the crown of the culvert. • A submerged inlet occurs where the headwater is greater than 1.2 times the culvert diameter.
Tailwater (TW)	The depth of water at the outlet of a culvert.

5.2 DESIGN CRITERIA

Design criteria are the standards by which a policy is carried out or placed into action. They form the basis for the selection of the final design configuration. There are a number of different culvert sizes, shapes and materials from which a designer can choose. The design selected should be the one that best integrates hydraulic efficiency, serviceability, structural stability, economics, environmental considerations, traffic safety and land use requirements.

Culverts are used in the following conditions:
- where they are more economical than a bridge,
- where bridges are not hydraulically required,
- where higher velocities can be tolerated,
- where greater stage increases can be tolerated, and
- where debris and ice are tolerable.

5.2.1 Policy

Policy is a set of goals that establish a definite course or method of action. These goals are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. The policies specific to culverts are listed below.
- All culverts should be hydraulically designed, however the minimum pipe size specified in Section 5.2.4 will sometimes dictate the ultimate design.
- Use 50 year design frequency criteria for minor culverts 48" or less in diameter. The overtopping flood need not be computed. A greater design frequency may be required if there is significant flood damage potential upstream, there are special traffic considerations, or to accommodate FEMA mapped floodplains.
- A risk assessment shall be completed for all major culverts 54" or larger. The 500-year flood or overtopping flood (if less than Q_{500}) shall be computed.
- Culvert location in both plan and profile should be investigated and designed to avoid sediment build-up in culvert barrels.
- The potential for culverts plugging with debris or ice shall be considered in the design.
- Material selection is based on determining the material type which best fulfills all of the engineering requirements for a specific installation. Factors to be considered are hydraulic performance, structural stability, serviceability, and economics. Abrasion and corrosion should be considered when determining serviceability requirements.
- Culverts shall be located and designed to present a minimum hazard to traffic and people.
- The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure. Design data and calculations should be assembled in an orderly fashion and retained for future reference.
- Culverts should be regularly inspected and maintained.

5.2.2 Site Criteria

Design criteria that are dependant on site factors include: structure type, length, location in plan, location in profile, overfill, debris and ice.
- The length of a culvert should be based on roadway clear zone and embankment geometry.
- Severe or abrupt changes in channel alignment upstream or downstream of culverts are not recommended.
- Small culverts with no defined channel are placed normal to centerline.
- Large culverts perpetuating drainage in defined channels should be skewed as necessary to minimize channel relocation and erosion.
- Culvert location in both plan and profile should be investigated and designed to avoid sediment build-up in culvert barrels. Consider having the invert of one barrel lower than the others in multiple barrel crossings.
- At most locations, the culvert profile will approximate the natural stream profile. Exceptions can be considered to: arrest stream degradation by utilizing a drop inlet or broken back culvert; or improve hydraulic performance by utilizing a slope tapered inlet.
- Culverts shall be located and designed to present a minimum hazard to traffic and people. Full recovery distance is desirable without guardrail. When safety grates are required, the potential for flood damage caused by the grate plugged with debris or ice shall be assessed.
- Minimum and Maximum Overfill
  - Minimum overfill at the shoulder P.I. for reinforced concrete pipe (RCP) and corrugated steel pipe (CSP) on centerline culverts is 1.25 feet to the top of rigid pavement and 1.75 feet to the top of flexible pavement.
  - For precast box culverts fill heights of less than 2.0 feet require a distribution slab.
  - Maximum overfill is controlled by the load tables.

- Survey information used in culvert design shall include topographic features, channel characteristics, fish migration needs, highwater information if available, existing structures, and other related site specific information.
- Debris and Ice
  The potential for plugging with debris or ice shall be considered. The source of the debris or ice and the potential flood damage resulting from plugged culverts are important. Options available to the designer include: attempt to pass the debris or ice through the culvert usually by increasing the culvert height or the placement of relief openings (preferred alternative); retain the debris or ice upstream of the culvert (may require frequent maintenance); non flared sloped end sections allow the ice and debris to ride up the sloped end; use a bridge.
- Where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris, the following information should be considered prior to a decision whether to attempt to pass or retain the debris:
  - determine the type and quantity of debris;
  - experience of upstream or downstream culverts in passing or retaining the debris;
  - experience of the subject roadway passing debris at the sag point;
  - large floatable debris will usually ride up the culvert end sections;
  - available access for maintenance to remove debris from the culvert entrance or the debris barrier;
  - assessment of damage due to debris clogging, if protection is not provided;
  - feasibility of relief opening, either in the form of a vertical riser or a relief culvert placed higher in the embankment.
  - review the HEC-9, *Debris Control Structures* (FHWA, 1971).

5.2.3 Design Limitations

There are several criteria that place limitations on the design of a culvert: allowable headwater, channel tailwater relationship, confluence tailwater relationship, outlet velocity, minimum velocity, temporary upstream ponding and flood frequency.

- Allowable headwater is the depth of water that can be ponded at the upstream end of the culvert which will be limited by one or more of the following: be non-damaging to upstream property, be non-damaging to the roadway, meet stage increase criteria set forth by regulatory agencies, and should not cause disruption to traffic flow.
- Channel tailwater relationship requires the evaluation of the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges which include the design and review discharges.
  - Usually a single section analysis is adequate for culvert design. Backwater curves can be calculated to transfer the tailwater elevation from the cross section to the culvert site.
  - Utilize a step backwater method such as provided in computer applications to determine tailwater elevations at sensitive locations.
  - Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall. $(d_c + D)/2$ where d_c is the critical depth of flow in feet and D is the culvert diameter in feet.
  - Use the headwater elevation of any nearby, downstream culvert or other control structure if it is greater than the channel depth.
- Consider the confluence tailwater relationship.
  - Evaluate the high water elevation that has the same frequency as the design flood if events are known to occur concurrently and are statistically dependent.
  - If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth.
- The maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with: channel stabilization (Channel Chapter), energy dissipation (Energy Dissipator Chapter). In general, outlet velocities less than 6 feet per second will not require energy dissipation or protection.
- The culvert should be designed to maintain a minimum self cleaning velocity. Use 2.5 feet per second for mean annual flood, (2 year frequency) when streambed material size is not known.
- If storage is being contemplated upstream of the culvert in order to reduce the peak outflow through the culvert, consideration shall be given to:
  - limiting ponding in urban areas to non-sensitive locations;
  - limiting ponding in rural areas to non-crop producing locations;
  - limiting the total area of flooding;
  - maintaining storage volume by removing sediment as required; and
  - ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

- Design recommendations for flood frequency, See Hydrology Chapter for Additional Information.
  - Use 50 year design frequency for minor culverts 48" or less in diameter. The overtopping flood need not be computed. A more conservative design frequency (I.E. 100 year flood event) may be required if there is significant flood damage potential upstream.
  - Minimum overtopping flood frequency for risk assessment is based on projected average daily traffic (ADT)

Projected ADT	Minimum Overtopping Flood Frequency
0 - 10	2 year
11 - 49	5 year
50 - 399	10 year
400 - 1499	25 year
1500 and up	50 year

  - Risk assessment shall be completed for all major culverts greater then 48". The 500-year flood or overtopping flood shall be computed, whichever is less.

Road Classification	Size	Design Frequency
All Centerline	> 48 inches	Need Risk Assessment
All Centerline	≤ 48 inches	50 year
Median Drain	15 inch minimum	50 year
Entrance	15 inch minimum	10 year

5.2.4 Design Features

Basic design features and considerations which must be considered include: culvert size and shape, number of barrels, material selection, end treatment for both inlet and outlet, improved inlets, safety and performance curves.

- The culvert size and shape selected shall be based on engineering and economic criteria related to site conditions. All culverts should be designed to provide adequate hydraulic capacity. However land use requirements and debris or ice potential may dictate a larger or different barrel geometry than required for hydraulic design alone. The following minimum sizes shall be used to avoid maintenance problems and clogging:

Type of Road	Minimum Size
Trunk Highway Centerline	24 inches
CSAH Centerline	18 inches
Local Roads Centerline	18 inches
Ramps, Loops, Rest Area	18 inches
Side Culverts	15 inches
Median Drains	15 inches
Entrances	15 inches

- Multiple barrel culverts shall fit within the natural dominant channel, or with minor widening of the channel. Widening the channel at a culvert to allow multiple barrels typically leads to conveyance loss through sediment deposition in some of the barrels. Multiple barrels are to be avoided where the approach flow is high velocity, particularly if supercritical flow is expected. These sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects.
- Where fish passage is required, special treatment is necessary to insure adequate low flows. Commonly when there are multiple barrels one barrel is lowered.
- Barrel material selection is based on the material type that best fulfills all of the engineering requirements for a specific installation. The following factors shall be considered: hydraulic performance, structural stability, serviceability, economics based on design life of structure, and replacement cost and difficulty of construction.
- Abrasion and corrosion are also considered when determining serviceability requirements. The culvert design sheet shall provide documentation for each centerline pipe installation indicating the engineering considerations that dictate the selection of the specific type of pipe.

- An apron end section is a concrete or metal structure attached to the end of a culvert for purposes of appearance, anchorage, and stabilization of the embankment near the waterway. The culvert inlet type and associated entrance coefficient, k_e values are given in Table 5.1.

Table 5.1 Inlet and Outlet End Treatments

Standard Plate	Entrance Coefficient, k_e
3022	0.7
3100	0.5
3110	0.5
3114	0.5
3122	0.5
3123	0.5
3125	0.2
3126	0.7
3127	0.7
3128	0.7
3129	0.5
3148	0.7
Bridge Details Manual	0.5

- When the potential for vehicle impact exists at the culvert ends consider vehicle safety during end treatment design, since some end treatments can be hazardous to errant vehicles.
  - Culvert ends located outside the clear zone do not need safety aprons or grates.
  - Culvert ends located within the clear zone should be treated in accordance with Mn/DOT Road Design Manual Guidelines.
  - If a safety apron is installed on the downstream end of a culvert, a safety apron, grate or trash rack should also be installed at the upstream end of the culvert.
- Improved inlets are an option for long culverts which will operate under inlet control. While improved inlets can increase the hydraulic performance of the culvert, they may also add to the total culvert cost. Therefore, these inlets should only be used when necessary. Three types of improved inlets are available: apron inlet with reducer, side tapered inlet, and slope tapered inlet.
- Performance curves are developed for culverts > 48" to evaluate the hydraulic capacity and outlet velocity of a culvert for various headwater depths. These curves display the consequence of high flow rates at the site and provide a basis for evaluating flood hazard.

5.2.5 Related Designs

There are additional criteria for designs related to culverts.

<u>Buoyancy Protection</u>

Inlet protection is usually necessary to anchor the inlet end of the culvert and provide buoyancy protection for all flexible culverts. Buoyancy is affected by steepness of the culvert slope, depth of the potential headwater (debris blockage may increase), flatness of the upstream fill slope, height of the fill, large culvert skews, or mitered ends. The standard apron end section is considered adequate protection for corrugated metal culverts 12" through 84". Standard Plates 3128 and 3148 include anchorage that provide some buoyancy protection. Concrete headwalls such as shown in Standard Plates 3125, 3126, and 3127 shall be specified as inlet protection for structural plate culverts 60" or greater in diameter.

<u>Multiple Use Culverts</u>

Consideration may be given to combining drainage culverts with other land use requirements necessitating passage under a highway such as animal passes, boat traffic or pedestrian underpasses:

- during the selected design flood the land use is temporarily forfeited, but available during lesser floods,
- two or more barrels are required with one situated so as to be dry during floods less than the selected design flood,
- shall be sized so as to insure it can serve its intended land use function up to and including a 2-year flood, and
- the height and width constraints shall satisfy the hydraulic or land use requirements, whichever is the larger.

Outlet Protection

Protection against scour at culvert outlets ranges from riprap placement to complex and expensive energy dissipation devices. The most common energy dissipation devices used by Mn/DOT is the riprap apron. Other outlet protection alternatives are listed below, for more details see HEC-14 (FHWA, 1983).

- A ring dissipator is used where right of way area is limited, debris is not a problem, Fr is >1, a riprap apron is not adequate, and moderate velocity reduction is needed.
- A drop box inlet was initially a grade control structure developed by SCS. It can be used at the culvert inlet to flatten the grade of culvert and thereby reduce the velocity.
- A riprap basin is used where adequate right-of-way is available, the Froude Number (Fr) is less than 3.0, only a moderate amount of debris is present, adequate riprap is available, and other methods are not appropriate or are more expensive.
- An impact basin is used when little debris potential exists, design discharge is less than 150 cfs, no tailwater is required for successful operation, and is economically feasible.
- A stilling basin (SAF Basin) utilizes a hydraulic jump to dissipate energy. This option is an economical mean of dissipating large amounts of energy, but a tailwater is required and the Froude number must be between 1.7 and 17.
- For outlet velocities less then or equal to 6 fps, check shear stress to determine if vegetation will be adequate. If vegetation is used consider temporary erosion control during and immediately following construction until vegetation becomes established.
- Riprap Apron for minor culverts (equivalent diameter ≤ 48"), the riprap apron detailed in Std. Plates 3133 and 3134 will generally be adequate. Geotextile fabric may be substituted for granular filter. Use the following guidelines:

Outlet Velocity, V_o	Riprap Specifications	Filter Specifications
$0 < V_o \leq 6$ fps	Riprap or stable vegetation	
$6 < V_o \leq 8$ fps	12" Class II riprap	6" granular or geotextile filter
$8 < V_o \leq 10$ fps	18" Class III riprap	9" granular or geotextile filter
$10 < V_o \leq 12$ fps	24" Class IV riprap	12" granular or geotextile filter
$V_o > 12$ fps	Consider other energy dissipator	

Improved Inlets

Culverts operating under inlet control generally flow part full. In many cases it is possible to use a smaller pipe having about the same cross sectional area as the area of flow by using special inlets such as the side tapered or slope tapered inlets. These types of inlets work well for box culverts and round culverts but do require special design and fabrication. Another method which utilizes suppliers' standard materials is also available and has been used successfully by Mn/DOT for many years. This method sizes the inlet by conventional means utilizing the inlet control nomographs. Then the size and location of the reducer is calculated in order to minimize the size of the culvert for the balance of the length. Two significant factors then need to be considered: the amount of reduction, and the location of the reducer.

The allowable amount of diameter reduction is determined by comparing critical depth of the smaller pipe with its diameter. Too much constriction of the flow area can cause a choking effect, creating orifice flow at the reducer throat, by sealing off the throat entrance to the smaller pipe with standing waves and other losses in the reducer.

The longitudinal location of the reducer is based on the assumption that the velocity of flow entering the reducer should be equal or greater than the critical velocity in the smaller pipe. Starting with the critical depth, the standard incremental energy equation is used in determining the length of the larger pipe and its associated velocity. The velocity used to determine the total length of the pipe, should be 10% greater than the critical velocity of the smaller pipe to help overcome any other losses that are developed by the flow contraction. Since the depth of flow crosses critical depth in the larger pipe at an approximate distance of 0.5 times the diameter from the inlet of the larger pipe, this length should be added to the total length of the pipe.

5.2.6 Design Methods

The designer has several computational methods to choose from when designing a culvert. These methods include: nomographs, hand calculations or computer applications. Computer applications are most commonly used and have the advantage of being able to rapidly perform iterations and compare different designs. Computer applications solve equations and give answers, but it does not mean the answers are correct. The designer still needs to understand culvert design and be able to assess the reasonableness of the solution. Occasional hand or nomograph computations are a good way to perform a quick check of computer results.

Culverts are designed by assuming either a constant discharge (peak flow) or routing a hydrograph. Constant discharge is utilized for most culvert designs. The analysis is performed using the peak discharge, however a range of discharges and a performance curve is recommended for the major culverts (diameter > 48"). Using a constant discharge will yield a conservatively sized structure where temporary storage is available, but is not considered in the design. Acceptable methods are detailed in the Hydrology Chapter and include: Regression Equations developed by USGS for ungaged streams, Log Pearson III analysis for gaged streams, SCS Method, and Rational Method for drainage areas less than 200 acres.

Flood routihg through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. Flood routing requires synthesis of a hydrograph and should be used when significant storage upstream will reduce the required culvert size, or storage capacity behind a highway embankment attenuates a flood hydrograph and reduces the peak discharge. When a culvert is initially down sized to take advantage of potential storage the designer is placing an obstruction in the drainage way. This should only be considered in locations that will not damage crops or other property. Control of the upstream right of way by purchase or easement may be necessary. Flood routing equations and procedures are detailed in the Storage Facilities chapter.

5.3 CULVERT ANALYSIS

An exact analysis of culvert flow is complex because the following are required:

- analyzing nonuniform flow with regions of both gradually varying and rapidly varying flow,
- determining how the flow type changes as the flow rate and tailwater elevations changes,
- applying backwater and drawdown calculations, energy and momentum balance,
- applying the results of hydraulic model studies, and
- determining if hydraulic jumps occur, and if they are inside or downstream of the culvert barrel.

The procedures in this chapter utilize the concepts of minimum performance and control sections. The concept of minimum performance means that although the culvert may operate more efficiently at times, (more flow for a given headwater level), it will never operate at a lower level of performance than calculated. Minimum performance is assumed by analyzing both inlet and outlet control and using the highest headwater.

The control section is the location where there is a unique relationship between the flow rate and the upstream depth of water. Inlet control is governed by the inlet geometry which includes the barrel shape, cross-sectional area, and inlet edge. Outlet control is governed by a combination of the culvert inlet geometry, the barrel characteristics, and the tailwater.

5.3.1 Inlet Control

For inlet control, the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet, either inside or outside of the pipe.

Headwater depth is measured from the inlet invert to the surface of the upstream pool. Inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area. Inlet edge configuration describes the entrance type. Some typical inlet edge configurations are apron inlet, beveled edge, mitered to conform to slope, socket end, square edge in a headwall, and thin edge projecting. Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical and arch. Check for an additional control section, if the geometry varies, such as in an improved inlet.

Three regions of flow are shown in the Figure 5.1, unsubmerged, transition and submerged.

<u>Unsubmerged</u>

For headwater below the inlet crown, the entrance operates as a weir. A weir is a flow control section where the upstream water surface elevation can be predicted for a given flow rate. The relationship between flow and water surface elevation must be determined by model tests of the weir geometry or by measuring prototype discharges. These tests are then used to develop equations. Appendix A of HDS-5 (FHWA, 1985) contains the equations which were developed from model test data. Figure 5.2 shows an unsubmerged inlet in inlet control.

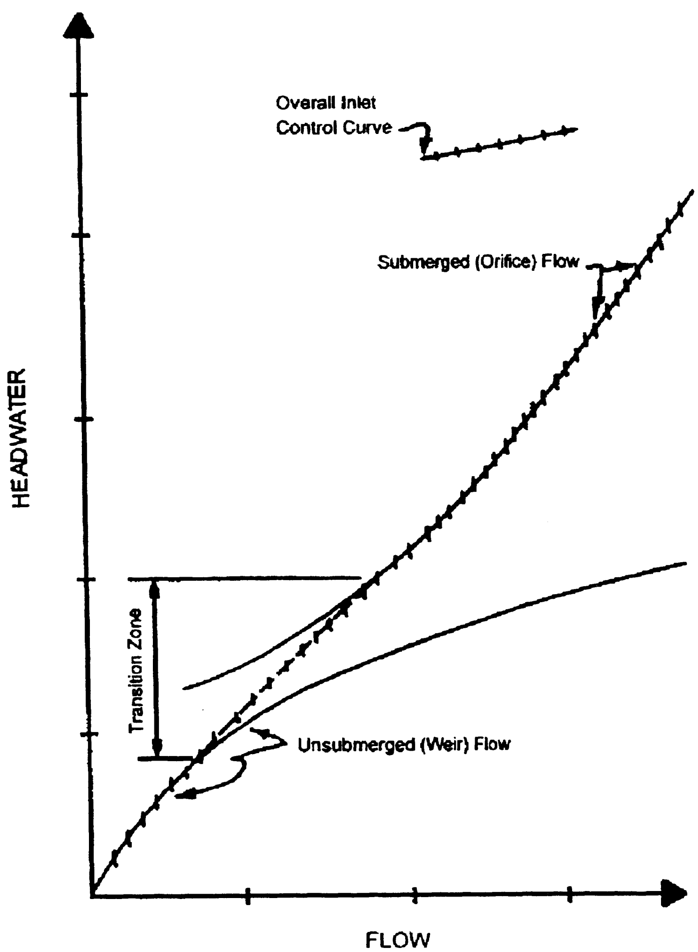


Figure 5.1 Unsubmerged, Transition and Submerged Flow
Source: HDS-5, Figure III-4 (FHWA, 1985)

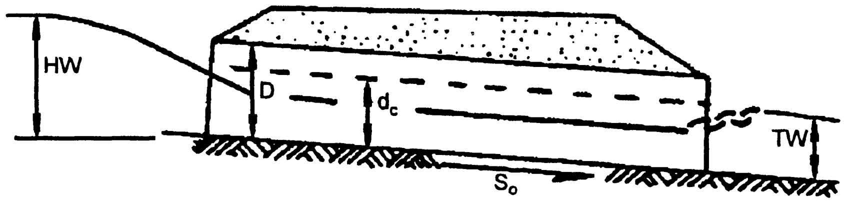


Figure 5.2 Unsubmerged Inlet in Inlet Control

Submerged

For headwaters above the inlet, the culvert operates as an orifice. An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section. The relationship between flow and headwater can be defined based on results from model tests. Appendix A of HDS-5 (FHWA, 1985) contains flow equations which were developed from model test data. Figure 5.3 shows a submerged inlet under inlet control.

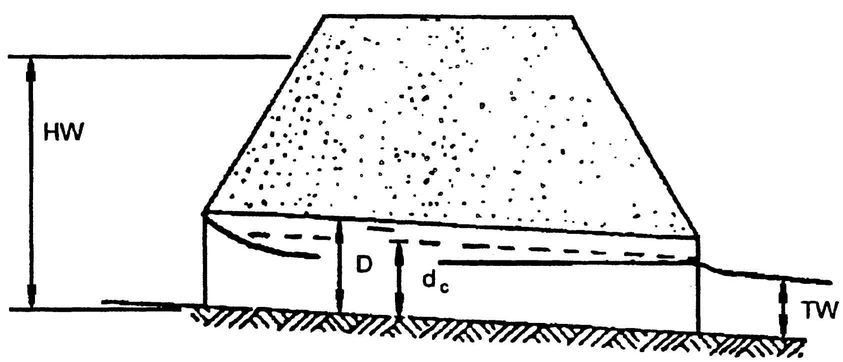


Figure 5.3 Submerged Inlet in Inlet Control

Transition Zone

The transition zone is located between the unsubmerged and the submerged flow conditions where the flow is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves. (Figure 5.1)

Nomographs

The inlet control flow versus headwater curves which are established using the above procedure are the basis for constructing the inlet control design nomographs. Note that in the inlet control nomographs, headwater (HW) is measured to the total upstream energy grade line including the approach velocity head. Culvert Design nomographs are provided in Appendix C.

5.3.2 Outlet Control

Outlet control has depths and velocity which are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). Tailwater (TW) is based on the downstream water surface elevation and is assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation. In a given culvert, the type of flow is dependent on all of the barrel factors: barrel roughness, barrel area, barrel length and barrel slope. The inlet control factors also influence culverts in outlet control.

In the analysis of outlet control hydraulics, full flow condition is assumed when TW depth is above the crown of the culvert. The outlet control nomographs are accurate for the analysis of full flow condition. Partial full condition is used when TW is below the crown of the culvert. A backwater calculation is necessary to accurately analyze partial flow conditions, however an approximate method utilizing $TW = (d_c+D)/2$ yields reasonable results, especially with pipes less than 48" diameter. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool.

Velocity

Velocity is computed by rearranging the continuity equation. Keep in mind that unless a round or arch pipe is full, the formula for determining area used in this equation is complex. The alternative is to use the design charts provided in Appendix C.

$$V = \frac{Q}{A} \tag{5.1}$$

Where: V = velocity (fps)
Q = flow rate (cfs)
A = cross sectional area of the flow area in the barrel (ft^2)

Head Losses

$$H_L = H_e + H_f + H_o + H_b + H_j + H_g \tag{5.2a}$$

Where:
g = acceleration due to gravity (32.2 ft/s^2)
H_b = bend losses (ft)
H_e = entrance loss (ft)
H_f = friction losses (ft)
H_g = losses at grates (ft)
H_j = junction losses (ft)
H_L = total energy head loss (ft)
H_o = exit loss (ft)
H_v = full flow velocity head (ft)
k_e = entrance loss coefficient (Table 5.2)
L = length of the culvert barrel (ft)
n = Manning's roughness coefficient (Table 5.3)
P = wetted perimeter of the barrel (ft)
R = hydraulic radius of the full culvert barrel (ft)
R = area/wetted parameter
V = average barrel velocity (ft/s)
V_d = channel velocity downstream of the culvert (ft/s)

Entrance Loss

$$H_e = k_e\left(\frac{V^2}{2g}\right) \tag{5.2b}$$

Friction Loss

$$H_f = \left(\frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \tag{5.2c}$$

Velocity Head

$$H_v = \frac{V^2}{2g} \tag{5.2d}$$

Exit Loss

$$H_o = 1.0\left[\left(\frac{V^2}{2g}\right) - \left(\frac{V_d^2}{2g}\right)\right] \tag{5.2e}$$

V_d is usually neglected, then:

$$H_o = H_v = \frac{V^2}{2g} \tag{5.2f}$$

Barrel Losses

$$H = H_e + H_o + H_f$$

Substituting in equations:

$$H = \left[1 + k_e + \left(\frac{29n^2L}{R^{1.33}}\right)\right]\left[\frac{V^2}{2g}\right] \tag{5.3}$$

Table 5.2 Entrance Loss Coefficients, k_e for Outlet Control, Full or Partly Full

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

[1] "End section conforming to fill slope" refers to the sections commonly available from manufacturers and may be made of either metal or concrete. From limited hydraulic tests they are equivalent in operation to a headwall under either inlet or outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.
Source: HDS-5 (FHWA, 1985)

Table 5.3 Mannings "n" Values for Culverts

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	Smooth	0.010 - 0.011
Concrete Box	Smooth	0.012 - 0.015
Spiral Rib Metal Pipe	Smooth	0.012 - 0.013
Corrugated Metal Pipes, Pipe-Arch and Box (Annular or Helical Corrugations, Manning's n varies with barrel size)	2⅔ by ½ inch Annular	0.022 - 0.027
	2⅔ by ½ inch Helical	0.011 - 0.023
	6 by 1 inch	0.022 - 0.025
	5 by 1 inch	0.025 - 0.026
	3 by 1 inch	0.027 - 0.028
	6 by 2 inch Structural Plate	0.033 - 0.035
	9 by 2 ½ inch Structural Plate	0.033 - 0.037
Corrugated Polyethylene	Smooth	0.009 - 0.015
Corrugated Polyethylene	Corrugated	0.018 - 0.025
Polyvinyl Chloride (PVC)	Smooth	0.009 - 0.011

The Manning's n values indicated in this table were obtained in the laboratory. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection and joint conditions.
Source: excerpt from Table 4 HDS-5 (FHWA, 1985, reprinted1998)

Stage Increase
The stage increase is the increase in water surface due to the constriction of the culvert. It is computed as the difference between the water surface elevation prior to installation and the head water just upstream of the culvert. Headwater must be measured sufficiently distant from the pipe to exclude local drawdown effects.

$$S.I. = HW - \left(TW + LS_o\right) \tag{5.4}$$

Energy Grade Line
The energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 5.4, the following relationship results:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L \tag{5.5}$$

Where: HW_o = headwater depth above the outlet invert (ft)
V_u = approach velocity (ft/s)
TW = tailwater depth above the outlet invert (ft)
V_d = downstream velocity (ft/s)
H_L = sum of all losses (Equation 5.2a)
L = Length of culvert barrel (ft)
S_o = slope of culvert invert (ft/ft)

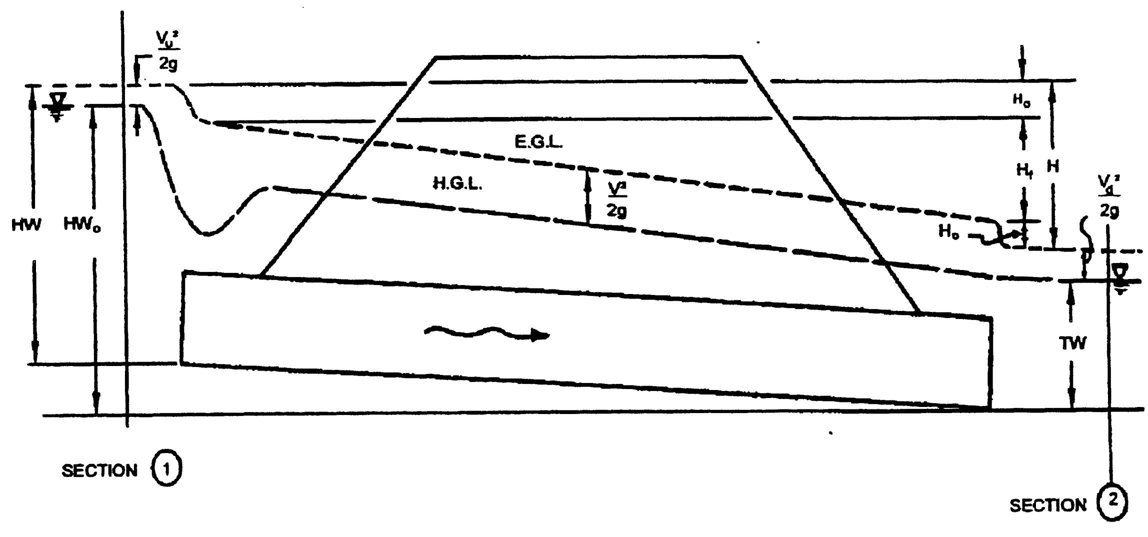


Figure 5.4 Full Flow Energy and Hydraulic Gradelines

Hydraulic Grade Line
The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel lines separated by the velocity head except at the inlet and the outlet.

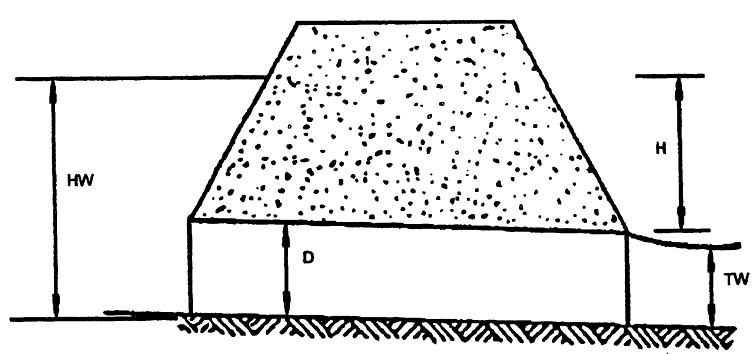


Figure 5.5 Flowing Full in Outlet Control, Where d_c > D

Nomographs (Full Flow)
The nomographs were developed assuming that the culvert barrel is flowing full. Two flow conditions occur in outlet control when the barrel is flowing full: TW > D, Flow Type IV (see Figure 5.4) or $d_c > D$, Flow Type VI (see Figure 5.5)
V_u is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert. V_d is small and its velocity head can be neglected. Equation 5.5 becomes:

$$HW = TW + H - S_o L \tag{5.6}$$

Where: HW = depth from the inlet invert to the energy grade line (ft)
H = is the value read from the nomographs (ft) or Equation 5.3
S_oL = drop from inlet to outlet invert (ft)

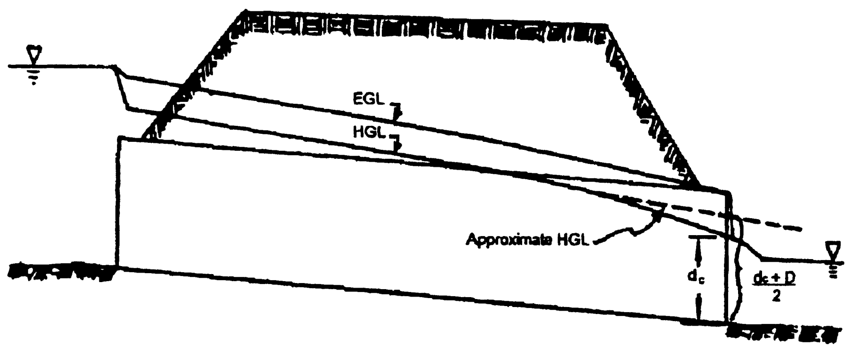


Figure 5.6 Effectively Full Flow in Outlet Control, Where TW < d_c

Nomographs (Partly Full Flow)
Equations 5.1 through 5.5 were developed for full barrel flow. The equations also apply to the flow situations which are effectively full flow conditions, when TW < d_c, (see Figure 5.6).

When the culvert barrel is partly full, backwater calculations may be required which begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel, full flow extends from that point upstream to the culvert entrance.

Based on model studies and numerous backwater calculations performed by the FHWA staff, it was found that the hydraulic grade line pierces the plane of the culvert outlet at a point one-half way between critical depth and the top of the barrel or $(d_c + D)/2$ above the outlet invert. TW should be used if higher than $(d_c + D)/2$. The following equation should be used for culverts that are partially full when determining the Headwater (HW) for outlet control:

$$HW = h_o + H - S_o L \tag{5.7}$$

Where: h_o = the larger of TW or $(d_c + D)/2$ (ft)

Adequate results are obtained down to a HW = 0.75D. For lower headwaters and major culverts, backwater calculations are recommended. Figures 5.7 and 5.8 show partially full flow conditions under outlet control with TW < d_c and TW > d_c respectively.

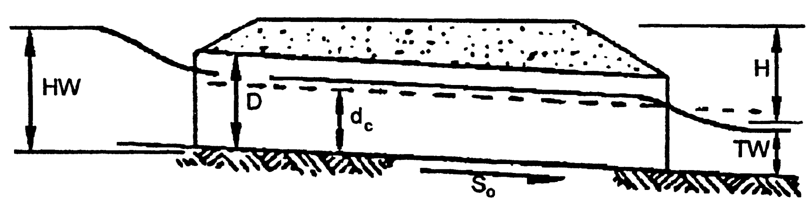


Figure 5.7 Partly Full Flow in Outlet Control, Where TW < d_c

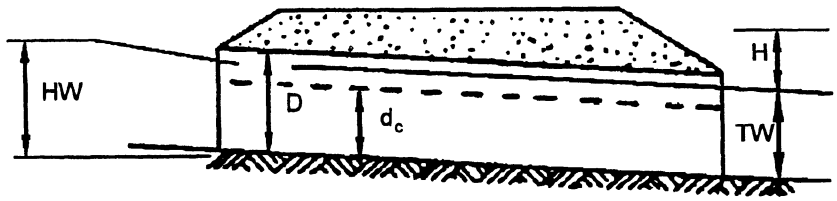


Figure 5.8 Partly Full Flow in Outlet Control ,Where TW > d_c

5.3.3 Outlet Velocity

Culvert outlet velocities shall be calculated to determine the need for erosion protection at the culvert exit. Since the culvert outlet velocity is usually higher than the natural stream velocity, energy dissipation may be necessary to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed. Outlet velocities for inlet and outlet control are computed as follows:

Inlet Control

- If water surface profile calculations are necessary, begin at d_c at the entrance and proceed downstream to the exit. Determine at the exit the depth and flow area. The velocity is calculated from Equation 5.1. While water surface profiles can be computed by hand, typically when water surface profile calculations are necessary a computer application will be selected to preform the iterative computations.
- Use normal depth and velocity. This approximation may be used since the water surface profile converges towards normal depth if the culvert is of adequate length. The normal depth velocity may be higher than the actual velocity at the outlet determined from running a water surface profile. Normal depths and velocities may be obtained from the open channel flow charts for circular pipe provided in Appendix C.

Outlet Control

The cross sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit.

- Critical depth is used when the tailwater is less than critical depth.
- Tailwater depth is used when tailwater is greater than critical depth, but below the top of the barrel.
- The total barrel area is used when the tailwater exceeds the top of the barrel.

5.3.4 Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir. Flow coefficients for flow overtopping roadway embankments are found Figure 5.9.

$$Q_r = C_d L \left(HW_r\right)^{1.5} \tag{5.8}$$

Where: Q_r = overtopping flow rate (cfs)
C_d = overtopping discharge coefficient = $k_t\ C_r$
k_t = submergence coefficient
C_r = discharge coefficient
L = length of the roadway crest (ft)
HW_r = the upstream depth, measured above the roadway crest (ft)

The length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth of the upstream pool above the roadway. The length is difficult to determine when the crest is defined by a roadway sag vertical curve. The length should be subdivided into a series of segments. The flow over each segment is calculated for a given headwater. The flows for each segment are added together to determine the total flow.

Total flow is calculated for a given upstream water surface elevation by adding the roadway overflow plus culvert flow. Performance curves for the culvert and the road overflow may be summed to yield an overall performance curve. A trial and error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway. Assume HW_r and solve for Q_r, then solve for Q through the culvert by computing HW-TW and solving for V and then Q. Summation of the Q's must balance with the total Q.

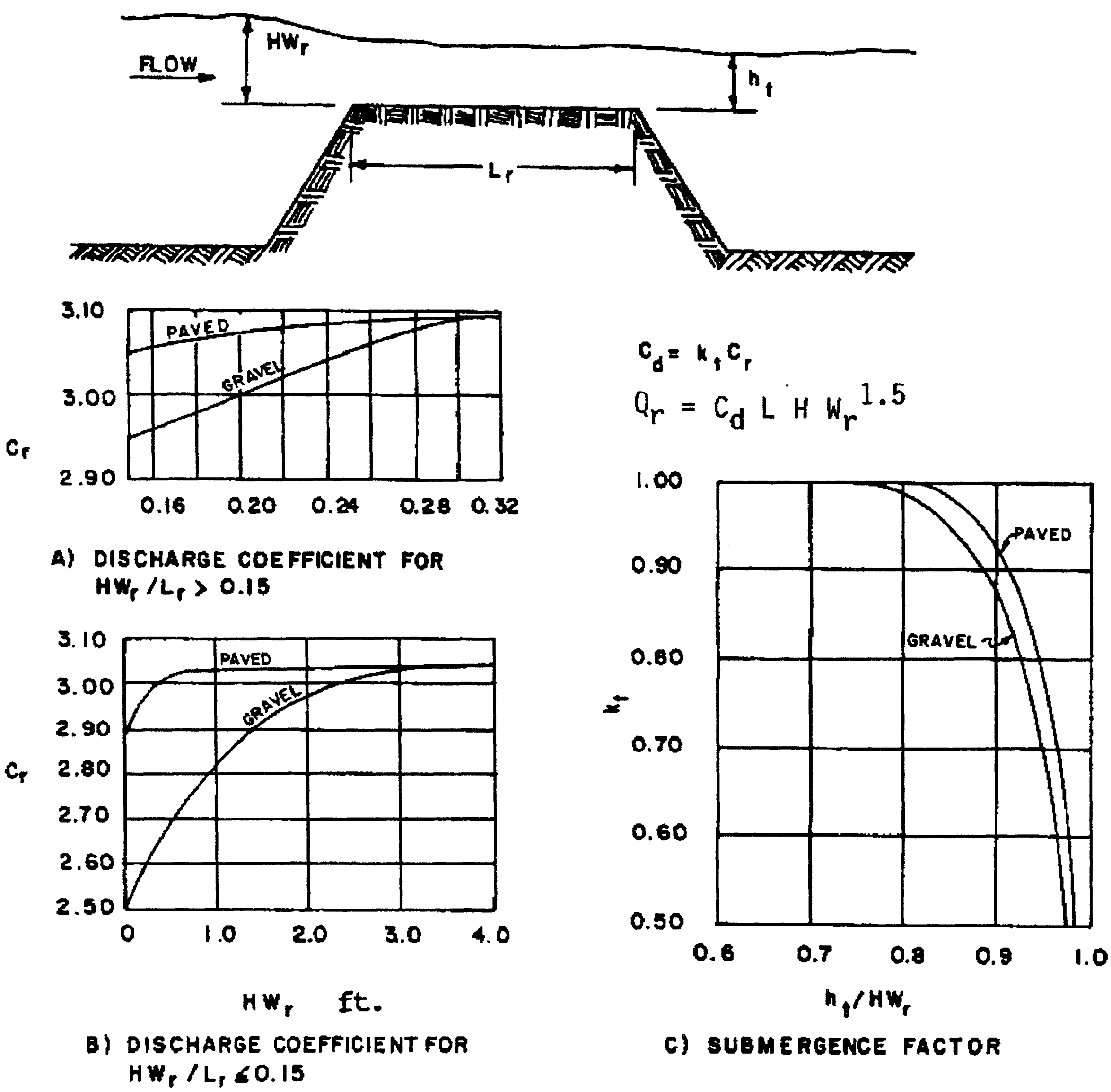


Figure 5.9 Discharge Coefficients for Roadway Overtopping
Source: HDS-5, Figure III-11 (FHWA, 1985)

5.3.5 Performance Curves

A performance curve is a plot of flow rate versus headwater depth or elevation. The culvert performance curve is made up of the controlling portions of the inlet, outlet and roadway overtopping performance curves (See Figure 5.10). The overall performance curve is the sum of the flow through the culvert and the flow across the roadway and can be determined by performing the following steps.

Step 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.

Step 2 Combine the inlet and outlet control performance curves to define a single performance curve for the culvert

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Equation 5.8 to calculate flow rates across the roadway.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve similar to the one shown in Figure 5.10.

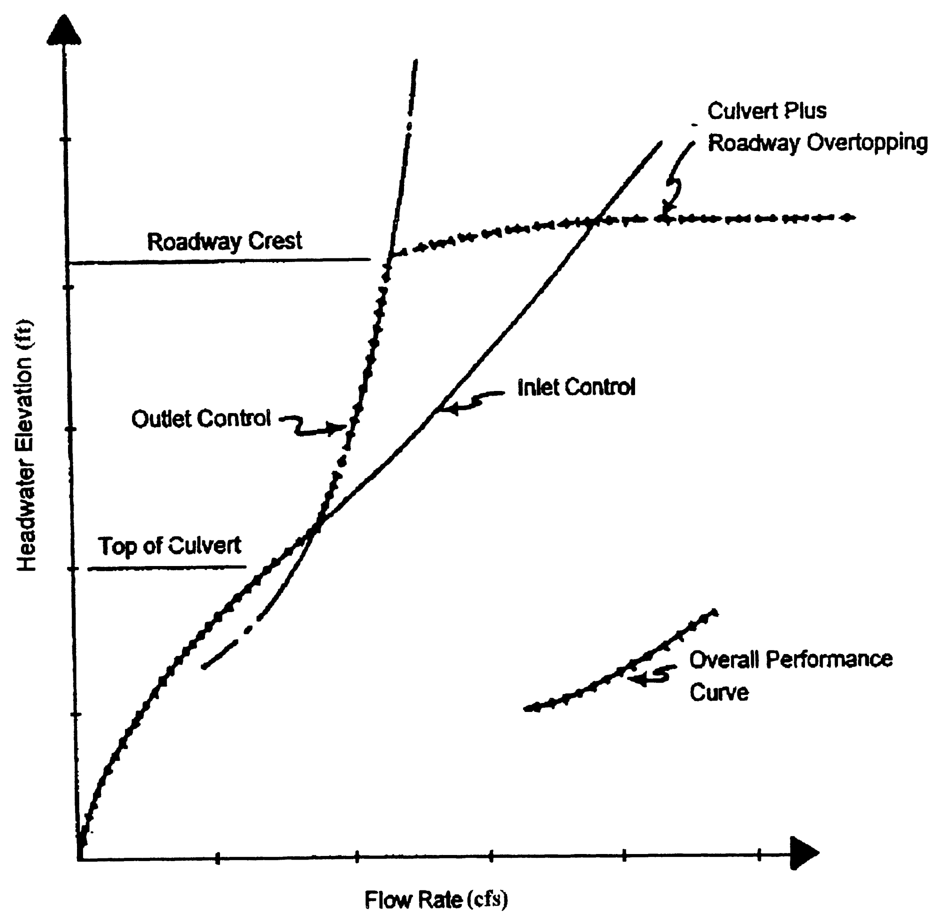


Figure 5.10 Overall Performance Curve
Source: HDS-5, Figure III-16 (FHWA, 1985)

5.4 DESIGN PROCEDURE

The following design procedure provides a convenient and organized method for designing culverts for a constant discharge, considering inlet and outlet control. The procedure does not address the affect of storage which is discussed in the Storage Chapter.

- The designer should be familiar with all the equations in Section 5.5 before using these procedures. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.
- The computation form has been provided in Figure 5.10 to guide the user. It contains blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected, and comments.
- The culvert design procedure adopted by Mn/DOT is less rigorous for culverts 48" diameter and less, than for greater than 48".

Step 1 **Determine Scope of Culvert Design**

Minor Culvert Designs (48" and less)
- Design Frequency 50 year
- Hydraulic Data on plans not necessary
- DNR Permits generally not necessary
- Overtopping calculations generally not necessary
- Tailwater (TW) elevation can be estimated from known condition or $(d_c+D)/2$
- Nomographs usually adequate for design
- Specify culvert material
- Determine pipe class or gage and bedding
- Estimate outlet velocity using pipe flow charts
- Design outlet protection
- Documentation is Culvert Design Sheet

Major Culvert Designs (> 48")
- Risk Assessment necessary
- Hydraulic Data on plans
- Permit usually required using Basic Flood (100 year frequency)
- Overtopping or Q_{500} necessary
- Compute tailwater (TW) (See Channel Chapter)
- Water surface profile necessary for outlet control with partial full flow
- Nomograph solution is OK for full flow conditions
- Computer method preferred for design
- Performance curve required
- Investigate opportunity for improved inlet if inlet control
- Compute outlet velocity
- Design outlet protection where necessary
- Specify culvert material
- Documentation includes risk assessment, computer run, performance curve and material selection

Step 2 **Assemble Site Data and Project File**

Table 5.4 Data Needs

TYPE OF DATA	MAJOR CULVERTS	MINOR CULVERTS
MAPS	USGS Quadrangle Maps Site and Location Maps Drainage Area Maps	Drainage Area Maps
PHOTOGRAPHS	Aerial and land photographs	Aerial and land photographs
PLANS	Existing and Proposed Roadway Plans and Profiles Embankment Cross Sections Inplace Structures	Existing and Proposed Roadway Plans and Profiles Embankment Cross Sections Inplace Structures
SURVEYS	Stream profile and cross sections Elevations of inplace structure Elevations of flood prone property Maximum observed highwater elevations Ordinary highwater if feasible	Stream profile Channel cross-section if confined channel Flood damage potential
REVIEW FILES	Design data for nearby structures Previous recommendations Highwater or flood data Maintenance problems	Previous recommendations Flooding or Maintenance Problems
STUDIES BY OTHERS	Flood Insurance Studies (DNR) Watershed Districts Overall Plan Water Planning Organizations Water Quality Studies (MPCA) Flood retention (SCS,COE,Counties) Future development (Cities & Counties) Public Ditch Authorities Land use	Any applicable studies Land use plans Future development plans
ENVIRONMENTAL CONSTRAINTS	Boat Passage (DNR) Fish Migration (DNR) Wild Life Passage (DNR) Domestic Animal Passage Wetland Mitigation Protected Waters (DNR) Right of Way Limitations Commitments in review Documents Elevation of Flood Prone Property	Fish migration (DNR) Protected waters (DNR) Wetland mitigation Elevation of flood prone property Other as available

Step 3 **Determine Hydrology**

A. See Hydrology Chapter.
B. Use procedure for major or minor culvert.

Step 4 **Select Design Discharge Q_d**

A. See Section 5.2.3 Design Limitations.
B. Determine flood frequency from criteria.
C. Determine Q from appropriate procedure.
D. Prorate Q to each barrel if more than one.

Step 5 **Compute Tailwater Elevation**

A. See Channel Chapter
B. Minimum data are cross section, "n" values, and slope of channel to compute the rating curve for channel.

Step 6 **Summarize Data On Design Form**

A. See Culvert Design Form (Figure 5.11).
B. Use data from steps 1-5.

Step 7 **Select Design Alternative**

See Section 5.2.4 Design Features.
Choose trial culvert material, shape, size, and entrance type.

Step 8 **Determine Inlet Control Headwater Depth (HW_i)**

Use the inlet control nomograph (Appendix C).

A. Use the appropriate nomographs for the material types and shapes being considered. The nomographs are self explanatory.
B. Calculate headwater depth (HW_i).
   - Multiply HW/D by D to obtain HW to energy gradeline.
   - For minor culverts neglect the approach velocity $HW_i = HW$.
   - For major culverts in confined channels include the approach velocity $HW_i = HW$ - approach velocity head.

Step 9 **Determine Outlet Control Headwater Depth At Inlet (HW_{oi})**

A. Compare tailwater depth calculated in Step 5 with the rise of the culvert. If tailwater (TW) ≥ to rise, nomographs will provide accurate results. Skip to step "E". If TW < D, a computer analysis is recommended for an exact solution for major culverts; nomographs will give approximate solution for minor culverts.
B. Calculate critical depth (d_c) using appropriate chart in Appendix C. Note that d_c cannot exceed D.
C. Calculate $(d_c + D)/2$.
D. Determine (h_o). h_o = the larger of TW or $(d_c + D)/2$.
E. Determine (k_e). k_e = entrance loss coefficient, Table 5.2.
F. Determine losses through the culvert barrel (H).
   Use nomograph (Appendix C) or Equation 5.3b if outside range.
   Locate culvert length (L) or (L_1):
   The nomographs can be used for different values of "n" by modifying the culvert length as follows:

$$L_1 = L\left(\frac{n_1}{n}\right)^2 \qquad (5.9)$$

Where: L_1 = adjusted culvert length, ft
L = actual culvert length, ft
n_1 = desired Manning n value
n = Manning n value on chart

G. Calculate outlet control headwater depth (HW_{oi}).
   - Use Equation 5.10, if the approach velocity (V_u) and the downstream velocity (V_d) are neglected:

$$HW_{oi} = H + h_o - S_o L \qquad (5.10)$$

   - use Equation 5.2a, 5.2e and 5.5 to include V_u and V_d.
   - If HW_{oi} is less than 1.2D and control is outlet control:
     - the barrel may flow partly full,
     - the approximate method of using the greater of tailwater or $(d_c + D)/2$ may not be applicable,
     - backwater profile calculations should be used to check the result,
     - if the headwater depth falls below 0.75D, the nomograph method shall not be used for major culverts.

Step 10 **Determine Controlling Headwater (H_{wc})**

- Compare HW_i and HW_{oi}, use the higher.
- Compare HW_c with allowable HW and adjust culvert size if necessary.

Step 11 **Compute Discharge Over The Roadway (Q_r) (Major Culverts when Appropriate)**

A. Assume the upstream depth over the roadway (HW_r), calculate length of roadway crest (L), and calculate the overtopping flowrate (Q_r). See Section 5.3.4 and Equation 5.8.

B. Calculate the flow in the culvert by using the equations in Section 5.3.2 and solving for V and then Q.

Step 12 **Compute Total Discharge (Q_t)**

Sum the flow over the road (Q_r) and the flow in the culvert (Q_c). This sum should equal the total flow (Q_t). If not, assume a new HW_r and make another iteration.

Step 13 **Calculate Outlet Velocity (V_o) And Depth (d_n) See Section 5.3.4**

If inlet control is the controlling headwater:

A. Calculate flow depth at culvert exit.
   - use normal depth (d_n)
   - use water surface profile

B. Calculate flow area (A).

C. Calculate exit velocity (V_o) = Q/A.

If outlet control is the controlling headwater:

A. Calculate flow depth at culvert exit.
   - use (d_c) if d_c > TW for minor culverts
   - use (d_c + TW/2) if d_c > TW for major culverts
   - use (TW) if d_c < TW < D
   - use (D) if D < TW

   where: TW = tailwater
   d_c = critical depth
   D = Pipe diameter (height)

B. Calculate flow area (A).

C. Calculate exit velocity (V_o) = Q/A.

Step 14 **Review Results**

Compare alternative design with constraints and assumptions. If any of the following are exceeded, repeat Steps 4 through 13:

- the barrel must have adequate cover, (See Site Criteria Section 5.2.2)
- the length shall be reasonably accurate,
- the proper end treatment is used, (See Design Features Section 5.2.4)
- the allowable headwater shall not be exceeded, and
- the allowable overtopping flood frequency shall not be exceeded.

Step 15 **Plot Performance Curve (Major Culverts Only)**

A. Repeat steps 4 through 13 with a range of discharges which include Q_{design}, Q_{100}, & Q_{OT} or Q_{500}.

B. Use the following upper limit for discharge:
   - Q_{100} if $Q_{OT} \leq Q_{100}$
   - Q_{500} if $Q_{OT} > Q_{500}$

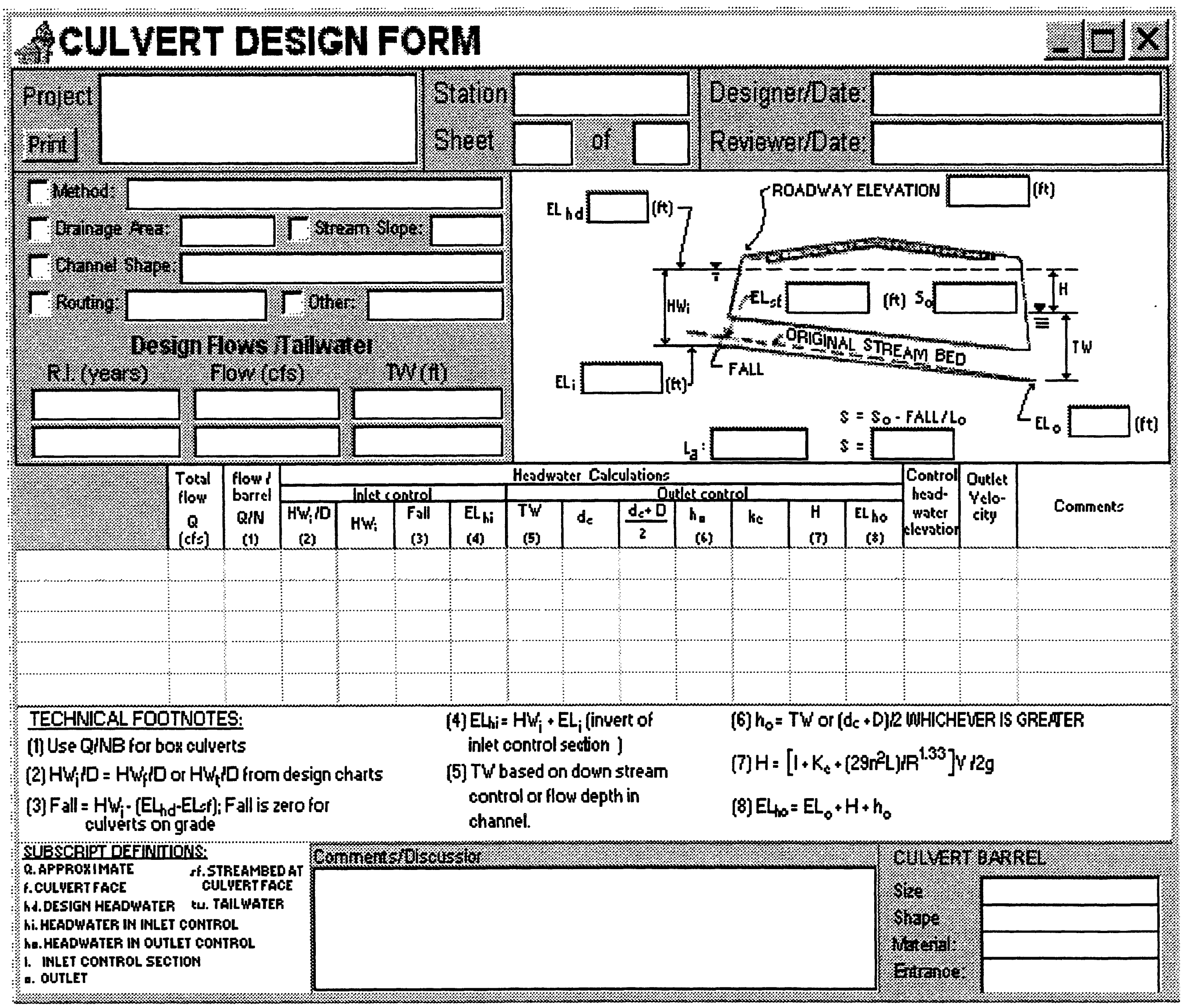
Step 16 **Related Designs**

Consider the following options (See Section 5.2.5).

- Tapered inlet or larger inlet with reducer if culvert is in inlet control, very long or has limited available headwater.
- Flow routing if a large upstream headwater pool exists (Storage Facilities Chapter).
- Energy dissipators if V_o exceeds design criteria. (See Energy Dissipator Chapter).
- Sediment control storage for sites with sediment concerns such as alluvial fans .
- Fishery passage (Consult with DNR).

Step 17 **Documentation**

- Culvert Design Form (Minor Culverts)
- Design Computations and letter of recommendation. (Major Culverts)
- Risk Assessment (Major Culverts)



CULVERT DESIGN FORM

Project | Station | Designer/Date:
Print | Sheet of | Reviewer/Date:

Method:
Drainage Area: Stream Slope:
Channel Shape:
Routing: Other:

Design Flows /Tailwater

R.I. (years)	Flow (cfs)	TW (ft)

	Total flow Q (cfs)	flow / barrel Q/N (1)	Headwater Calculations: Inlet control HWi/D (2)	HWi	Fall (3)	ELhi (4)	Outlet control TW (5)	dc	(dc+D)/2	ho (6)	ke	H (7)	EL ho (8)	Control head-water elevation	Outlet Velo-city	Comments

TECHNICAL FOOTNOTES:

(1) Use Q/NB for box culverts

(2) $HW_i/D = HW_f/D$ or HW_t/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) $H = [1 + K_e + (29n^2L)/R^{1.33}]V^2/2g$

(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

Size:
Shape:
Material:
Entrance:

Figure 5.11 Culvert Design Form
Source: Pallas, Inc., 1996

5.5 REFERENCES

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Federal Highway Administration (FHWA), 1973. *Design Charts For Open Channel Flow.* Hydraulic Design Series No. 3 (HDS-3), Hydraulics Branch, Bridge Division, Office of Engineering, Washington, D.C. 20590.

Federal Highway Administration (FHWA), 1983. *Hydraulic Design of Energy Dissipators for Culverts and Channels.* Hydraulic Engineer Circular No. 14 (HEC-14), Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590.

Federal Highway Administration (FHWA), 1985, reprinted 1998. *Hydraulic Design of Highway Culverts.* Hydraulic Design Series No. 5 (HDS-5), FHWA-IP-85-15, Washington, D.C.

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Chapter 6 ENERGY DISSIPATOR

6.1 INTRODUCTION

The failure or damage of many culverts and detention basin outlet structures can be traced to unchecked erosion. Erosive forces which are at work in the natural drainage network are often exacerbated by the construction of a highway or by other urban development. Interception and concentration of overland flow and constriction of natural waterways may result in increased erosion potential. To protect the culvert and adjacent areas, it is sometimes necessary to employ an energy dissipator.

Throughout the selection and design process, the designer should keep in mind that the primary objective is to protect the highway structure and adjacent area from excessive damage due to erosion. One way to help accomplish this objective is to return the flow to the downstream channel in a condition which approximates the natural flow regime. This also implies guarding against over design or employing dissipation devices which reduce flow conditions substantially below the natural or normal channel conditions.

This chapter provides design procedures which are based on FHWA Hydraulic Engineering Circular Number 14 (HEC-14) *Hydraulic Design of Energy Dissipators for Culverts and Channels*, September 1983, revised in 1995.

6.1.1 Definition

An energy dissipator is any device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits. In this chapter, the terms internal and external are used to indicate the location of the dissipator in relationship to the culvert. An external dissipator is located outside of the culvert and an internal dissipator is located within the culvert barrel.

6.1.2 Concept Definitions

Brink Depth	Depth of flow at the outlet of a pipe.
Equivalent Depth (d_E)	Equivalent depth is an artificial depth which is calculated for culverts which are not rectangular so that a reasonable Fr can be determined.
External Dissipator	An external dissipator is located outside of the culvert.
Froude Number (Fr)	The Froude number is a flow parameter.
Internal Dissipator	An internal dissipator is located within the culvert barrel.
Mean Riprap Diameter (d_{50})	Average rock diameter of armament material. The d_{50} is frequently used to specify the size of the rock.

6.2 DESIGN CRITERIA

Following sections include criteria for application of various energy dissipators. Detailed information is provided for the most widely used energy dissipators in later sections. Additional information on energy dissipators including procedures and sample calculations are provided in HEC-14 (FHWA, 1995).

6.2.1 Natural Scour Holes

Erosion occurring at the end downstream end of a hydraulic structure will sometimes form a natural scour hole which will dissipate energy and reduce velocity. Design procedures and equations are available for both cohesive and cohesionless materials. Investigations indicate that the scour hole geometry varies with tailwater conditions with the maximum scour geometry occurring at tailwater depths less than half the culvert height and that the maximum depth of scour (h_s) occurs at a location approximately 0.4 L_s downstream of the culvert outlet where L_s is the length of scour.

Natural scour holes may be considered where:

- undermining of the culvert outlet will not occur or it is practicable to be checked by a cutoff wall,
- the expected scour hole will not cause costly property damage, and
- there is no nuisance effect.

See HEC-14 (FHWA, 1995) for further information on natural scour holes.

6.2.2 Riprap Apron

For major culverts riprap must be designed individually for each site. For minor culverts (equivalent diameter ≤ 48"), the riprap apron detailed in Standard Plates 3133 and 3134 will generally be adequate. For outlet velocities less than or equal to 6 fps, check shear stress to determine if vegetation will be adequate. If vegetation is used consider temporary erosion control during and immediately following construction until vegetation becomes established. Use the following guidelines:

Outlet Velocity, V_o	Riprap Specifications	Filter Specifications
$0 < V_o \le 6$ fps	Riprap or stable vegetation	
$6 < V_o \le 8$ fps	12" Class II riprap	6" granular or geotextile filter
$8 < V_o \le 10$ fps	18" Class III riprap	9" granular or geotextile filter
$10 < V_o \le 12$ fps	24" Class IV riprap	12" granular or geotextile filter

6.2.3 Internal Ring Dissipator

There are two types of internal ring dissipator available. Internal ring dissipator are used where:

- the culvert shape is round,
- a scour hole at the culvert outlet is unacceptable,
- the right-of way is limited,
- debris is not a problem, and
- riprap apron is not adequate.

See Section 6.4 for design procedures for the use of internal ring dissipators.

<u>Increased Resistance</u>

This type utilizes roughness elements to increase resistance inside the pipe near the culvert outlet. See Standard Plate 5010 for detail of elements.

- Use for slopes less than 4%.
- Use for culvert flowing partially full with inlet control.
- Can force full flow near the culvert outlet without creating additional headwater.
- Five roughness rings at culvert outlet are necessary.
- Diameter of outlet may be the same or larger than the diameter of the culvert.

<u>Tumbling Flow</u>

This type utilizes the same rings as the above dissipator but can handle higher velocities.

- Use with slopes greater than 4% but less than 25%.
- Five roughness rings at culvert outlet are necessary.
- The diameter of pipe where rings are located is larger than diameter of culvert.
- Standard reinforced concrete pipe (RCP) increasers are recommended to increase pipe size.
- Outlet velocity is approximately critical velocity.

6.2.4 Increased Resistance Box Culverts

The primary method to increase resistance in box culverts has been to place roughness elements on the flowline of the box culvert. The spacing and height of the elements are variable. They can be used independently or in conjunction with a drop weir placed at the inlet to reduce the slope of the culvert. The methodology presented in HEC-14 (FHWA, 1995) is sufficiently broad to allow the roughness elements to be placed on the bottom only or to extend up the side and across the top. The result would be a design similar to the ring dissipator for round pipes.

- Slope should not exceed 6 %.
- Roughness elements should be sharp edge with the element height (h) not exceeding 10% of the flow depth.
- Using element spacing (L) of 10 times element height (h) maximizes the Manning's n value. (L/h) = 10
- The first element, h_1, should be 2 times the height of other elements.
- The minimum number of rows is 5.
- A gap of 2 ± inches in the center of the element is recommended to assist the passing of sediment.

See HEC-14 (FHWA, 1995) for further information regarding increased resistance box culverts.

6.2.5 Riprap Basins

Riprap basins are preformed scour holes that are lined with riprap. The design is based on a study sponsored by the Wyoming Highway Department and conducted by Colorado State University. Consideration may be given to utilizing articulated concrete as an alternate to riprap if conditions are appropriate. Manufacturers recommendations should be consulted when comparing the size of articulated concrete to a riprap d_{50}.

- The basin is preshaped and lined with riprap.
- The depth (h_s), length (L_s), and width (W_s) of the scour hole are related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (y_o), and tailwater depth (TW).
- When (TW/y_o) is less than 0.75 and h_s/d_{50} is greater than 2.0, the scour hole operates very efficiently as an energy dissipator.
- When TW/y_o is greater than 0.75, the scour hole is shallower and longer, thus riprap may be required at the channel downstream of the rock lined basin.

See Section 6.5 for design procedures on the use of riprap basins.

6.2.6 Impact Dissipator

The use of the impact type energy dissipator (USBR Type VI) has largely been replaced by the use of the internal ring dissipator at Mn/DOT. However, use of the impact dissipator can be considered for severe conditions with discharges up to 400 cfs and outlet velocities up to 50 ft/sec.

- An impact basin has greater capacity for dissipating energy than the natural hydraulic jump.
- Tailwater is not necessary, but a moderate depth will improve performance.
- Riprap should be placed downstream of the basin for a length of at least four conduit widths.
- This dissipator is not recommended at locations where debris or ice buildup may cause substantial clogging.

See HEC-14 (FHWA, 1995) for further information on impact dissipators.

6.2.7 Stilling Basins

St. Anthony Falls (SAF) Basin and USBR Type II, III, and IV stilling basin types are presented in HEC-14 (FHWA, 1995). Stilling Basins are used where:

- the outlet scour hole is not acceptable,
- debris is present, and
- the culvert outlet velocity (V_o) is high, Fr > 3.

See HEC-14 (FHWA, 1995) for further information regarding stilling basins.

6.3 ENERGY DISSIPATOR ANALYSIS

An exact theoretical analysis of flow at culvert outlets is extremely complex and generally not attempted because the following data would be required:

- analyzing non-uniform and rapidly varying flow,
- applying energy and momentum balance,
- determining where a hydraulic jump will occur,
- applying the results of hydraulic model studies, and
- consideration of temporary upstream storage effects.

The design procedures presented in this Chapter are based on model studies that were done to calibrate the equations and charts for scour hole estimating and energy dissipator design. HEC-14 (FHWA, 1995) is the base reference and contains full explanation of all the equations and procedures used in this Chapter.

6.3.1 Design Parameters

The dissipator type selected for a site must be appropriate to the location. Selection of dissipator type is made in accordance with the criteria for each dissipator as detailed in Section 6.2. In order to determine the type of energy dissipator which is the most appropriate for a given site, some design information is necessary. Much of this information should already be available from the culvert design. The following information will be necessary.

<u>Flood Frequency</u>

The flood frequency used in the design of the energy dissipator device should be the same flood frequency as the culvert design. In general, this will be:

- 50 year discharge rate (Q_{50}) for minor culverts
- the lesser of overtopping discharge rate (Q_{OT}) or 100 year discharge rate (Q_{100}) for major culverts.

<u>Tailwater Depth</u>

The hydraulic conditions downstream should be evaluated to determine a tailwater depth and the maximum velocity for a range of discharges. For calculating maximum outlet velocity, a low tailwater is usually the most critical whereas for calculating the HW elevation, a high tailwater may be the most critical.

- Open channels - Use methods provided in the Channel Chapter.
- Pond or small body of water - Since the water surface will be rising during the runoff event, it's conservative to assume low tailwater, which will maximize the outlet velocity.
- Lake or large body of water - Use normal or controlled lake level elevation.
- A joint probability analysis comparing drainage areas of the culvert and the receiving water may be desirable in some cases. See the discussion on joint probabilities analysis in the Storm Drain Chapter.

<u>Outlet Velocity</u>

It is necessary to calculate the culvert outlet velocity in order to determine the need for erosion protection at the culvert exit. The flow depth and Froude number may also be required if erosion protection is needed. For a discussion on how to compute outlet velocities for inlet and outlet control, see the Culvert Chapter.

6.3.2 Culvert Outlet Conditions

The outlet flow conditions are established during the culvert design. These parameters may require a closer analysis for energy dissipator design.

<u>Outlet Depth (d_o)</u>

For culverts with supercritical flow, normal depth at the outlet is appropriate unless culvert length (L) < 50 d_o, then a water surface profile should be calculated. For culverts with subcritical flow, the brink depth should be used instead of critical depth at locations with low tailwater. Figures 6.8 and 6.9 are curves for determining outlet depth for subcritical flow conditions.

<u>Area (A_o)</u>

The cross sectional area of flow at the culvert outlet should be calculated based on (d_o).

Outlet Velocity (V_o)
The culvert outlet velocity should be calculated as follows:

$$V_o = \frac{Q}{A_o} \tag{6.1}$$

Where: Q = discharge (cfs)
A_o = area of flow at the outlet (ft^2)

Equivalent Depth (d_E)
Equivalent depth is an artificial depth which is calculated for culverts which are not rectangular so that a reasonable Fr can be determined.

$$d_E = \left(\frac{A_o}{2}\right)^{0.5} \tag{6.2}$$

Where: A_o = area of flow at the outlet (ft^2)

Froude Number (Fr)
The Froude number is a flow parameter that has traditionally been used to design energy dissipators and is calculated using:

$$Fr = \frac{V_o}{\left(gd_o\right)^{0.5}} \tag{6.3}$$

Where: g = acceleration due to gravity (32.2 ft/s^2)
d_o = d_E for rectangular shape (ft)

6.3.3 Erosion Assessment

In determining whether energy dissipation is necessary, it may be helpful to estimate the potential erosion at a culvert outlet. An assessment could then be made to determine if the scour hole is acceptable. Estimating scour at culvert outlets is difficult because of the many complex factors affecting erosion. These factors include discharge, culvert size and shape, soil type, duration of flow, culvert height above the bed, culvert slope, tailwater depth and vegetative cover. In addition, the magnitude of the total erosion can consist of local scour and/or channel degradation. Maintenance history, site reconnaissance, and data on soils, flows, and flow duration provide the best estimate of the potential erosion hazard at a culvert outlet. Analytical methods to estimate erosion at culvert outlets in both cohesive and non-cohesive soils are available in HEC-14 (FHWA, 1995).

6.4 INTERNAL RING DISSIPATORS

There are two types of design for ring dissipators which can be used to reduce the velocity in round reinforced concrete pipe. The first is the increased resistance type and the second is the tumbling flow type. Basic information and a design procedure will be given for each type. The derivation of the methodology can be found in HEC-14 (FHWA, 1995).

6.4.1 Increased Resistance Round Pipes

The methodology described in this section involves using roughness elements to increase resistance and induce velocity reductions. Increasing resistance may cause a culvert to change from partial flow to full flow in the roughened zone. Velocity reduction is accomplished by increasing the wetted surfaces as well as by increasing drag and turbulence by the use of roughness elements. The details shown on Standard Plate No. 5010 can be used to provide the resistance on 24" to 84" diameter pipes.

If the requirement is for outlet velocities between critical and normal velocity, designing increased resistance into the barrel is a viable alternative. The most obvious situation for application of increased barrel resistance is a culvert flowing partially full with inlet control. The objective is to force full flow near the culvert outlet without creating additional headwater.

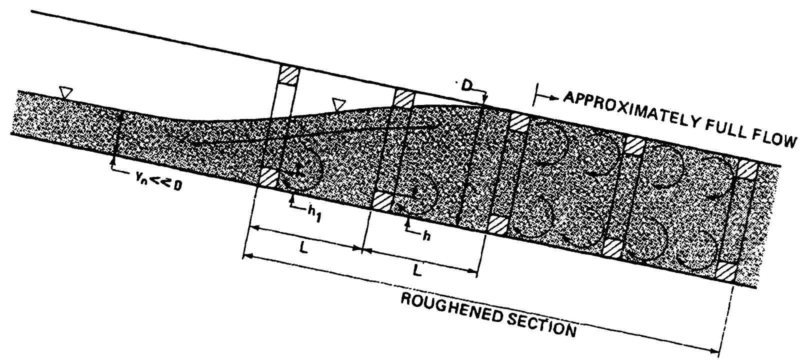


Figure 6.1 Conceptual Sketch of Roughness Elements to Increase Resistance Source: HEC-14 (FHWA, 1983)

The following criteria should be followed:

- Use on culverts with slopes less than 4% operating under inlet control.
- Use five rows of roughness elements.
- The height of each element should be 5% to 10% of the diameter.
- The diameter of the outlet may be the same or larger than the diameter of the culvert.
- Doubling the height of the first ring (Type 2 Table on Standard Plate No. 5010) is effective in triggering full flow in the roughened zone.

Studies of pertinent rough pipe flow data (Morris, 1963) have concluded that there are three flow regimes and each has a different resistance relationship. The three regimes are quasi-smooth flow, hyperturbulent flow, and isolated roughness flow. Quasi-smooth flow occurs only when roughness elements are spaced very close ($L/h \leq 2$) and is not important for this discussion. Hyperturbulent flow occurs when roughness elements are sufficiently close so each element is in the wake of the previous element and rough surface vortices are the primary source of the overall friction drag. Isolated roughness flow occurs when roughness spacing is large and overall resistance is due to drag on the culvert surface plus form drag on the roughness elements. Equations to solve for Manning's n for the two flow regimes are as follows:

<u>Isolated Roughness Flow</u>

$$n_{IR} = n\left(\frac{D_i}{D}\right)^{1/6}\left[1 + 67.2C_D\left(\frac{L_r}{P}\right)\left(\frac{h}{L}\right)\right]^{1/2} \qquad (6.4)$$

Where:
- n_{IR} = overall Manning's n for isolated roughness flow
- n = Manning's n for the culvert surface without roughness rings
- D = nominal diameter of culvert (ft)
- h = height of roughness rings (ft)
- D_i = inside diameter of roughness rings (ft) = D - 2h
- C_D = drag coefficient, which has a constant value of 1.9 for sharp edge rectangular roughness shapes
- L = spacing between roughness elements (ft)
- L_r/P = ratio of total peripheral length of roughness elements to total wetted perimeter

Hyperturbulent Flow

$$n_{HT} = \frac{0.0736 D_i^{1/6}}{\left(1.75 - 2\log_{10}\frac{L}{r_i}\right)} \tag{6.5}$$

Where: n_{HT} = Manning's n for hyperturbulent flow
h = height of roughness rings (ft)
D_i = inside diameter of roughness rings (ft) = D - 2h
L = spacing between roughness elements (ft)
r_i = pipe radius based on inside diameter of roughness rings (ft)

Step 1 Compute $n/D^{1/6}$, where "n" is Manning's coefficient for smooth culvert and D is the diameter in feet.

Step 2 Select L/D_i in the range 0.5 to 1.5. Try to use standard laying length of pipe for L and Standard Plate 5010 for D_i which is the inside diameter of the ring. This will not be possible for minor culverts as the L will be less than the normal laying length. Special design laying lengths will be necessary.
Compute inside ring diameter, D_i = D - 2h

Step 3 Select h/D_i in the range 0.05 to 0.10. Use sharp edged roughness rings.

Step 4 Determine the flow regime from Figure 6.5. The flow regime will be "isolated roughness" (I.R.), if the point defined by the L/D_i and h_r/D_i ratios is above the $n/D^{1/6}$ value. If the point is below, the flow is "hyperturbulent" (H.T.). Isolated roughness is the most common for large culverts.

Step 5 Determine the rough pipe resistance ($n_r = n_{IR}$ or n_{HT})

A. For isolated roughness flow obtain (n_{IR}/n) from Figure 6.6 or from Equation 6.4.
If gaps are to be left in the roughness rings so that L_r/P is much less than 1.0, Equation 6.4 must be used because Figure 6.6 is based on L_r/P = 1.0.

B. For hyperturbulent flow obtain $n_r = n_{HT}$ from Figure 6.2 or from Equation 6.5.

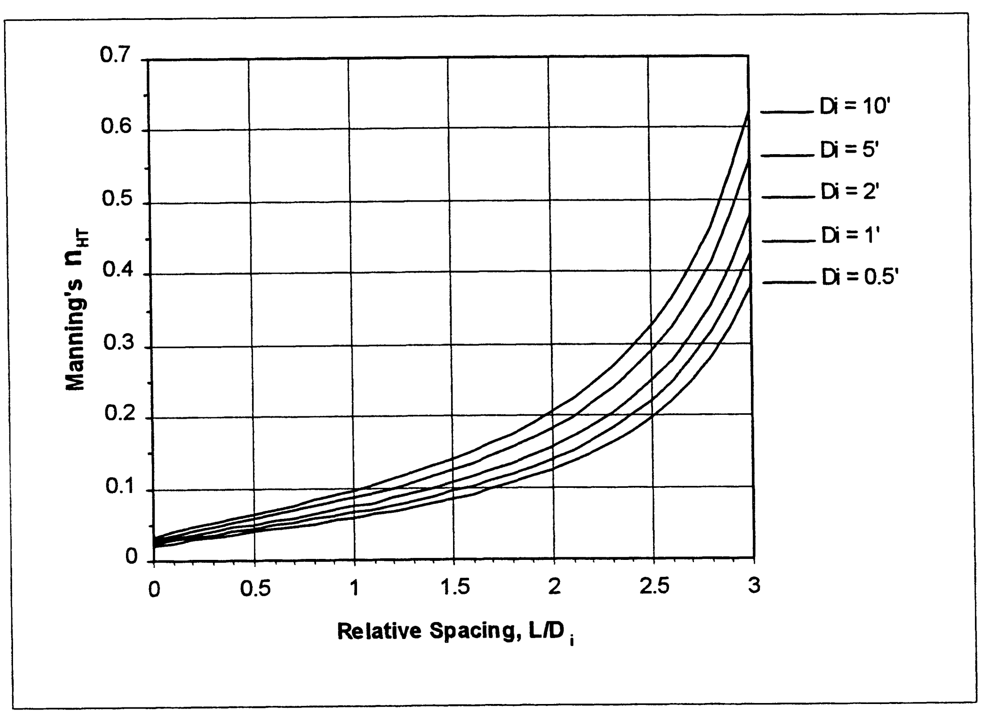


Figure 6.2 Resistance Curves for Hyperturbulent Flow

Step 6 Compute full flow characteristics based on D and n:

$$Q_{FULL} = \left(\frac{0.46}{n_r}\right) D_i^{8/3} S_o^{1/2} \qquad V_{FULL} = \left(\frac{0.59}{n_r}\right) D_i^{2/3} S_o^{1/2} \qquad (6.6)$$

Where: Q = discharge (cfs)
V = velocity (fps)
n_r = rough pipe resistance
D_i = inside diameter of roughness rings (ft)
S_o = slope of pipe (ft/ft)

Step 7 Determine outlet velocities:

A. If Q_{FULL} = Design Q,
$V_{OUTLET} = V_{FULL}$

B. If Q_{FULL} is less than Design Q, the culvert is likely to flow full and result in increased headwater requirements. In this case, a complete hydraulic analysis of the culvert is necessary to compute the outlet velocity which will be greater than V_{FULL} from Step 6. To avoid this situation, use an oversize diameter, D and D_i, for the roughened section of the culvert and repeat steps 1 through 6 above.

C. If Q_{FULL} is greater than Design Q, use Figure 6.4 to compute the velocity.

Step 8 Evaluate acceptability of outlet velocity and repeat design steps if necessary. Acceptable outlet velocity is a site determination that must be made by the designer. It is anticipated that the use of roughness rings may not eliminate the need for riprap but complement or minimize the use of riprap protection. If the outlet velocity is not acceptable, the recommended order of consideration is:

A. If Q_{DESIGN} is less than Q_{FULL}, increase h/D_i to approach full flow. A solution can usually be attained with one iteration by approximating the resistance from n = $[0.59D_i^{2\backslash3}\ S_o^{1/2}]/V_{DESIRED}/1.15$ and using an estimated value of D_i slightly greater than expected. With n_r known, selecting a corresponding h/D_i from Figure 6.2 or Figure 6.6 is relatively straightforward.

B. If V_{FULL} is still too high, increase D for the roughened section to make possible higher values of h/D_i and correspondingly higher values of n_r; i.e., use an oversized culvert with diameter, D, to allow for the new D_i, in the rough section and repeat Steps 1 through 7 above.

C. Use a tumbling flow design as described in Section 6.4.2.

D. Use another type of dissipator either in lieu of or in addition to the roughness rings.

Step 9 Determine the size and spacing of roughness rings.

A. Prepare summary of the parameters.

Where: $D_i = D - 2h$ inside diameter of roughness rings (ft)
$h = (h/D_i)D_i$ height of roughness rings (ft)
$L = (L/D_i)D_i$ spacing between roughness elements (ft)
$h_1 = 2h$ height of first roughness ring (ft)
$D = D_i + 2h$ nominal diameter of culvert barrel (ft)

B. Use five roughness rings including the oversized first ring. If an oversized diameter is used, provide an approach length of one diameter before the first ring.

6.4.2 Tumbling Flow Circular Pipes

Tumbling flow in circular culverts can be attained by inserting circular rings inside the barrel, Figure 6.3. This concept is very similar to the Increased Resistance concept described previously; the primary difference being that it can be used for culverts on slopes between 4% and 25%. The variables that determine whether or not tumbling flow will occur are roughness height (h), spacing (L), slope (S_o), discharge (Q), and the diameter (D_1). Practical design limits can be assigned to h/D_1 and L/D_1 to further simplify the functional relationship. Based on qualitative laboratory observations, tumbling flow is easiest to maintain when:

- L/D_1 is between 1.5 and 2.5 — $L/D_1 \approx 2.0$ (Tolerance ± 25%)
- h/D_1 is between 0.10 and 0.15 — $h/D_1 \approx 0.125$ (Tolerance ± 20%)
- slope is between 4% and 25% — $4\% < S_o < 25\%$

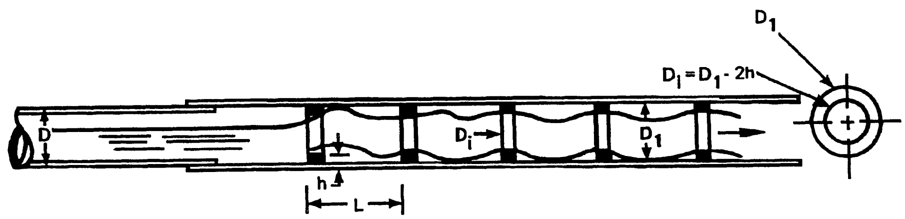


Figure 6.3 Definition Sketch For Tumbling Flow in Culverts
Source: HEC-14 (FHWA, 1983)

The basic design relationship for tumbling flow in steep circular culverts is:

$$0.21 < \frac{Q}{\left(gD_1^5\right)^{1/2}} < 0.32 \qquad \text{and} \qquad 1.6\left(\frac{Q^2}{g}\right)^{0.2} < D_1 < 1.9\left(\frac{Q^2}{g}\right)^{0.2} \qquad (6.7)$$

Where: Q = discharge (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)
D_1 = pipe diameter (ft)

Five roughness rings (Standard Plate No. 5010) at the outlet end of the culvert are sufficient to establish tumbling flow. The diameter computed from Equation 6.7 is for the roughened section only, and will not necessarily be the same as the rest of the culvert. If a larger diameter is necessary for the roughened section, precast increasers may be used to transition from the culvert diameter to the larger roughened diameter. A length of at least one diameter of the larger pipe should be placed upstream of the first ring.

The outlet velocity for tumbling flow is approximately critical velocity. It can be computed by determining the critical depth (d_c) for the inside diameter of the roughness rings. Chart 4 in Appendix C can be used to determine critical depth using D_i. Figure 6.4 can be used to determine the critical area (A_c). Outlet velocity can be computed from Equation 6.1, substituting V_c for V_o and A_c for A_o.

Step 1 Check culvert control. If inlet control governs, tumbling flow may be a good choice for dissipating energy.

Step 2 Determine the diameter (D_1), of the roughened section of pipe to sustain tumbling flow using Equation 6.7.

Step 3 Compute h and L from:
$h/D_1 = 0.125 \pm 20\%$
$L/D_1 = 2.0 \pm 25\%$

Step 4 Compute the internal diameter of the roughness rings
$D_i = D_1 - 2h$

Step 5 Determine the critical depth (y_c), from Chart 4 in Appendix C using design discharge for Q and D_i for diameter.

Step 6 Compute y_c/D_i

Step 7 Determine A_c from Table 6.1. Setting $d = y_c$ and $A = A_c$, use the known (d/D) to determine (A/D^2).

Step 8 Compute the outlet velocity
$V_o \approx V_c = Q/A_c$

Step 9 Evaluate acceptability of outlet velocity, and document results.

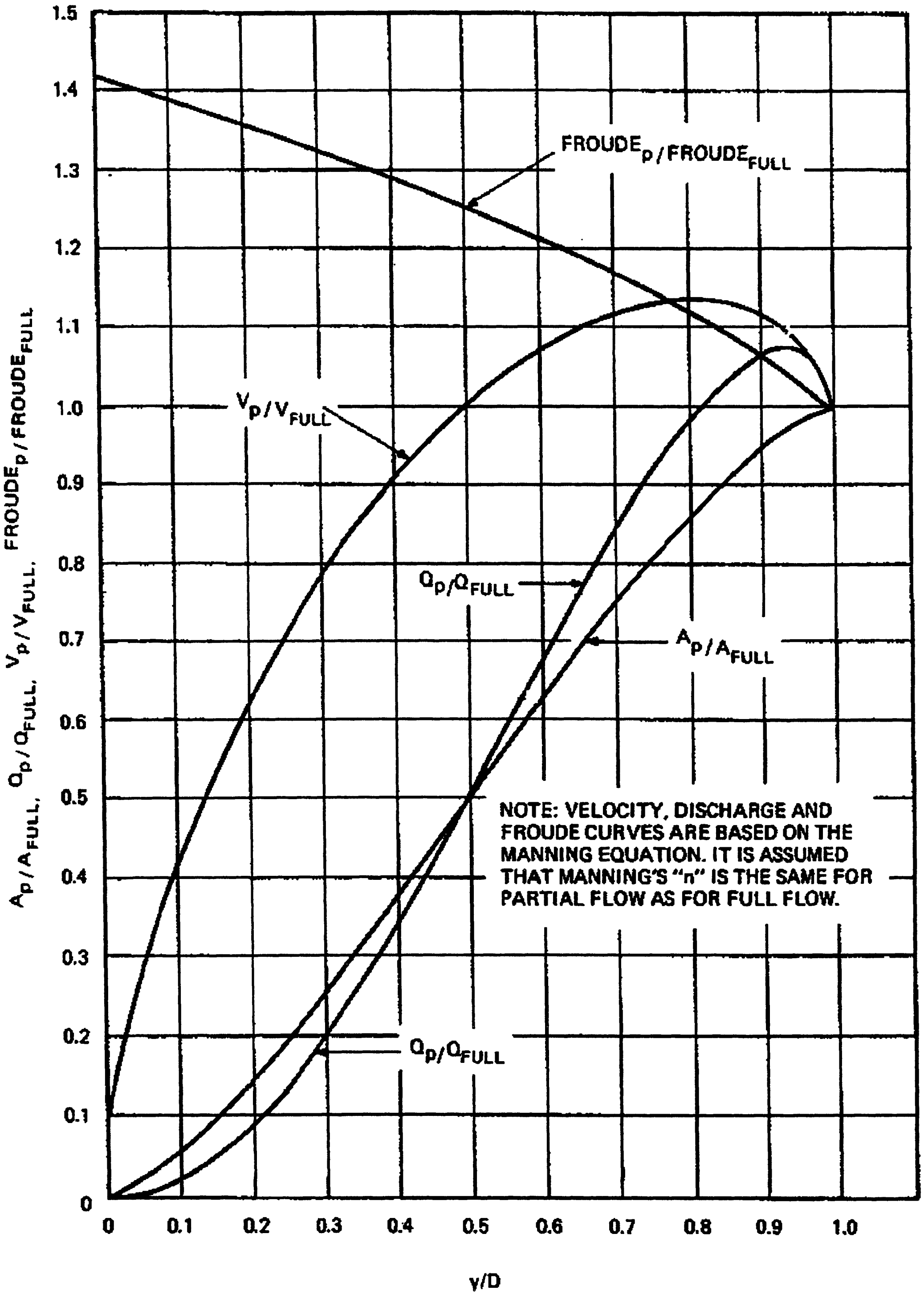


Figure 6.4 Hydraulic Elements Diagram for Circular Culverts Flowing Part Full
Source: HEC-14, Figure VII-C-3 (FHWA, 1983)

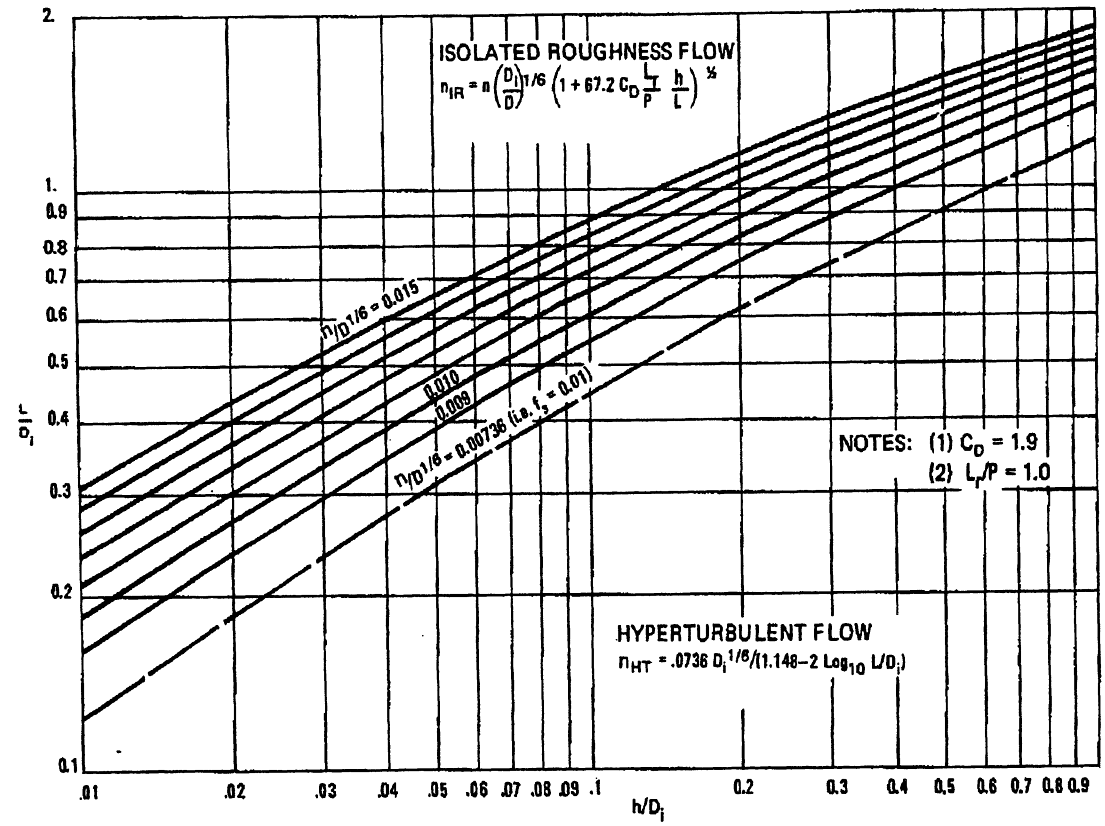


Figure 6.5 Flow Regime Boundary Curves
Source: HEC-14, Figure VII-C-6 (FHWA, 1983)

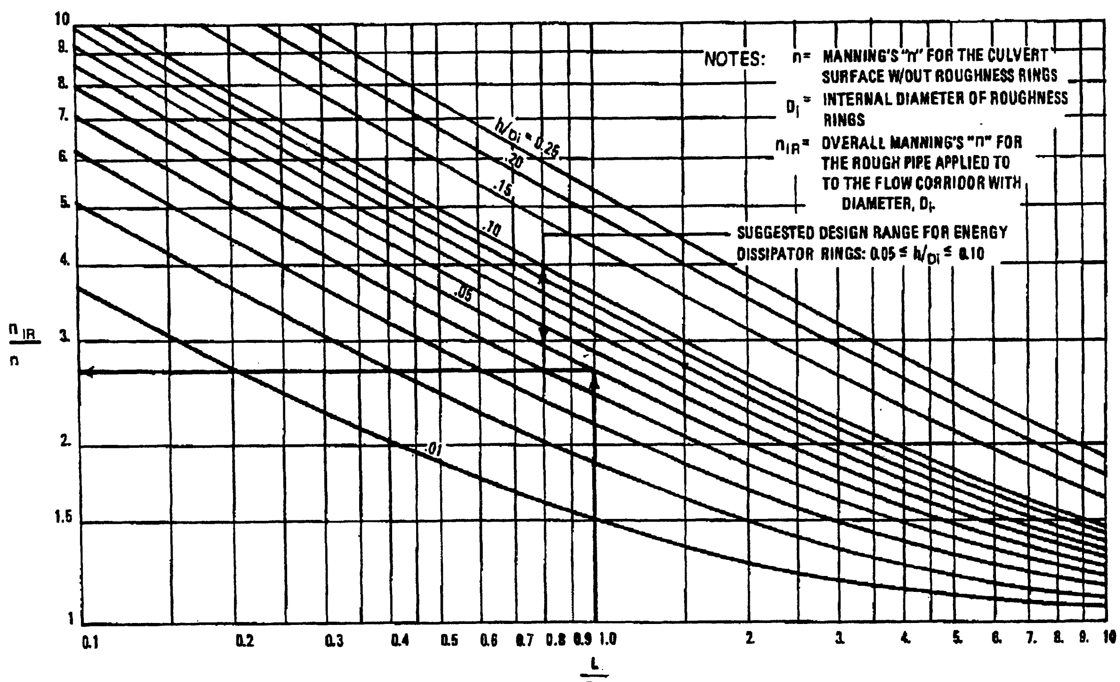


Figure 6.6 Relative Resistance Curves for Isolated Roughness Flow
Source: HEC-14, Figure VII-C-4 (FHWA, 1983)

Table 6.1 Uniform Flow in Circular Sections Flowing Partly Full

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{QN}{D^{8/3}S^{1/2}}$	$\frac{QN}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{QN}{D^{8/3}S^{1/2}}$	$\frac{QN}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
.0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4586	0.2676	0.279	1.311
.0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.79	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0929	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0985	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.1024	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.1097	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.1152	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	.01206	0.1206	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.1259	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.1612	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.1364	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.1416	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.1466	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.1516	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.1566	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.1614	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.1662	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.1709	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.1759	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.11802	0.1802	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1847	2.09	0.83	0.6969	0.3041	1.468	0.770
0.34	0.2355	0.1891	0.1891	2.05	0.84	0.7043	0.3038	1.473	0.753
0.35	0.2450	0.1935	0.1935	2.00	0.85	0.7115	0.3033	1.477	0.736
0.36	0.2546	0.1978	0.1978	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.2020	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.2062	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.2102	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	.02142	0.2142	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.2182	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.2220	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.2258	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.2295	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.2331	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.2366	1.590	0.96	0.7749	0.2829	0.498	0.553
0.47	0.3627	0.2401	0.2401	1.559	0.97	0.7785	0.2778	0.496	0.535
0.48	0.3727	0.2435	0.2435	1.530	0.98	0.7817	0.2735	0.494	0.517
0.49	0.3827	0.2468	0.2468	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

d = depth of flow A = area of flow Q = discharge in cfs by Manning's formula N = Manning's coefficient
D = diameter of pipe R = hydraulic radius S = slope of the channel bottom and of the water surface SOURCE: HEC-14 (FHWA, 1983)

6.5 RIPRAP BASIN

The riprap basin should be considered for energy dissipation for those locations where the standard riprap apron or other energy dissipators are inadequate. The riprap basin design is based on laboratory data obtained from full scale prototype installations. Following are the principal features of the basin :

- The basin is preshaped and lined with riprap of median size, d_{50} (ft).
- Constructing the floor at a depth of h_S below the invert, where h_S (ft) is the approximate depth of scour that would occur in a thick pad of riprap of size located at the outfall of the culvert if subjected to the design discharge.
- Size d_{50} so that $2 < h_S/d_{50} < 4$.
- Size the length of the dissipating pool to be $10(h_S)$ or $3(W_o)$ whichever is larger for a single barrel.
- Size the overall length of the basin is $15(h_S)$ or $4(W_o)$, whichever is larger.
- Angular rock results were approximately the same as the results of rounded material.
- Layout details are shown on Figure 6.11.

High Tailwater ($TW/d_o > 0.75$)

- The high velocity core of water emerging from the culvert retains its jet like character as it passes through the basin.
- The scour hole is not as deep as with low tailwater and is generally longer.
- Riprap may be required for the channel downstream of the rock-lined basin.

6.5.1 Riprap Basin Design Procedures

The following variable definitions are for use with the riprap basin design procedures.

Where: d_{50} = median diameter size of riprap (ft)
y_o = brink depth, also referred to as d_o (ft)
V_o = velocity at brink (fps)
y_E = equivalent depth at the brink (ft)
Fr_o = Froude number at brink
D = height of box culvert (ft)
A_o = Wetted area associated with y_o (ft^2)
Q = Discharge rate (cfs)
h_S = scour hole depth (ft)
L_S = length of the dissipating pool (ft)
W_o = pipe diameter, box barrel width, or arch span of culvert (ft)
L_B = length of basin (ft)
d_B = critical depth at basin exit (ft)
V_B = Basin exit velocity (fps)

Step 1 **Determine Input Flow Properties**

Estimate flow properties at the brink of the culvert. (y_o, V_o, Fr_o)

A. For mild slopes (subcritical flow conditions) Figures 6.8 and 6.9 can be utilized to find the brink depth (y_o) for rectangular and round culverts. V_o is obtained by dividing Q by A_o (continuity equation)

B. For steep slopes assume normal depth at the brink. Velocity is calculated using Manning's equation.

C. Compute the brink Froude number, $Fr_o = V_o/(gy_e)^{0.5}$ using the equivalent depth at the brink, $y_E = (A/2)^{0.5}$.

Step 2 **Check tailwater (TW)**

Determine if $TW/y_o \leq 0.75$. If $TW/y_o > 0.75$, follow high tailwater design procedures outlined below.

Step 3 **Determine riprap size (d_{50})**

A. Use Figure 6.10.

B. Select d_{50}/y_e. Start with d_{50} values that are locally available and correspond with existing Mn/DOT riprap specifications (class II, class III, class IV).

C. Obtain h_S/y_e using Froude number (Fr) and Figure 6.10.

D. Check if $2 < h_S/d_{50} < 4$ and repeat until a d_{50} is found within the range.

Step 4 **Size Basin**

A. Size basin as shown in Figure 6.11.

B. Determine length of the dissipating pool, L_S. $L_S = 10h_S$ or $3W_o$ whichever is larger.

C. Determine length of basin, L_B. $L_B = 15h_S$ or $4W_o$ whichever is larger.

D. Thickness of riprap:
Approach $= 3d_{50}$ or $1.5\ d_{max}$
Remainder $= 2d_{50}$ or $1.5\ d_{max}$

Step 5 **Determine Basin Exit Velocity (V_B)**

A. Basin exit depth, d_B = critical depth at basin exit.

B. Basin exit velocity, $V_B = Q/(W_B.d_B)$.

C. Compare V_B with the average normal flow velocity in the natural channel, V_d.

Step 6 **High Tailwater Design**

A. Design a basin for low tailwater conditions, Steps 1-5.

B. Compute equivalent circular diameter D_E for brink area from: $A = \pi D_E^2/4 = d_o(W_o)$

C. Estimate centerline velocity at a series of downstream cross sections using Figure 6.12.

D. Size riprap using HEC-11 *Design of Riprap Revetment* (FHWA, 1989) or the Channel Chapter.

Step 7 **Design Filter**

Unless the streambed material is sufficiently well graded use a filter under the riprap. Follow Mn/DOT Specifications.

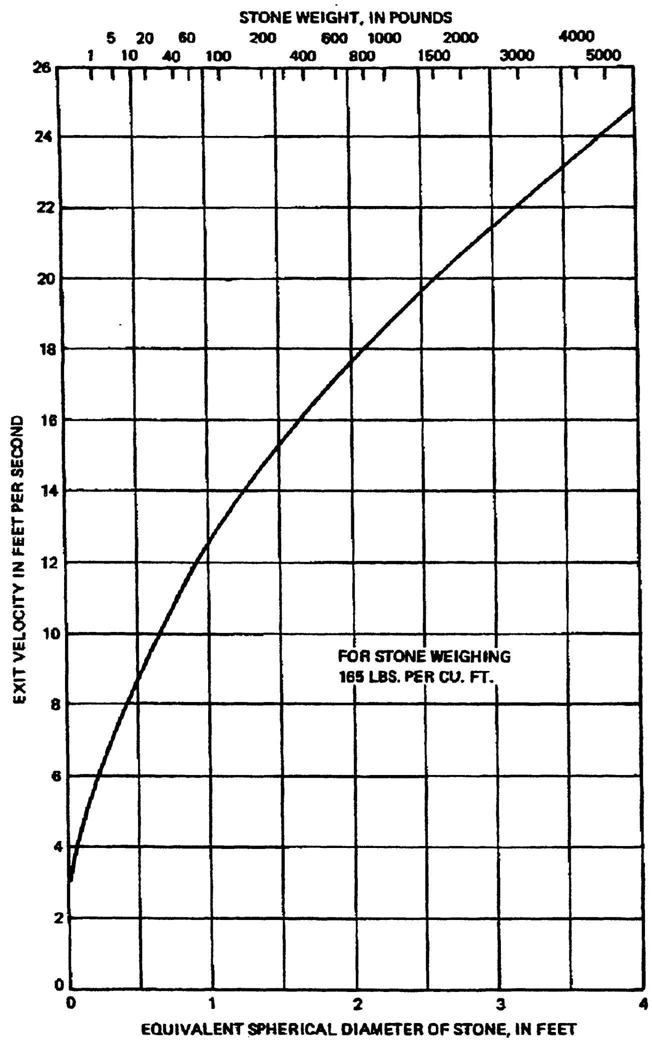


Figure 6.7 Riprap Size Versus Exit Velocity
Source: HEC-14 (FHWA, 1983)

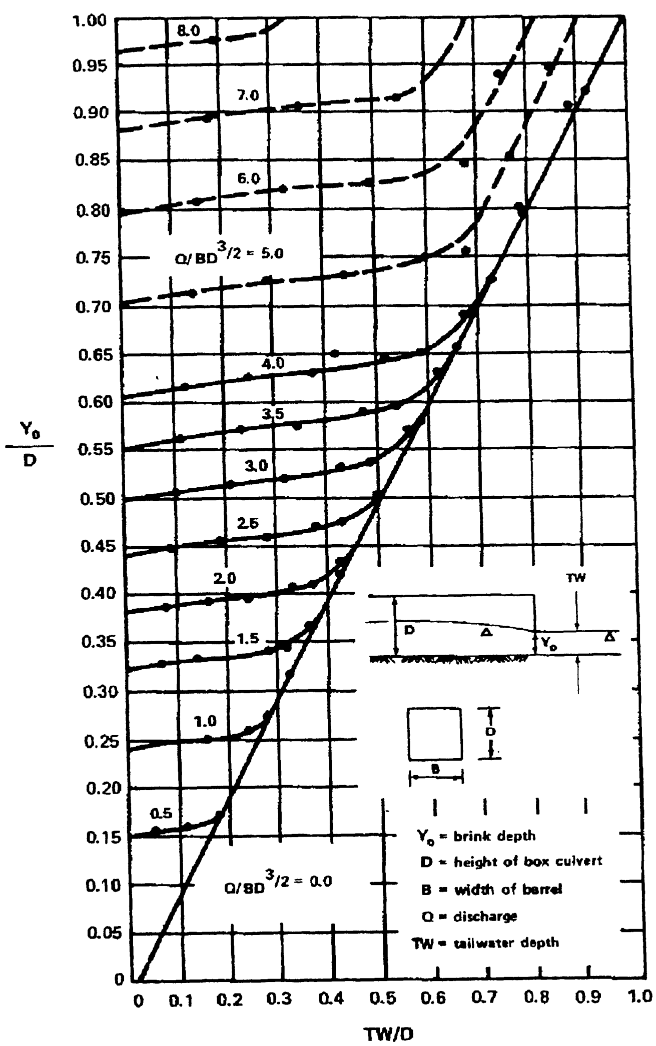


Figure 6.8 Dimensionless Rating Curves for Rectangular Culvert Outlets
Source: HEC-14, Figure III-9 (FHWA, 1983)

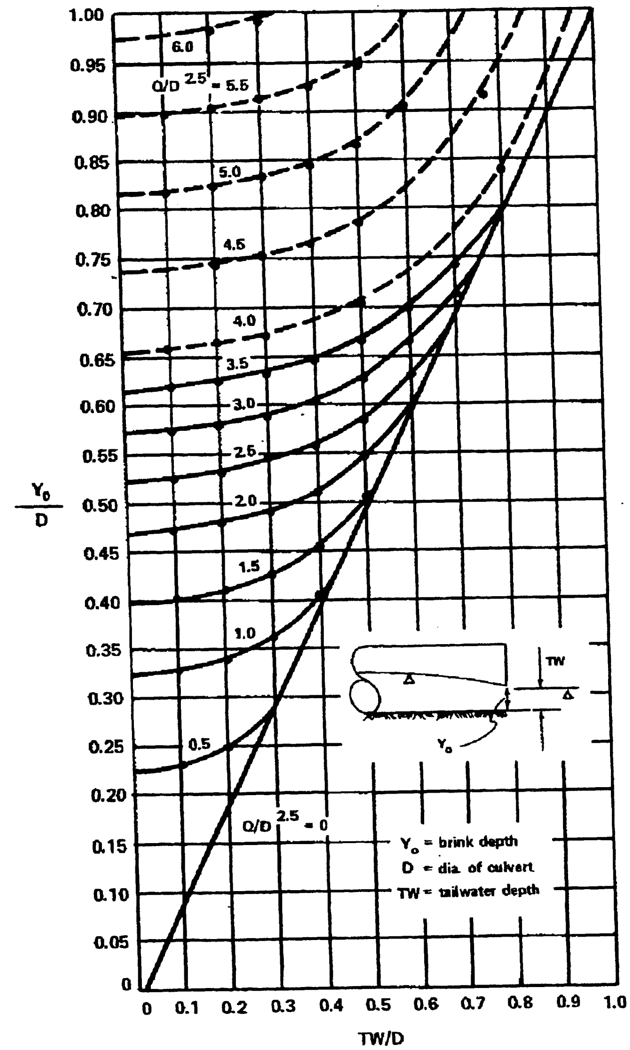


Figure 6.9 Dimensionless Rating Curves for Circular Culvert Outlets
Source: HEC-14, Figure III-10 (FHWA, 1983)

Figures 6.8 and 6.9 are dimensionless rating curves which indicate the effect on brink depth of tailwater for culverts on mild or horizontal slopes. Use these figures to determine the brink depth for culvert outlets. For circular culverts of known geometry (D = diameter), tailwater (TW), and Discharge (Q); compute $\frac{TW}{D}$ and $\frac{Q}{D^{5/2}}$. Use Figure 6.9 to look up $\frac{Y_o}{D}$, then compute Y_o. For rectangular culverts of known geometry (B = barrel width and D = box height), tailwater (TW), and Discharge (Q); compute $\frac{TW}{D}$ and $\frac{Q}{BD^{3/2}}$. Use Figure 6.8 to look up $\frac{Y_o}{D}$, then compute Y_o.

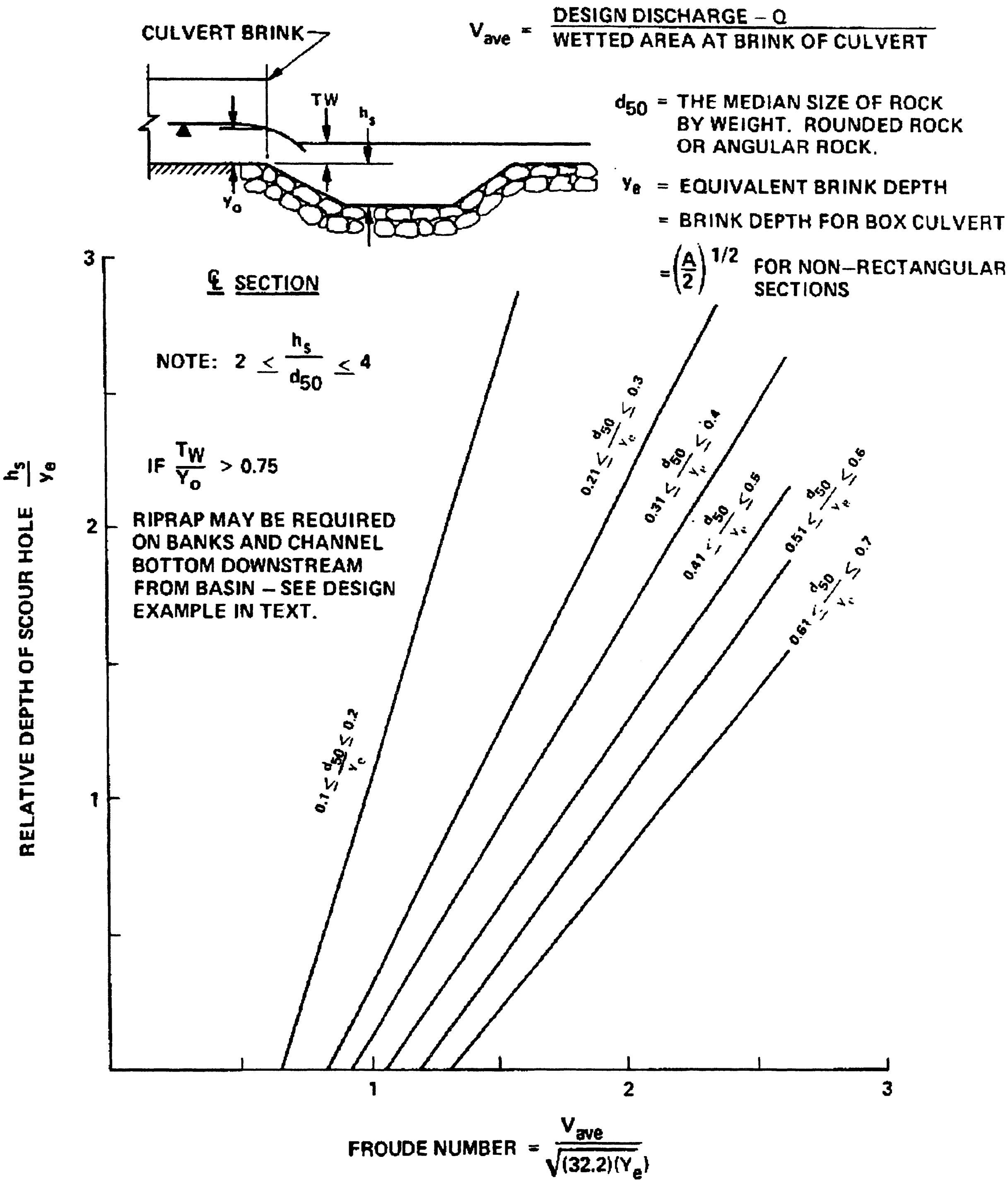
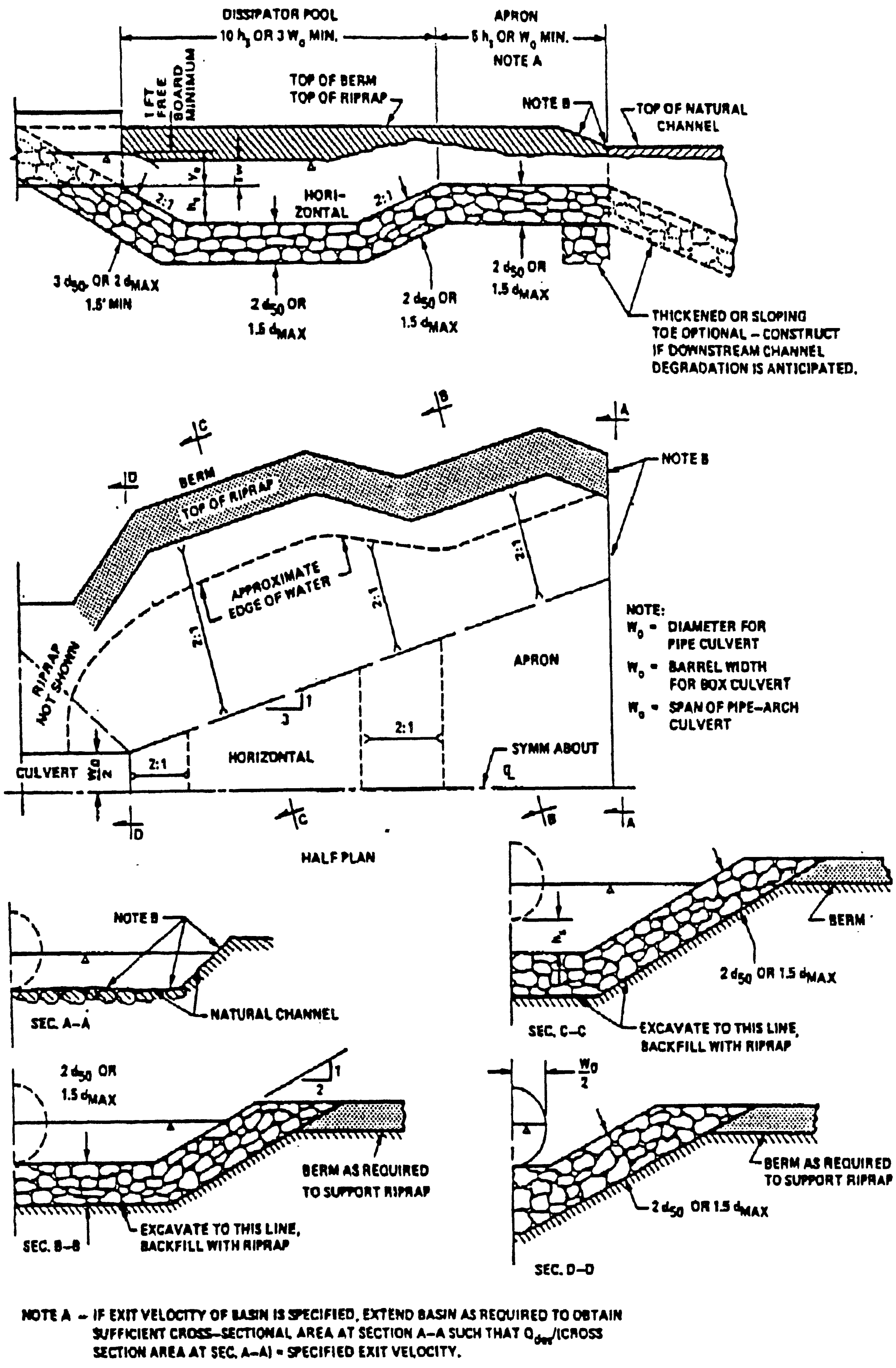


Figure 6.10 Riprapped Basin Depth of Scour
Source: HEC-14, Figure XI-2 (FHWA, 1983)

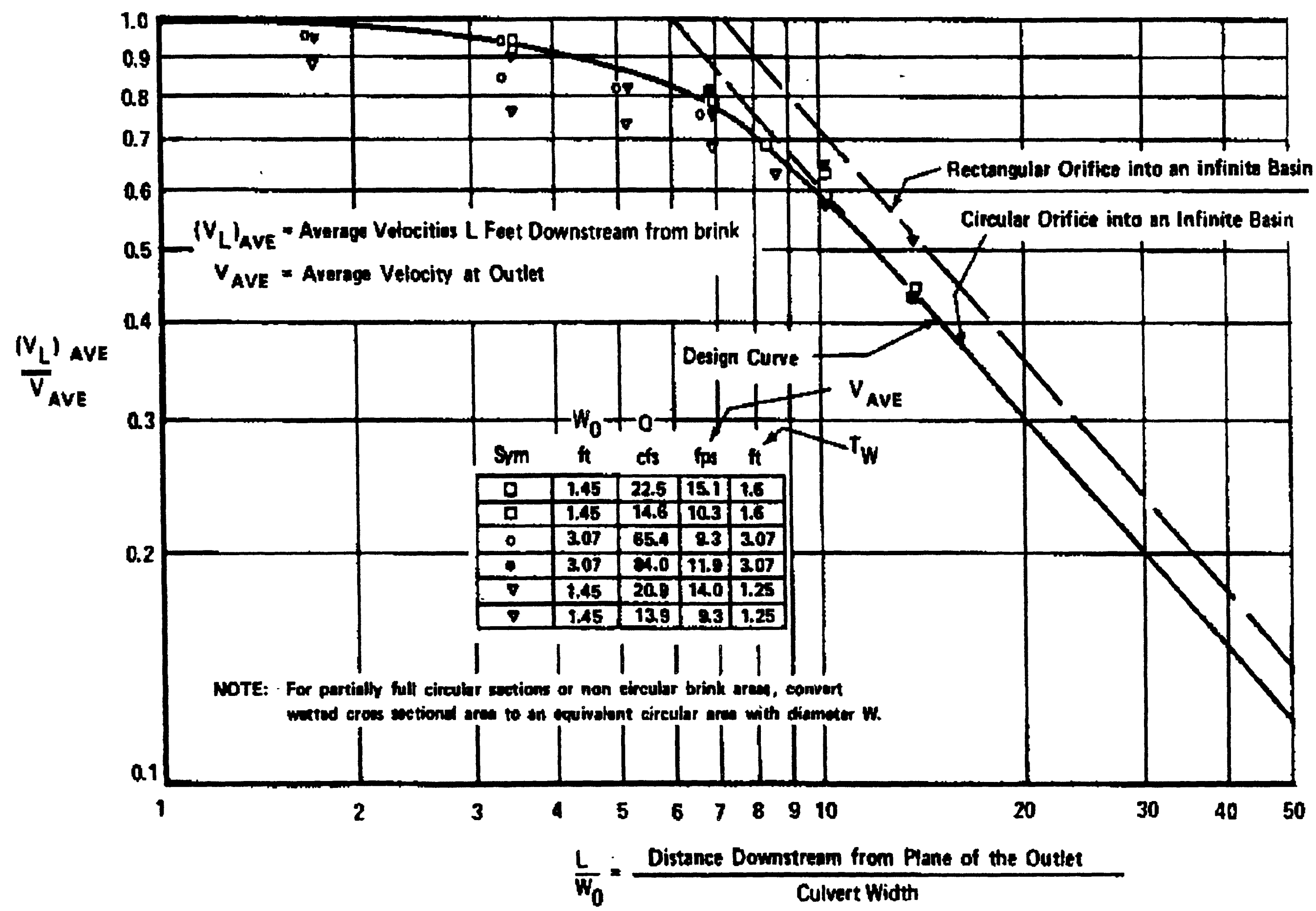


NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS–SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.

Figure 6.11 Details of Riprapped Energy Dissipator
Source: HEC-14, Figure XI-1 (FHWA, 1983)

Figure 6.12 gives the distribution of centerline velocity for flow from submerged outlets to be used for predicting channel velocities downstream from culvert outlet where high tailwater prevails. Velocities from this figure are used with Figure 6.7 for sizing riprap.



Sym	W_0 ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
o	3.07	65.4	9.3	3.07
•	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▼	1.45	13.9	9.3	1.25

Figure 6.12 Velocity Distribution for High Tailwater
Source: HEC-14, Figure XI-3 (FHWA, 1983)

6.6 REFERENCES

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Chapter 7 STORAGE FACILITIES

7.1 INTRODUCTION

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual.

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decrease downstream channel erosion,
- control of sediment deposition,
- improved water quality through stormwater filtration.

Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements,
- maintenance of historic low flow rates by controlled discharge from storage.

7.1.1 Detention and Retention Facilities Definition

Urban stormwater storage facilities are generally referred to as detention (dry pond) or retention (wet pond) facilities. Dry ponds are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. They are designed to completely drain after the design storm has passed. Wet ponds are designed to contain a permanent pool of water usually for water quality purposes.

7.1.2 Concept Definitions

Following are discussions of concepts which are important in storage design:

Outlet Structures	Outlet structures selected for storage facilities typically include a principal outlet and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet structures can take the form of combinations of drop inlets, pipes, weirs, and orifices. The principal outlet is intended to convey the design storm.
Stage-storage Curve	A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir.
Stage-discharge Curve	A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets: principal outlet and emergency overflow.
Inflow, I	Discharge into a storage facility at a specific time.
Outflow, O	Discharge out of a storage facility at a specific time.
Hydrograph	The hydrograph is a graph of the time distribution of flow rate at a single point.

7.2 DESIGN CRITERIA

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Possible dispersed or on-site storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. Although there are no known strict rules covering ponding adjacent to roadways, ponding within the clear zone and adjacent R/W should be evaluated for safety on a project by project basis. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis.

The design criteria for storage facilities should include:
- location,
- outflow rate,
- storage volume,
- grading and depth requirements,
- outlet structure.

7.2.1 Location

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is important for the designer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis. Local stormwater management should be coordinated with the local entities such as watershed districts, municipalities and counties, judicial and county ditch authorities.

7.2.2 Storage

Storage volume shall normally be provided to attenuate the post-development peak discharge rates to pre-developed peak discharge rates for the design storm, or other design storms depending on what the downstream system is designed for. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project.

Dry Ponds

Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions.

Wet Ponds

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 4 feet is generally reasonable. Where aquatic habitat is required, wildlife experts should be contacted for site specific criteria relating to such things as depth, habitat, and bottom and shore geometry.

7.2.3 Flow Rate

For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The sizing of a particular outlet structure shall be based on results of hydrologic routing calculations. Control structure outflow rates shall usually approximate pre-developed peak runoff rates for the design storm or what is required by the local regulatory agency. It is recommended that emergency overflow be provided for events greater than the design storm. In addition to controlling the peak discharge from the outlet structures, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. The location and timing of the structure should be coordinated with the local drainage offices.

7.3 STORAGE FACILITY ANALYSIS

The following data will be needed to complete storage design and routing calculations:

- Inflow hydrograph for selected design storms.
- Stage-storage curve for proposed storage facility (Figure 7.1).
- Stage-discharge curve for all outlet control structures (Figure 7.2).

A routing procedure is used to route the inflow hydrograph through the storage facility with different basin (stage-storage curve) and outlet geometry (stage-discharge curve) until the desired outflow hydrograph is achieved.

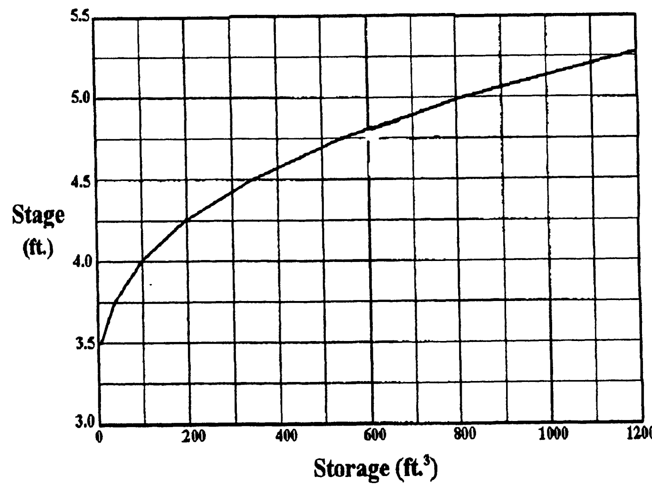


Figure 7.1 Example Stage-Storage Curve

7.3.1 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and the double-end area, frustum of a pyramid or prismoidal formulas. Only the more commonly used double-end area formula is provided. To convert from ft³ to acre-ft, divide number by 43560.

The double-end area formula is expressed as:

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2}\right] d \qquad (7.1)$$

Where: $V_{1,2}$ = storage volume between elevations 1 and 2 (ft^3)
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)
d = change in elevation between points 1 and 2 (ft)

7.3.2 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets: principal outlet and emergency overflow. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal outlet. The emergency overflow is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. The stage-discharge curve should take into account the discharge characteristics of both the principal outlet and emergency overflow. Care should be taken to consider downstream tailwater effects, especially in cases of multiple ponds where backwater may occur.

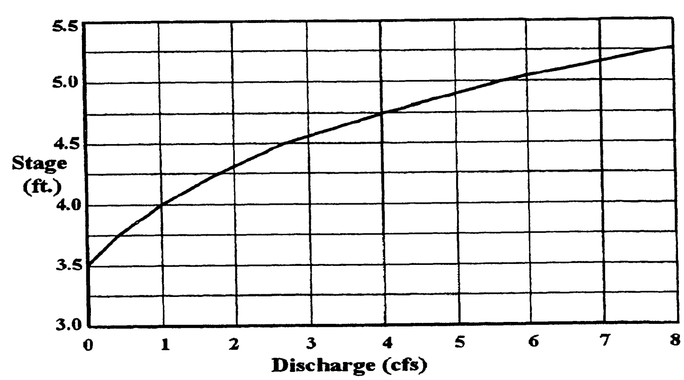


Figure 7.2 Example Stage-Discharge Curve

7.3.3 Generalized Routing Procedure

A general procedure for using the above data in the design of storage facilities is presented below. This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

Step 1 Compute inflow hydrograph for runoff from the design storm using the procedures outlined in the Hydrology Chapter. Determine allowable peak outflow rate.

Step 2 If necessary perform preliminary calculations to evaluate detention storage requirements for the hydrograph from Step 1. If storage requirements are satisfied for runoff from the design storms, runoff from lower return period storms is assumed to be controlled.

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

Step 4 Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed peak discharges from the design storm exceeds the allowable peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to step 3.

Step 6 Consider emergency overflow from runoff due to a larger storm than the design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems.

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

7.4 OUTLET HYDRAULICS

Equations are provided for sharp-crested weirs, broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlet works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data. When analyzing release rates the tailwater influence of the principal spillway culvert on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening.

7.4.1 Sharp Crested Weirs

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below.

Sharp Crested Weir with No End Contractions

A sharp-crested weir with no end contractions is illustrated in Figure 7.3. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] L H^{1.5} \tag{7.4}$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

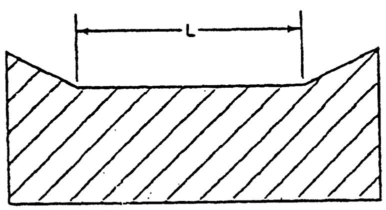


Figure 7.3 Sharp-Crested Weir (No End Contractions)

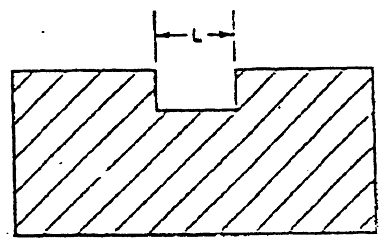


Figure 7.4 Sharp-Crested Weir, (Two End Contractions)

Sharp Crested Weir with Two End Contractions

A sharp-crested weir with two end contractions is illustrated in Figure 7.4. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[3.27 + 0.4\left(\frac{H}{Hc}\right)\right](L - 0.2H)\, H^{1.5} \tag{7.5}$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

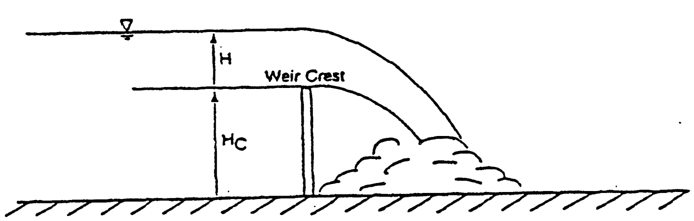


Figure 7.5 Sharp-Crested Weir and Head

Submerged Sharp Crested Weir

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_s = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \tag{7.6}$$

Where: Q_s = submergence flow (cfs)
Q_f = free flow (cfs)
H_1 = upstream head above crest (ft)
H_2 = downstream head above crest (ft)

7.4.2 Broad Crested Weirs

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \tag{7.7}$$

Where: Q = discharge (cfs)
C = broad-crested weir coefficient
L = broad-crested weir length (ft)
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 7.1.

Table 7.1 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (ft)

Measured Head, H[1] (ft)	Breadth Of The Crest Of Weir (ft)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Source: Brater and King (1976).

7.4.3 V-notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5} \tag{7.8}$$

Where: Q = discharge (cfs)
θ = angle of v-notch (degrees)
H = head on apex of notch (ft)

7.4.4 Orifices

An orifice is an opening through which water may flow. Generally the walls are assumed to be thin with a square edge, though variations in geometry are possible. The entrance coefficient will vary with geometry, but is typically given a value of 0.6 for circular orifices with square edged entrances. If the orifice discharges as a free outfall then the head, H is measured from the upstream water surface to the center of the orifice. If the orifice is submerged the head on the pipe is the difference between the upstream and downstream water surfaces. Pipes smaller than 12" in diameter may be analyzed as a submerged orifice if H/D is greater than 1.5. Pipes with a diameter larger than 12" should be analyzed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

$$Q = CA\left(2gH\right)^{0.5} \tag{7.9}$$

Where: Q = discharge (cfs)
C = entrance coefficient (typically C = 0.6)
A = cross-section area of pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
D = diameter of pipe (ft)
H = head on pipe (ft)

7.5 DESIGN PROCEDURE

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs. Also, the storage indicator method can be used which makes calculations simple. All storage facilities shall be designed and analyzed using reservoir routing calculations. Software available for final routing includes TR20, HEC-1 and HEC-RAS. Many commercial programs are also available but the engineer/technician operating the programs should possess the background and skill necessary to correctly interpret the results.

7.5.1 Preliminary Detention Calculation of Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 7.6. This procedure is not recommended for final design but only where preliminary estimates are needed of storage volume.

Use inflow hydrograph to determine Q_i and T_i. The allowable peak outflow rate, Q_o must be estimated. The peak discharge for pre-existing conditions is commonly used.

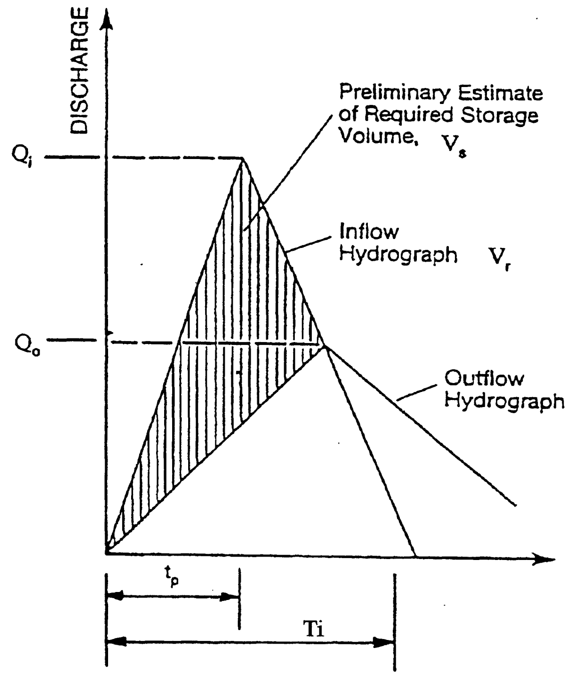


Figure 7.6 Triangular Shaped Hydrographs
(For Preliminary Estimate Of Required Storage Volume)

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5\,Ti\left(Q_i - Q_O\right) \tag{7.10}$$

Where: V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_o = peak outflow rate (cfs)
T_i = duration of basin inflow (sec)

7.5.2 Storage Indicator Method

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or modified Puls method. This method begins with the continuity equation which states that the inflow minus the outflow equals the change in storage (I - O = ΔS). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 7.11. In Equation 7.11 subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

$$\frac{\Delta S}{\Delta t} = \frac{(I_1+I_2)}{2} - \frac{(O_1+O_2)}{2} \tag{7.11}$$

Where: ΔS = change in storage (ft^3)
Δt = time interval (minutes)
I = inflow (ft^3)
O = outflow (ft^3)

Equation 7.11 can be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right hand side of the equation, as shown in Equation 7.12. Now, the equation with two unknowns, S_2 and O_2, can be solved with one equation. The following procedure can be used to perform routing through a reservoir or storage facility using Equation 7.12.

$$\frac{(I_1+I_2)}{2} + \left(\frac{S_1}{\Delta t}+\frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t}+\frac{O_2}{2}\right) \quad (7.12)$$

Where: I_1 = Instantaneous inflow to the pond at the beginning of the incremental time period, Δt
I_2 = Instantaneous inflow at the end of the time period, Δt
O_1 = Instantaneous outflow at the beginning of the time period, Δt
O_2 = Instantaneous outflow at the end of the time period, Δt
S_1 = Storage volume in the pond at the beginning of the incremental time period, Δt.
S_2 = Storage volume in the pond at the end of the incremental time period, Δt
Δt = Incremental routing time interval selected to subdivide hydrograph into finite time elements

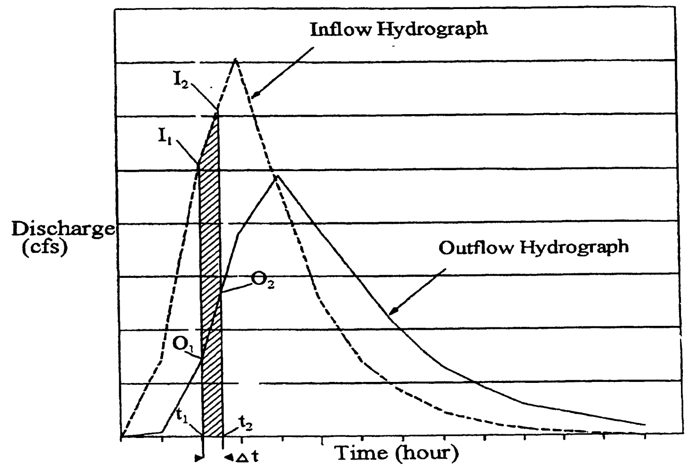


Figure 7.7 Routing Hydrograph Schematic

Step 1 Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

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

Step 3 Use the stage-storage and stage-discharge data from Step 1 to develop a storage indicator numbers table that provides storage indicator values, $(S/\Delta t) + O/2$, versus stage. A typical storage indicator numbers table contains the following column headings:

STORAGE INDICATOR TABLE

(1) Stage (ft)	(2) Discharge O (cfs)	(3) Storage S (ft^3)	(4) $O_2/2$ (cfs)	(5) $S_2/\Delta t$ (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)

A. The discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.

B. The subscript 2 is arbitrarily assigned at this time.

C. The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Step 4 Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column (6). An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed.

Step 5 A supplementary curve of storage (column 3) vs. $S_2/\Delta t + O_2/2$ (column 4) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of storage vs. time can be developed from this curve.

Step 6 The routing can now be performed by developing a routing table for the solution of Equation 7.12. A typical storage indicator numbers table contains the following column headings:

ROUTING TABLE

(1) Time (Hour)	(2) Inflow (cfs)	(3) $(I_2 + I_1)/2$ (cfs)	(4) $S_1/\Delta t + O_1/2$ (cfs)	(5) O_1 (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)	(7) O_2 (cfs)

A. Columns (1) and (2) are obtained from the inflow hydrograph.

B. Column (3) is the average inflow over the time interval.

C. The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.

D. The left side of the Equation 7.12 is determined algebraically as columns (3) + (4) - (5). This value equals the right side of equation 7.12 or $S_2/\Delta t + O_2/2$. And is placed in column (6).

E. Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (column 7).

F. Column (6) ($S_2/\Delta t + O_2/2$) and column (7) (O_2) are transported to the next line and become $S_1/\Delta t + O_1/2$ and O_1 in columns (4) and (5), respectively. Because ($S_2/\Delta t + O_2/2$) and O_2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.

G. Columns (3), (4), and (5) are again combined and the process is continued until the storm is routed.

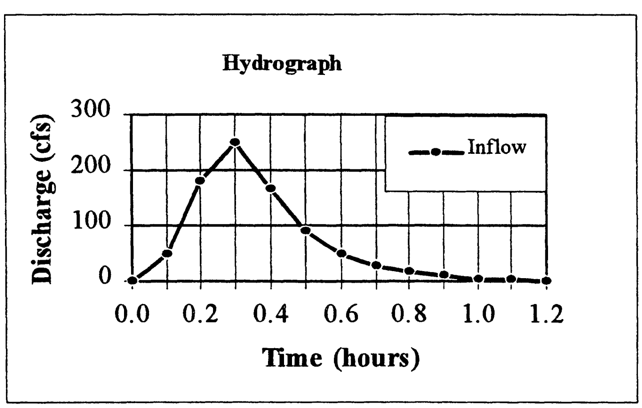
H. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.

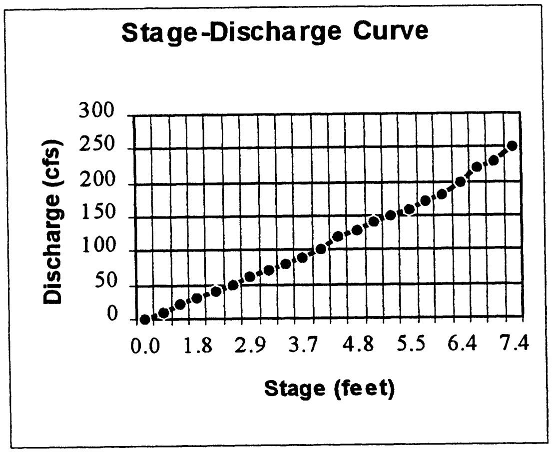
I. The designer needs to make sure the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

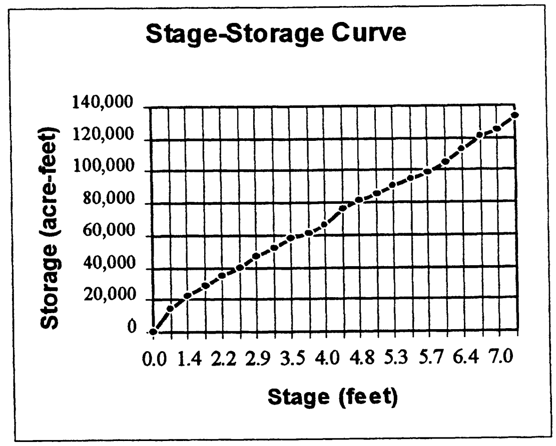
Step 7 Plot O_2 (column(7)) versus time (column (1)) to obtain the outflow hydrograph.

7.5.3 Routing Example

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. An inflow hydrograph with a peak discharge of 250 cfs is provided. The inflow hydrograph will typically be developed using hydrologic methods from the Hydrology Chapter. Stage-discharge and stage-storage curves are developed for the storage facility.







Once the stage-discharge and stage-storage curves are determined for the storage facility a storage indicator table is developed. For a each stage (column 1) a corresponding discharge (column 2) is taken from the stage-discharge curve and storage (column 3) is taken from the stage-storage curve. The remainder of the storage indicator table is filled out according to the procedure given in Section 7.5.2, Step 3.

STORAGE INDICATOR TABLE

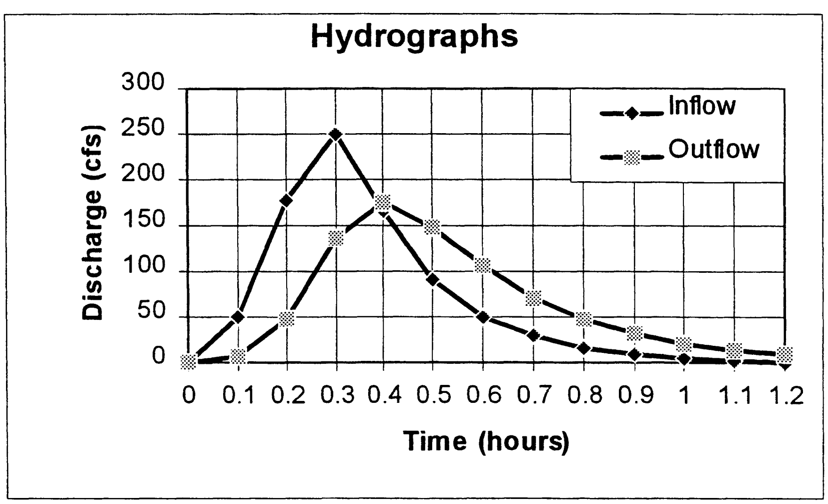
(1) Stage (ft)	(2) Discharge, O (cfs)	(3) Storage, S (ft^3)	(4) $O_2/2$ (cfs)	(5) $S_2/\Delta t$ (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)
0.0	0	0	0	0	0
0.9	10	13872.45	5	38.53	43.53
1.4	20	21842.07	10	60.67	70.67
1.8	30	28354.82	15	78.76	93.76
2.2	40	34990.65	20	97.20	117.20
2.5	50	40048.98	25	111.25	136.25
2.9	60	46902.95	30	130.29	160.29
3.2	70	52126.21	35	144.80	179.80
3.5	80	57420.97	40	159.50	199.50
3.7	90	60990.76	45	169.42	214.42
4.0	100	66405.76	50	184.46	234.46
4.5	120	75592.76	60	209.98	269.98
4.8	130	81202.94	65	225.56	290.56
5.0	140	84984.20	70	236.07	306.07
5.3	150	90718.14	75	251.99	326.99
5.5	160	94582.35	80	262.73	342.73
5.7	170	98479.98	85	273.56	358.56
6.0	180	104389.4	90	289.97	379.97
6.4	200	112387.1	100	312.19	412.19
6.8	220	120521.0	110	334.78	444.78
7.0	230	124639.5	115	346.22	461.22
7.4	250	132980.1	125	369.39	494.39

After the storage indicator table is completed, a routing table is developed. The inflow hydrograph is entered time (column 1) and inflow (column 2). The remainder of the table is filled out according to the procedure given in Section 7.5.2, Step 6. The outflow (column 7) is then analyzed to make sure the design criteria were met.

ROUTING TABLE

(1) Time (Hour)	(2) Inflow (cfs)	(3) $(I_2 + I_2)/2$ (cfs)	(4) $S_1/\Delta t + O_1/2$ (cfs)	(5) O_1 (cfs)	(6) $S_2/\Delta t + O_2/2$ (cfs)	(7) O_2 (cfs)
0	0	0	0	0	0	0
0.1	50	25	0	0	25	5.74
0.2	178	114	25	5.74	133.26	48.43
0.3	250	214	133.26	48.43	298.83	135.33
0.4	165	207.5	298.83	135.33	371	175.81
0.5	90	127.5	371.00	175.81	322.69	147.94
0.6	50	70	322.69	147.94	244.75	105.79
0.7	29	39.5	244.75	105.79	178.46	69.32
0.8	16	22.5	178.46	69.32	131.64	47.58
0.9	9	12.5	131.64	47.58	96.56	31.19
1.0	5	7	96.56	31.19	72.37	20.73
1.1	3	4	72.37	20.73	55.64	14.46
1.2	1	2	55.64	14.46	43.18	9.92

Typically a storage facility will be analyzed for a series of storm events and drainage conditions. The designer is interested in pre-development and post-development hydrographs. A common design criteria is that the pre-development peak discharge will not exceed the post-development peak discharge after being routed through the storage facility. The designer may evaluate multiple events, the design event (10-50 year) is analyzed and a major flood event (typically 100 year) is checked to insure the storage facility does not cause flood damage. The results from the calculations are compared against the design criteria, if the storage facility fails to meet the design criteria, it must be re-designed by changing the geometry and/or outlet design. This is an iterative procedure, and continues until all design goals are met.



7.6 REFERENCES

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Chapter 8 STORM DRAINAGE SYSTEMS

8.1 INTRODUCTION

This chapter provides guidance for the planning and design of pavement drainage systems. The complete system will be referred to as a storm drain system and will normally consist of curbs and/or gutters, inlets or catch basins, laterals or leads, trunk lines or mains, junction chambers, manholes, and ponds. Most aspects of storm drain design such as system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, and hydraulic grade line calculations are included.

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- the wide roadway sections, flat grades, both in longitudinal and transverse directions, shallow water courses, absence of side channels;
- the more costly property damage which may occur from ponding of water, or from flow of water through developed areas;
- the fact that the roadway section must not only carry traffic, but also act as a channel to convey water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic.

The most serious effects of an inadequate roadway drainage system are:

- damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property,
- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway and increased potential for accidents,
- weakening of base and subgrade due to saturation from frequent ponding of long duration.

8.1.1 Definition

A storm drain is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a channel, water body, or piped system. It consists of one or more pipes connecting one or more inlets. A storm drain may be a closed-conduit, open-conduit, or some combination of the two. The terminology "storm sewer" which has been in general use for many years, is gradually being replaced with the term "storm drain" to differentiate between sanitary sewers and storm drains. Storm drain will be used throughout this manual.

The purpose of a storm drain is to collect storm water runoff from the roadway and convey it to an outfall. Storm drain design generally consists of three major parts:

- system planning which includes data gathering and outfall location;
- pavement drainage which includes pavement geometrics and inlet spacing;
- location and sizing of the mains and manholes.

8.1.2 Concept Definitions

Following are discussions of concepts which will be important in a storm drainage analysis and design. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Bypass	Flow which goes past an inlet on grade and is carried in the street or channel to the next inlet downgrade. Is also referred to as carryover or runby.
Check Storm	The use of a lesser frequency event, such as a 50 year storm, to assess the flood hazard at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.
Combination Inlet	A drainage inlet is usually composed of a curb-opening inlet and a grate inlet, but may include a grate inlet and a slotted drain inlet.
Crown	The crown, sometimes known as soffit, is the top inside of a pipe.

Culvert	A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have one or two inlets connected to it to convey drainage from the median area.
Curb-Opening Inlet	A drainage inlet consisting of an opening in the roadway curb.
Drop Inlet	A drainage inlet with a horizontal or nearly horizontal opening.
Equivalent Cross Slope	An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.
Flanking Inlets	Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets are to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
Flow (Q)	Flow refers to a quantity of water which is being conveyed .
Frontal Flow	The portion of the flow which passes over the upstream side of a grate.
Grate Inlet	A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale or channel.
Grate Perimeter	The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.
Gutter	That portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, usually 2.0′ at a cross-slope of say 0.06 ft/ft, and the parking lane, shoulder, or pavement at a cross-slope of a lesser amount, say 0.02 ft/ft. A uniform gutter section has one constant cross-slope.
Hydraulic Grade Line (HGL)	The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).
Inlet	The term "inlets" refers to all types of inlets such as apron inlets, grate inlets, curb inlets, and slotted inlets. The term catch basin has been dropped from this manual and replaced with the appropriate type of inlet.
Inlet Efficiency	The ratio of flow intercepted by an inlet to total flow in the gutter.
Invert	The invert is the inside bottom of the pipe.
Lateral Line	A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 15 inches or less in diameter and is tributary to the trunk line.
Manhole	Hydraulic structure that is included in a storm drain system to provide access to storm drain pipes for inspection and cleanout. Manhole structures are the same as inlet structures except for the castings and cover. Manhole structures are also referred to as access holes.
Pressure Head	Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.
Runby	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets can be designed to allow a certain amount of runby for one design storm and larger or smaller amounts for other storms. Runby is also referred to as bypass or carry-over.

Sag Point/ Major Sag Point	A low point in a vertical curve. A major sag point refers to a low point that can overflow only if water can pond to a depth of two feet or more.
Scupper	A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.
Side-Flow Interception	Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.
Slotted Drain Inlet	A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow. Two types in general use are the vertical riser and the vane type.
Storm Drain	A storm drain is a closed conduit that conveys storm water that has been collected by inlets to an outfall. It generally consists of laterals or leads, and trunk lines or mains. Culverts connected to the storm drainage system are considered part of the system.
Splash-Over	Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.
Spread (T)	The width of stormwater flow in the gutter measured laterally from the roadway curb.
Trunk Line	A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures or manholes. A trunk line is sometimes referred to as a "main".
Velocity Head	Velocity head is a measure of the kinetic energy of flowing water expressed as a height or head of water, ($V^2/2g$).

8.2 DESIGN CRITERIA

Design criteria establishes the standards by which a policy is placed into action. They form the basis for the selection of the final design configuration. Listed below are the design criteria which shall be considered for storm drain systems.

8.2.1 Policy

Policy is a set of goals that establish a definite course or method of action and are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. The following policies are specific to storm drain systems.

Storm Drains

- Storm drain systems should have adequate capacity so that they can accommodate runoff that enters the system for the design frequency.
- Storm drain systems should be designed with future development in mind if it is appropriate.
- The storm drain system for a major vertical sag point that can't overflow elsewhere until the depth of water is two feet or greater, should have a greater level of flood protection to decrease the depth of ponding on the roadway and bridges.
- Where feasible, storm drains shall be designed to avoid existing utilities. The recommended minimum distance is 10 feet when the storm drain system is parallel to the waterline and 1 foot when the storm drain system crosses a waterline.
- Attention shall be given to the storm drain outfall design to insure that the potential for erosion is minimized.
- Drainage system design should be coordinated with the proposed staging of large construction projects in order to maintain an outlet throughout the construction project period.
- The placement and hydraulic capacities of storm drainage structures and conveyances should be designed to take into consideration potential damage to adjacent property and to minimize traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.
- Storm drain placement and capacity should be consistent with local storm water management plans.

Pavement Drainage

- Minimum pavement cross slope should be selected to ensure drainage of pavement. For multilane pavements there is often an 0.005 ft/ft increase in the cross slope for each additional lane beyond the first lane from the crown. Specific cross slope design criteria are provided in the Road Design Manual.
- In municipal areas, a minimum time of concentration of seven minutes is recommended for calculation of runoff from paved areas, all other areas should be calculated on a case by case basis.
- Curbs or dikes, inlets, or flumes are used where runoff from the pavement would erode fill slopes and/or to reduce right-of-way needed for shoulders, channels, etc.
- Where storm drains are necessary, pavement sections are usually curbed.

Gutter

- Gutter grades are recommended to be at least 0.5 percent for curbed pavement with a minimum grade of 0.35 percent.
- Composite gutter sections have greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred.
- Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the shoulder, parking lane, or pavement section.

Inlets

- Drainage inlets are sized and located to limit the spread of water on traffic lanes in accordance with the design criteria specified in Table 8.1.
- The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades.
- All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel.
- Curb boxes are not recommended at locations where grate inlets on grade are utilized.
- Curb inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities. When grate inlets are used at sag locations, assume they are half plugged with debris and size accordingly.
- Mn/DOT Standard Plate Number 4125 frame is preferred when combination grate/curb-opening inlets are utilized at sag points.
- In locations where significant ponding may occur, such as at underpasses or sag vertical curves in depressed sections, recommended practice is to place flanking inlets on each side of the low point inlet in the sag.

Manholes

- The maximum spacing of storm drain access structures whether manholes or inlets, should be approximately 400 feet for 12 inch through 54 inch diameter storm drains and approximately 600 to 800 feet for 60 inch and larger diameter storm drains.
- Minimum manhole diameter is limited by the maximum pipe size and the deflection angle of the pipes.

Pipes

- A minimum velocity of 3 fps is desirable in a storm drain in order to prevent sedimentation from occurring in the pipe.
- Water tight joints are used if velocities are high, or if the pipe is placed in the ground water table.

Bridge Decks

- Many bridges do not require any drainage structures. Equation 8.4 can be utilized to determine the maximum length of deck permitted without drainage structures.
- Zero gradients, sag vertical curves and superelevation transitions with flat pavement sections should be avoided on bridges.
- The minimum desirable longitudinal slope for bridge deck drainage is 0.5 percent.
- Runoff should be handled in compliance with applicable stormwater quality regulations.

Detention Storage

- Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and channels, and other detention storage facilities. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate.
- Detention storage should be considered in highway drainage design where:
  - existing downstream conveyance facilities are inadequate to handle peak flow rates from highway storm drainage facilities;
  - where the highway would contribute to increased peak flow rates and aggravate downstream flooding problems;
  - as a technique to reduce the right-of-way, construction, and operation costs of outfalls from highway storm drainage facilities;
  - or as a treatment technique to improve water quality by removing sediment and/or pollutants. (See the Storage Facilities chapter)

Roadside and Median Ditches

- Large amounts of runoff should be intercepted before it can reach the highway in order to minimize the deposition of sediment and other water transported debris on the roadway, and to reduce the amount of water carried in the gutter section.
- Slope median areas and inside shoulders toward a center depression to prevent runoff volume from running across the pavement.
- Surface channels should have adequate capacity for the design runoff volume and should be located and shaped in a manner that does not present a traffic hazard.
- Channels should have a vegetative lining when possible, based upon design velocity criteria. Appropriate linings may be necessary where vegetation will not control erosion.

8.2.2 Design Frequency and Allowable Spread

The design storm frequency selected for pavement drainage should be consistent with the frequency selected for other components of the storm drain system. The design frequency for pavement drainage is tied to the allowable water spread on the pavement and is related to ADT. Mn/DOT has established criteria for design frequency and water spread as shown in Table 8.1.

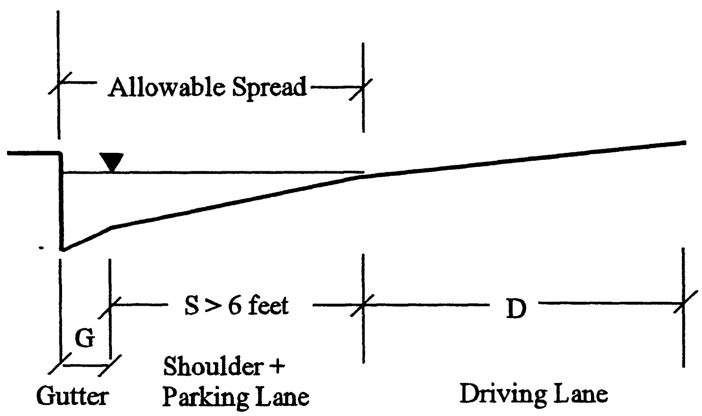
The factor that governs how much water can be tolerated in the curb and gutter section and on the adjacent roadway is known as water spread. Water is allowed to spread onto the roadway area within tolerable limits because it is usually not economically feasible to keep it within a narrow gutter width. The designer keeps track of the water spread by calculation, and when the allowable spread is reached, an inlet is proposed to intercept a portion of the flow. Placement of inlets is used to prevent gutter flow from exceeding the allowable water spread criteria.

Ramps, loops, turn lanes, acceleration and deceleration lanes should generally be designed for the same frequency as the mainline. Exceptions to the table values will be permitted for up to ½ driving lane for existing conditions. Where the speed limit is 35 miles per hour (MPH) or less and there are no shoulders or parking lanes, allowable spread can encroach up to ½ of the driving lane for short periods of time. High capacity inlets such as slotted vane drains should be considered at locations where excessive spread may cause inconvenience or safety hazards to the motorists.

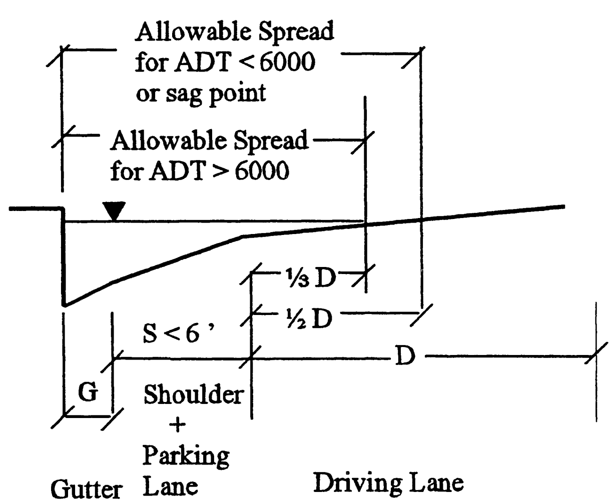
Table 8.1 Design Frequency for Storm Drains

PROJECTED 2-WAY ADT (vehicles per day, VPD)	DESIGN FREQUENCY (Year)	ALLOWABLE SPREAD[1]
> 6000	10	P, S, or ⅓ D
	50 Year at sag point[2]	P, S, or ½ D
2000-6000	10	P, S, or ½ D
1000-1999	5	P, S, or ½ D
< 1000	3	P, S, or ½ D

[1]P = Parking lane, S = Shoulder of 6′ or more, D = Driving lane if there is no shoulder.
[2]Sag Point refers to a true sag where flooding of 2' or more can occur.



Allowable spread when shoulder width is equal to or greater then 6 feet.



Allowable spread when the sum of the shoulder and parking lane is less than 6 feet.

8.3 SYSTEM PLANNING

System planning prior to commencing the design of a storm drain system is essential. The design of any storm drainage system involves the accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

The typical elements of the design process are listed below.

- Data collection
- Coordination with other agencies
- Preliminary sketch
- Inlet location and spacing
- Plan layout of storm drain system
  - locate main outfall
  - determine direction of flow
  - locate existing utilities
  - locate connecting mains
  - locate manholes
- Size the pipes
- Develop the energy and hydraulic grade line
- Prepare the plan
- Provide documentation

8.3.1 Required Data

The designer should be familiar with land use patterns, the nature of the physical development in the area(s) to be served by the storm drainage system, local stormwater management plans, and the ultimate pattern of drainage (both overland and by storm drains) to existing outfall locations. Furthermore, the designer should consider the location of the outfall and applicable water quality rules and regulations.

Onsite review and actual topographic survey data are often needed for storm drain system design. In addition, photogrammetric mapping has become an important method of obtaining the data required for drainage design, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps, available from the United States Geological Survey (USGS), Natural Resource Conservation Service (NRCS), many municipalities, some county governments, and even private developers are also valuable sources of the kind of data needed for a proper storm drainage design.

Developers and governmental planning agencies should be consulted regarding drainage plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer is to anticipate these changes and consider them in the storm drainage design. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

8.3.2 Cooperative Projects

Where a mutual economic benefit and a demonstrated need exists, Mn/DOT endorses cooperative storm drain projects with cities and municipalities. Early coordination with the governmental entity involved is necessary to determine the scope of the project. Each cooperative project must be initiated by a resolution adopted by the governing body of the municipality either requesting the improvements and/or indicating its willingness to share the cost of a state project, or indicating the municipality's intention to make certain improvements and requesting state cost participation in the municipal project. The policy regulating cooperative construction agreements is contained in the Mn/DOT Policy Manual under Cooperative Construction Projects With Municipalities. An explanation of the various methods of cost proration computation is contained therein.

8.3.3 Preliminary Considerations

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, overhead sign foundation locations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunk line layouts, and a clear definition of the outfall location and characteristics. The preliminary sketch should be reviewed along with the traffic staging plans and soil recommendations to identify conflicts that could arise during construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments, and refinements. Unless the proposed system is very simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

When designing storm drain systems, planning is needed to avoid utilities and deep cuts. Avoid unnecessary excavation to minimize adverse impacts upon utilities, reduce cost and potential negative environmental impacts.

In some cases, traffic must be maintained or temporary bypasses constructed and temporary drainage provided for during the construction phase. Further consideration should be given to the actual trunk line layout and its constructibility. For example, will the proposed location of the storm drain interfere with inplace utilities or disrupt traffic? Some instances may dictate a trunk line on both sides of the roadway with very few laterals while other instances may call for a single trunk line. Storm system layout is usually designed to minimize cost but may be controlled by other physical features.

It is generally not a good idea to decrease pipe size in a downstream direction regardless of the available pipe gradient because of potential plugging with debris. However for large pipes, 36 inches diameter and above, substantial savings can result if the gradient will allow pipe size reductions. In those cases, reductions can be made with smooth transitions such as commercially available reducers, at the discretion of the designer.

8.4 HYDROLOGY

The Rational method is the most common method in use for design of storm drains when momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 200 acres or less. Drainage systems involving detention storage and pumping stations require the development of a runoff hydrograph. Other hydrological methods include unit hydrograph procedures, or SCS TR-55 procedures. Refer to the Hydrology Chapter for a discussion of these methods.

8.4.1 Rational Method

The Rational Equation is written as follows:

$$Q = CIA = \left(\sum CA\right)I \tag{8.1a}$$

Where: Q = peak runoff rate (cfs)
C = runoff coefficient
I = rainfall intensity (inches/hour)
A = drainage area (acres)

Runoff Coefficient

The runoff coefficient is a dimensionless value representing characteristics of the watershed that affect how much of the rain will become runoff. Coefficient selection is based on land use and soil conditions. The weighted C value is to be based on a ratio of the drainage areas associated with each C value. The runoff coefficients for various types of surfaces are provided in Table 3.7.

$$Weighted\ C = \frac{A_1C_1 + A_2C_2 + \ldots + A_nC_n}{A_1 + A_2 + \ldots + A_n} \tag{8.1b}$$

Rainfall Intensity

Rainfall intensity (I) is an average rainfall intensity for a duration equal to the time of concentration and for a select recurrence interval. Rainfall intensity is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. Intensity is a rate of rainfall over an interval of time such that intensity multiplied by duration equals total amount of rain. The value of I for various concentration times and recurrence frequencies is provided in Section 3.5.4.

Time of Concentration

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. The designer is usually concerned about two different times of concentration, one for inlet spacing and the other for pipe sizing. There is a major difference between the two times.

- Inlet Spacing
  The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. For pavement drainage, when the total time of concentration to the upstream inlet is less than seven minutes, a minimum t_c of seven minutes should be used to estimate the intensity of rainfall. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. Runby travel time between inlets is not considered. In the case of a constant roadway grade and relatively uniform contributing drainage area, the time of concentration for each succeeding inlet could also be constant.

- Pipe Sizing
  The time of concentration for pipe sizing is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. It generally consists of two components, the time to flow to the inlet which can consist of overland and channel or gutter flow, and the time to flow through the storm drain to the point under consideration.

The preferred method of calculating overland flow time is the kinematic wave approach. Channel travel time of concentration can be developed using Manning's equation. Travel time within the storm drain pipes can be estimated by the relation:

$$t_t = \frac{L}{60V} \tag{8.2}$$

Where: t_t = travel time (min)
L = length of pipe in which runoff must travel (ft)
V = estimated or calculated velocity (ft/s)
Velocity is based upon normal depth of flow for the design discharge.

To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I). There could be exceptions to this generality, for example where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c and the one that produces the larger discharge controls. Note that when computing the peak discharge with the shorter t_c, not all the area from the basin with the longest t_c will contribute runoff.

$$A_c = A\left(\frac{t_{c1}}{t_{c2}}\right) \tag{8.3}$$

Where: A_c = contributing area
A = area of the basin with longest time of concentration (t_{c2})
t_{c1} = smaller time of concentration
t_{c2} = larger time of concentration

8.4.2 Detention

When the Rational Method is utilized, detention is accommodated to some extent by the process of building a design flow moving downstream through the storm drainage system as described later in this chapter. As the time of concentration is recalculated at each manhole by adding in the time of travel between manholes, a new lower rainfall intensity and new discharge is calculated. In a sense, the effect of detention by the carrier pipe itself is reflected in the design.

A more direct application of detention involves the use of detention ponds, usually located in the contributing watersheds and often immediately upstream of the entrance to the storm drainage system. By introducing detention ponds, the designer is able to attenuate the peak of the runoff hydrograph, thus reducing the immediate design discharge rate. Estimation of the effects of detention requires a reservoir routing procedure such as that presented in the Storage Facilities Chapter.

8.5 PAVEMENT DRAINAGE

Roadway features considered during gutter, inlet, and pavement drainage calculations include:

- longitudinal slope,
- cross slope,
- curb and gutter sections,
- roadside and median ditches,
- pavement texture, and
- bridge decks.

The pavement width, cross slope, and profile control the time it takes for storm water to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow which can be carried in the gutter section.

8.5.1 Longitudinal Slope

A minimum longitudinal gradient is more important for curbed pavement than for uncurbed pavement since the curbed section is susceptible to the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge. To assure adequate drainage a minimum gutter grade of 0.35 percent is desirable.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent is desirable within 50 feet of the level point in the curve. This is accomplished where the length of the curve divided by the algebraic difference in grades is equal to or less than 167. Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

8.5.2 Cross Slope

Pavement cross slope should be adequate to provide proper drainage. The design rate of cross slope will vary with surface type, and road classification. The Mn/DOT Road Design Manual contains accepted values for pavement cross slopes.

On multi-lane divided highways, each roadway may be crowned separately or all lanes of each roadway may flow in one direction. Consider the following when selecting the cross slope direction:

- From a drainage and wet pavement safety standpoint, the preferred alternate is the crowned cross section with drainage both ways from the crown. This cross section will drain the pavement the quickest and the difference between the high and low points will be minimized. This alternate requires drainage design for both sides of the highway, and may complicate at-grade intersection traversability because of the several ups and downs of the cross section, although this can be lessened by transitioning to flatter cross slopes through the intersection. Therefore this alternate is most suited for divided highways with wide depressed medians and full or partial control of access.
- An uni-directional slope towards the outside edge will reduce traversability problems at at-grade intersections. It will require drainage design for one side of each roadway. With each lane contributing runoff the potential for hydroplaning is increased. During freeze-thaw periods, a safety problem can be created when snow, which has been plowed to the median, melts and drains across the travel lanes. Slippery conditions can be created when the melting runoff refreezes.

A careful check should be made of designs to minimize the number and length of flat pavement sections in super elevation transition areas where the cross slope is approaching zero. Since cross slope is the dominant factor in removing water from the pavement, the designer should be aware that hydroplaning is a potential hazard to be considered in these areas especially when combined with flat profile grades and wide pavement sections.

8.5.3 Pavement Texture
The pavement texture is an important consideration for roadway surface drainage. Although the hydraulic design engineer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macro texture provides a channel for water to escape from the tire-pavement interface and reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining new portland cement concrete pavements while it is still in the plastic state. Retexturing of an existing portland cement concrete surface can be accomplished through pavement grooving. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high speed conditions.

8.5.4 Curb and Gutter
Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. The curbs serve several purposes which include containing the surface runoff within the roadway and away from adjacent properties, preventing erosion, providing pavement delineation, and enabling the orderly development of property adjacent to the roadway.

The Road Design Manual is recommended for guidance in selecting a particular curb and gutter. A curb and gutter forms a triangular channel that can be an effective hydraulic conveyance facility which can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. Where curbs are used, composite gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface. This is the width the hydraulic engineer is most concerned about in curb and gutter flow, and limiting this width becomes a very important design criterion.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the highway, in order to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water which must be carried in the gutter section.

8.5.5 Roadside and Median Ditches
Roadside ditches are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside ditches cannot be used on most urban arterials. 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. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted in the ditch as appropriate. It is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

8.5.6 Bridge Decks
Drainage of bridge decks is similar to other curbed roadway sections. Deck drainage is often less efficient, because cross slopes are flatter, and small drainage inlets or scuppers have a higher potential for clogging by debris. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5 percent. If a sag curve cannot be avoided, the sag should not occur directly over a water body unless it is a treatment pond. Deck drainage can generally not be discharged directly to a water body without providing a pond or other means of intercepting a hazardous spill. Runoff should be handled in compliance with applicable stormwater quality regulations.

The gutter spread should be checked to insure compliance with the design criteria in Section 8.2. A large number of bridges will not require any drainage structures at all. To determine the length of deck permitted to achieve the allowable spread, the following equation which is based on a uniform cross slope can be utilized:

$$L = \frac{24400\left(S_x^{1.67}\right)\left(S^{0.5}\right)\left(T^{2.67}\right)}{CnIW} \quad \textbf{(8.4)}$$

Where: S = Longitudinal slope (ft/ft)
S_x = cross slope (ft/ft)
W = width of drained deck (ft)
C = runoff coefficient
I = rainfall intensity (in/hr)
n = Manning's n
T = Allowable spread (ft)

The use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. For bridges that require deck drains, it may be necessary to provide a drainage system on the bridge to convey water off the bridge.

8.5.7 Median and Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated super-elevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. Locating the center of the grate inlet 2.0′ from the edge of barrier as shown in Figure 8.1, has the advantage of allowing approximately 1′ for storage of snow and ice during snow removal operations.

8.5.8 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems, it is often necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes.

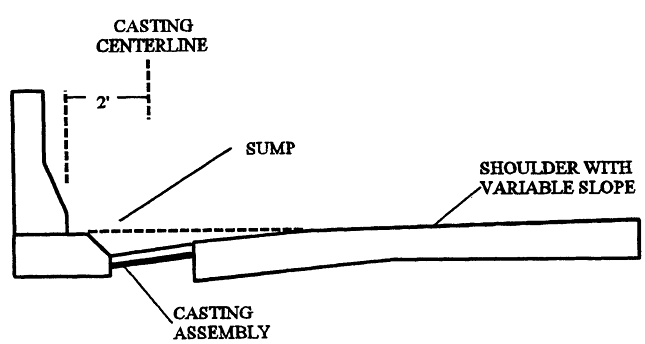


Figure 8.1 Median Barrier Inlet

8.6 GUTTER FLOW

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane, or pavement section. Typical gutter section shapes include: uniform cross slope channels, composite gutter sections, and V shape gutter sections. Either nomographs (see Figures 8.2, 8.3 and 8.4), programable calculators or computer programs can be utilized to perform gutter flow calculations.

$$Q = \frac{0.56}{n} S_x^{1.67} S^{0.5} T^{2.67} \tag{8.5}$$

Where: Q = gutter flow (cfs)
S_x = cross-slope (ft/ft)
S = longitudinal slope (ft/ft)
T = spread (ft)
n = Manning's n

Table 8.2 Manning's n for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002

Source: HDS-3 (FHWA, 1961)

Two sets of procedures are provided, CONDITION 1 where gutter flow (Q) is known and spread (T) is calculated and CONDITION 2 where spread (T) is known and gutter flow (Q) is calculated.

8.6.1 Uniform Cross Slope Procedure

The nomograph in Figure 8.3 is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 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.

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

CONDITION 2: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 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 Q_n from the intersection of that line on the capacity scale.

Step 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 (Q_n) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

8.6.2 Composite Gutter Sections Procedure

Figure 8.4 can be used to find the flow in a gutter section with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter section cross slope (S_w), gutter section width (W), Manning's n, total flow (Q), and a trial value of the curbed channel capacity above the gutter section (Q_s).

Step 2 Calculate the gutter flow (Q_w) in the gutter section over width, W, using the equation:

$$Q_w = Q - Q_s \tag{8.6}$$

Where: Q_w = gutter flow (cfs)
Q = total flow (cfs)
Q_s = capacity above the gutter section (cfs)

Step 3 Calculate the ratios Q_w/Q and S_w/S_x and use Figure 8.4 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, longitudinal slope (S), and cross slope (S_x) to find the actual value of Q_s from Figure 8.3.

Step 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: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters, including spread (T), spread above the gutter section (T_s), cross slope (S_x), longitudinal slope (S), gutter section cross slope (S_w), gutter section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 8.3 to determine the capacity of the curbed channel above the gutter section flow (Q_s). Use the procedure for uniform cross slopes - CONDITION 2, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 8.2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total flow using the equation:

$$Q = \frac{Q_s}{(1 - E_o)} \tag{8.7}$$

Where: Q = total curbed channel flow rate (cfs)
Q_s = flow capacity of the curbed channel above the gutter section (cfs)
E_o = ratio of frontal flow to total flow (Q_w/Q)

Step 5 Calculate the gutter flow (Q_w) in width (W), using Equation 8.6.

NOTE: Figure 8.3 can also be used to calculate the flow in a composite gutter section.

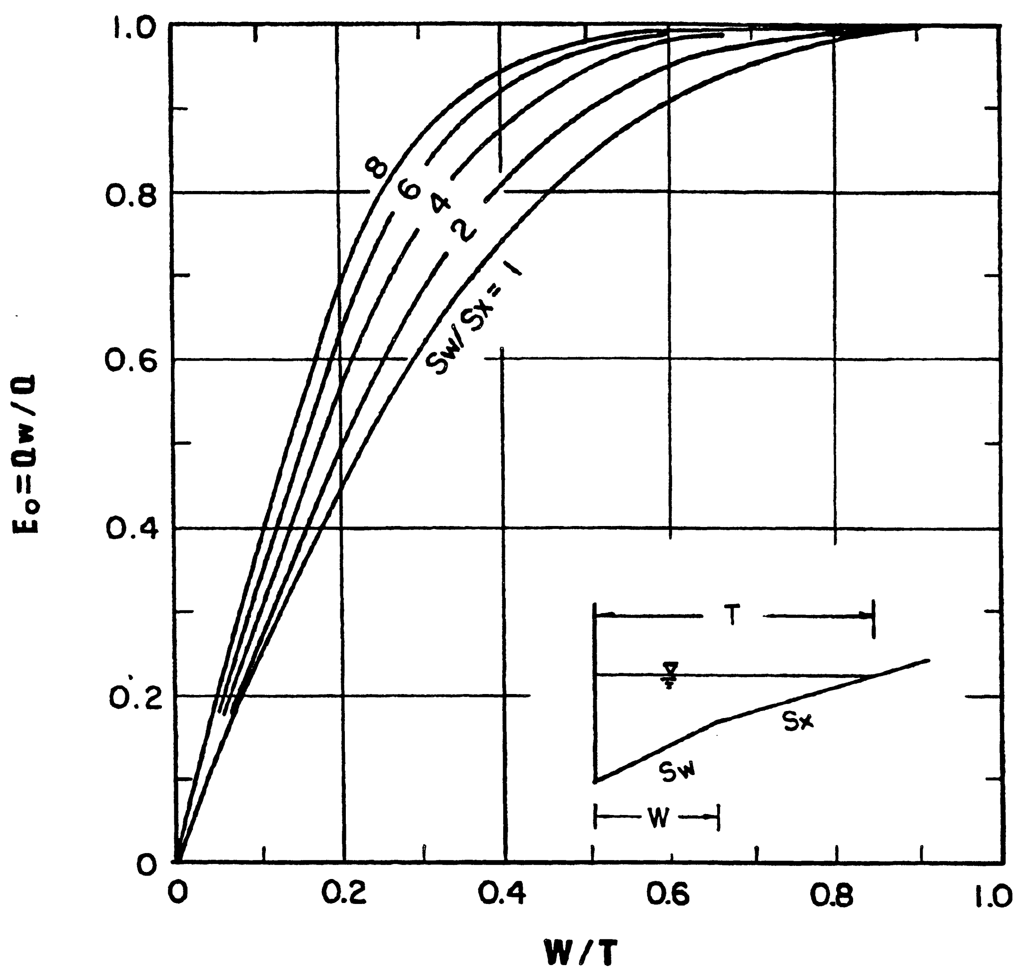


Figure 8.2 Ratio Of Frontal Flow To Total Gutter Flow
Source: HEC-12 (FHWA, 1984)

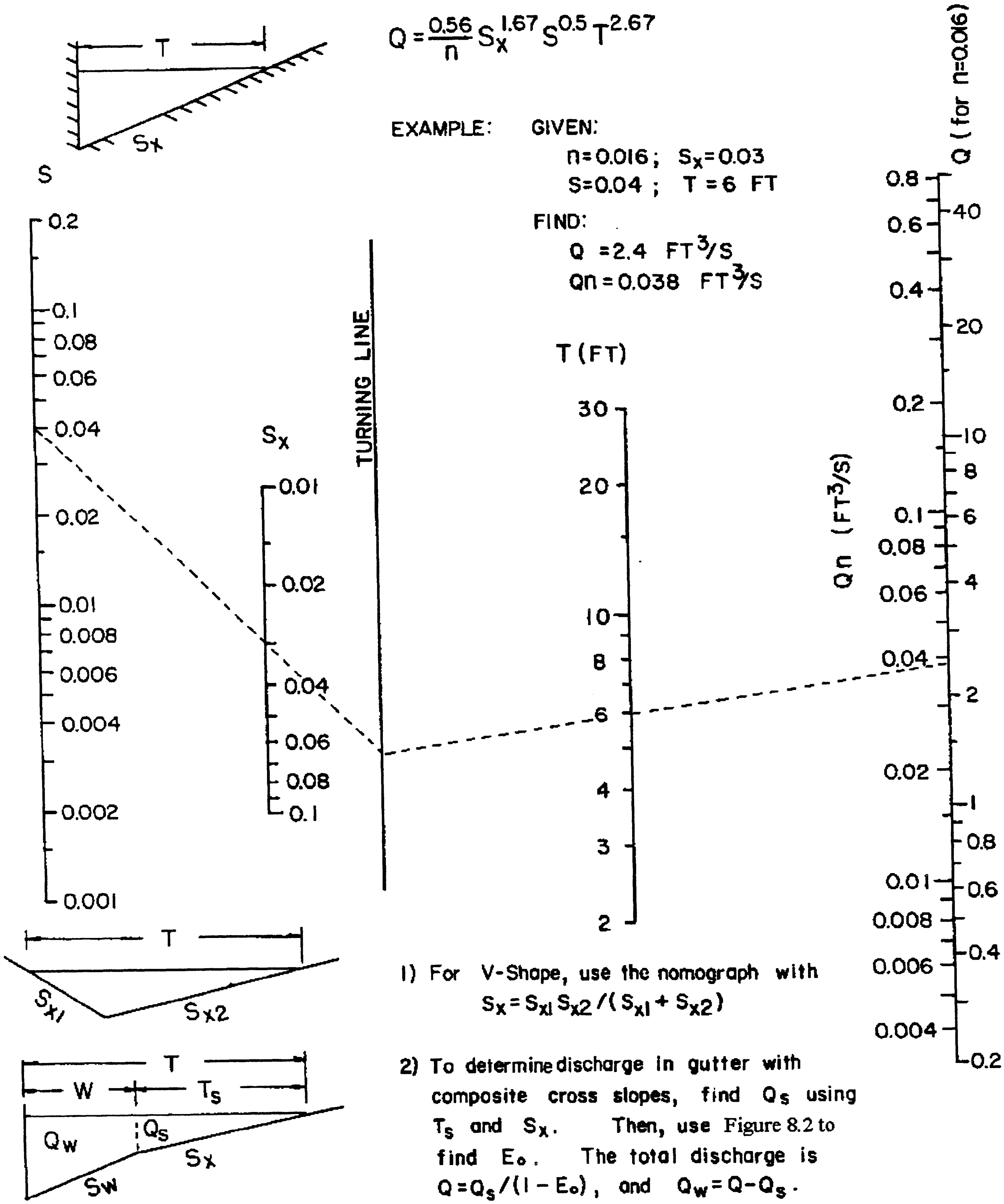


Figure 8.3 Flow in Triangular Gutter Sections
Source: HEC-12 (FHWA, 1984)

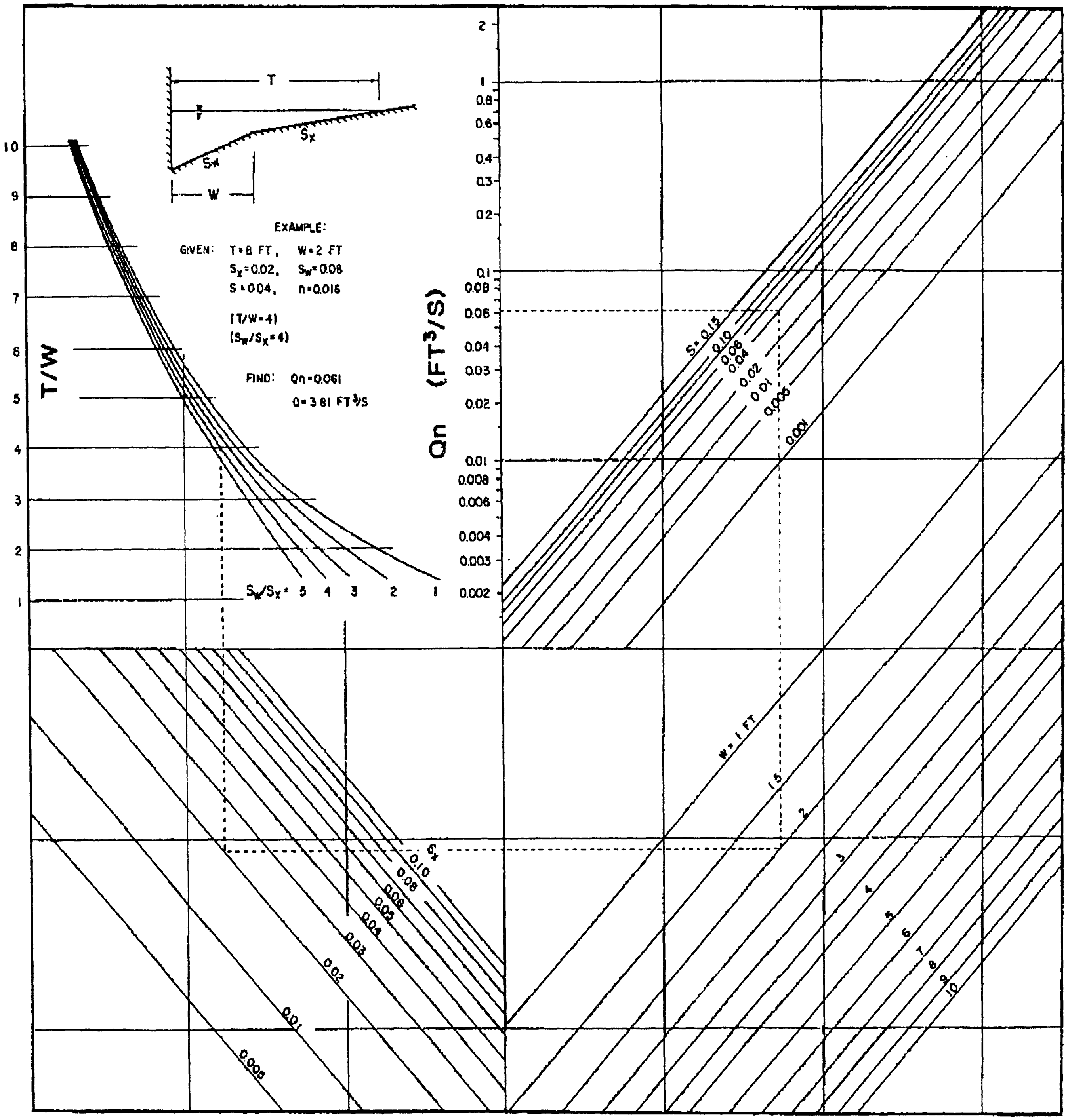


Figure 8.4 Flow in Composite Gutter Sections
Source: HEC-12 (FHWA, 1984)

8.6.3 V Type Gutter Sections Procedure

Figure 8.3 can also be used to solve V Type channel problems. The spread (T) can be calculated for a given flow (Q) or the flow can be calculated for a given spread. This method can be used to calculate flow conditions in the triangular channel adjacent to concrete median barriers. (See Figure 8.5)

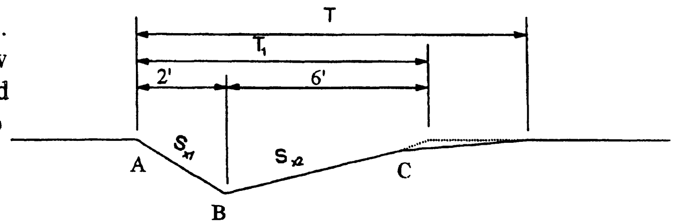


Figure 8.5 V Type Gutter Section

CONDITION 1: Given gutter flow (Q), find spread (T).

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), Manning's n, total flow (Q).

Step 2 Calculate S_x

$$S_x = \frac{S_{x1} S_{x2}}{\left(S_{x1} + S_{x2}\right)} \tag{8.8}$$

Where: S_x = cross slope (ft/ft)
S_{x1} = cross slope of section 1 (ft/ft)
S_{x2} = cross slope of section 2 (ft/ft)

Step 3 Solve for T_1 using the nomograph on Figure 8.3.
T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}.

Step 4 To find the actual spread, solve for depth at points B and C in Figure 8.5.

Step 5 Solve for the spread on the pavement.

Step 6 Find the actual total spread (T).

CONDITION 2: Given spread (T), find gutter flow (Q).

Step 1 Determine input parameters such as longitudinal slope (S), cross slope (S_x), Mannings n, and allowable spread.

Step 2 Calculate S_x using Equation 8.8.

Step 3 Using Figure 8.3 or Equation 8.5 to solve for Q.

8.7 INLETS

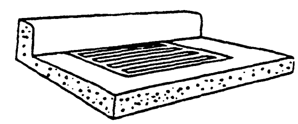
Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets should be bicycle safe unless located on highways where bicycles are not permitted. This section includes various types of inlets commonly in use and provides recommendations on the use of each type.

Table 8.3 lists the various types of inlets used by Mn/DOT. Comparisons are made between the various types for each of the inlet characteristics listed. This table should be reviewed prior to selecting a specific type of inlet or grate.

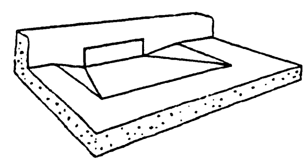
Table 8.3 Inlet Comparisons

Standard Plate	Curb Box	Bike Safe	Handle Debris	Sag Point	Max. Grade
4151 (811)	yes	yes	yes	yes[3]	2%
4152 (814)	no	yes	no	no	6%
4153 (815)	no[1]	no	yes	yes[3]	none
4154 (816)	no[1]	yes	yes	yes[3]	none
4021[2]	n/a	yes	yes	yes	0%
slotted inlet 3136, 3137, 3138	n/a	yes	yes	yes	n/a

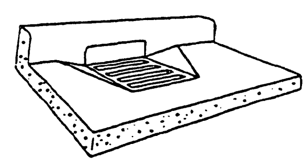
[1] Use curb box only when used at sag point.
[2] Use low point (LP) type only in sag point.
[3] Use with Frame 4125 at sag points.



Grate Inlet



Curb-Opening Inlet



Combination Inlet

Inlet Types

8.7.1 Inlet Types

Inlets used for the drainage of highway surfaces can be divided into four major classes.

Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Since they are susceptible to clogging with debris, the use of standard grate inlets alone at sag points should be limited to minor sag point locations without debris potential. Special design (oversize) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets are definitely recommended. Grates should be bicycle safe where bike traffic is anticipated and should be structurally designed to handle the appropriate loads when subject to traffic.

Curb-Opening Inlets

These inlets are vertical openings in the curb covered by a top slab. They are best suited for use at sag points and flat grades since they can convey large quantities of water and debris. They are generally not recommended for use on steep continuous grades.

Combination Inlets

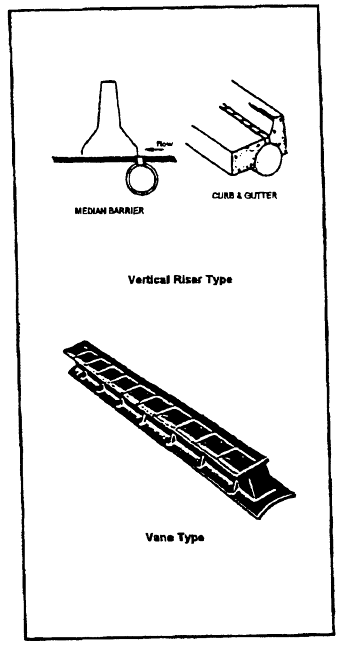
Various types of combination inlets are in use. Curb opening and grate combinations are in common use by Mn/DOT. Slotted inlets may also be used in combination with grates. In general, the vertical riser type is used for longitudinal placement upstream of the grate, while the vane type is used transversely to the grate. Engineering judgement is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each. The gutter grade, cross slope and proximity of the inlets to each other will be deciding factors. Combination grate and curb-opening inlets are desirable in sags because they can provide additional capacity in the event of plugging.

Slotted Drain Inlets
These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets. The two types of slotted inlets in general use are the vertical riser type and the vane type.

8.7.2 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (Section 8.2). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or runby. Examples of such locations are as follows:

- sag points in the gutter grade,
- upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections,
- immediately upstream and downstream of bridges,
- immediately upstream of cross slope reversals,
- on side streets at intersections,
- at the end of channels in cut sections,
- behind curbs, shoulders, or sidewalks to drain low areas,
- where necessary to collect snow melt.



Slotted Inlets

Inlets should be placed in the correct position, typically in the gutterline. In superelevated roadways it can be challenging to place inlets so that spread criteria is met, and that pavement flow is not excessive. If possible, inlets should not be located in the path where pedestrians are likely to walk.

8.7.3 Inlet Spacing

Inlets are often required to collect runoff at locations with small or insignificant drainage areas, these should be plotted on the plan first. Next, it is best to start locating inlets from the crest of the profile and work down grade to the sag points. The location of the first inlet from the crest is found by determining the length of pavement and the area back of the curb sloping toward the roadway which will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel which will meet the design criteria as specified in Section 8.2. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet can be calculated with an alternate form of the Rational Equation as follows:

$$L = \frac{43560 Q_t}{CIW} \qquad \textbf{(8.9)}$$

Where: L = distance from the crest (ft)
Q_t = maximum allowable flow (cfs)
C = composite runoff coefficient for contributing drainage area
W = width of contributing drainage area (ft)
I = rainfall intensity for design frequency (in/hr)

If the drainage area contributing to the first inlet is irregular in shape, trial and error computations will be necessary to match a design flow with the maximum allowable flow. To space successive down grade inlets, it is necessary to compute the amount of flow which will be intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the runby. The runby from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Table 8.5 is an inlet spacing computation sheet which can be utilized to record the spacing calculations, alternatively inlet spacing can be performed using a computer application.

8.7.4 Grate Inlets on Grade

The capacity of a grate inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted.

On steep slopes, a portion of the frontal flow may tend to splash over the end of the grate. Mn/DOT grate 4151 is the most prone for splash over particularly on grades steeper than 2%. When using different grate lengths or types than those listed below, consult the manufacturer to get the inlet grate capacity (Neenah, 1987), or refer to HEC-12 (FHWA, 1984) for additional information on computing V_o. Although FHWA grates are not the same as the grates utilized by Mn/DOT two of them are sufficiently similar to be considered equivalent hydraulically.

Grate Type	Mn/DOT Grate Standard Plate No.	Equivalent FHWA Grate	Splash-Over Velocity, V_o [1]
Parallel Bar	4153	P-1-7/8	8 fps
Curved Vane	4152 and 4154	Curved Vane	6 fps
Reticuline	None	Reticuline	4 fps

[1] Splash over velocity for grates 2 foot in length.

Frontal Flow Ratio

The ratio quantifies how much of the total gutter flow runs directly over the front edge of the grate. The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by Equation 8.10a. Figure 8.2 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad \textbf{(8.10a)}$$

Side Flow Ratio

The ratio quantifies how much of the total gutter flow does not pass over the front edge of the grate, but passes along the side of the grate. The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \qquad \textbf{(8.10b)}$$

Intercepted Frontal Flow Ratio

This ratio is equivalent to frontal flow interception efficiency. Figure 8.7 is a nomograph to solve for velocity in a triangular gutter section with known cross slope, longitudinal slope and spread. R_f cannot exceed 1.0. The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\left(V - V_o\right) \qquad \textbf{(8.10c)}$$

Intercepted Side Flow Ratio

The ratio is equivalent to the side flow interception efficiency of the grate. Figure 8.6 provides a graphical solution to Equation 8.10d. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = \frac{1}{\left[1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)\right]} \qquad \textbf{(8.10d)}$$

Where: E_o = ratio of frontal flow to total gutter flow
Q_w = flow in width W (cfs)
Q = total gutter flow (cfs)
W = width of depressed gutter or grate (ft)
T = total spread of water in the gutter (ft)
Q_s = ratio of side flow to total gutter flow
R_f = ratio of frontal flow intercepted to total frontal flow
V = velocity of flow in the gutter (fps)
V_o = gutter velocity where splash-over first occurs (fps)
R_s = ratio of side flow intercepted to total side flow
L = length of the grate (ft)
S_x = cross slope

To determine the efficiency of the grate at capturing gutter flow the frontal and side flow interception efficiencies are summed. The efficiency (E) of a grate is expressed as:

$$E = R_f E_o + R_s\left(1 - E_o\right) \tag{8.11}$$

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

$$Q_i = EQ = Q\left[R_f E_o + R_s\left(1 - E_o\right)\right] \tag{8.12}$$

Where: Q_i = flow intercepted by grate (cfs)
E = grate efficiency
Q = total gutter flow (cfs)
R_f = ratio of frontal flow intercepted to total frontal flow
E_o = ratio of frontal flow to total gutter flow
R_s = ratio of side flow intercepted to total side flow

Inlet interception capacity has been investigated by the FHWA. The grates tested in an FHWA research study are described in *Drainage of Highway Pavements*, HEC-12 (FHWA, 1984). These grates are not the same as the grates utilized by Mn/DOT, though two of them are sufficiently similar for on grade installations: FHWA P-1-7/8 and Mn/DOT grate 4153, Curved Vane and Mn/DOT 4152 or 4154.

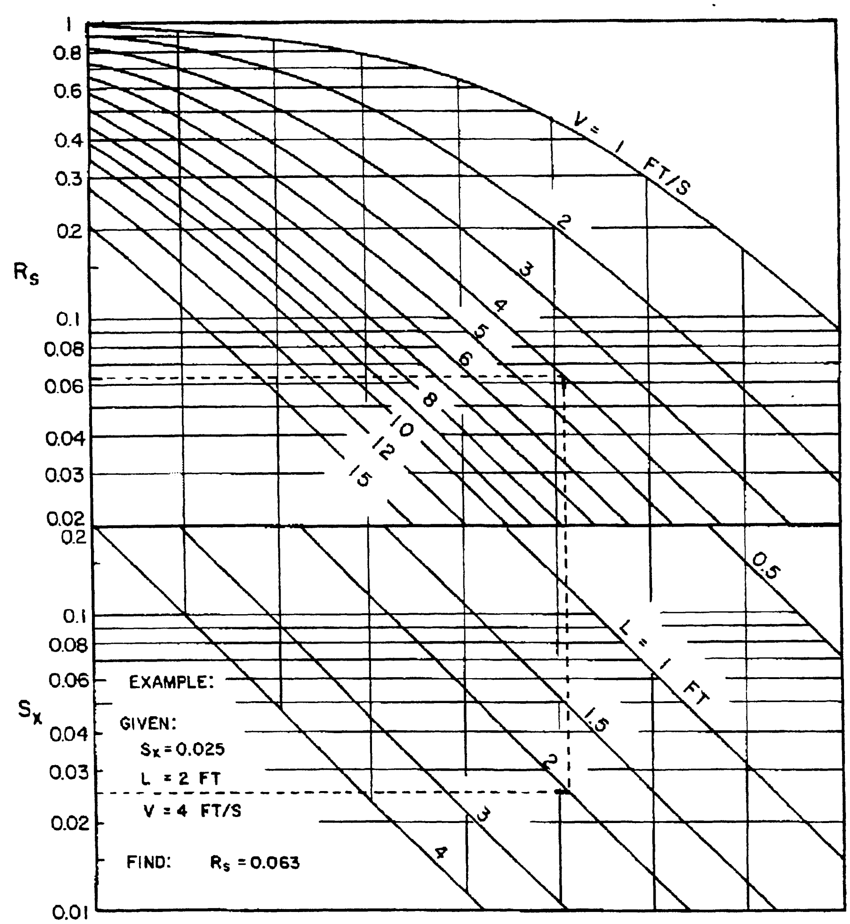


Figure 8.6 Grate Inlet Side Flow Interception Efficiency
Source: HEC-12 (FHWA, 1984)

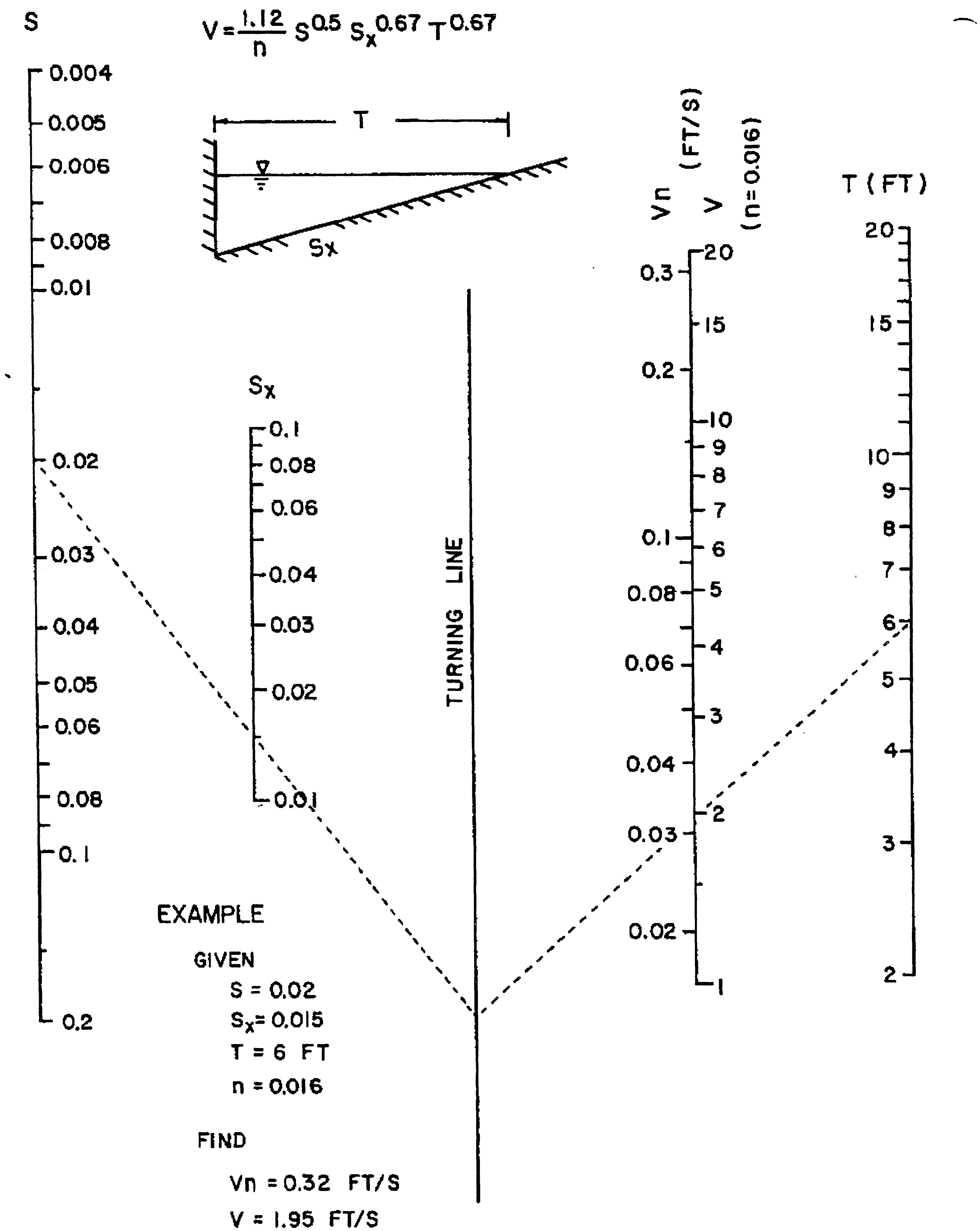


Figure 8.7 Velocity in Triangular Gutter Sections
Source: HEC-12 (FHWA, 1984)

8.7.5 Grate Inlets in Sag

Although curb opening inlets are generally preferred to grate inlets at a sag point, grate inlets can be used successfully. For minor sag points where debris potential is limited, grate inlets without a curb box can be utilized. An example of a minor sag point might be on a ramp as it joins a mainline. Curb boxes in addition to a grate are preferred at sag points where debris is likely such as on a city street. For major sag points such as on divided high speed highways, a curb opening inlet is preferable to a grate inlet because of its hydraulic capacity and debris handling capabilities. Where this is not practical such as adjacent to median barriers, special design grate inlets can be used successfully. In such instances, it is good practice to assume half the grate is plugged with debris. Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place a minimum of one flanking inlet on each side of the sag point inlet.

A grate inlet in a sag operates as a weir at smaller depths and as an orifice for greater depths. Between these depths, a transition from weir to orifice flow results in interception capacity less than that computed by either the weir equation or the orifice equation.

<u>Weir</u>

The capacity of a grate inlet operating as a weir is:

$$Q_i = C_w P d^{1.5} \qquad \textbf{(8.13)}$$

Where: P = perimeter of grate disregarding the side against curb (ft)
C_w = weir coefficient (3.0)
d = depth of water above grate (ft)

<u>Orifice</u>

The capacity of a grate inlet operating as an orifice is:

$$Q_i = C_o A (2gd)^{0.5} \qquad \textbf{(8.14)}$$

Where: C_o = orifice coefficient (0.67)
A = clear opening area of the grate (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
d = depth of water above grate (ft)

Figure 8.8 is a plot of Equations 8.13 and 8.14 for various grate sizes. Figure 8.8 was developed with the assumptions that there was no curb box, sump or plugging. The sump is not included when computing depth of water above grate, d.

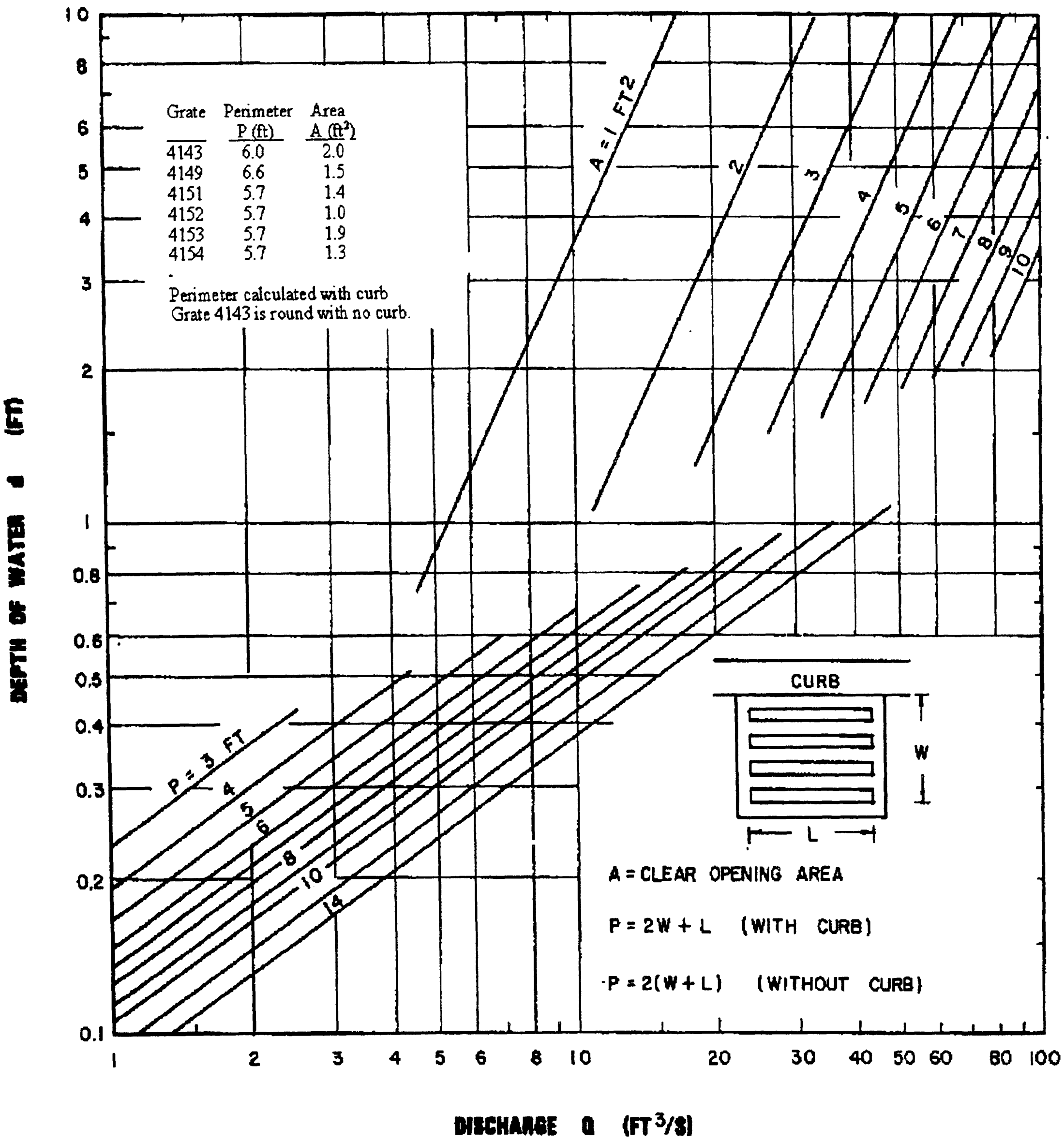


Figure 8.8 Grate Inlet Capacity In Sump Conditions
Source: HEC-12 (FHWA, 1984)

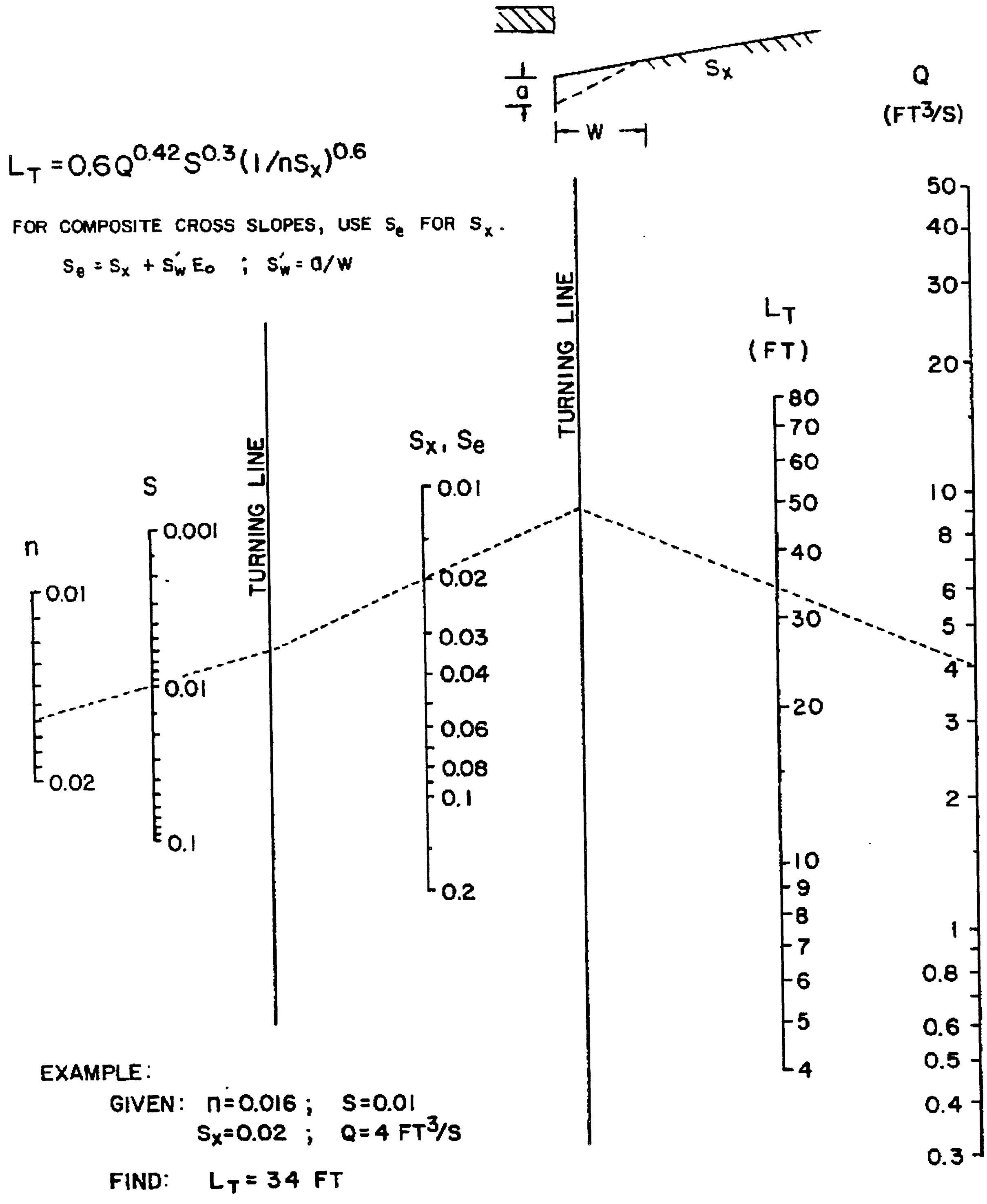


Figure 8.9 Curb Opening and Longitudinal Slotted Drain Inlet Length For Total Interception
Source: HEC-12 (FHWA, 1984)

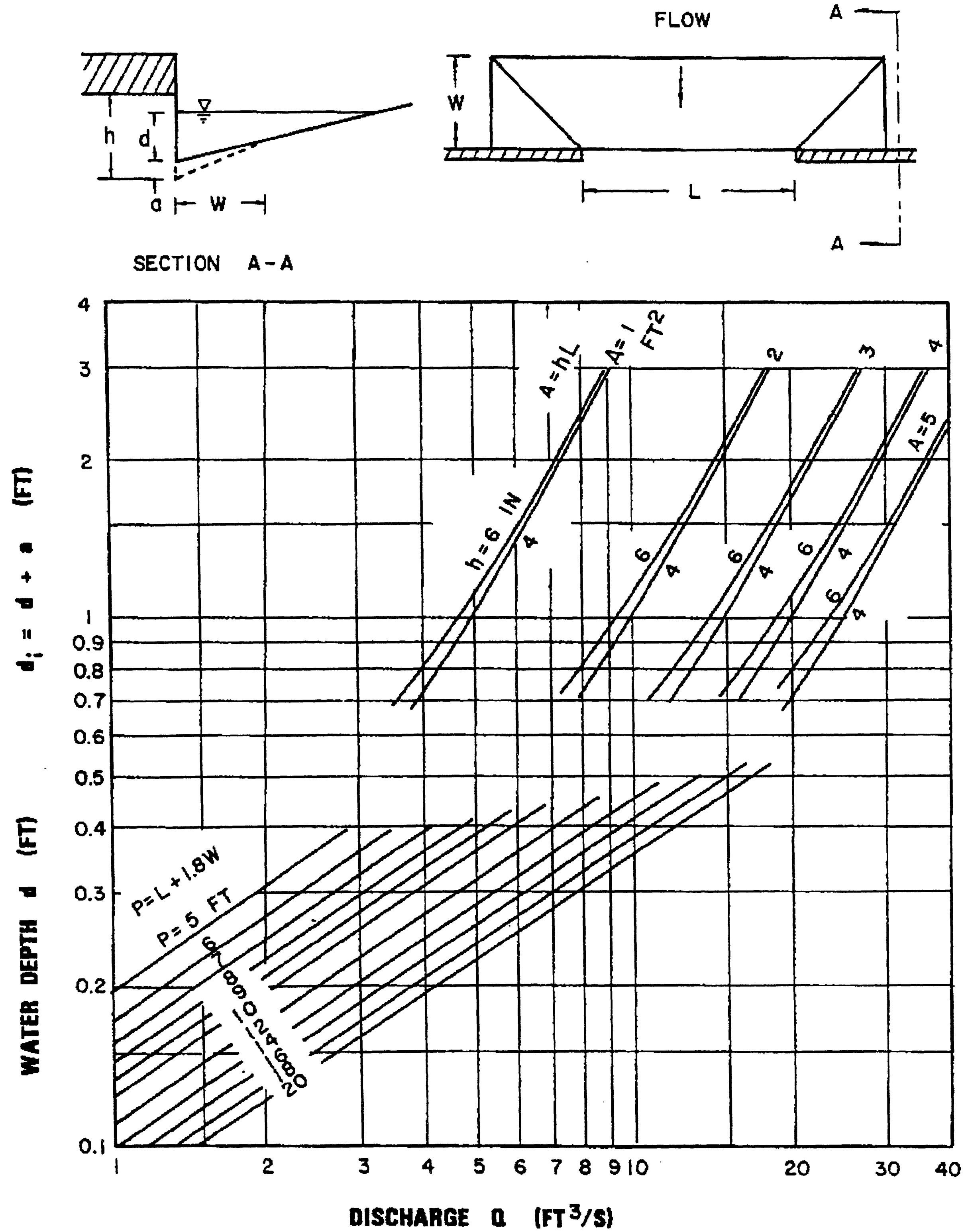


Figure 8.10 Depressed Curb-Opening Inlet Capacity In Sump Locations
Source: HEC-12 (FHWA, 1984)

8.7.6 Curb Inlets in Sag

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

Weir

Curb opening inlets operate as a weir when $d \leq h$. The equation for the interception capacity of a depressed curb opening inlet operating as a weir is:

$$Q_i = C_W(L + 1.8W)d^{1.5} \qquad \textbf{(8.15a)}$$

The weir equation for curb-opening inlets without depression becomes:

$$Q_i = C_W L d^{1.5} \qquad \textbf{(8.15b)}$$

Where: C_w = weir coefficient (2.3)
L = length of curb opening (ft)
W = width of depression (ft)
d = depth of water at curb measured from the normal cross slope gutter flow line (ft)

Orifice

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 h. Equation 8.16 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression. The interception capacity can be computed by:

$$Q_i = C_o A\left[2g\left(d_i - \frac{h}{2}\right)\right]^{0.5} \qquad \textbf{(8.16)}$$

Where: C_o = orifice coefficient (0.67)
h = height of curb-opening orifice (ft)
A = clear area of opening, (ft^2)
d_i = depth at lip of curb opening (ft)
g = acceleration due to gravity (32.2 ft/s^2)

8.7.7 Slotted Inlets on Grade

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets should generally be connected into inlet structures so they will be accessible to maintenance forces in case of plugging or freezing.

Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right of way is narrow and existing drainage systems must be supplemented. In some cases, curbs can be eliminated as a result of utilizing slotted inlets.

The length of a slotted inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \tag{8.17}$$

Where: K = coefficient (0.6)
L_T = slotted inlet length required to intercept 100% of the gutter flow (ft)
Q = gutter flow (cfs)
S = longitudinal slope
S_x = cross slope
n = Manning's n

The efficiency of slotted inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \tag{8.18}$$

Where: L = slotted inlet length (ft)
E = slotted inlet efficiency
L_T = slotted inlet length required to intercept 100% of the gutter flow (ft)

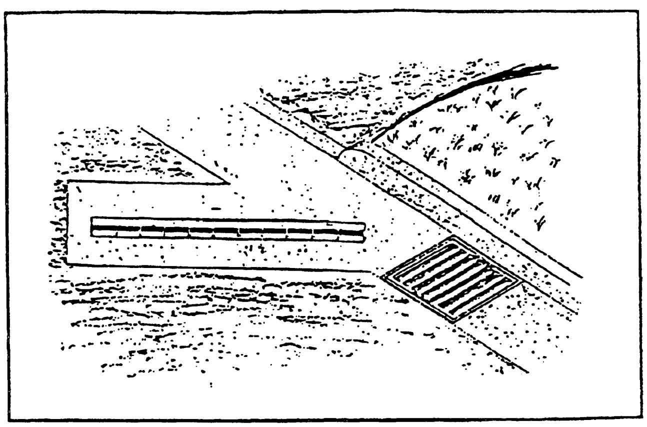
The length of inlet required for total interception by a slotted inlet in a composite section can be found by the use of an equivalent cross slope, S_e.

$$S_e = S_x + S'_w E_o \tag{8.19}$$

Where: S_x = pavement cross slope
S_w = gutter cross slope
$S'_w = S_w - S_x$
E_o = ratio of flow in the gutter section to total curbed channel flow, Q_w/Q

Transverse Placement

At locations where it is desirable to capture virtually all of the flow in the curbed section, a slotted vane drain can be installed in conjunction with a grate inlet. Tests have indicated that when the slotted vane drain is installed perpendicular to the flow, it will capture from 0 to 0.5 cfs per lineal foot of drain on longitudinal slopes of 0% to 6%. Capacity curves are available from the manufacturer. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder. Note that a slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet.



Slotted Vane Drain

8.7.8 Slotted Inlets in Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to intercept debris in sags. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 0.2 feet, dependent on slot width and length. At depths greater than about 0.4 feet, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW(2gd)^{0.5} \quad \textbf{(8.20)}$$

Where: W = width of slot (ft)
L = length of slotted drain inlet (ft)
d = depth of water at slot (ft)
g = acceleration due to gravity (32.2 ft/s^2)

The interception capacity of slotted inlets at depths between 0.2 feet and 0.4 feet can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet. Figure 8.11 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

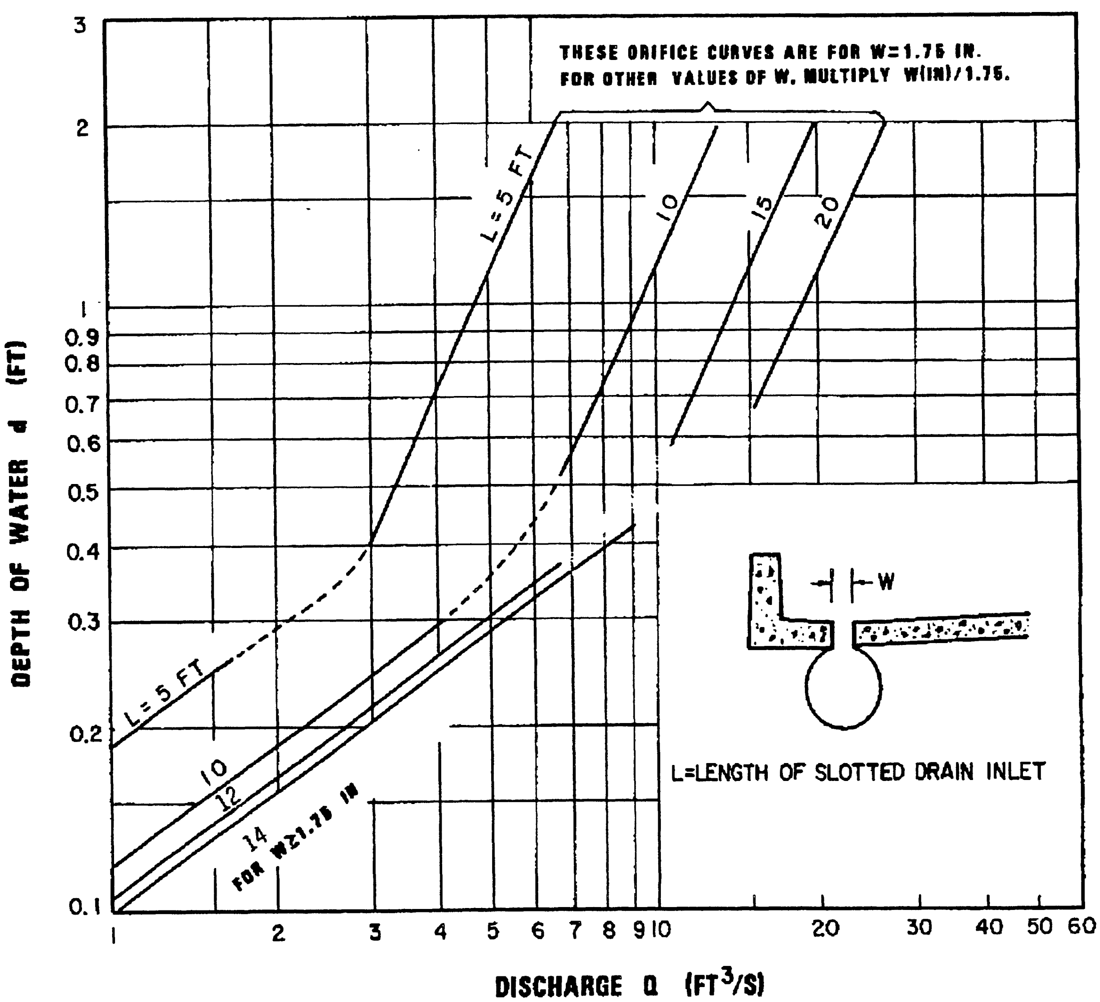


Figure 8.11 Slotted Drain Inlet Capacity In Sump Locations
Source: HEC-12 (FHWA, 1984)

8.7.9 Combination Inlet

The interception capacity of a combination grate/curb opening inlet on a continuous grade is not appreciably greater than that of a grate alone. Although Mn/DOT has used combination inlets for many years, we are now recommending that the curb box be dropped for all inlets placed on grade. The original intent of the curb box was to collect runoff in the event the grate became clogged with debris. However, experience has shown that the curb box tends to plug with debris and dirt, thus making it ineffective at capturing runoff and increasing maintenance costs. At minor sag points, combination inlets are recommended. The curb box is much less apt to plug and will provide some relief if the grate should become clogged. The capacity of a combination inlet in a sag is essentially the same as the grate alone in weir flow conditions unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the total capacity of grate and curb opening.

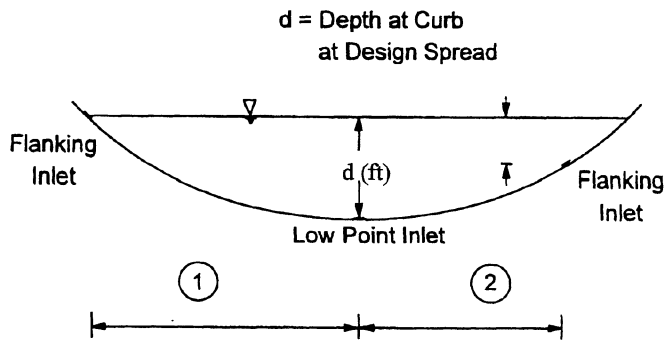


Figure 8.12 Flanking Inlets at Sag Point

8.7.10 Flanking Inlets

At major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed so they will limit spread on low gradient approaches to the sag and act in relief of the sag inlet if it should become clogged or if the design spread is exceeded. For major sag points, the flanking inlets are added as a safety factor, and not considered as intercepting flow to reduce the runby to the sag point. They are installed to assist the sag point inlet in the event of plugging.

Table 8.4 shows the spacing required for various depth at curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in feet and A is the algebraic difference in approach grades. The AASHTO policy on geometrics specifies K values for various design speeds in miles per hour (MPH) and a maximum K of 167.

Table 8.4 Flanking Inlet Locations

Distance to flanking inlet in sag vertical curve locations using depth at curb criteria.												
Speed (MPH)	20	25	30	35	40	45	50	55	60	*	65	70
d (ft)↓ K→	20	30	40	50	70	90	110	130	160	167	180	220
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	162
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

NOTES:
1. $x = (200dK)^{0.5}$, where x = distance from the low point in feet.
2. Drainage maximum K = 167
3. d = depth at curb and does not include sump in feet.

Source: HEC-12, Table 5 (FHWA, 1984)

8.7.11 Inlet Spacing Computation Procedures

In order to design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, super elevation diagrams, and contour maps are necessary. The inlet computation sheet, Table 8.5 can be used to document the computations. Alternatively several computer applications are available which are capable of performing inlet spacing computations. The assumptions and procedures incorporated in a computer application must be carefully evaluated before the designer uses the application. A step by step procedure is as follows:

Step 1		Complete the blanks on top of the sheet to identify the job by S.P., route, date and your initials.
Step 2		Mark on the plan the location of inlets which are necessary even without considering any specific drainage area. See Section 8.7.2 Inlet Locations for additional information.
Step 3		Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.
Step 4		Select a trial drainage area approximately 300′ to 500′ below the high point and outline the area including any area that may come over the curb. (Use drainage area maps.) Where practical, large areas of behind the curb drainage should be intercepted before it reaches the highway.
Step 5	*Col 1* *Col 2*	Describe the location of the proposed inlet by number and station in Columns 1 & 2. Identify the curb and gutter type in the Remarks Column 19. A sketch of the cross section should be provided in the open area of the computation sheet.
Step 6	*Col 3*	Compute the drainage area in acres and enter in Column 3.
Step 7	*Col 4*	Select a C value from Tables 3.7 or compute a weighted value based on area and cover type as described in the Hydrology Chapter and enter in Column 4.
Step 8	*Col 5*	Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 8.4 and the Hydrology Chapter. The minimum time of concentration should be 7 minutes. Enter value in Column 5.
Step 9	*Col 6*	Using the Intensity-Duration-Frequency curves, Section 3.5.4, select a rainfall intensity at the t_c for the design frequency. Enter in Column 6.
Step 10	*Col 7*	Calculate Q by multiplying Column 3 X Column 4 X Column 6. Enter in Column 7.
Step 11	*Col 8*	Determine the gutter slope at the inlet from the profile grade - check effect of superelevation. Enter in Column 8.
Step 12	*Col 9* *Col 13*	Enter cross slope adjacent to inlet in Column 9 and gutter width in Column 13. Sketch composite cross slope with dimensions.
Step 13	*Col 11*	For the first inlet in a series (high point to low point) enter Column 7 in Column 11 since no previous runby has occurred yet.
Step 14	*Col 12* *Col 14*	Using Figure 8.3 or 8.4 or a computer model, determine the spread T and calculate the depth d at the curb by multiplying T times the cross slope(s). Compare with the allowable spread as determined by the design criteria in Section 8.2.2. If Column 12 is less than the curb height and Column 14 is near the allowable spread, continue on to Step 16. If not OK, expand or decrease the drainage area to meet the criteria, and repeat Steps 5 through 16. Continue these repetitions until column 14 is near the allowable spread then proceed to step 15.
Step 15	*Col 15*	Calculate W/T and enter in Column 15.

Step 16	*Col 16*	Select the inlet type and dimensions and enter in Column 16.
Step 17	*Col 17*	Calculate the Q intercepted (Q_i) by the inlet and enter in Column 17. Utilize Figure 8.2, 8.3 or 8.4 to define the flow in the gutter. Utilize Figure 8.6, 8.7 and 8.8 and Equation 8.12 to calculate Q_i for a grate inlet and Figure 8.9 and 8.10 to calculate Q_i for a curb opening inlet.
Step 18	*Col 18*	Calculate the runby by subtracting Column 17 from Column 11 and enter into Column 18.
Step 19	*Col 1-4*	Proceed to the next inlet down grade. Select an area approximately 300′ to 400′ below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.
Step 20	*Col 5*	Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21	*Col 6*	Determine the intensity based on the time of concentration determined in Step 20 and enter it in Column 6.
Step 22	*Col 7*	Determine the discharge from this area by multiplying Column 3 X Column 4 X Column 6. Enter the discharge in Column 7.
Step 23	*Col 11*	Determine total gutter flow by adding Column 7 and Column 10. Column 10 is the same as Column 18 from the previous line.
Step 24	*Col 12* *Col 14*	Determine spread (T) based on total gutter flow (Column 11) by using Figure 8.3 or 8.4. If spread (T) in Column 14 exceeds the allowable spread, reduce the area and repeat Steps 19-23. If spread (T) in Column 14 is substantially less than the allowable spread, increase the area and repeat Steps 18-24.
Step 25	*Col 16*	Select inlet type.
Step 26	*Col 17*	Determine Q_i, See instruction in Step 17.
Step 27	*Col 18*	Calculate the runby by subtracting Column 17 from Column 7. This completes the spacing design for this inlet.
Step 28		Go back to Step 19 and repeat Step 19 through Step 27 for each subsequent inlet. If the drainage area and weighted "C" values are similar between each inlet, the values from the previous grate location can be reused. If they are significantly different, recomputation will be required.

INLET COMPUTATION SHEET

DATE________ SP________ ROUTE________

COMPUTED BY________ SHEET____OF____

LOCATION		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			REMARKS
INLET NO.	STAT.	DRAIN AREA "A" (acres)	RUNOFF COEF. "C"	TIME OF CONC. "T_C" (min)	Rain Intens. "I" (in/hr)	Q=CIA (cfs)	GRADE "S_o" (ft/ft)	CROSS SLOPE S_x (ft/ft)	PREV. RUNBY (cfs)	TOTAL GUTTER FLOW (cfs)	DEPTH "d" (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (cfs)	RUNBY "Q_r" (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Table 8.5 Inlet Spacing Computation Sheet

8.8 MANHOLES AND INLET STRUCTURES

Manholes (MH) are utilized to provide access to continuous underground storm drains for inspection and cleanout. Where feasible, grate inlets may be used in lieu of manholes for access so that the benefit of extra stormwater interception is achieved with minimal additional cost. Typical locations where manholes should be specified are:

- where two or more storm drains converge,
- at intermediate points along tangent sections,
- where pipe size changes,
- where an abrupt change in alignment occurs, and
- where an abrupt change of the grade occurs.

Manholes should not be located in traffic lanes; however, when it is impossible to avoid locating a manhole in a traffic lane, care should be taken to insure it is not in the normal wheel path. There are various types of manholes available which are listed in Table 8.6. Usually the type selected is dependent on the storm drain pipe size and depth of the manhole.

Table 8.6 Manhole and Inlet Structure Types

Standard Plate	Design Type	Manhole Diameter	Maximum Depth	Maximum Size Pipe	Minimum Size Pipe	Comments
4000	A	48"	variable	30 "	12"	Masonry field constructed, includes cone section and barrel.
4002	C	48"	5′	variable	12"	Masonry field constructed cone section, typically used with single pipe.
4003	N	30"	4′	18"	12"	Precast 30" catch basin, typically used with single pipe.
4005	F	48"	variable	27"	12"	Precast Type A and B cone sections
4006	G	48"	5′	21"	12"	Precast cone section, typically used with single pipe.
	H	27"	3′	15"	12"	Precast 27" catch basin, typically used with single pipe.
4008	I	24"	variable	diameter	24"	Sectional concrete pipe (Tee) with 24" diameter riser.
4009	J	48"	N/A	diameter	42"	Sectional concrete pipe (Tee) with 48" diameter riser and Type A cone section.
4020	-	48" to 120"	variable	variable	12"	Manholes larger than 120" need to be evaluated on a case by case basis.
4024	SD	48"	4'	24"	12"	Precast 48" shallow depth catch basin with cover. Maximum Depth measured to bottom of cover

8.8.1 Height

Precast manhole sections can be made for installation of pipes up to 60 inch diameter. Larger sizes are limited by the precast segment height of 8 feet. Manholes with installation of larger pipe sizes will require either field construction (cast-in-place or masonry), tee structures or a special precast design.

Pay heights should be computed according to the methodology provided in the Minnesota Department of Transportation Standard Specification for Construction.

8.8.2 Spacing

The maximum spacing of access structures whether manholes or inlets should be approximately 400 feet for 12 inch through 54 inch diameter storm drains and approximately 600 to 800 feet for 60 inch and above. Where self cleaning velocities of at least 3 feet per second (fps) are assured, the distance between access points is not as critical as for storm drains on very flat grades where sedimentation could be a problem. A minimum velocity of 3 feet per second (fps) is recommended where feasible.

8.8.3 Sizing

When determining the minimum manhole size required for various pipe sizes and locations, two conditions must be met.

- The manhole or inlet structure must be large enough to accept the maximum pipe as shown in Table 8.7.
- A minimum leg width between pipe holes of 6" measured on the inside of the manhole must be maintained. Knowing the relative locations of any two pipes, apply Equation 8.21 and determine minimum manhole size. If there are more then two pipes, check every combination of pipes to determine the most critical pair.

$$\frac{180(P_1 + P_2 + 12)}{\pi D \Delta} \leq 1 \qquad \textbf{(8.21)}$$

Where: P_1 = manhole perimeter removed for pipe 1 hole (inches)
P_2 = manhole perimeter removed for pipe 2 hole (inches)
Δ = angle between the pipe center lines (degrees)
D = inner diameter of the manhole (inches)
look up P_1 and P_2 in Table 8.7

Manholes must be sized to ensure constructability. Consider any special features such as flexible pipe connections that may result in the need to up size a manhole. For instance, connections sometimes used with plastic pipe may require additional space. Complex configurations, where numerous pipes enter a single structure, particularly with unusual pipe configurations or varying inverts will need to be evaluated on a case by case basis. The values provided in Table 8.7 are based on pre-cast structures, It may be possible to fit slightly larger pipes into a cast-in-place or masonry structure.

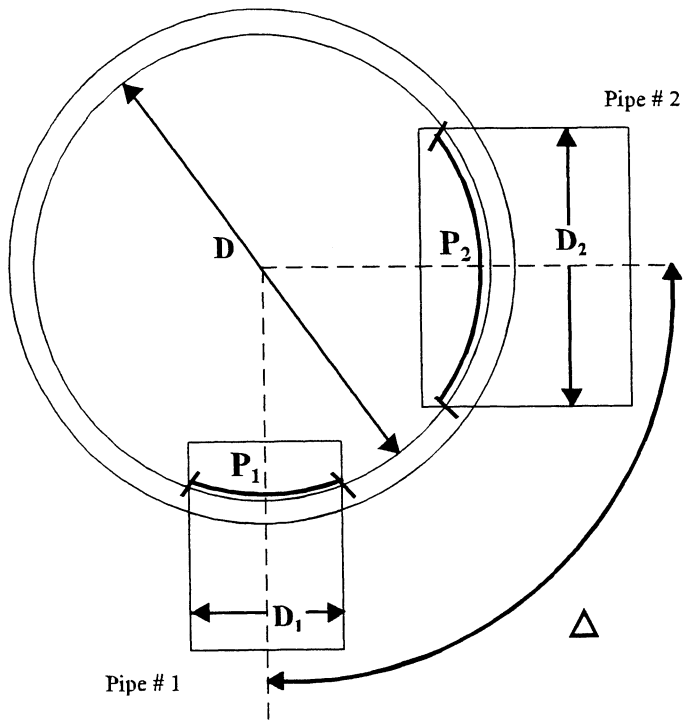


Figure 8.13 Manhole Diagram

Table 8.7 Minimum Manhole Size and Pre-cast Manhole Perimeter Removed to Install Pipe

Pipe Inner Diameter (inches)	Required hole size for C wall concrete pipe (inches)	Minimum Manhole Inner Diameter (inches)	Manhole Perimeter, P (inches) Removed to Install Pipe													
			MH I.D. 27"	MH I.D. 30"	MH I.D. 48"	MH I.D. 54"	MH I.D. 60"	MH I.D. 66"	MH I.D. 72"	MH I.D. 78"	MH I.D. 84"	MH I.D. 90"	MH I.D. 96"	MH I.D. 102"	MH I.D. 108"	MH I.D. 120"
12	20	27 [1]	22.52	21.89	20.63	20.49	20.39	20.32	20.27	20.23	20.19	20.17	20.15	20.13	20.12	20.09
15	24	27 [1]	29.56	27.82	25.13	24.87	24.69	24.56	24.47	24.40	24.34	24.29	24.26	24.23	24.20	24.16
18	26	30 [2]		31.45	27.48	27.13	26.89	26.72	26.60	26.51	26.43	26.38	26.33	26.29	26.26	26.21
21	30	48			32.41	31.81	31.42	31.14	30.94	30.79	30.68	30.59	30.51	30.45	30.40	30.32
24	34	48			37.78	36.78	36.15	35.72	35.41	35.18	35.00	34.87	34.75	34.66	34.59	34.47
27	38	48 [3]			43.85	42.15	41.15	40.49	40.03	39.69	39.43	39.23	39.07	38.94	38.83	38.67
30	42	54				48.12	46.52	45.53	44.84	44.35	43.98	43.70	43.47	43.29	43.14	42.91
33	46	54				55.05	52.42	50.90	49.90	49.20	48.68	48.28	47.97	47.72	47.52	47.21
36	48	60					55.64	53.75	52.54	51.70	51.09	50.63	50.27	49.98	49.74	49.38
42	55	66						65.02	62.59	61.04	59.96	59.17	58.57	58.09	57.71	57.13
48	64	72							78.83	75.06	72.76	71.20	70.05	69.18	68.50	67.50
54	70	84									82.75	80.20	78.44	77.15	76.16	74.74
60	78	90										94.36	91.05	88.80	87.16	84.91
66 [4]	84	102												100.48	97.85	94.46
72 [4]	90	108													110.11	104.84
78 [4]	98	120														116.42

Notes: Pipes larger then 60" and manholes larger then 120", will need to be evaluated on a case by case basis.

1 Design H minimum size based on one pipe connection

2 Design N minimum size based on one pipe connection

3 One exception is that the Design SD, Precast Shallow Depth Catch Basin is a 48" structure where the maximum pipe size is 24" Inner Diameter.

4 MH for pipes larger then 60" require field construct, Tee structures, or special precast design method.

8.9 STORM DRAINS

After the preliminary locations of inlets, connecting pipes, and outfalls with tailwater have been determined, the next logical step is the computation of the rate of discharge to be carried by the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream reach by reach to the point the storm drain connects with other drains or the outfall. At manholes where the pipe size is increased, it is recommended the pipe invert in the manhole be lowered to match crowns or at least 80% of the difference in pipe sizes.

The rate of discharge at any point in the storm drain is not the sum of the design inlet flow rates of all inlets above that storm drain section. It is generally less than this total. The time of concentration is most influential and as it grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment. The Hydrology Chapter contains a detailed discussion on time of concentration.

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's Formula is recommended for capacity calculations. The exceptions are depressed sections and underpasses where ponded water can be removed only through the storm drain system. In these situations, a 50 year frequency design should be used to design the storm drain which drains the sag point. The main storm drain which drains the depressed section should be designed by computing the hydraulic grade line and keeping the water surface elevations below the grates and/or established critical elevations.

8.9.1 Design Procedures

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data: location of storm drains, direction of flow, location of manholes, location of existing utilities such as water, gas, or underground cables.

Step 3 Determine drainage areas and runoff coefficients, and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by multiplying A x C x I.

Step 4 Size the pipe to convey the discharge by varying the slope and pipe size as necessary. The storm drain systems are normally designed for full gravity flow conditions using the design frequency discharges.

Step 5 Calculate travel time in the pipe to the next inlet or manhole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA), multiply by the new rainfall intensity to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Check the design by calculating the hydraulic grade line (HGL) as described in Section 8.10. The design procedure should include the following:

- Storm drain design computation can be made on forms as illustrated in Table 8.11.
- All computations and design sheets should be clearly identified. The designer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

8.9.2 50 Year Sag Point

As indicated above, the storm drain which drains a major sag point should be sized to accommodate the runoff from a 50 year frequency rainfall. This can be done by actually computing the runby occurring at each inlet during a 50 year rainfall and accumulating it at the sag point. The inlet at the sag point as well as all storm drain pipes leading from the sag point to the outlet must be sized to accommodate this additional runby within the criteria established. Another method which is approximate, assumes that during a 50 year rainstorm, the on grade inlets will intercept the 10 year flow and the runby will consist of the difference between the 50 year and the 10 year runoff. A step by step procedure for this approximate method follows.

Step 1 Total the CA contributing to the sag point inlet.

Step 2 From the IDF curve, determine the rainfall intensity for both I_{50} and I_{10} for the time of concentration computed in the storm drain pipe at the sag point. Subtract I_{10} from I_{50}.

Step 3 Multiply the total CA by the difference of I_{50} - I_{10}. This is the 50 year runby. Size the sag inlet to accommodate this additional flow.

Step 4 Convert the 50 year runby to an equivalent CA by dividing it by I_{10} in the pipe at the sag point.

Step 5 Add the equivalent CA to the total CA.

Step 6 Design the pipe from the sag point for the Q resulting from the CA in Step 5 multiplied by I_{10} and continue down line adding CA from additional inlets.

8.9.3 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's Formula and it is expressed by the following equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{8.22a}$$

In terms of discharge, the above formula becomes:

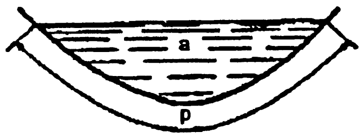
$$Q = VA = \frac{1.486}{n} A R^{2/3} S^{1/2} \tag{8.22b}$$

For storm drains flowing full, the above equations become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{8.22c}$$

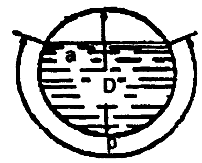
Where: V = mean velocity of flow (ft/s)
S = the slope of the energy grade line
R = hydraulic radius (ft)
area of flow divided by the wetted perimeter (A/P)
n = Manning's roughness coefficient
Q = rate of flow (cfs)
A = cross sectional area of flow (ft^2)
D = diameter of pipe (ft)

Figure 8.14 has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.



a = Cross-sectional area of waterway
p = wetted perimeter
R = a/p = Hydraulic radius

Section of Any Channel



For pipes full or half full
R = D/4

Section of Circular Pipe

V = Average or mean velocity in m/s
Q = a V = Discharge of pipe or channel in m³/s
n = Coefficient of roughness of pipe or channel surface
S = Slope of hydraulic gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)

Hydraulic Elements of Channel Sections

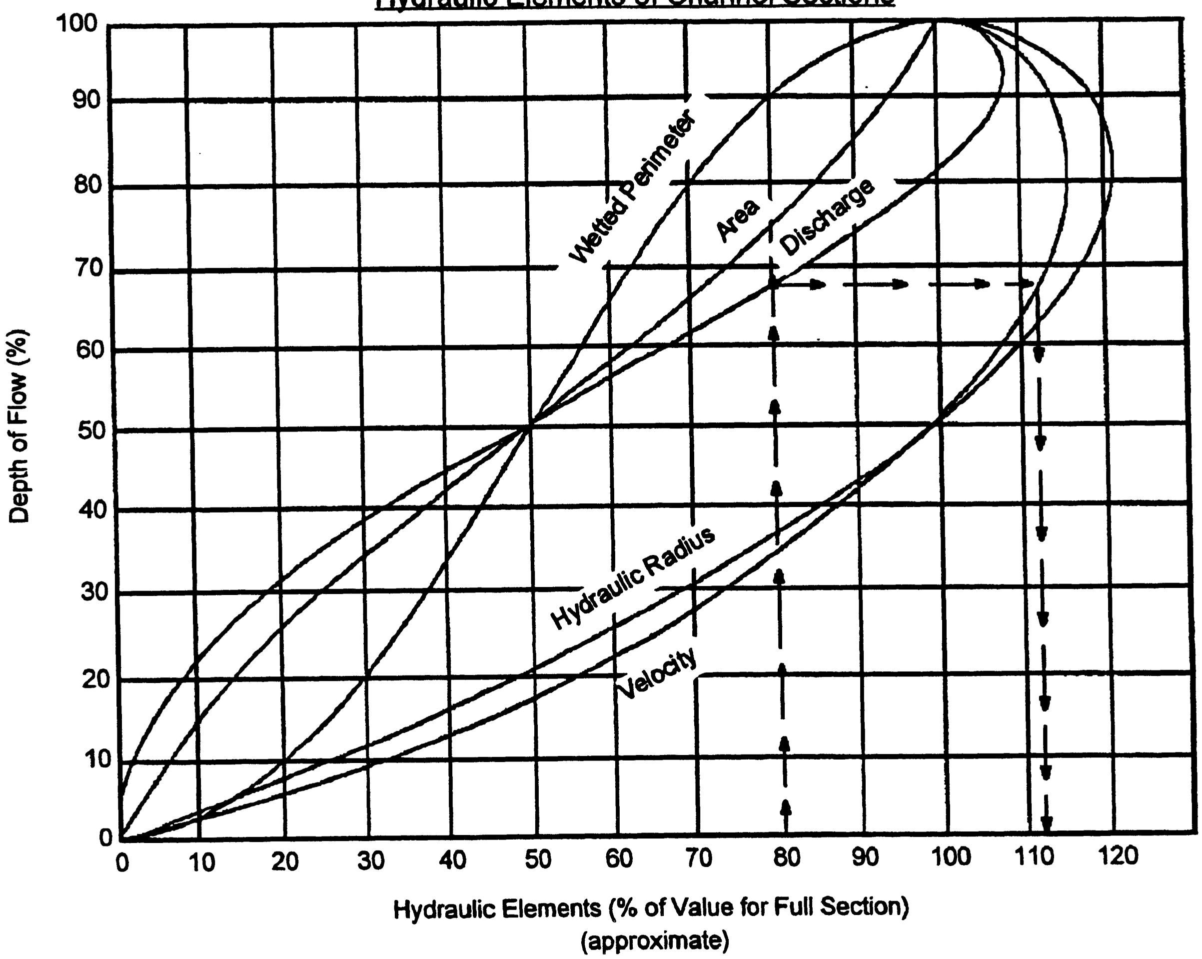


Figure 8.14 Values Of Hydraulic Elements Of Circular Section for Various Depths of Flow
Source: HEC-22 (FHWA 1996)

8.9.4 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 3 feet per second at full flow or lower. For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. Minimum slopes required for a velocity of 3 fps can be calculated by the Manning formula or by using values given in Table 8.8.

$$S = \frac{(nV)^2}{2.208R^{4/3}} \qquad (8.23)$$

Where: S = slope of the energy grade line
n = Manning's n
V = mean velocity of flow (fps)
R = hydraulic radius (ft)
area of flow divided by the wetted perimeter (A/P)

Table 8.8 Minimum Grades

Minimum Grades to Ensure 3 fps for Full Flow (ft/ft)		
Pipe Size (in)	Q Full (cfs)	Grade (ft/ft)
12	2.36	.0037
15	3.68	.0028
18	5.30	.0022
21	7.22	.0018
24	9.43	.0015
27	11.93	.0013
30	14.73	.0011
33	17.82	.00097
36	21.21	.00086
42	28.86	.00070
48	37.70	.00059
54	47.71	.00050
60	58.90	.00044
66	71.27	.00038
72	84.82	.00034

8.9.5 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available and are the preferable means of changing direction in pipes 48" and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures.

8.10 HYDRAULIC GRADE LINE

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This gradeline aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating during a flood of design frequency.

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another. A special concern with storm drains 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 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, frequent verification of the existence of the desired flow condition should be made.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm drain system, the HGL calculation must begin at the outlet end of the existing system, and proceed upstream through this inplace system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. Usually it is helpful to compute the energy grade line (EGL) first, then the velocity head ($V^2/2g$) is subtracted to obtain the HGL.

8.10.1 Tailwater

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or $(d_c+D)/2$, whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses, and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm which causes peak discharges on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. Table 8.9 will aid in the evaluation of joint probabilities.

Table 8.9 Joint Probability Analysis

Area Ratio	Frequencies for Coincidental Occurrence			
	10 Year Design		100 Year Design	
	Mainstream	Tributary	Mainstream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Source: US Army Corps of Engineers, Norfolk District, 1974

8.10.2 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \quad \textbf{(8.24)}$$

Where: V = average outlet velocity (fps)
V_d = channel velocity downstream of outlet (fps)
g = acceleration due to gravity (32.2 ft/sec^2)

Note that when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

8.10.3 Bend Loss

The bend loss coefficient is minor but can be evaluated using the formula:

$$h_b = 0.0033\Delta\left(\frac{V_o^2}{2g}\right) \quad \textbf{(8.25)}$$

Where: Δ = angle of curvature in degrees
V_o = velocity (ft/s)
g = acceleration due to gravity (32.2 ft/sec^2)

8.10.4 Pipe Friction Losses

The friction slope is the hydraulic gradient in ft/ft for that run. The friction loss is simply the hydraulic gradient multiplied by the length of the run in feet. The head losses due to friction may be determined by the formula:

$$H_f = S_f L \quad \textbf{(8.26)}$$

Energy losses from pipe friction may be determined by rewriting the Manning Equation with terms as previously defined:

$$S_f = \left[\frac{Qn}{1.486AR^{2/3}}\right]^2 \quad \textbf{(8.27)}$$

The Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f = 2.87\frac{n^2V^2L}{D^{4/3}} = \frac{29n^2L}{R^{4/3}}\left(\frac{V^2}{2g}\right) \quad \textbf{(8.28)}$$

Where: H_f = total head loss due to friction (ft)
S_f = friction slope of hydraulic grade line (ft/ft)
L = length of pipe (ft)
Q = rate of flow (cfs)
n = Manning's roughness coefficient
A = cross sectional area of flow (ft^2)
R = hydraulic radius (ft)
V = mean velocity (ft/s)
g = acceleration due to gravity (32.2 ft/sec^2)
D = diameter of pipe (ft)

8.10.5 Manhole Losses

The head loss encountered in going from one pipe to another through a manhole is commonly represented as being proportional to the velocity head at the outlet pipe. A head loss coefficient, K is used to signify this constant of proportionality. K is calculated by multiplying correction factors together, Equation 8.29. The head loss coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection., the energy loss is approximated as $K(V_o^2/2g)$. Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o C_D C_d C_Q C_p C_B \qquad \textbf{(8.29)}$$

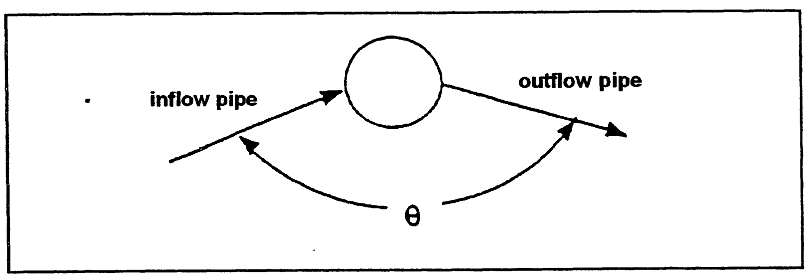
Where: K = adjusted loss coefficient
K_o = initial head loss coefficient based on relative manhole size
C_D = correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth (non-pressure flow only)
C_Q = correction factor for relative flow
C_B = correction factor for benching
C_p = correction factor for plunging flow

Relative Manhole Size

K_o is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes.

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \qquad \textbf{(8.30)}$$

Where: θ = the angle between the inflow and outflow pipes (degrees)
b = manhole diameter (ft)
D_o = outlet pipe diameter (ft)



Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth in the manhole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases.

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad \textbf{(8.31)}$$

Where: D_i = incoming pipe diameter (ft)
D_o = outgoing pipe diameter (ft)

Flow Depth

The correction factor for flow depth is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. Water depth in the manhole is approximated as the level of the hydraulic gradeline at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5\left(\frac{d}{D_o}\right)^{0.6} \tag{8.32}$$

Where: d = water depth in manhole above outlet pipe (ft)
D_o = outlet pipe diameter (ft)

Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. As can be seen from the Equation 8.33, C_Q is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1-2\sin\theta)\left(1-\frac{Q_i}{Q_o}\right)^{0.75} + 1 \tag{8.33}$$

Where: C_Q = correction factor for relative flow
θ = the angle between the inflow and outflow pipes (degrees)
Q_i = flow in the inflow pipe (cfs)
Q_o = flow in the outlet pipe (cfs)

Plunging Flow

This correction factor corresponds to the effect of another inflow pipe, plunging into the manhole, on the inflow pipe for which the head loss is being calculated. The correction factor is only applied when h > d.. The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{(h-d)}{D_o}\right] \tag{8.34}$$

Where: C_p = correction for plunging flow
h = vertical distance of plunging flow from center of outlet pipe (ft)
D_o = outlet pipe diameter (ft)
d = water depth in manhole (ft)

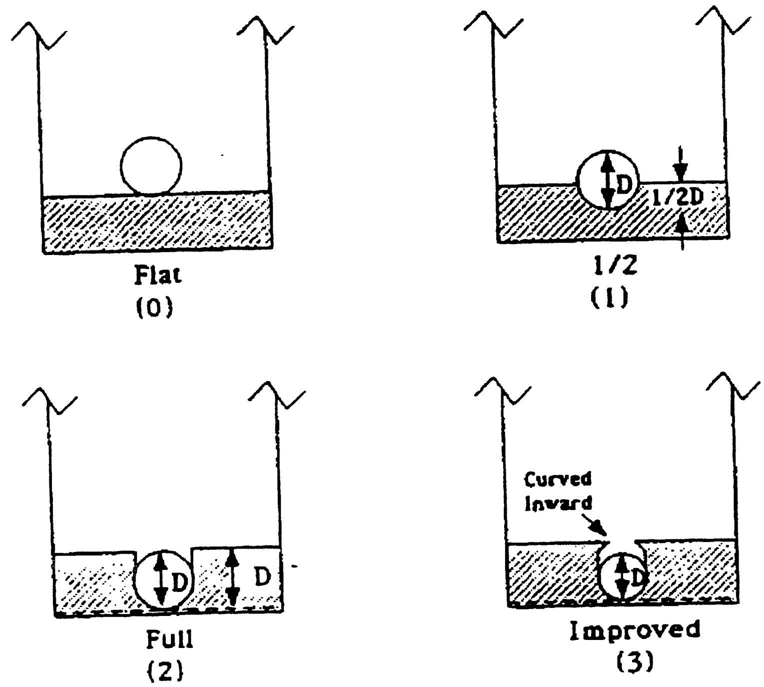
Benching

Benching tends to direct flows through the manhole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed. The correction for benching in the manhole, C_B, is obtained from Table 8.10.

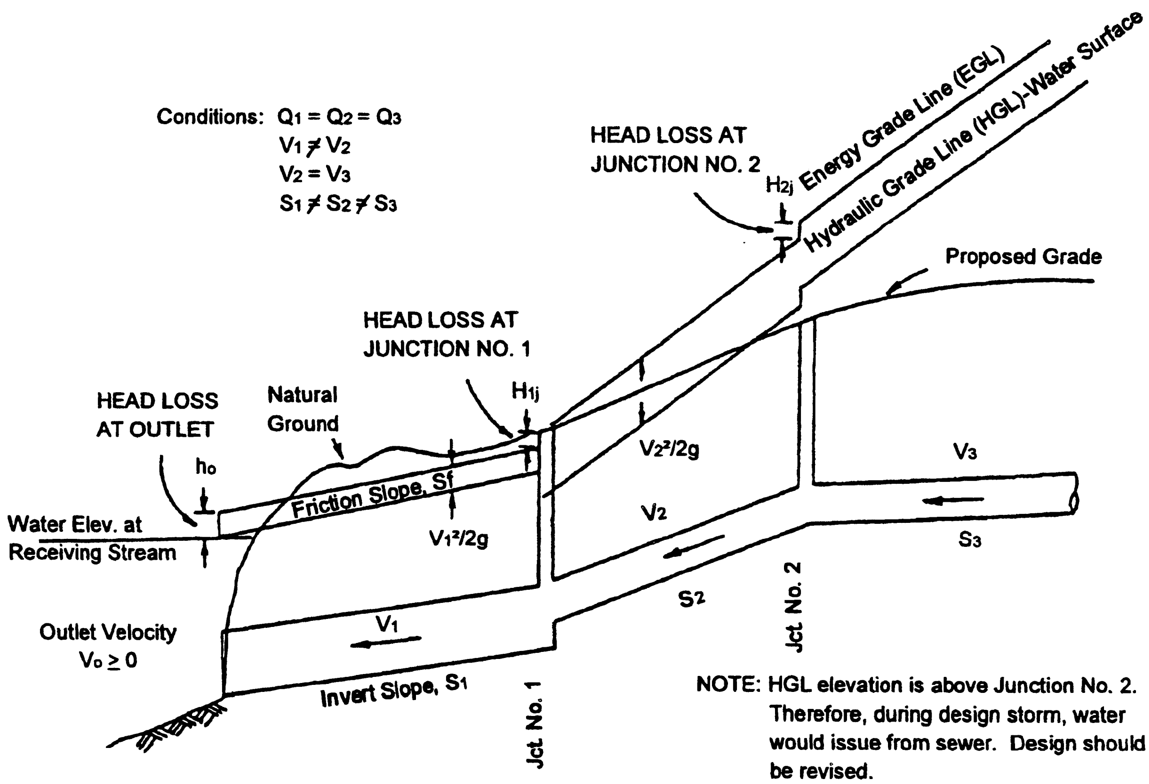
Table 8.10 Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged[1]	Unsubmerged[2]
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Type J	0.40	0.02

[1] pressure flow, $d/D_o > 3.2$
[2] free surface flow, $d/D_o < 1.0$



IMPROPER DESIGN



PROPER DESIGN

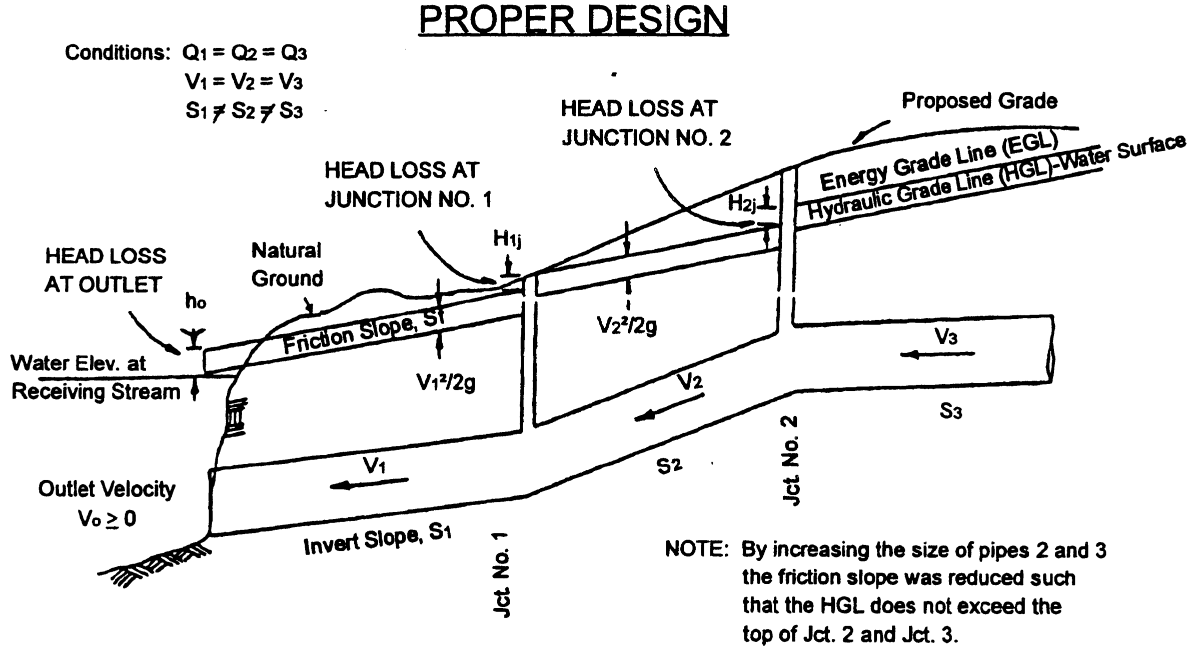


Figure 8.15 Use Of Energy Losses In Developing A Storm Drain System
Source: Model Drainage Manual - Metric Edition, (AASHTO, 1999)

8.10.6 Hydraulic Grade Line (HGL) Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic gradeline are included in this chapter. Any computer program used for design of storm drains should include a HGL analysis and a pressure flow simulation. A step by step procedure is given to manually compute the HGL. Table 8.11 can be used to document the procedure.

If the HGL 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. The process is repeated throughout the storm drain system. If all HGL elevations are acceptable then the hydraulic design is adequate. If the HGL exceeds an inlet elevation, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 8.15 for a sketch depicting the use of energy losses in developing a storm drain system.

Step 1 Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Column 2 the tailwater elevation if the outlet will be submerged during the design storm otherwise refer to the tailwater discussion in Section 8.10.1 for procedure.

Step 3 Enter in Column 3 the diameter (D_o) of the outflow pipe.

Step 4 Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Column 5 the length (L_o), of the outflow pipe.

Step 6 Enter in Column 6 the outlet velocity of flow (V_o).

Step 7 Enter in Column 7 the velocity head, $V_o^2/2g$.

Step 8 Enter in Column 8 the exit loss, H_o, as computed by Equation 8.24.

Step 9 Enter in Column 9 the friction slope (SF_o) in ft/ft of the outflow pipe. This can be determined by using the Equation 8.27. Note: Assumes full flow conditions.

Step 10 Enter in Column 10 the friction loss (H_f) which is computed by multiplying the length (L_o) in Column 5 by the friction slope (SF_o) in Column 9. On curved alignments, calculate bend losses by using Equation 8.25 adding the answer to the friction loss.

Step 11 Enter in Column 11 the initial head loss coefficient, K_o, based on relative manhole size as computed by Equation 8.30.

Step 12 Enter in Column 12 the correction factor for pipe diameter, C_D, as computed by Equation 8.31.

Step 13 Enter in Column 13 the correction factor for flow depth, C_d, as computed by Equation 8.32. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.

Step 14 Enter in Column 14 the correction factor for relative flow, C_Q, as computed by Equation 8.33.

Step 15 Enter in Column 15 the correction factor for plunging flow, C_p, as computed by Equation 8.34. The correction factor is only applied when h > d.

Step 16 Enter in Column 16 the correction factor for benching, C_B, as determined in Table 8.10.

Step 17 Enter in Column 17 the value of K as computed by Equation 8.29.

Step 18 Enter in Column 18 the value of the total manhole loss, $K(V_o^2/2g)$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Column 19 the sum of Column 2 (TW elevation) and Column 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to $(d_c + D)/2$.

Step 20 Enter in Column 20 the sum of the friction head (Column 10), the manhole losses (Column 18), and the energy grade line (Column 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Column 21) throughout the system by subtracting the velocity head (Column 7) from the EGL (Column 20).

Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow.

- The above procedure applies to pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

Station	TW	D_o	Q_o	L_o	V_o	$V_o^2/2g$	H_o	SF_o	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_o 2+7	EGL_i 10+18+19	HGL EGL-7	TOC Elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Table 8.11 Hydraulic Grade Line Computation Form

8.11 REFERENCES

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Federal Highway Administration (FHWA), 1996. *Urban Drainage Design Manual.* Hydraulic Engineering Circular No. 22 (HEC-22). Report No. FHWA-SA-96-078, Washington, D.C. 20590.

Neenah Foundry Company, 1987. *Neenah Inlet Grate Capacities.* Neenah, WI 54956.

Appendix A: RISK ASSESSMENT

Risk analysis and/or assessment is based on the theory that roads should not all be designed for an arbitrary design frequency. Instead the design selected for an encroachment should be supported by analysis of design alternatives with consideration given to capital cost and risks; and other economic, engineering, social and environmental concerns.

Mn/DOT has developed a risk assessment procedure which is an attempt to screen projects and determine the level of analysis required. The purpose of the questionnaire is to determine if a risk analysis is required. It is not a comprehensive design check list nor should it replace good engineering judgement. Culverts that are 48 inches or larger will require an assessment to determine whether or not an economic analysis is necessary to determine the frequency of design flood. The form should be filled out and signed by the engineer making the hydraulic recommendation and placed the documentation file.

The procedure, included in Appendix A, consists of a DATA REQUIREMENTS section and a LTEC DESIGN section. Start with the first question and follow directions included in the form. All questions do not have to be answered. Figures A and B are provided for use in answering question 2d. The form has a column on the right hand side that is titled LTEC DESIGN. LTEC refers to the Least Total Economic Cost. If any checks are made in the LTEC column then the designer must proceed with a Risk Analysis or document justification of why the Risk Analysis is not needed.

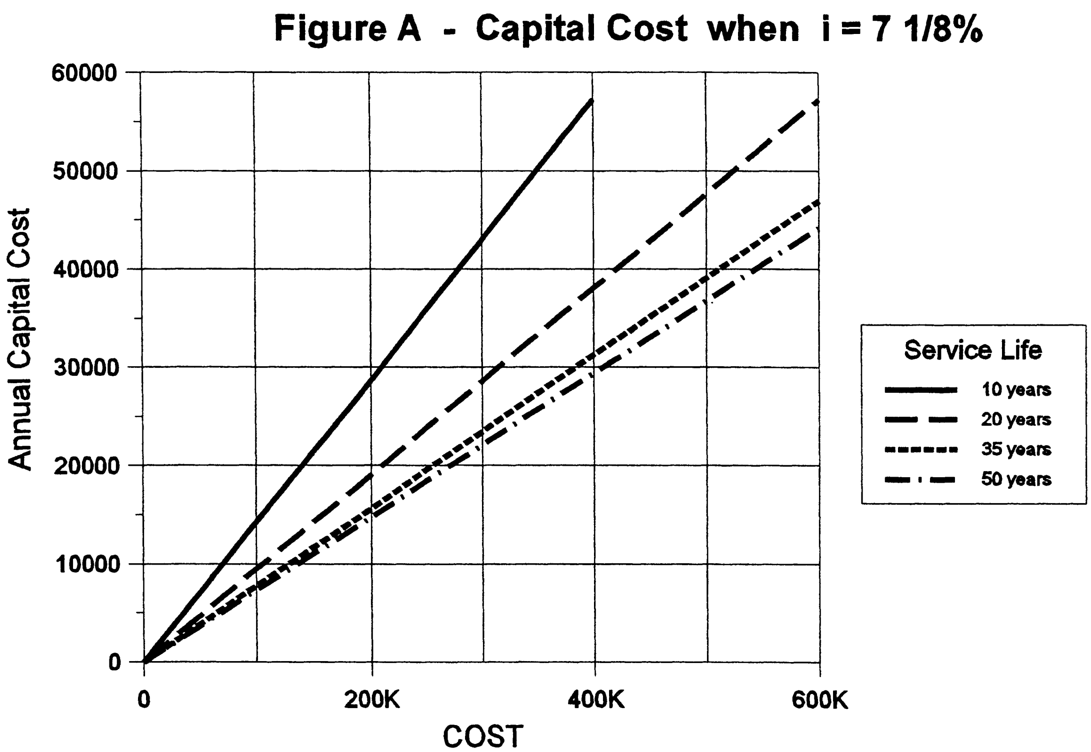


Figure A - Capital Cost when i = 7 1/8%
60000
50000
40000
30000
20000
10000
0
Annual Capital Cost
0
200K
400K
600K
COST
Service Life
10 years
20 years
35 years
50 years

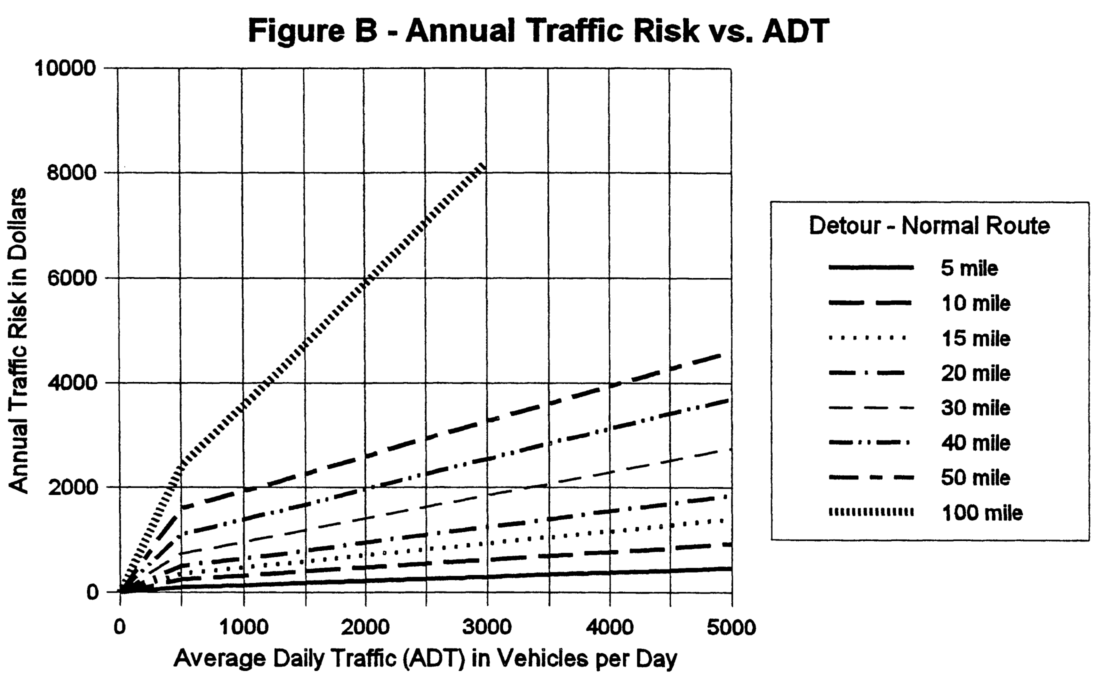


Figure B - Annual Traffic Risk vs. ADT
10000
8000
6000
4000
2000
0
Annual Traffic Risk in Dollars
0
1000
2000
3000
4000
5000
Average Daily Traffic (ADT) in Vehicles per Day
Detour - Normal Route
5 mile
10 mile
15 mile
20 mile
30 mile
40 mile
50 mile
100 mile

RISK ASSESSMENT
FOR
ENCROACHMENT DESIGN

Date ______________________________

District ______________________________

County ______________________________

Vicinity of ______________________________

____

DATA REQUIREMENTS

Sec. __________ T __________ R __________

1. Location of Crossing: Roadway ____________________ C.S. __________ M.P. __________

2. Name of Stream: ____________________ Bridge No. Old: __________ New: __________

3. Current ADT ____________________; Projected ADT ____________________

4. Practicable detour available Yes ________ No ________

   If no is checked, please explain: ______________________________

   ____

   If there is no practicable detour available, then the use of the road must be analyzed. Considerations such as emergency vehicle access, emergency supply and evacuation route, and the need for school bus, milk and mail routes should be studied. Factors to consider for this analysis include design frequency, depth, duration, and frequency of inundation if appropriate, and available funding.

5. Hydraulic Data: (Fill in as appropriate)

   Approximate Flowline Elevation __________

   Q_2 = ________________ TW_2 Elevation ____________________

   Q_5 = ________________ TW_5 Elevation ____________________

   Q_{10} = ________________ TW_{10} Elevation ____________________

   Q_{25} = ________________ TW_{25} Elevation ____________________

   Q_{50} = ________________ TW_{50} Elevation ____________________

   Q_{100} = ________________ TW_{100} Elevation ____________________

   Circle Design Frequency

Reasons for selecting Design Frequency:____

____

____

6. Magnitude and Frequency of the smaller of "Overtopping" or "500 yr." (Greatest)flood:

   ____________________ cfs   ________________ year frequency

7. Low member elevation ____

8. Minimum roadway overflow elevation if appropriate__________________________________

9. Elevation of high risk property, i.e. residences____________________________________

   Other buildings ____

10. Horizontal location of overflow:

    At structure ____________(See 12); Not at structure____________________________

11. Type of proposed structure:

    Bridge _______________(See 12); Culvert(s)__________________________________

12. If the proposed structure is a bridge with the sag point located on the bridge and there is ice and debris potential, strong consideration should be given to using Q_{50} as design discharge with 3′ of clearance between the 50 year tailwater stage and low member.

LTEC DESIGN

	LTEC DESIGN
1. BACKWATER DAMAGE - Major flood damage in this context refers to shopping centers, hospitals, chemical plants, power plants, housing developments, etc.	
1a. Is the overtopping flood greater than the 100 yr. flood?	
Yes__(Go to 1 b.); No__ (Go to 1 e.)	
1b. Is the overtopping flood greater than the "greatest" flood (500 yr. frequency)?	
Yes__(Go to 1 d.); No__ (Go to 1 c.)	
1c. Is there major flood damage potential for the overtopping flood?	
No__(Go to 1 e.)	YES __ (Go to 1 e.)

1d. Is there major flood damage potential for the greatest flood (500 year frequency)?	LTEC DESIGN
No ___ (Go to 1 e.)	YES ___ (Go to 1 e.)
1e. Will there be flood damage potential to residence(s) or other buildings during a 100 yr. flood?	
Yes ___ (Go to 1 f.); No___ (Go to 2)	
1f. Could this flood damage occur even if the roadway crossing wasn't there?	
Yes ___ (Go to 1 g.); No___ (Go to 1 h.)	
1g. Could this flood damage be significantly increased by the backwater caused by the proposed crossing?	
Yes ___ (Go to 1 h.); No___ (Go to 2)	
1h. Could the stream crossing be designed in such a manner so as to minimize this potential flood damage?	
Yes ___ (Go to 1 i.); No___ (Go to 2)	
1i. Does the value of the building(s) and/or its contents have sufficient value to justify further evaluation of risk and potential flood damage?	YES ___ (Go to 2)
No ___ (Go to 2)	
2. TRAFFIC RELATED LOSSES	
2a. Is the overtopping flood greater than the "greatest" flood (500 yr. frequency)?	
Yes ___ (Go to 3); No___ (Go to 2 b.)	
2b. Does the ADT exceed 50 vehicles per day?	
Yes ___ (Go to 2 c.); No___ (Go to 3)	
2c. Would the (duration of road closure in days) multiplied by the (length of detour minus the length of normal route in miles) exceed 20?	
Yes ___ (Go to 2 d.); No___ (Go to 3)	
2d. Does the annual risk cost for traffic related costs exceed 10% of the annual capital costs?	
No ___ (Go to 3) (See figures A and B for assistance)	YES___ (Go to 3)

3. ROADWAY AND/OR STRUCTURE REPAIR COSTS

LTEC
DESIGN

3a. Is the overtopping flood less than a 100 year frequency flood?

Yes__ (Go to 3 b.); No__ (Go to 3 i.)

3b. Compare the tailwater (TW) elevation with the roadway sag point elevation for the overtopping flood. Check the appropriate category.

__When TW is above the sag point (Go to 4)

__When TW is between 0 and .5′ below sag point (Go to 3 c.)

__When TW is between .5′ and 1.0′ below sag point (Go to 3 d.)

__When TW is 1.0′ and 2.0′ below sag point (Go to 3 e.)

__When TW is more than 2.0′ below sag point (Go to 3 g.)

3c. Does the embankment have a good erosion resistant vegetative cover?

Yes__ (Go to 3 i.); No__ (Go to 3 d.)

3d. Is the shoulder constructed from erosion resistant material such as paved, coarse gravel, or clay type soil?

Yes__ (Go to 3 i.); No__ (Go to 3 e.)

3e. Will the duration of overtopping for the 25 year flood exceed 1 hour?

Yes__ (Go to 3 f.); No__ (Go to 3 i.)

3f. Is the embankment constructed from erosion resistant material such as a clay type soil?

Yes__ (Go to 3 i.); No__ (Go to 3 g.)

3g. Is the overtopping flood less than a 25 year frequency flood?

Yes__ (Go to 3 h.); No__ (Go to 3 i.)

	LTEC DESIGN
3h. Will the cost of protecting the roadway and/or embankment from severe damage caused by overtopping exceed the cost of providing additional culvert or bridge capacity? No __ (Go to 3 i.);	YES (Go to 3 i.)
3i. Is there damage potential to the structure caused by scour, ice, debris or other means during the lesser of the overtopping flood or the 100 year flood? Yes __ (Go to 3 j.); No__ (Go to 4)	
3j. Will the cost of protecting the structure from damage exceed the cost of providing additional culvert or bridge water capacity? No __ (Go to 4);	YES__ (Go to 4)
4. Will the capital cost of the structure exceed $500,000? No __ (Go to 5);	YES__ (Go to 5)
5. In your opinion, are there any other factors which you feel should require further study through a risk analysis? No __ (Go to 6);	YES__ (Indicate)
6. If there are no ✓'s in the LTEC Design column on the right, proceed with the design, selecting the lowest acceptable grade line and the smallest waterway opening consistent with the constraints imposed on the project. The risk assessment has demonstrated that potential flood damage costs, traffic related costs, roadway and/or structure repair costs are minor and therefore disregarded for this project. One or more ✓'s in the LTEC Design column indicates further analysis in the category checked may be required utilizing the LTEC design process or justification why it is not required.	

JUSTIFICATION ____

____

____

____

I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of Minnesota

____

Registration Number:______________________ Date:______________________

Appendix B TP-40 RAINFALL INTENSITY CURVES

Rainfall Curves provided are from the U.S. Weather Bureau's *Technical Publication No. 40* (Hershfield, 1961). Additional rainfall curves for different frequencies and durations are available in TP-40.

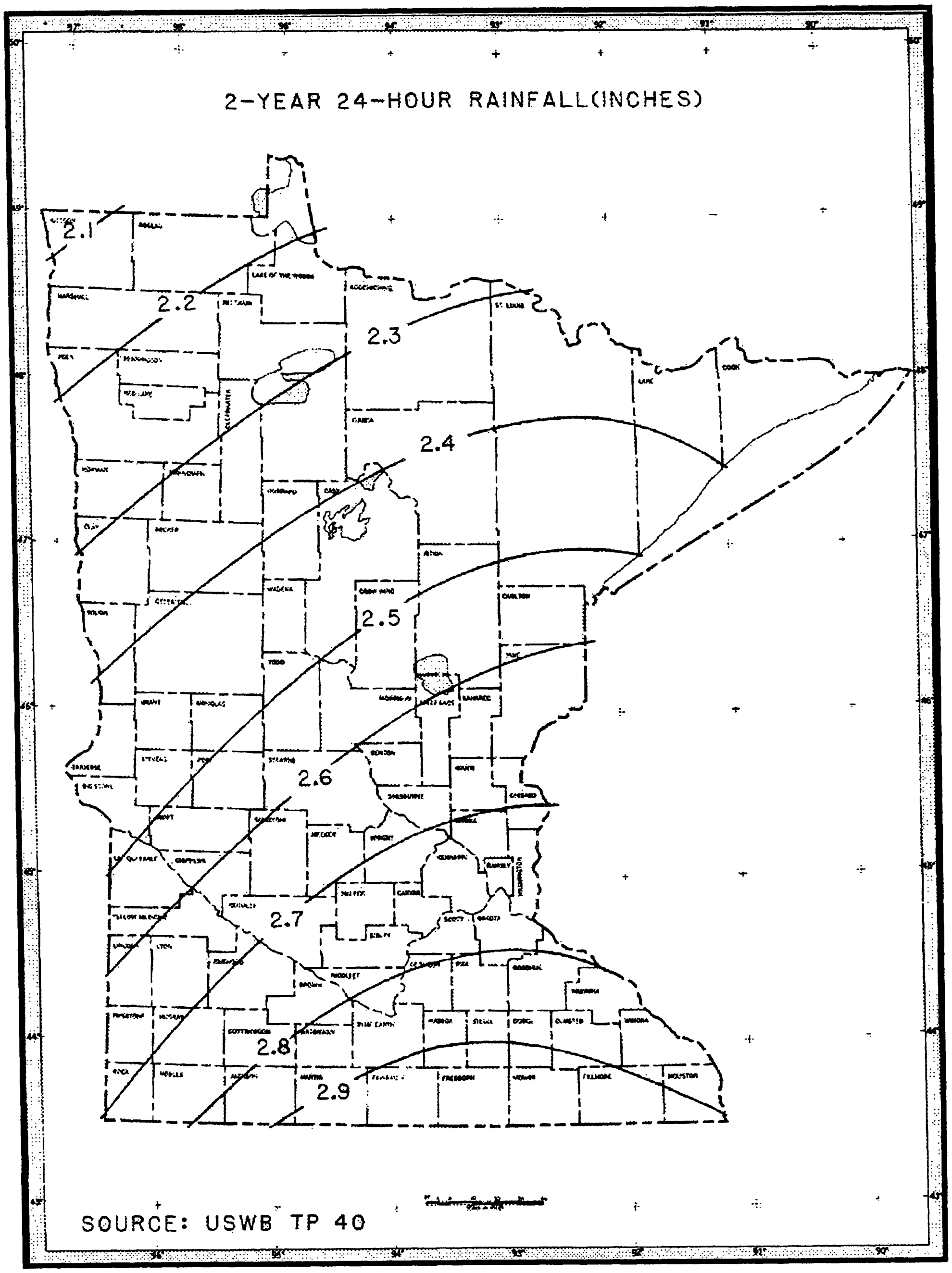


Figure B.1 TP-40 Rainfall Curves for 2-year 24-hour

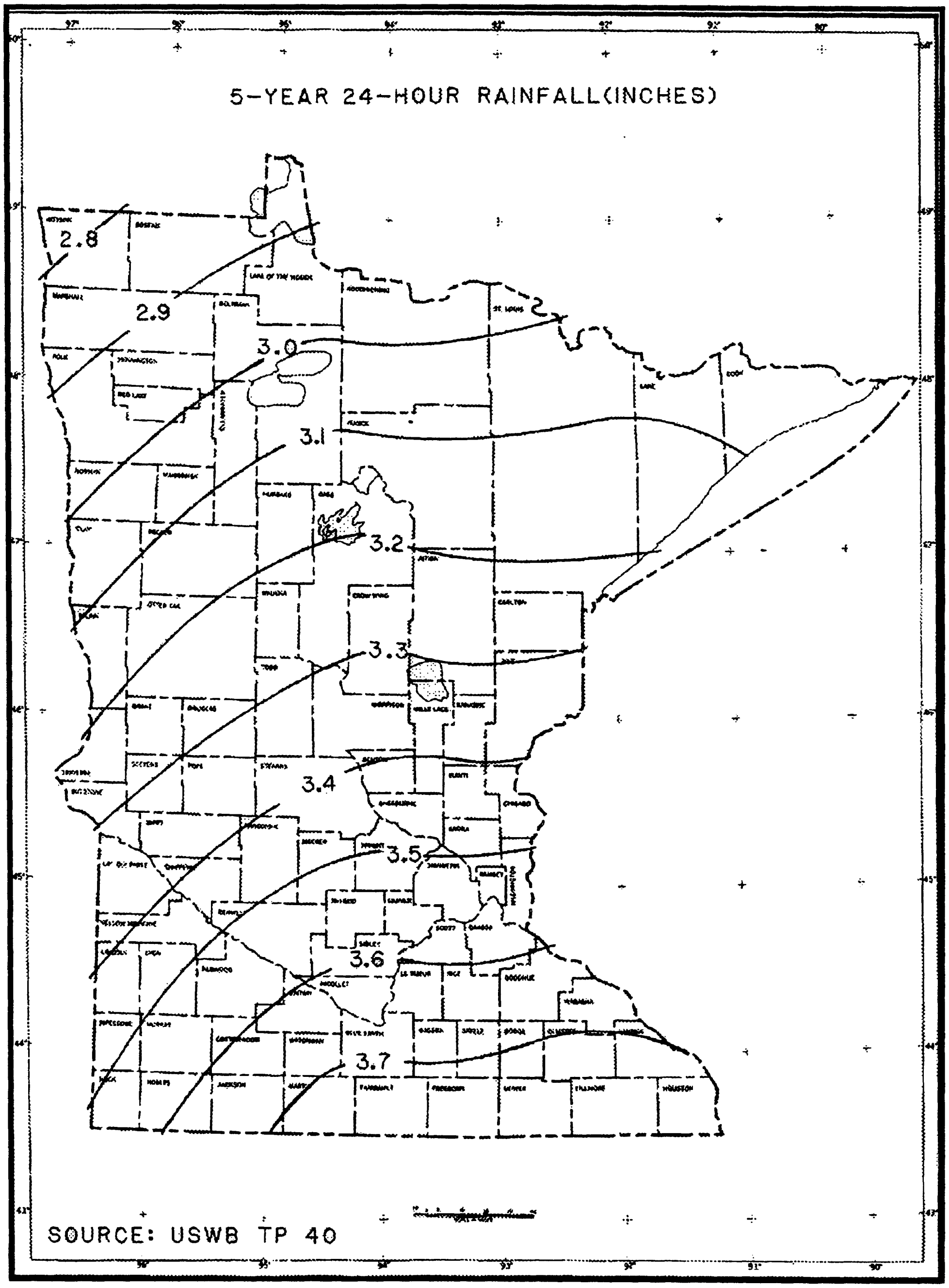


Figure B.2 TP-40 Rainfall Curves for 5-year 24-hour

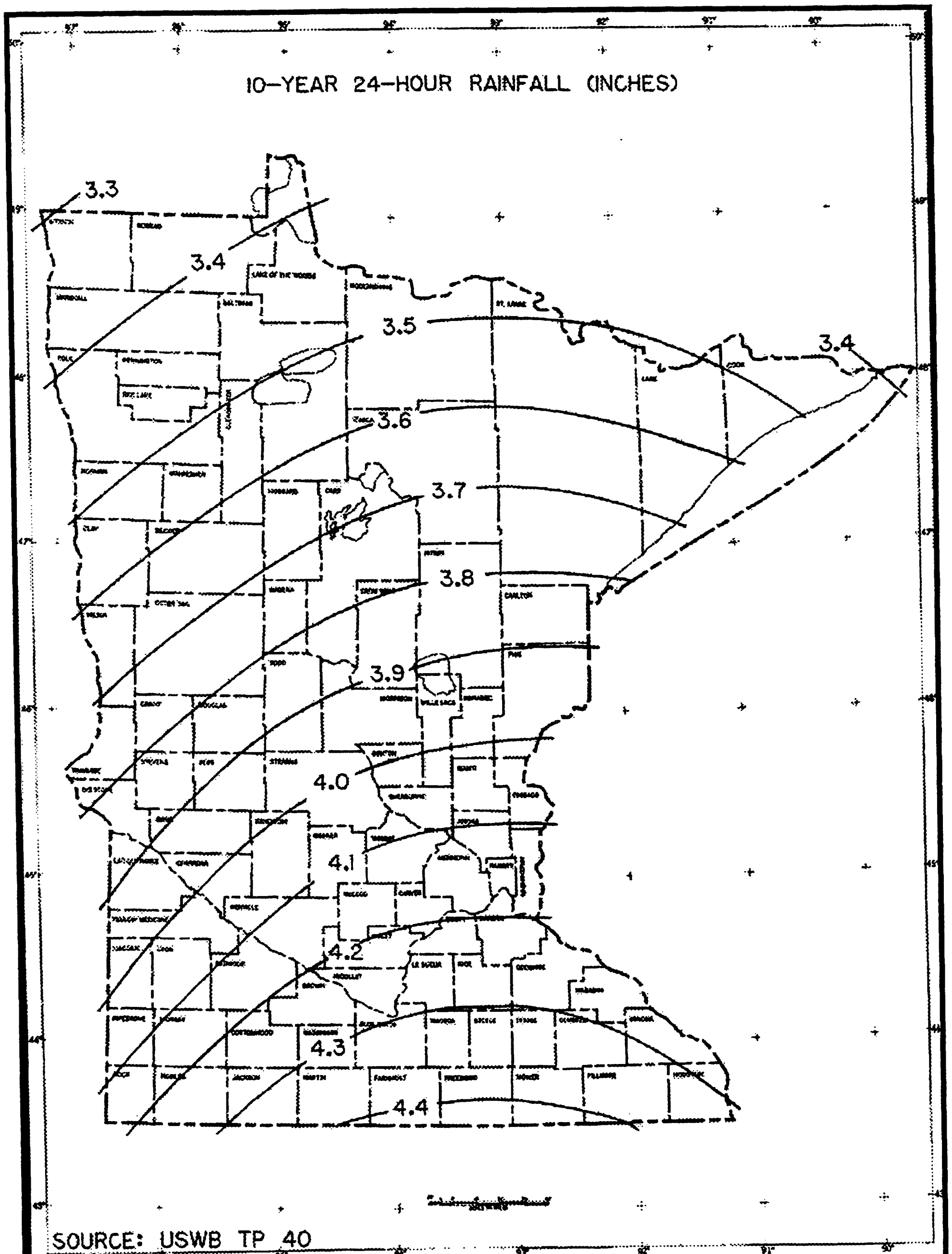


Figure B.3 TP-40 Rainfall Curves for 10-year 24-hour

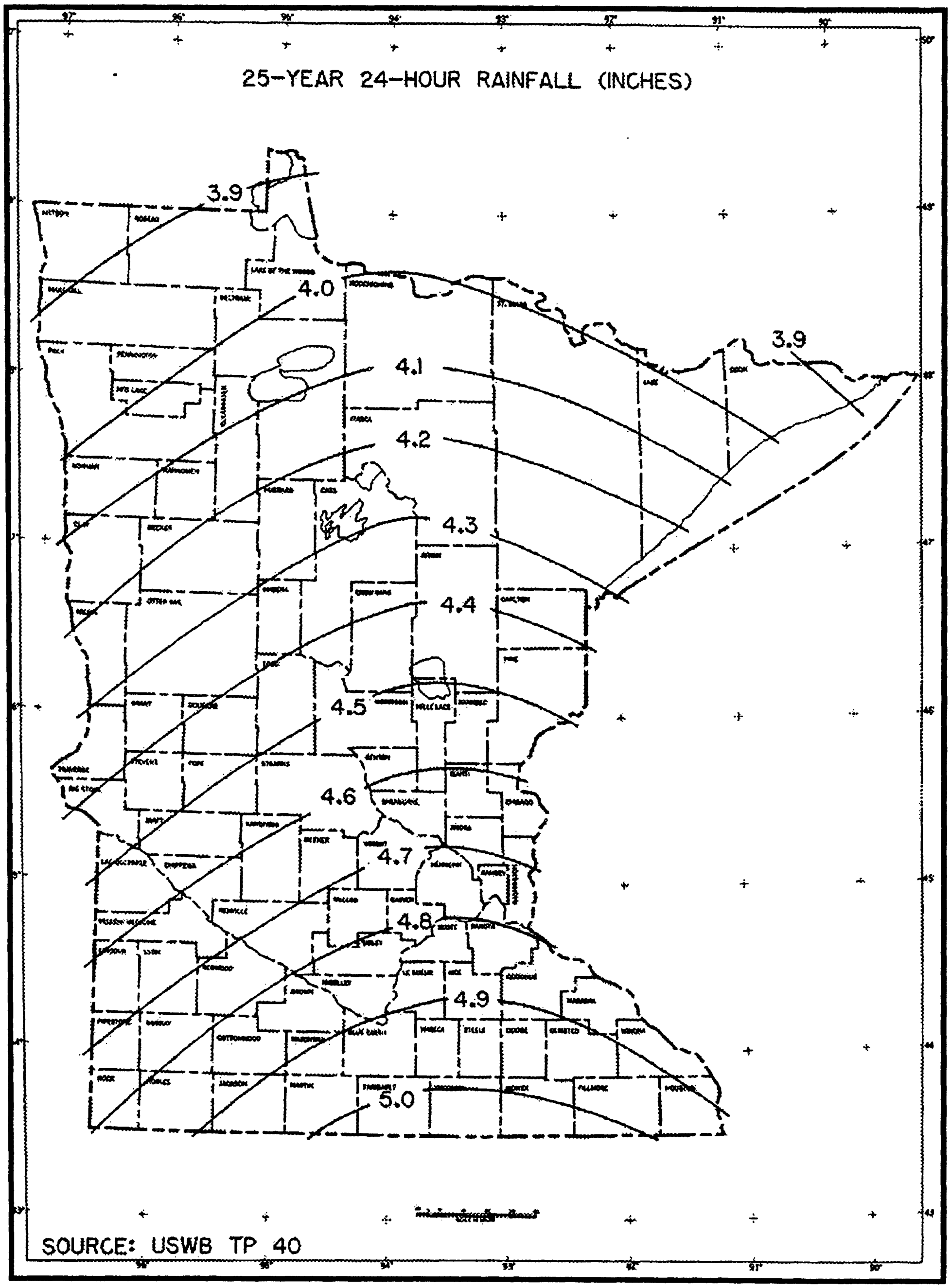


Figure B.4 TP-40 Rainfall Curves for 25-year 24-hour

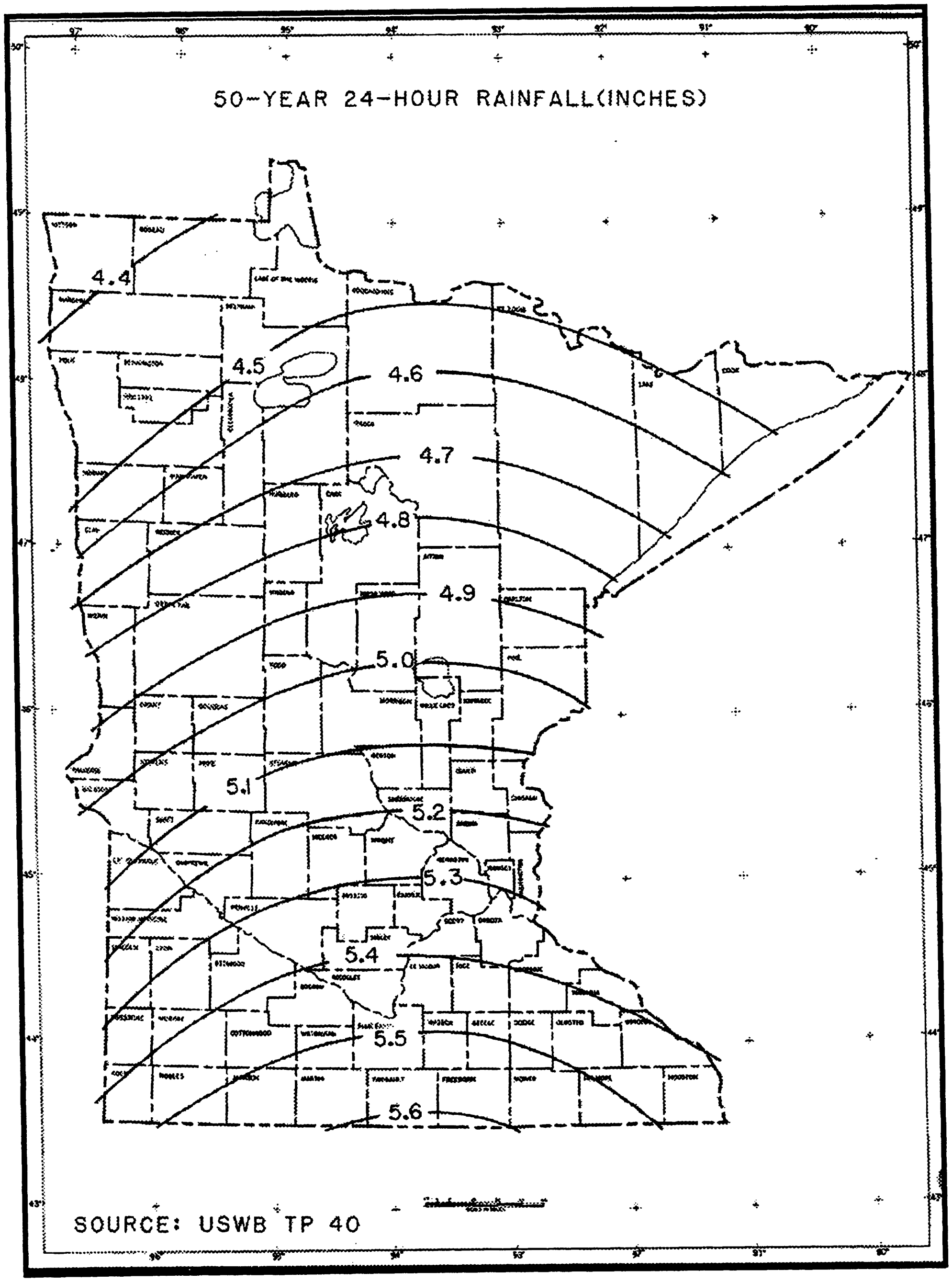


Figure B.5 TP-40 Rainfall Curves for 50-year 24-hour

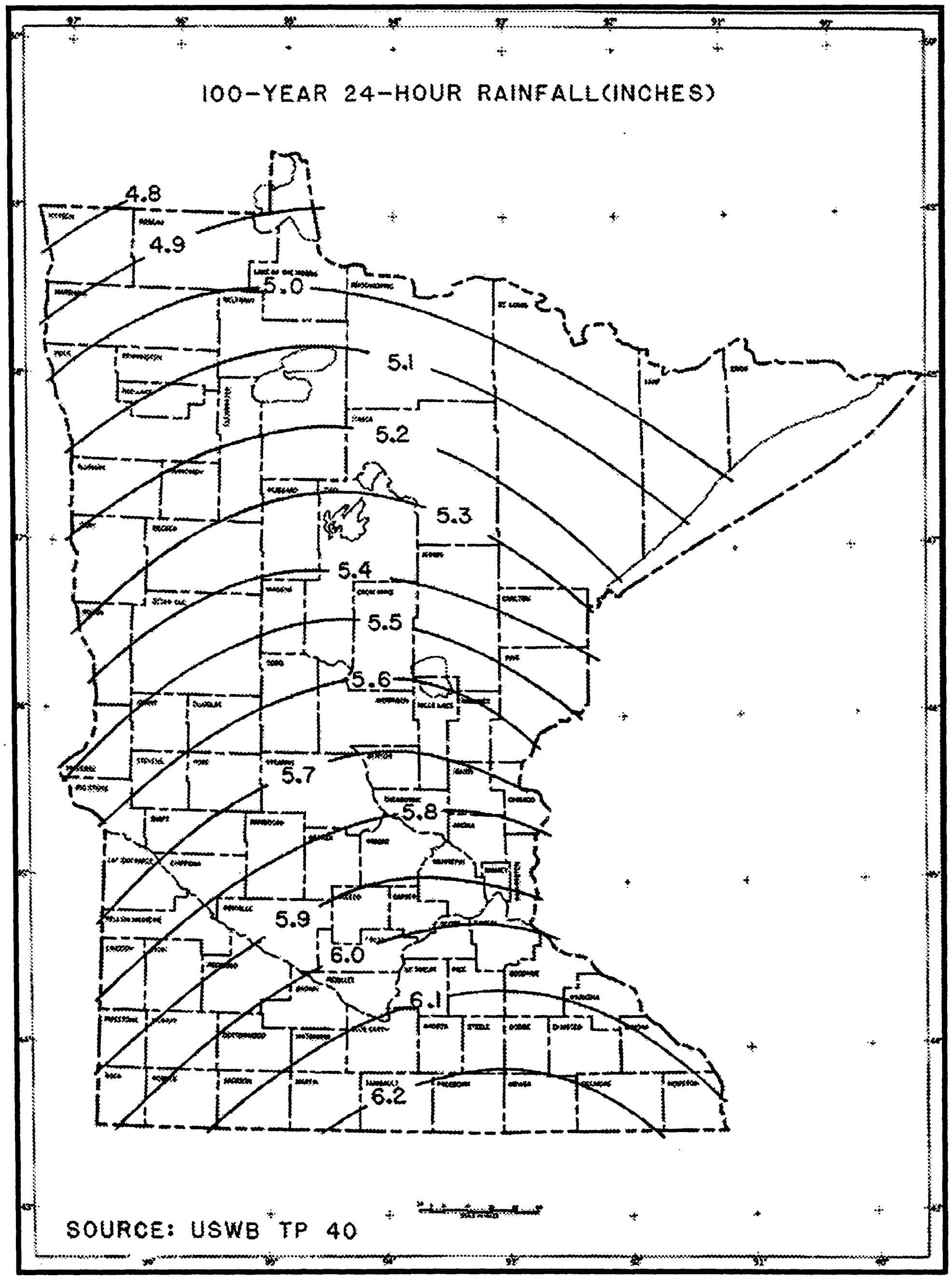


Figure B.6 TP-40 Rainfall Curves for 100-year 24-hour

Appendix C PIPE FLOW DESIGN CHARTS

HDS-5, *Hydraulic Design of Highway Culverts* (FHWA, 1985) contains pipe design charts. Some of these charts have been reproduced in this appendix. An index of the design charts provided is given below, not all charts included in the FHWA manual are reproduced in this document.

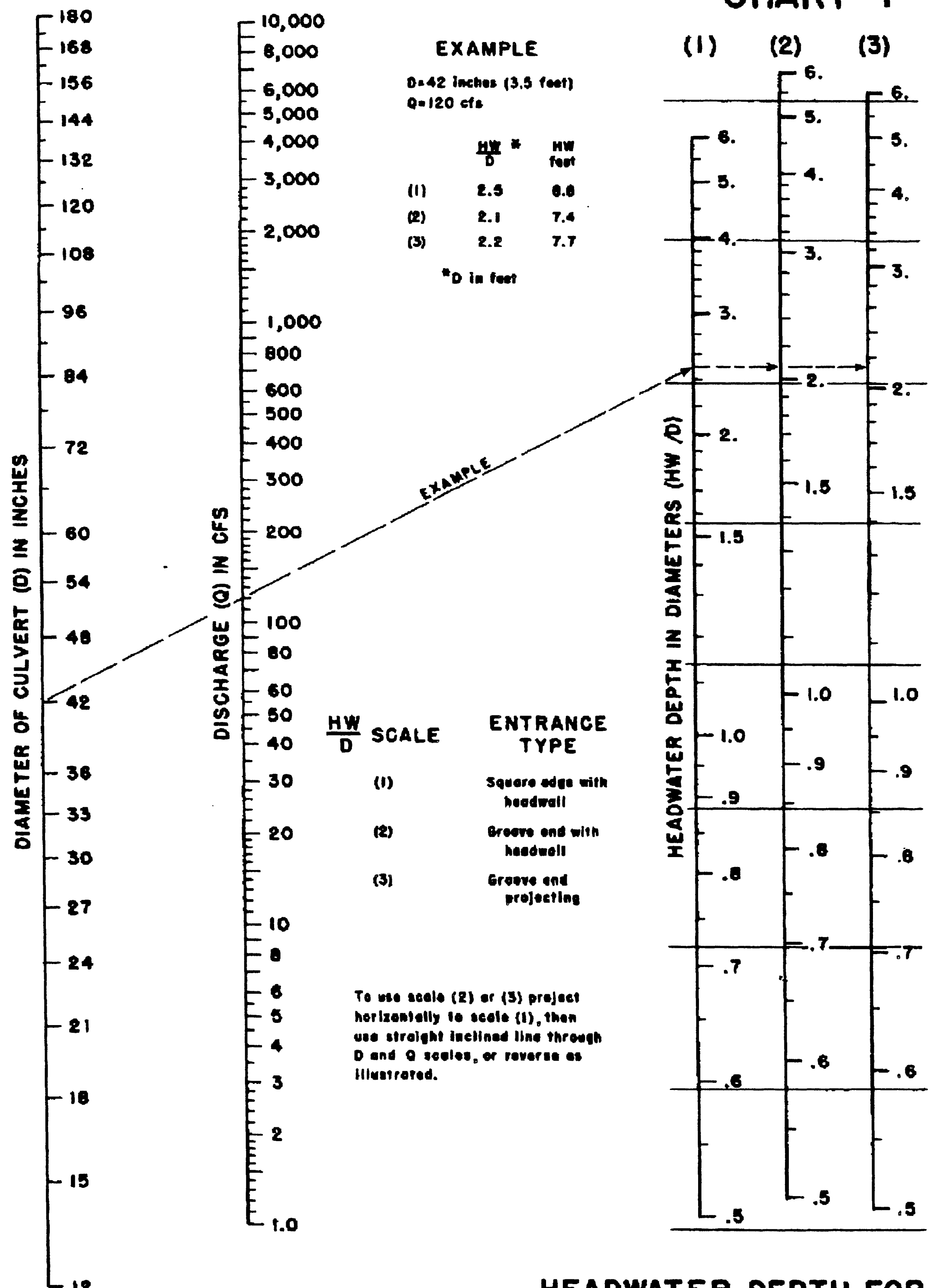
Design curves were developed for common conduit shapes, sizes and inlet edge configurations constants using a set of design equations. The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs.

A constant slope value of 2 percent (0.02) was selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent except for mitered inlets.

In formulating inlet and outlet control design nomographs, a certain degree of error is introduced into the design process. This error is due to the fact that the nomograph construction involves graphical fitting techniques resulting in scales which do not exactly match the equations. Checks by the authors and others indicate that all of the nomographs from HDS-5 (FHWA, 1985) have a precision of ± 10 percent of the equation values in terms of headwater (inlet control) or head loss (outlet control).

Chart Number	Shape	Control Section	Material	Type
1	Circular	Inlet	Concrete	
2	Circular	Inlet	Metal	
3	Circular	Inlet	Metal	Beveled Ring Control
4	Circular	Critical		
5	Circular	Outlet	Concrete	n = 0.012
6	Circular	Outlet	Metal	n = 0.024
7	Circular	Outlet	Metal	n = 0.0328 to 0.0302
8	Box	Inlet	Concrete	
9	Box	Inlet	Concrete	Wingwalls 18° to 33.7° and 45°
10	Box	Inlet	Concrete	90° Headwall, Beveled Edges
11	Box	Inlet	Concrete	Skewed Headwalls, Beveled Edges
12	Box	Inlet	Concrete	Flared Wingwalls, Normal and Skewed
13	Box	Inlet	Concrete	Offset Flared Wingwalls, Inlet Top beveled edge
14	Box	Critical	Concrete	Rectangular
15	Box	Outlet	Concrete	n = 0.012
34	Pipe Arch	Inlet	Metal	
35	Pipe Arch	Inlet	Metal	18 in. Corner Radius
36	Pipe Arch	Inlet	Metal	31 in. Corner Radius
37	Pipe Arch	Critical		Standard
38	Pipe Arch	Critical		Structural Plate
39	Pipe Arch	Outlet	Metal	n = 0.024
40	Pipe Arch	Outlet	Metal	18 in. Corner Radius
60	Roadway	Overtopping		Discharge Coefficients

CHART 1



HEADWATER SCALES 2&3
REVISED MAY 1964

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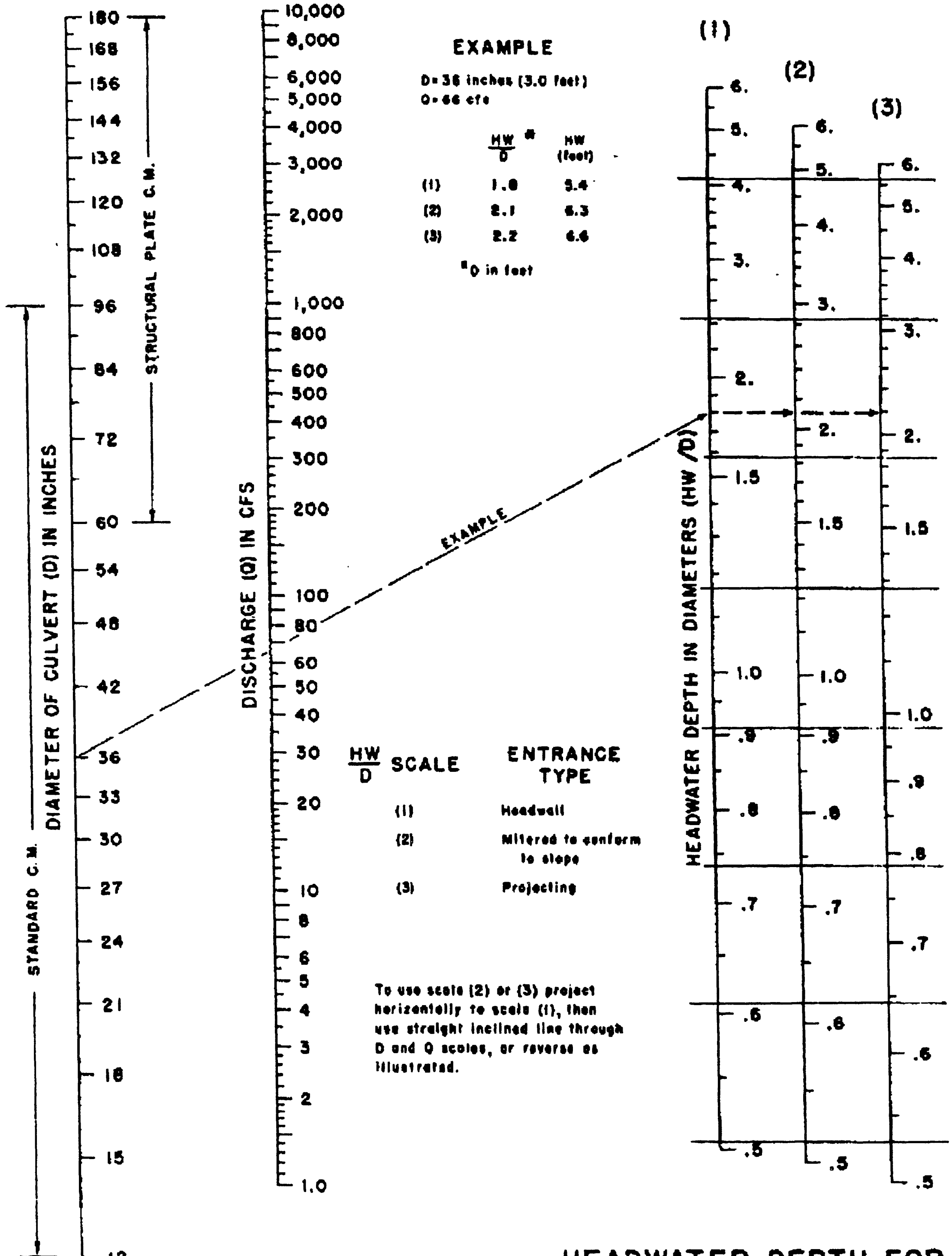
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART 2



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

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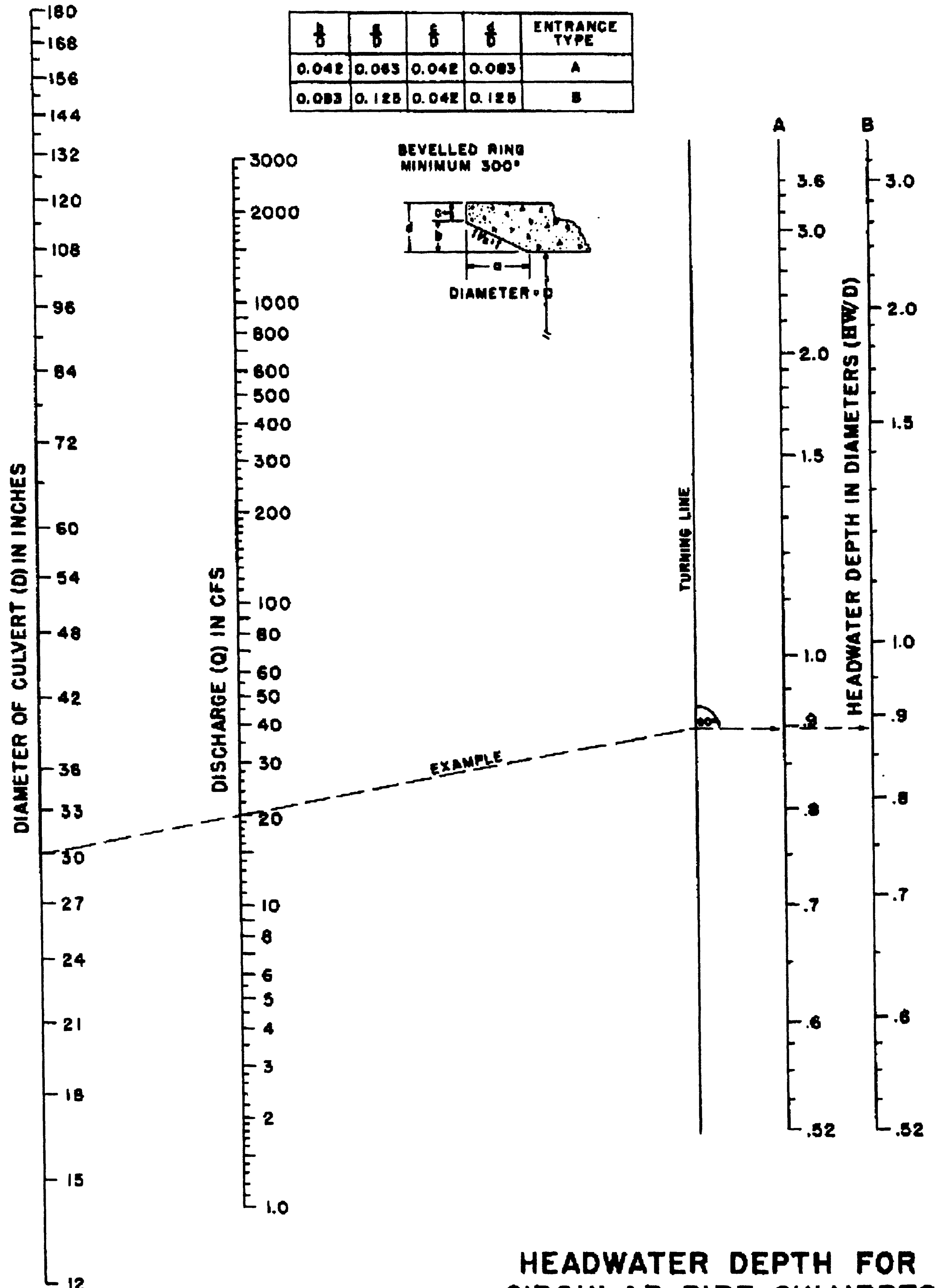
CHART 3

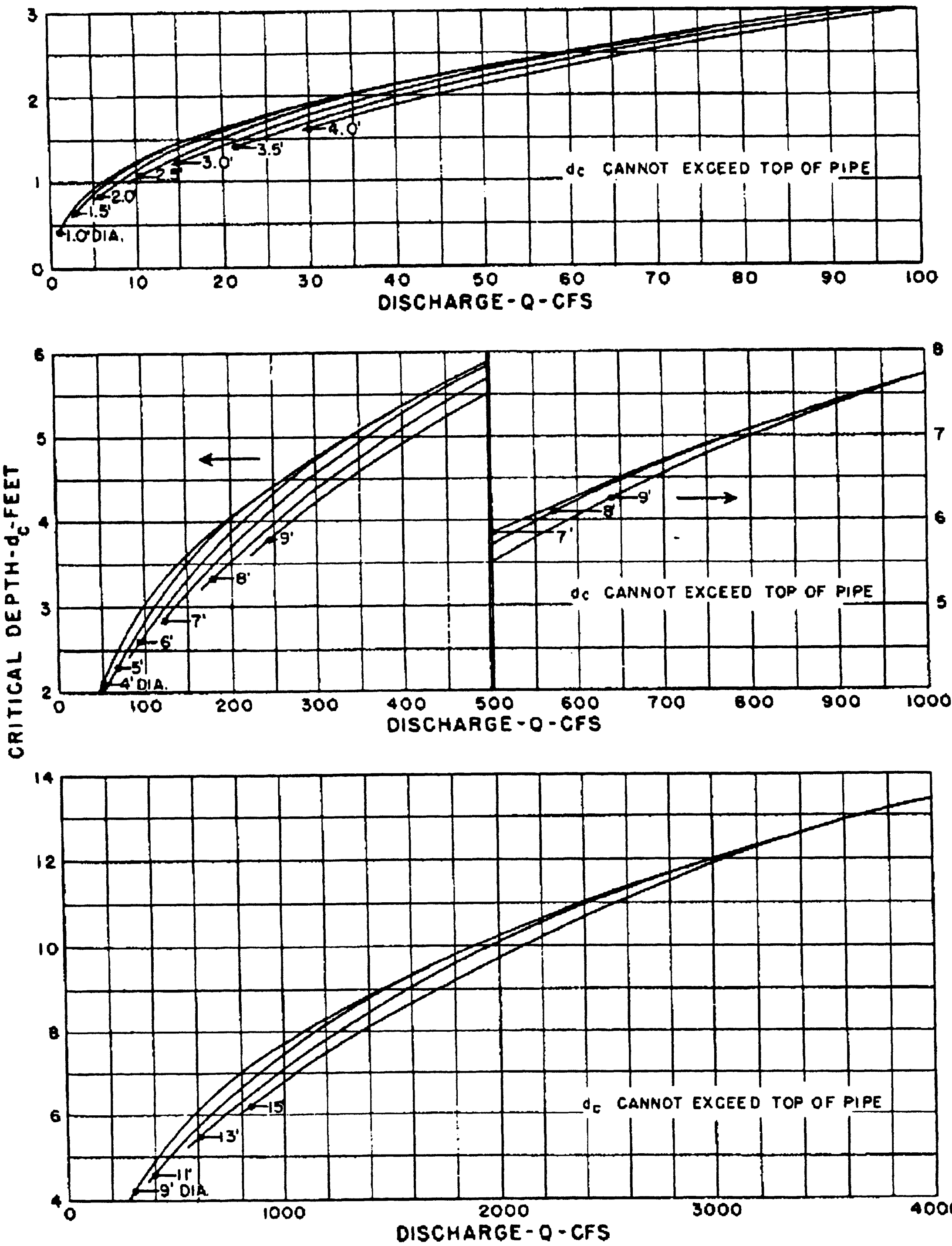
a/D	b/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

FEDERAL HIGHWAY ADMINISTATION
MAY 1973

CHART 4



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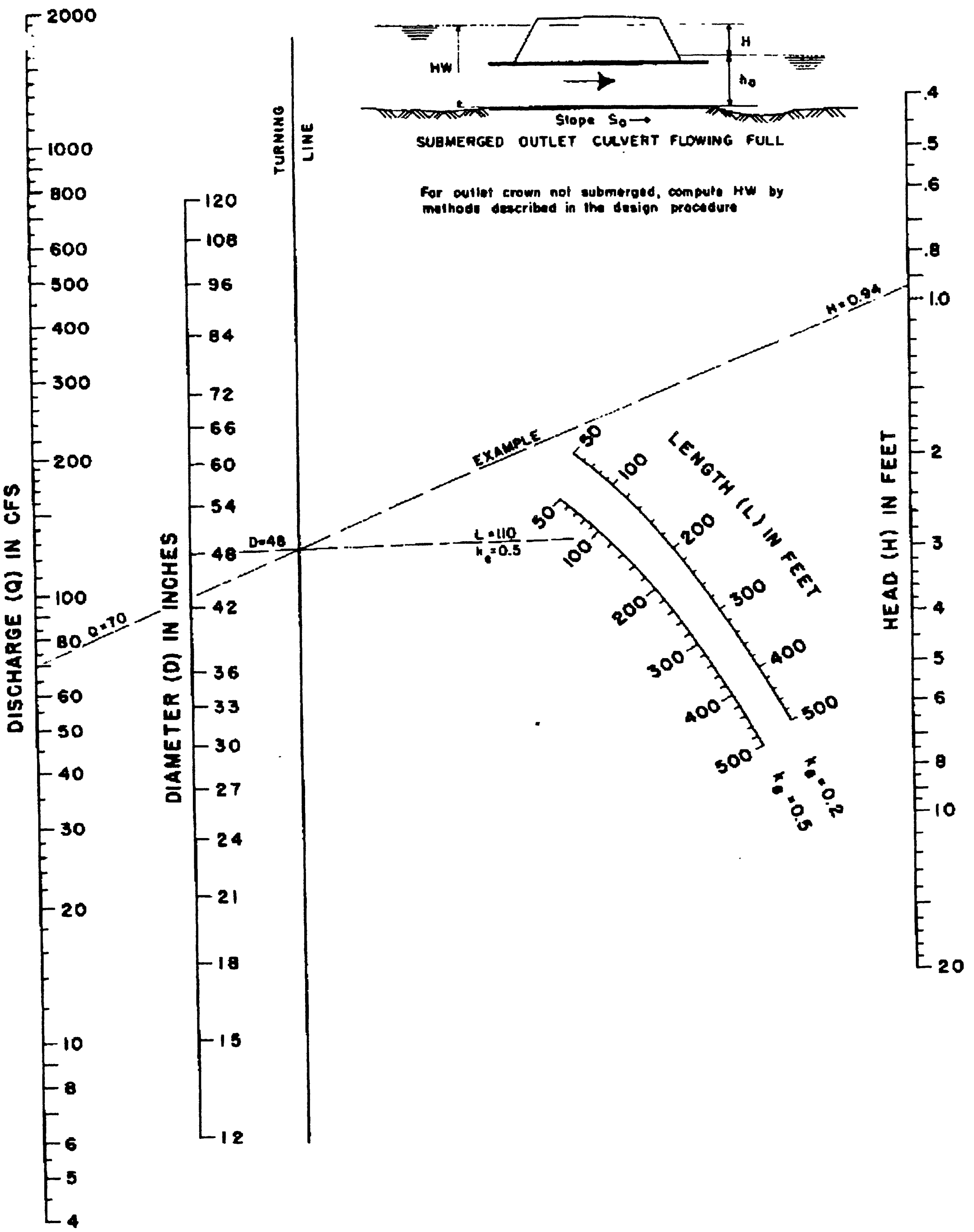
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

CHART 5



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

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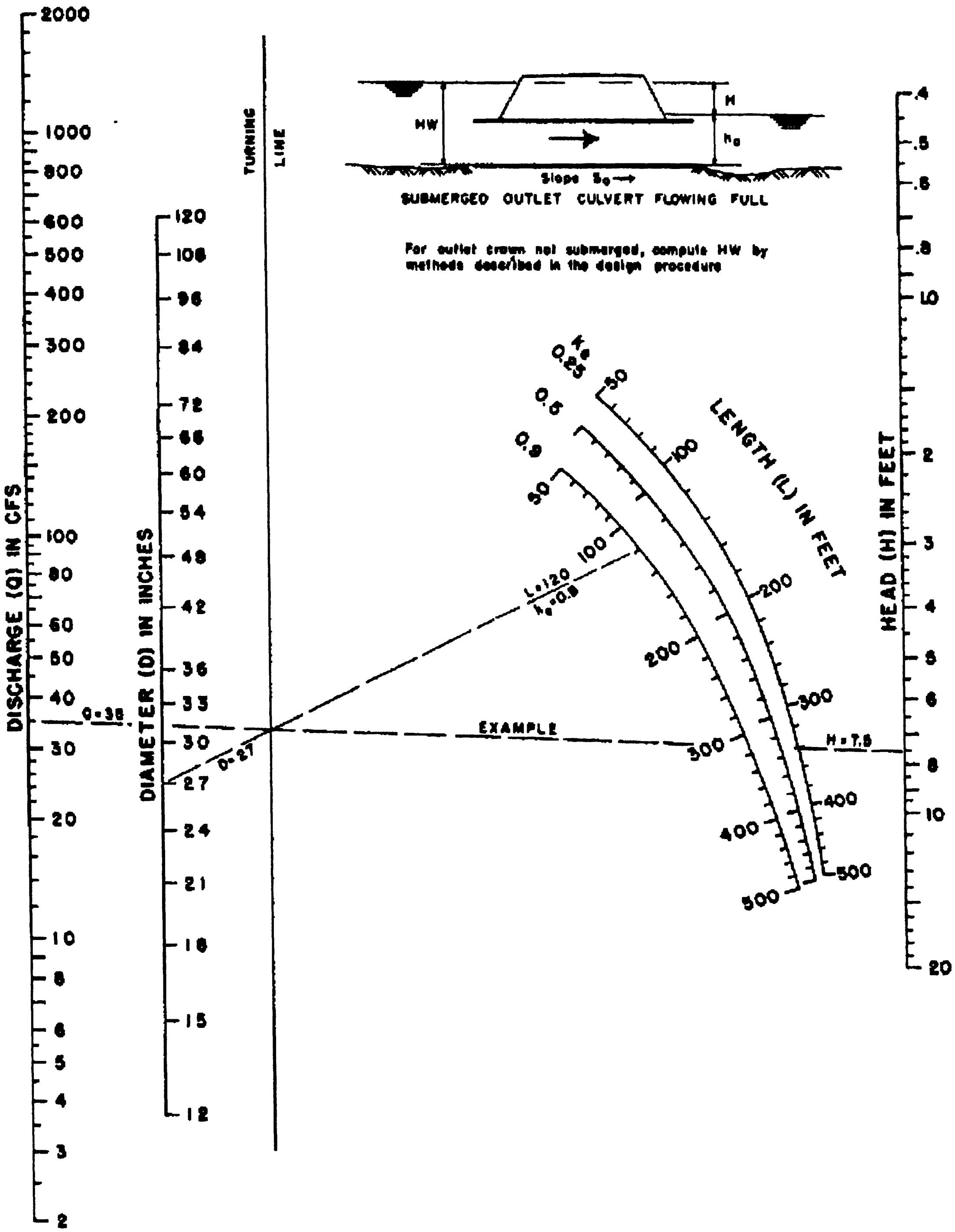
CHART 6



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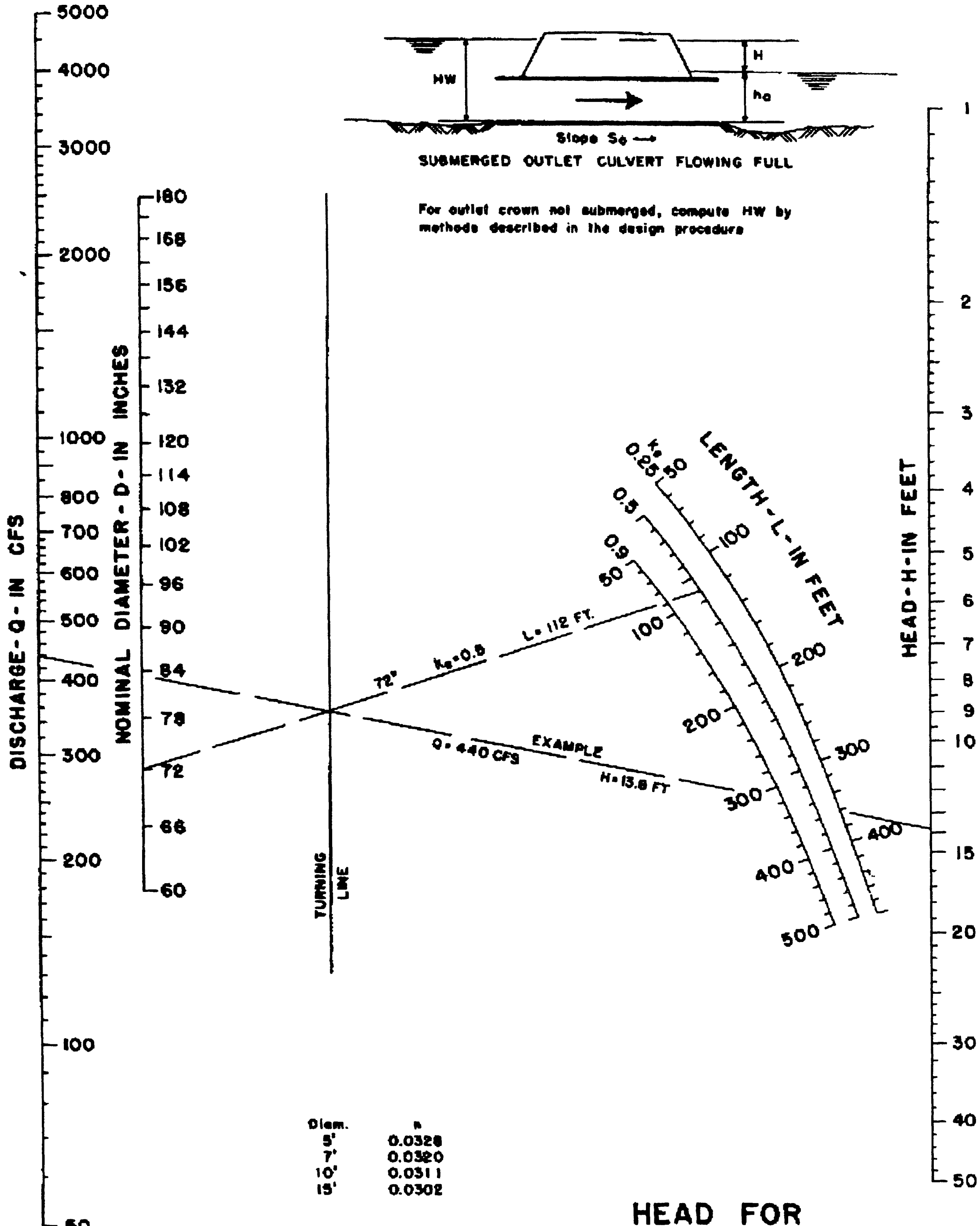
HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

CHART 7



Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

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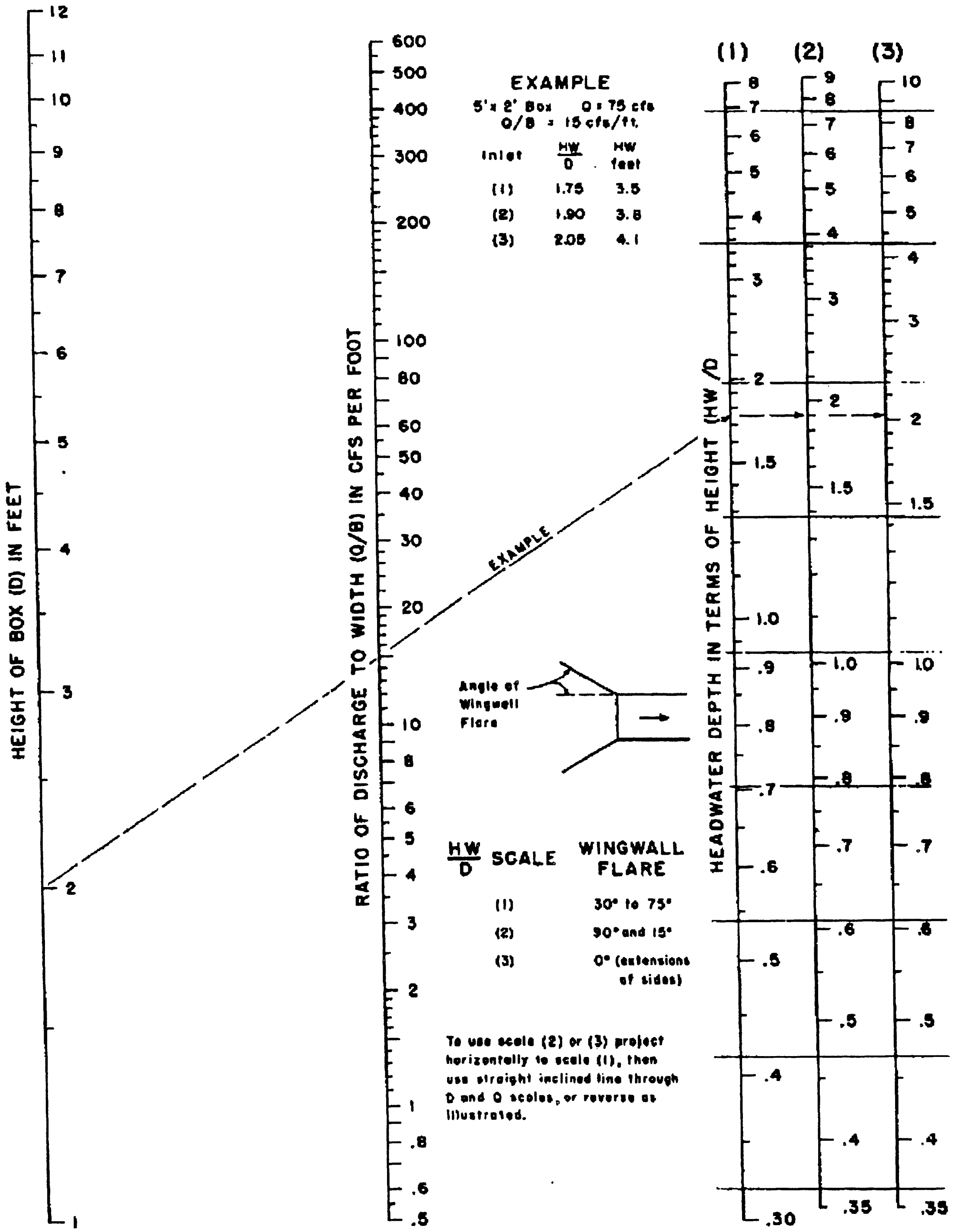
CHART 8



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

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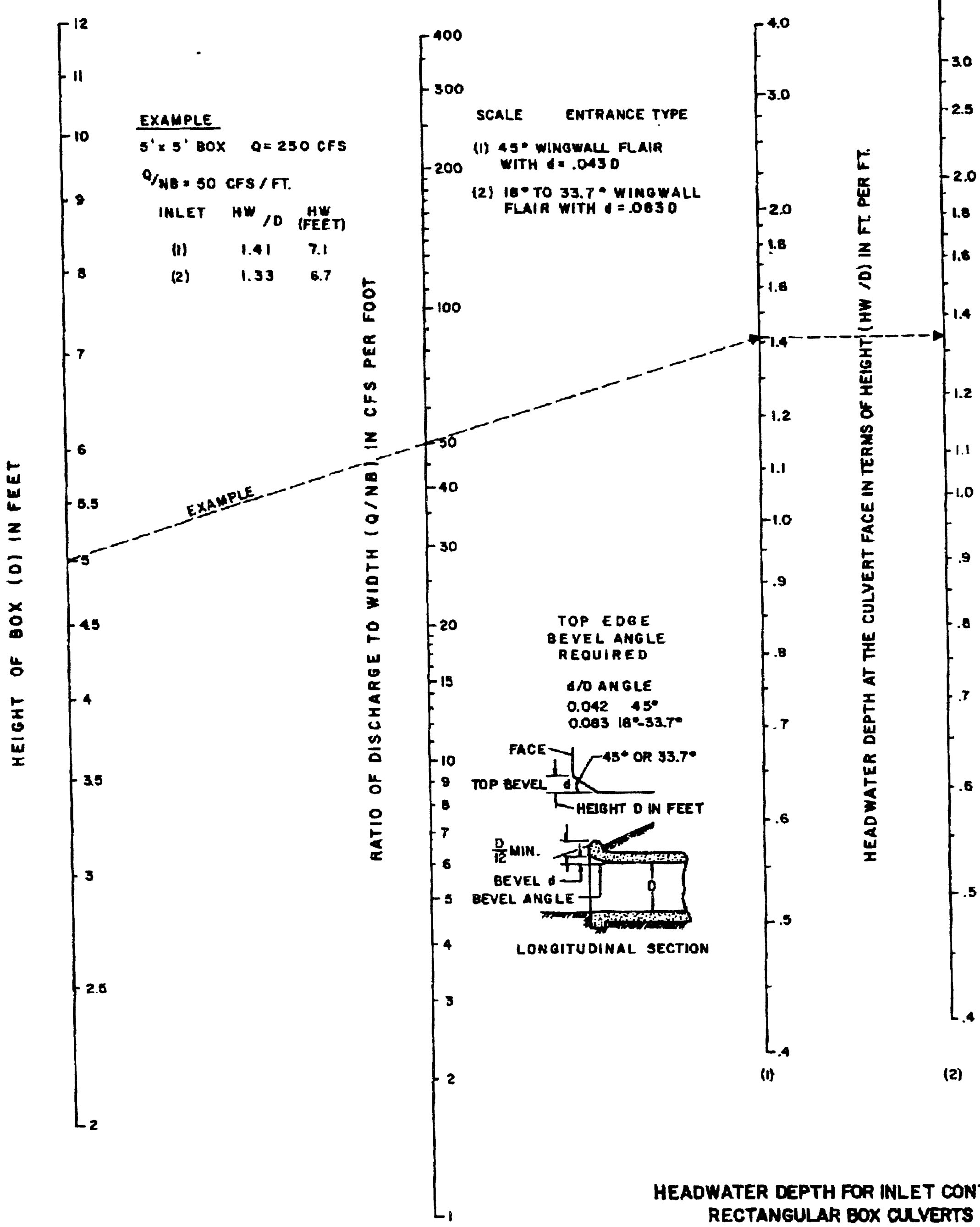
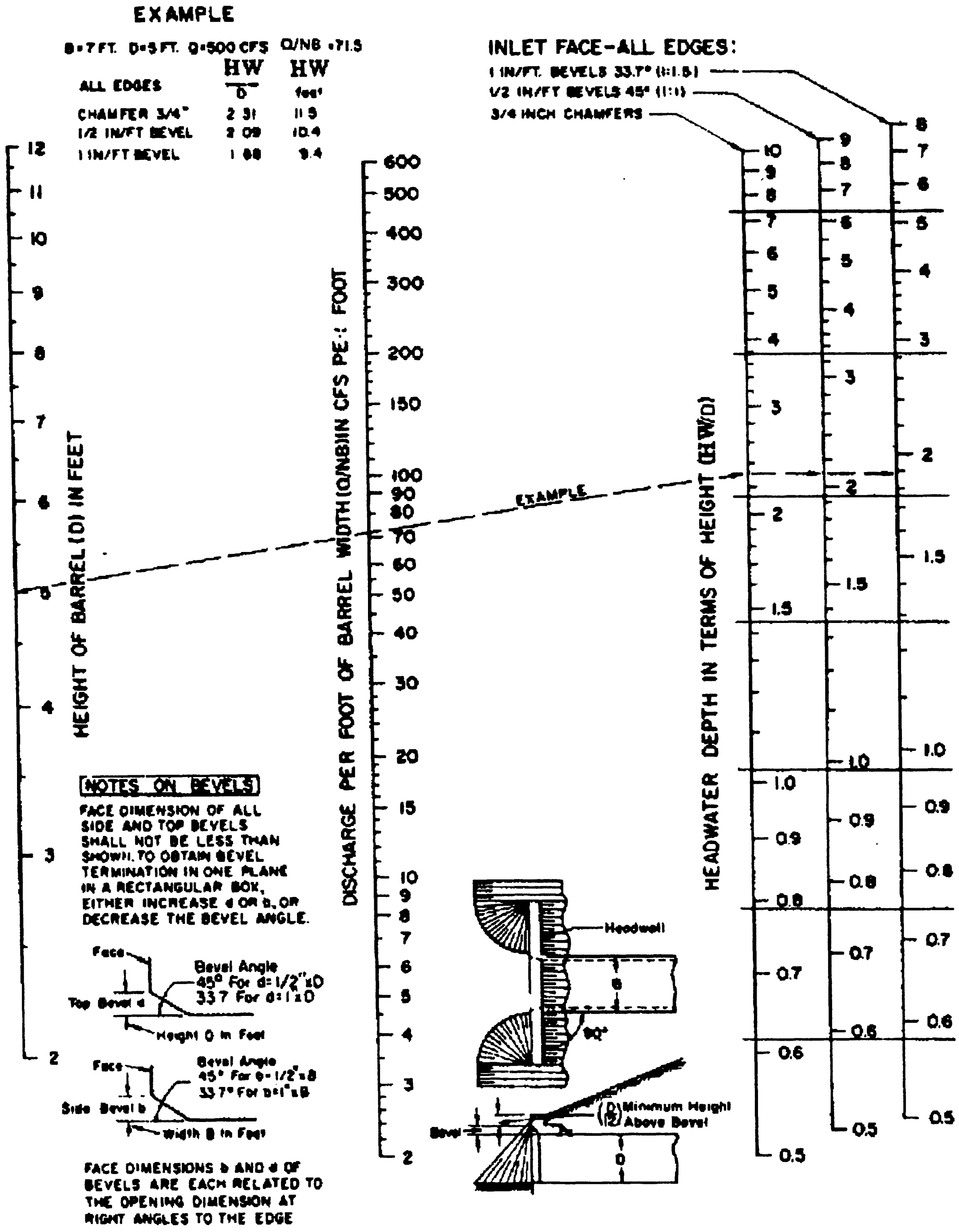
CHART 9



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

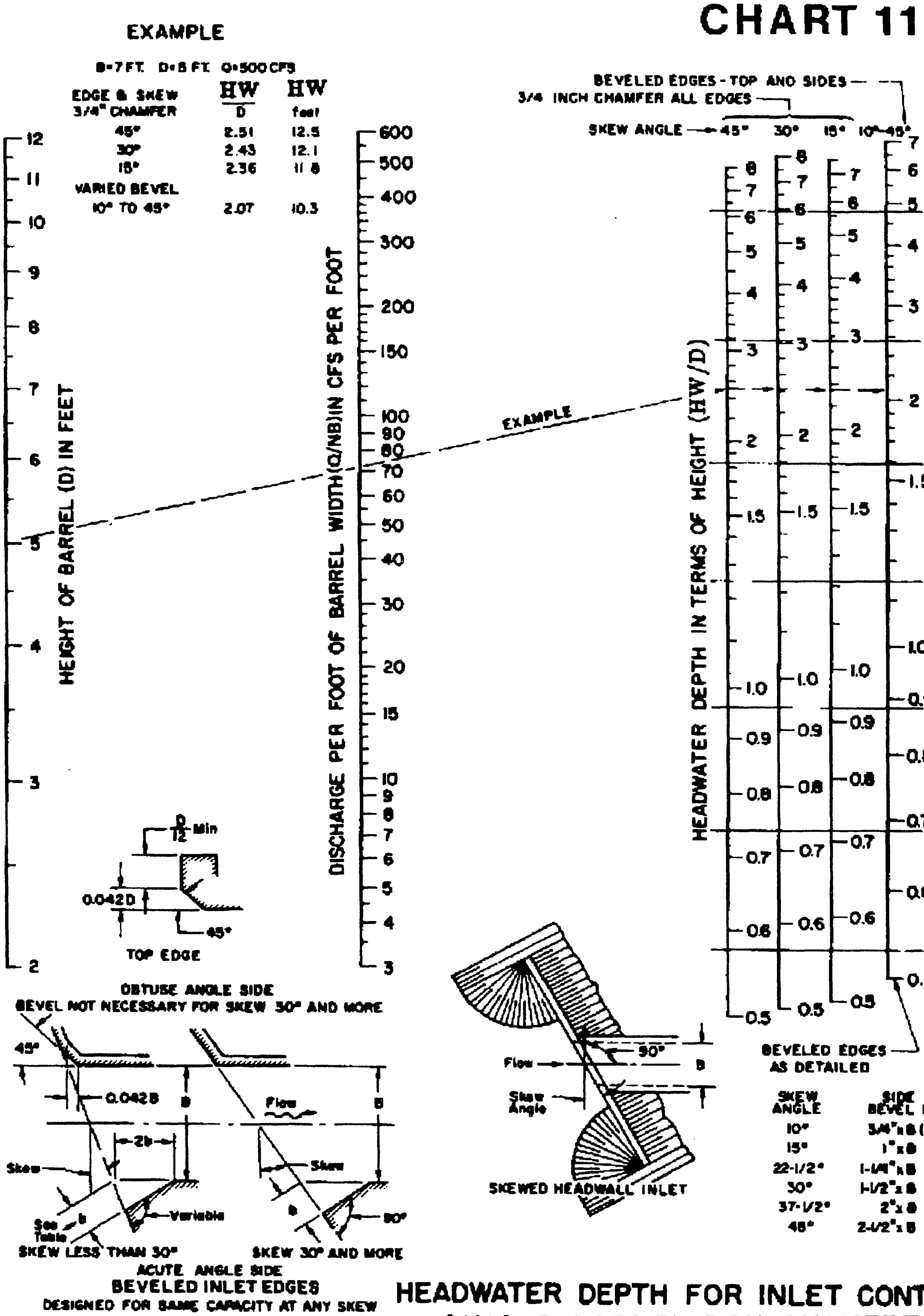
CHART 10



FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

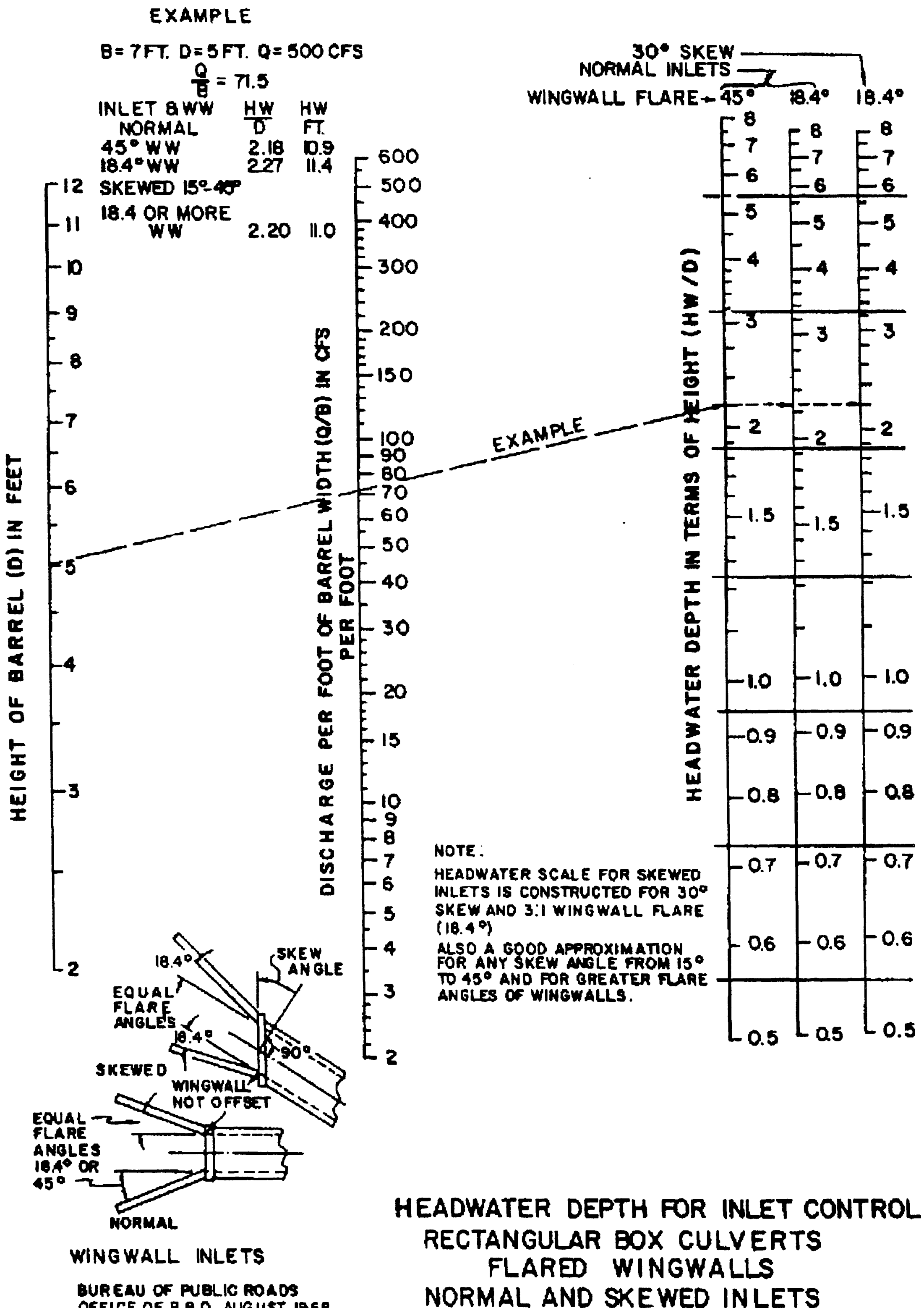
CHART 11



FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

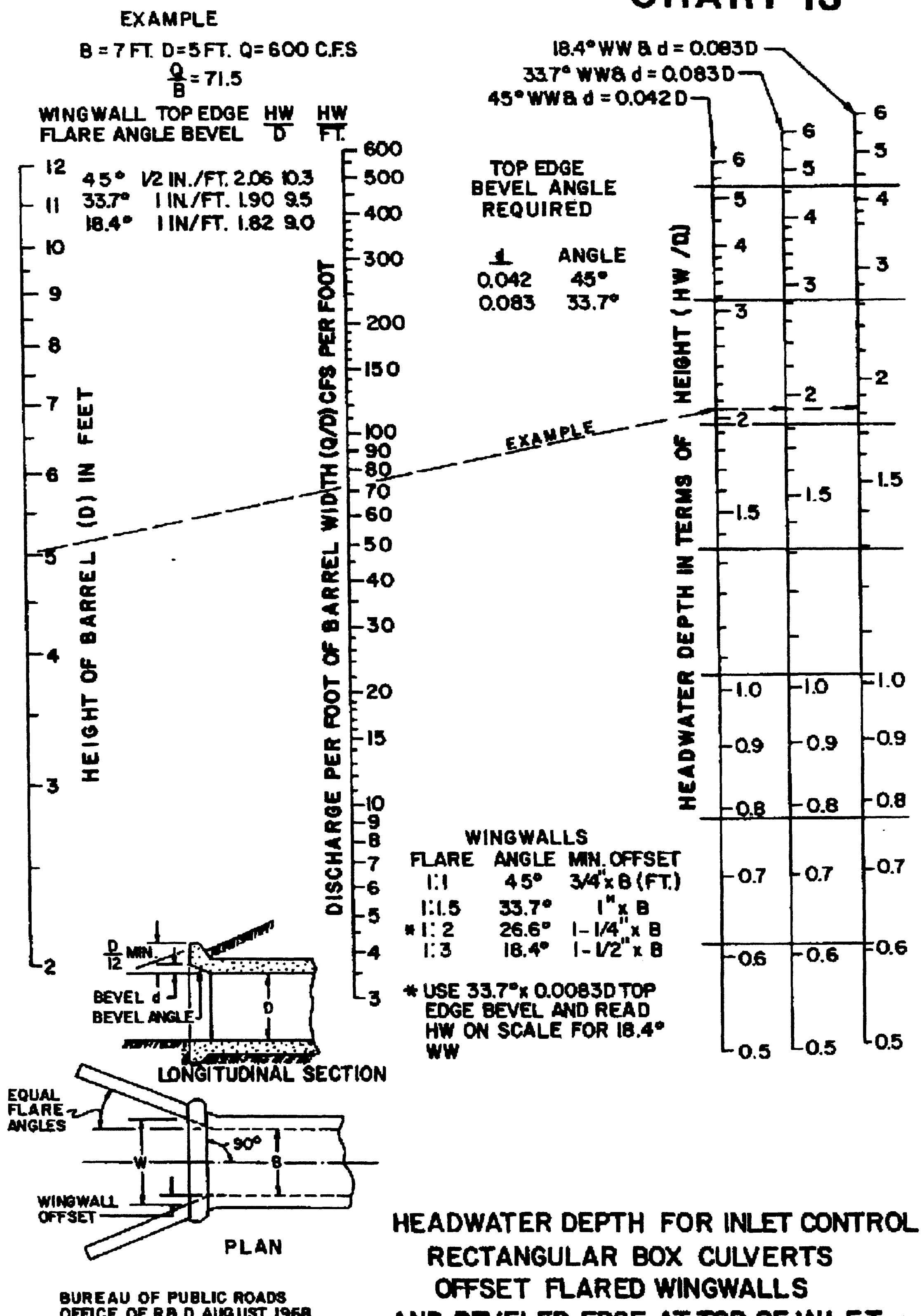
HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

CHART 12



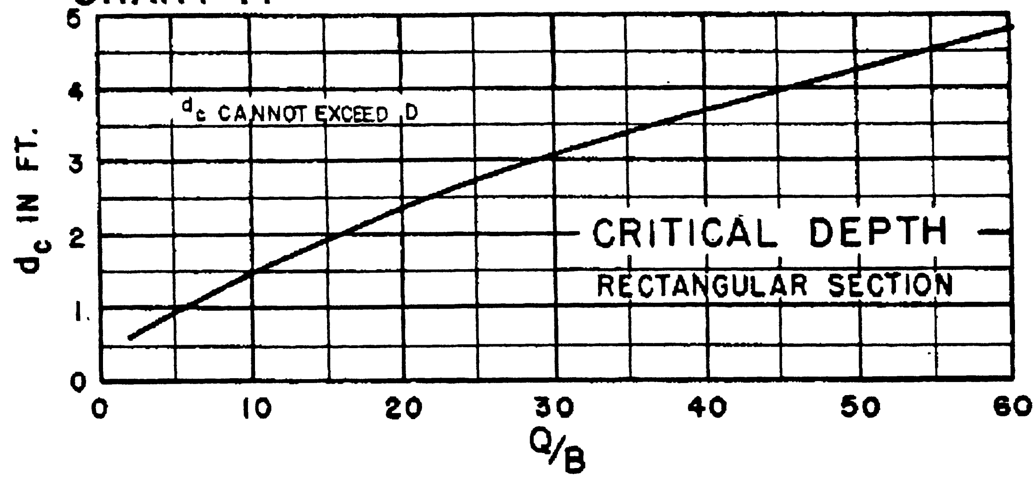
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING

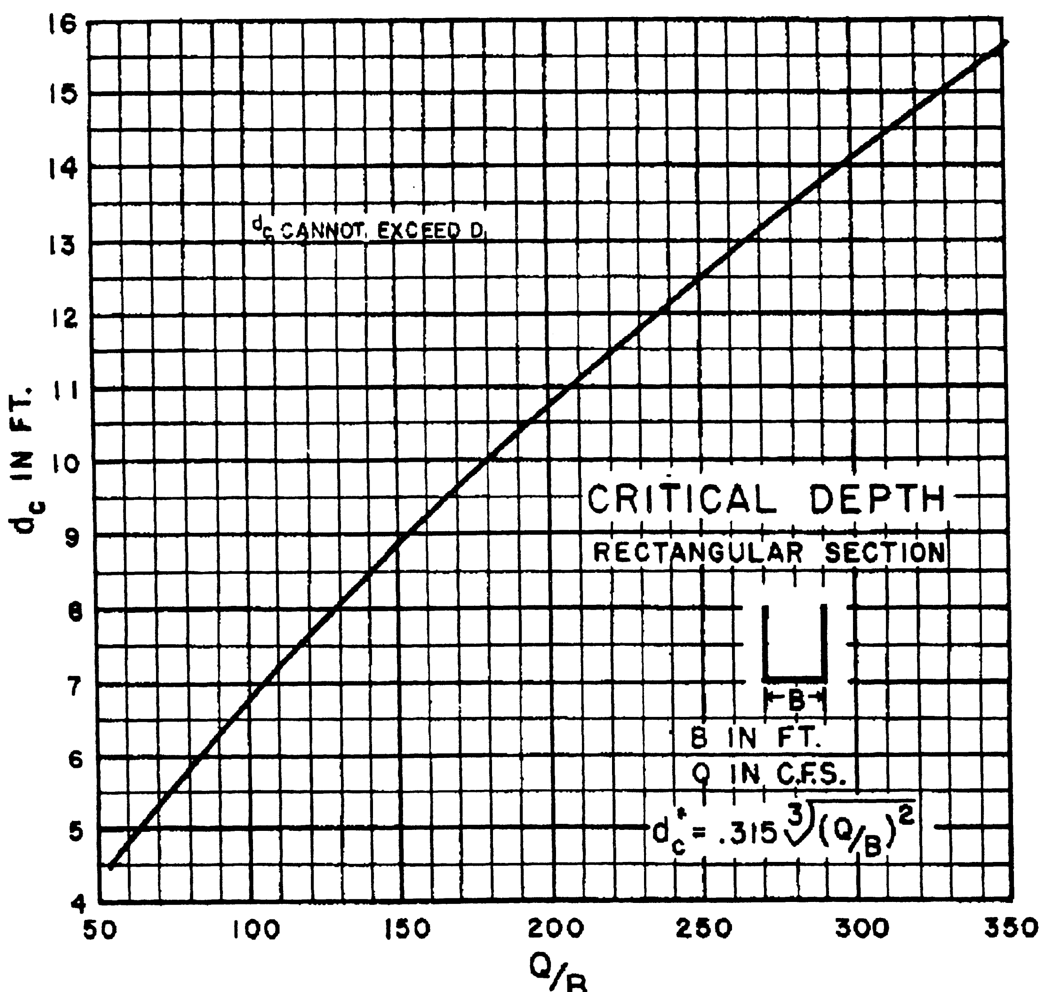
CHART 13



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET

CHART 14





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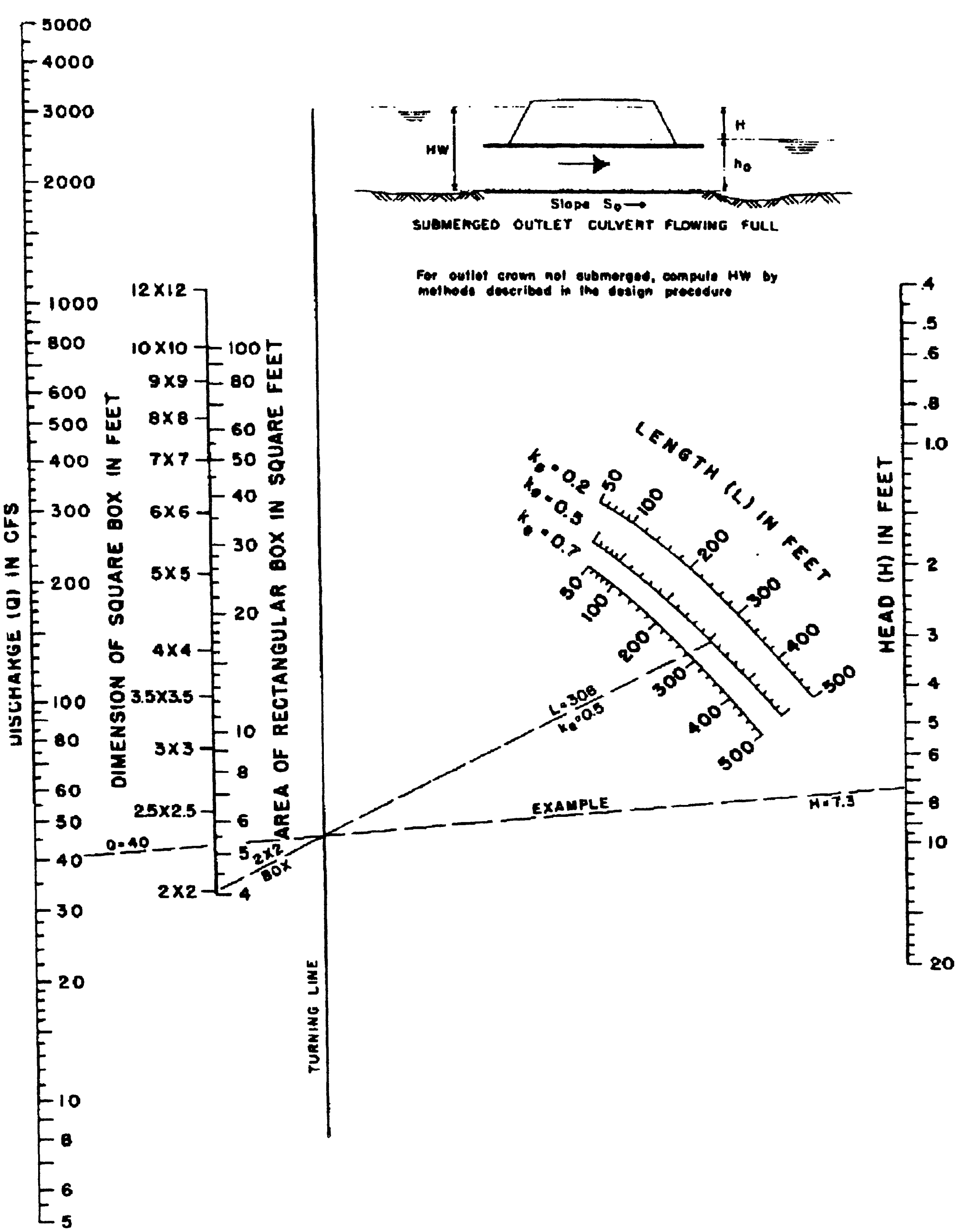
CRITICAL DEPTH
RECTANGULAR SECTION

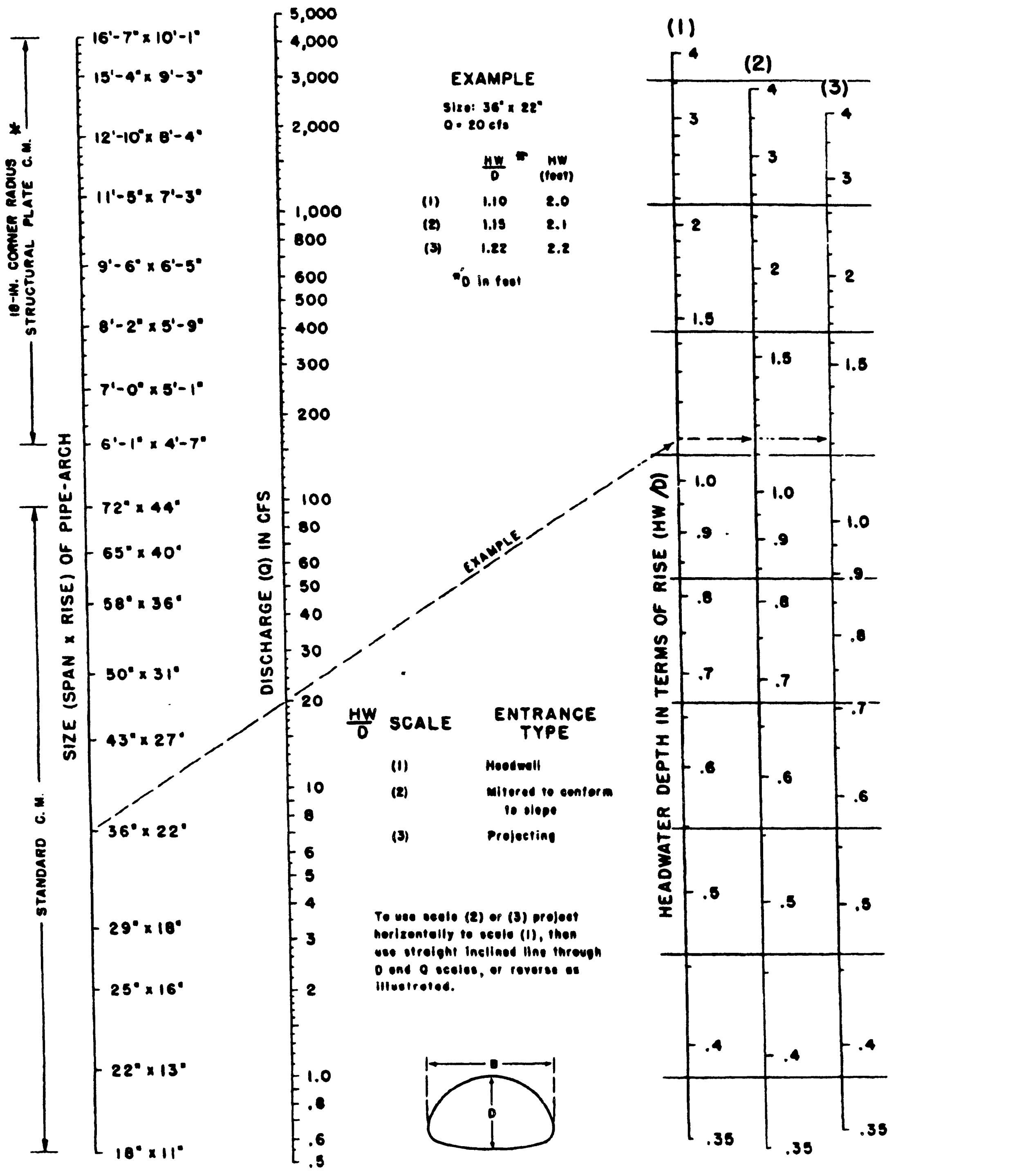
CHART 15



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

CHART 34



*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

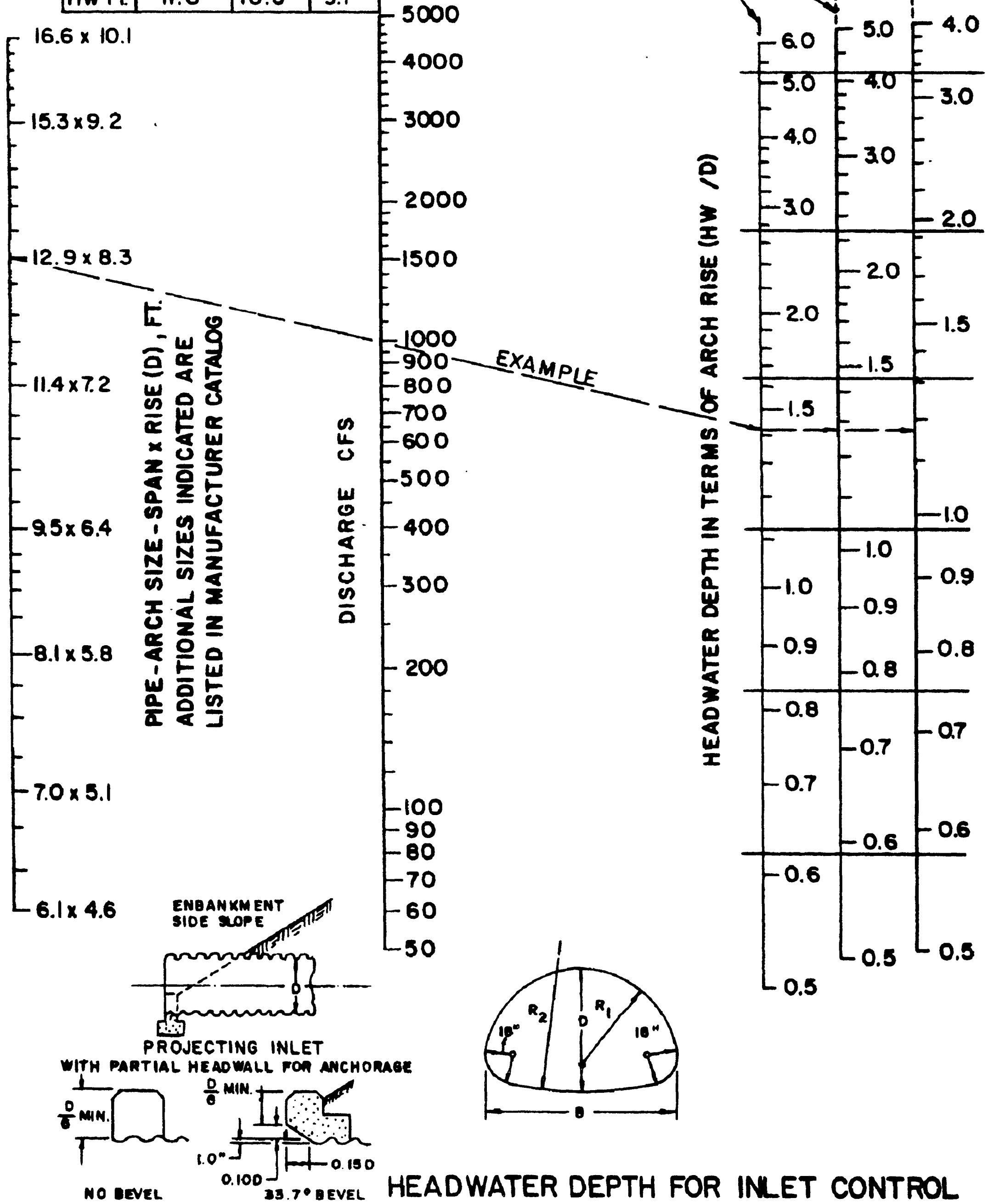
BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

CHART 35

EXAMPLE
SIZE 12.9' x 8.3' Q=1000 CFS

	PROJECT	HEADWALL NO BEV.	HEADWALL BEVEL
HW / D	1.42	1.27	1.17
HW Ft.	11.8	10.5	9.7



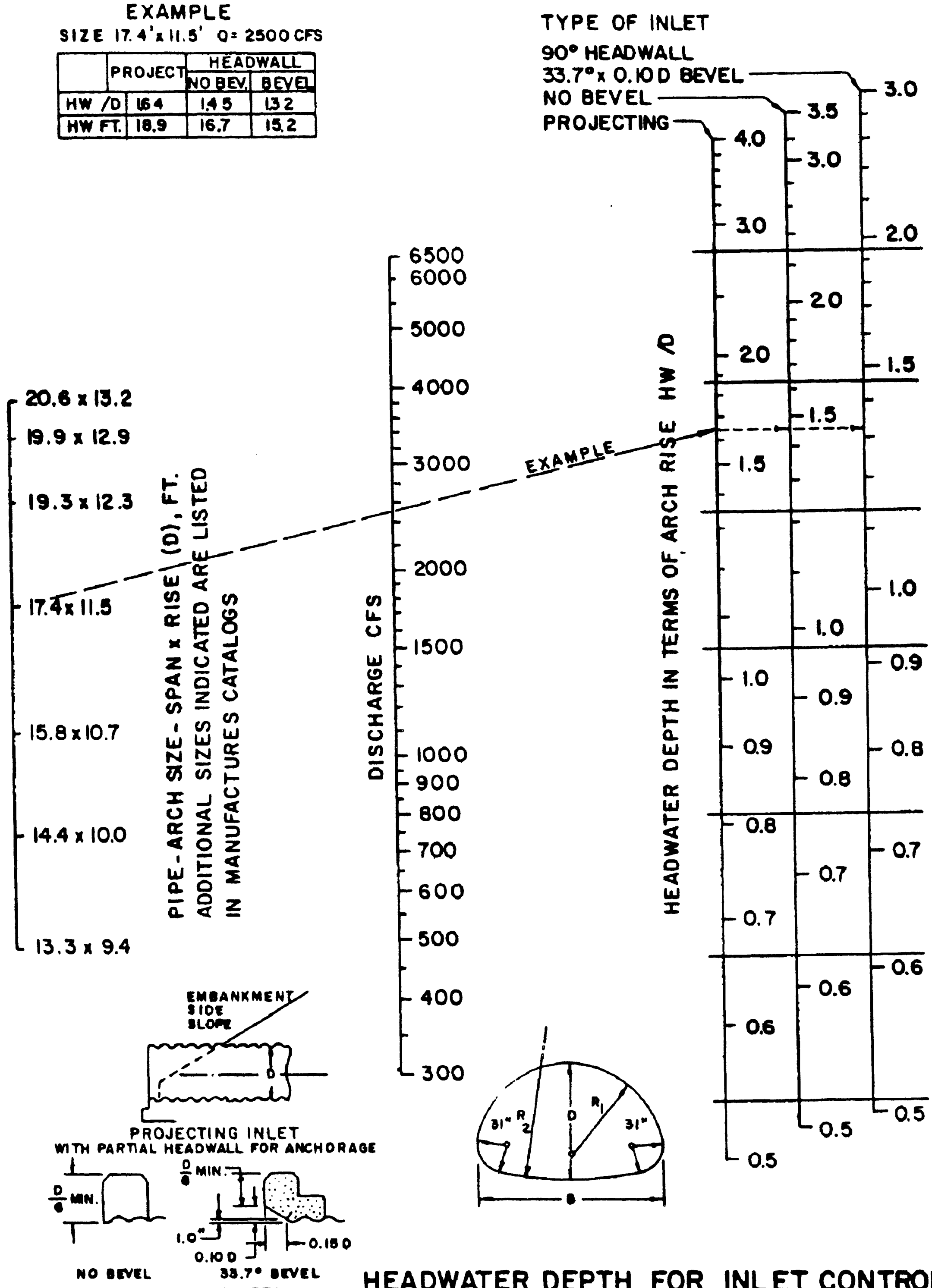
BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1968

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

CHART 36

EXAMPLE
SIZE 17.4'x 11.5' Q= 2500 CFS

	PROJECT	HEADWALL NO BEV.	HEADWALL BEVEL
HW /D	1.64	1.45	1.32
HW FT.	18.9	16.7	15.2

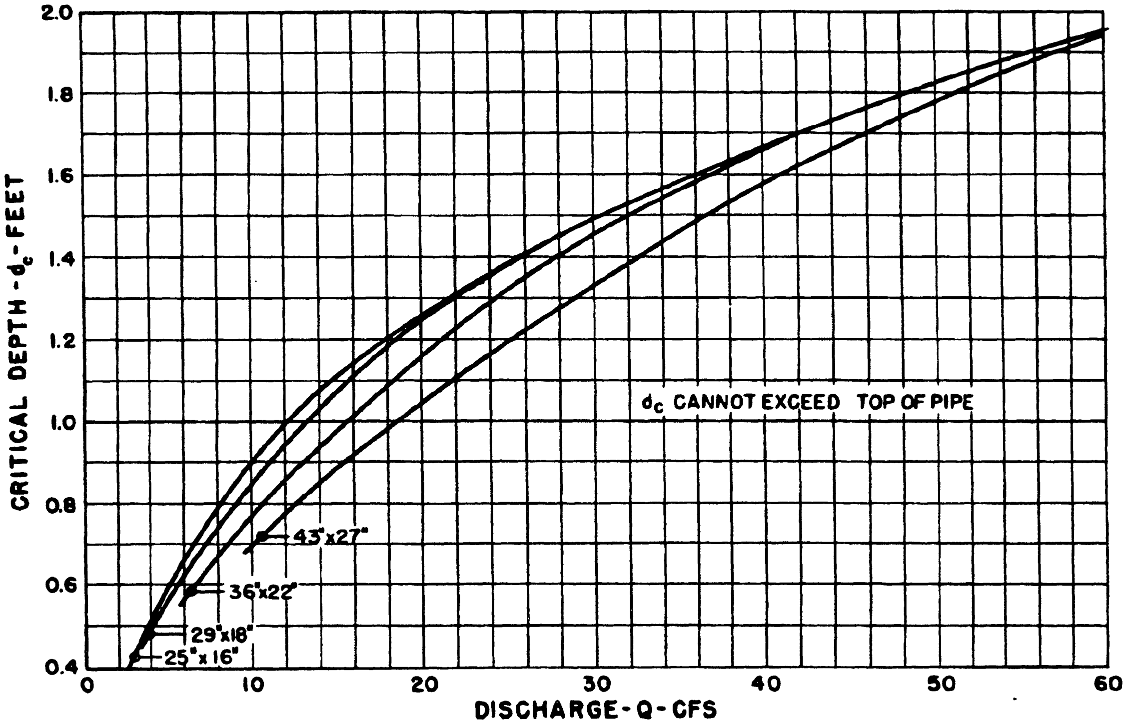


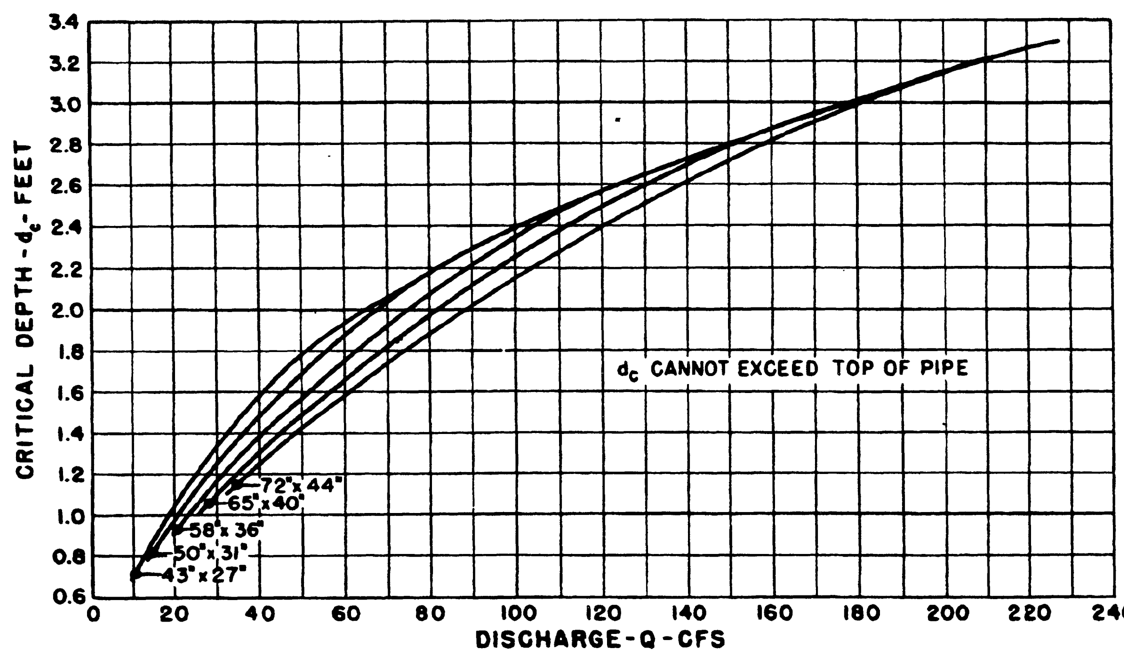
BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1968

HEADWATER DEPTH FOR INLET CONTROL STRUCTURAL PLATE PIPE-ARCH CULVERTS

31-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

CHART 37





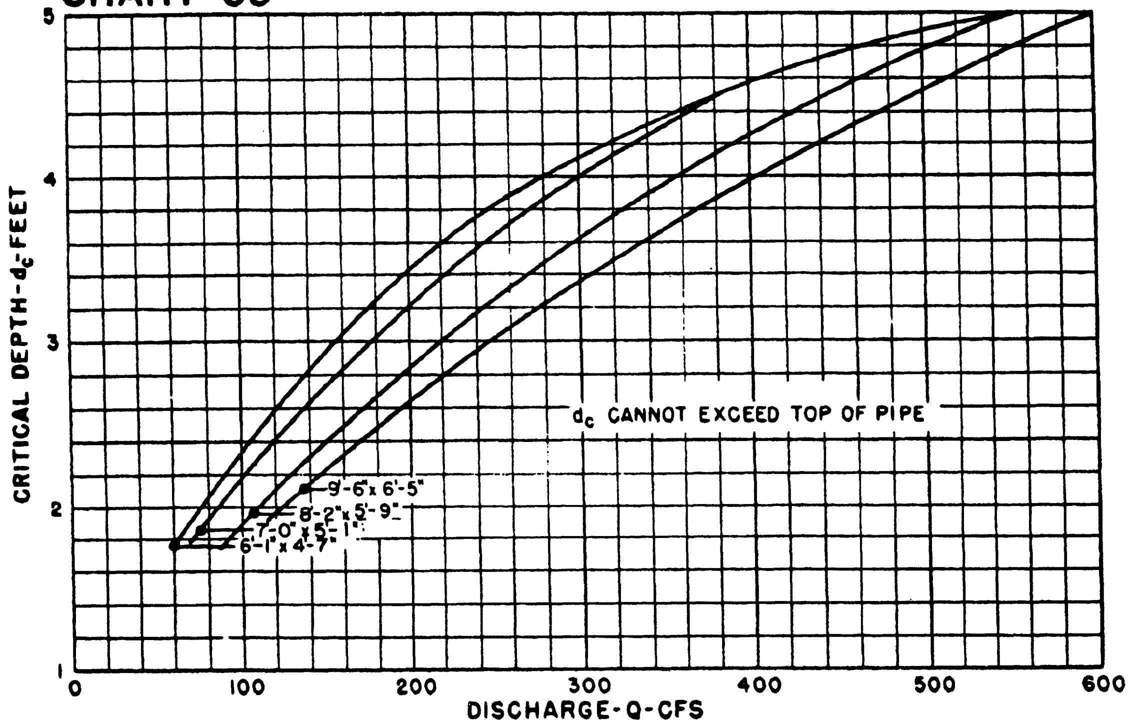
BUREAU OF PUBLIC ROADS

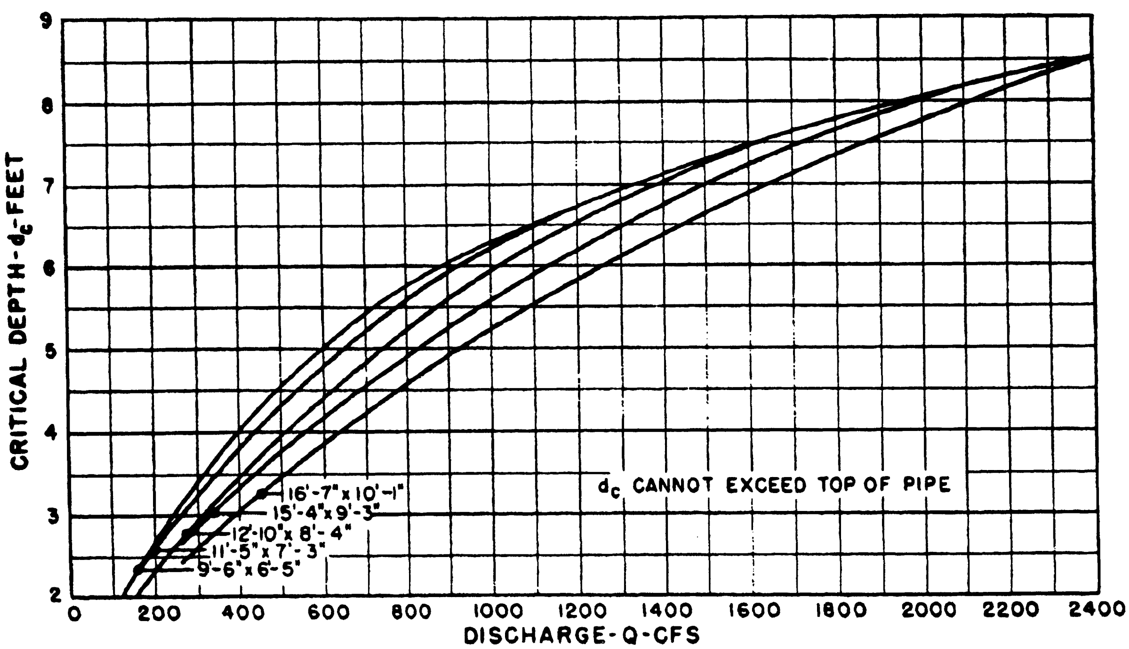
JAN. 1964

CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

Chart 37 can be used to approximate critical depth for comparably sized RCP-ARCH pipe

CHART 38





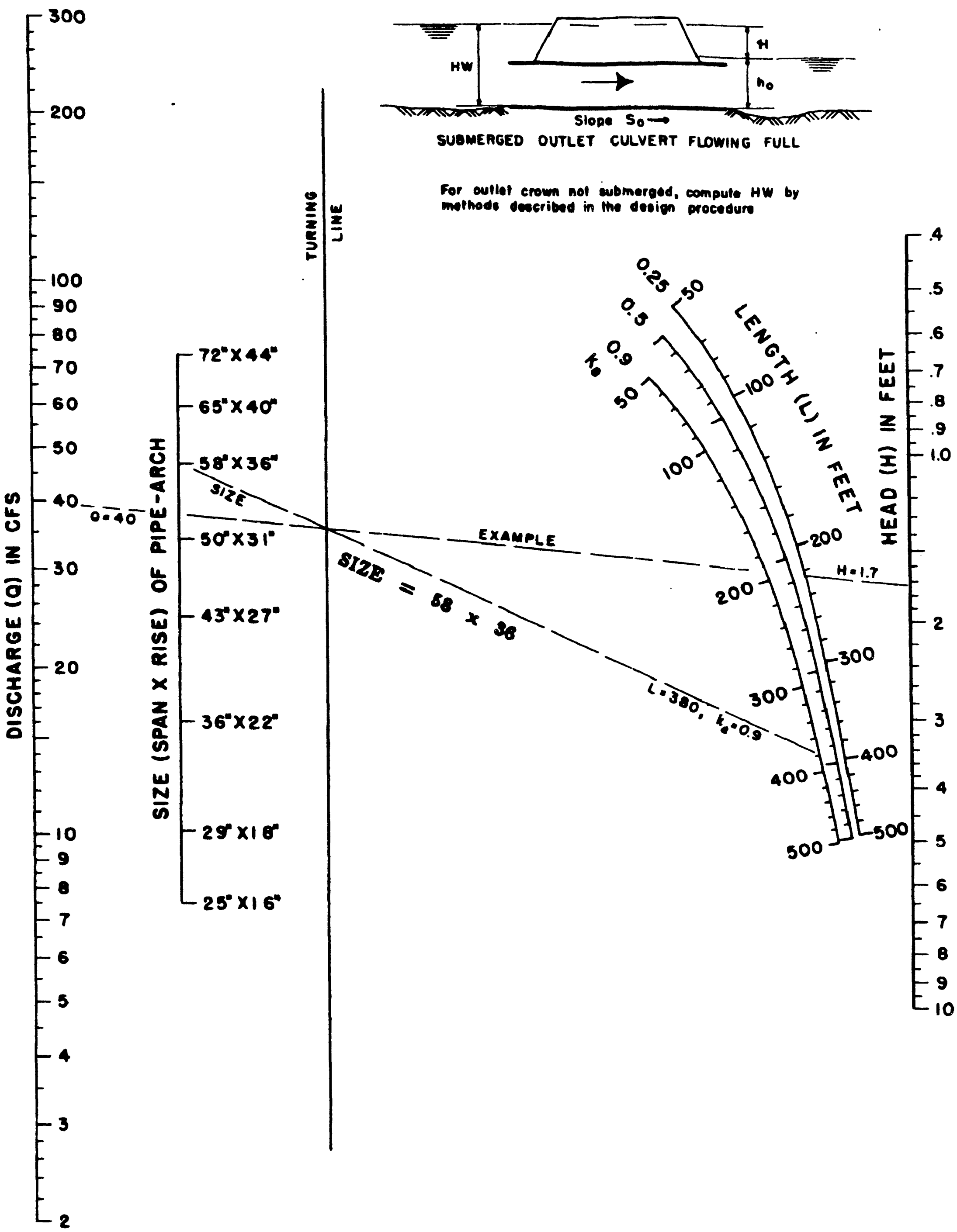
BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS

Chart 38 can be used to approximate critical depth for comparably sized RCP-ARCH pipe

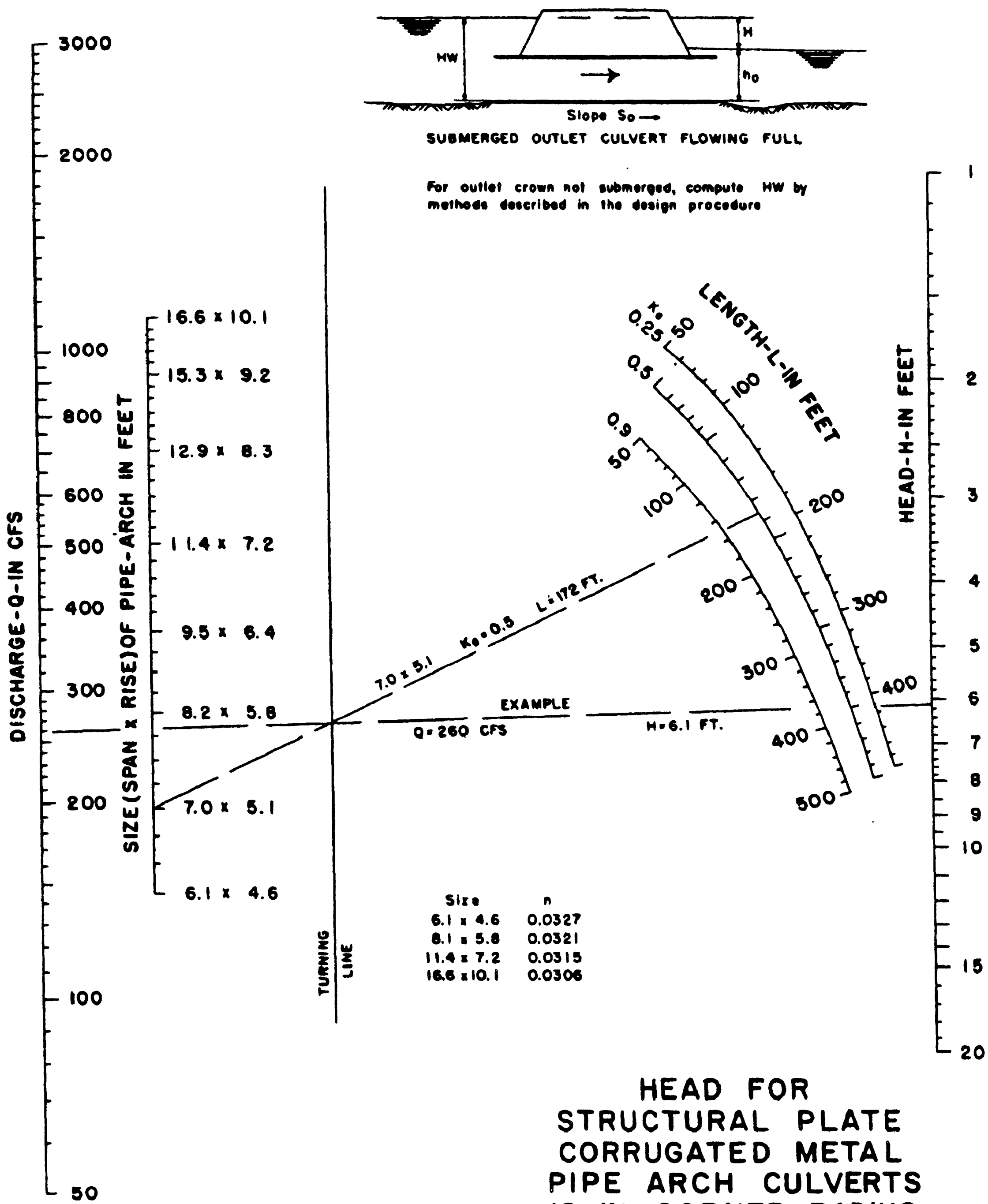
CHART 39



HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL $n = 0.024$

BUREAU OF PUBLIC ROADS JAN. 1963

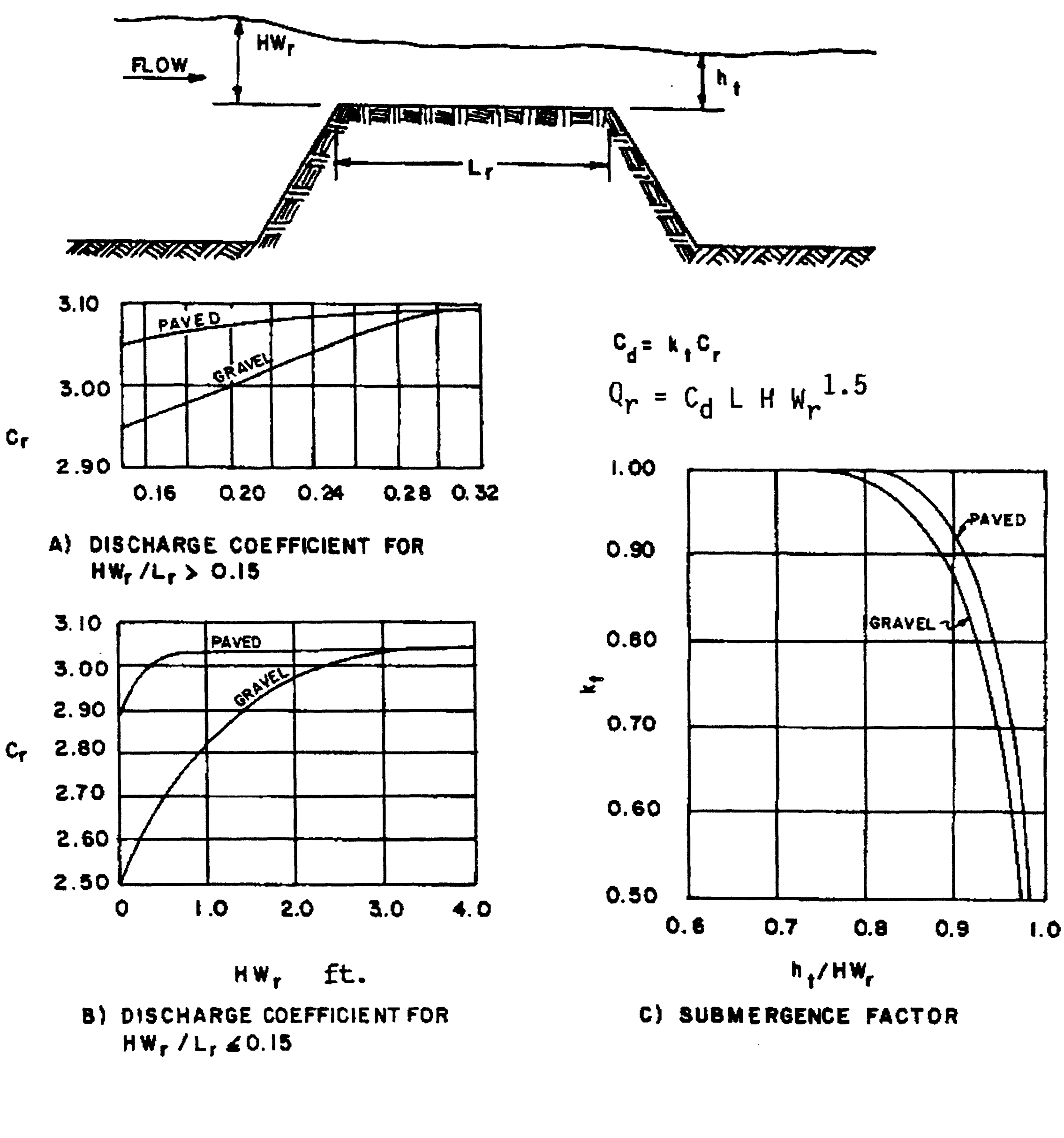
CHART 40



HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n=0.0327 TO 0.0306

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 60



DISCHARGE COEFFICIENTS
FOR ROADWAY OVERTOPPING

Appendix D OPEN CHANNEL FLOW CHARTS

HDS-3, *Design Charts for Open Channel Flow* (FHWA, 1961) contains pipe design charts. Some of these charts have been reproduced in this appendix. An index of the design charts provided is given below, not all charts included in the FHWA manual are reproduced in this document.

This publication contains charts which provide direct solution of the Manning equation for uniform flow in open prismatic channels of various cross sections. The charts fall into two major groups: The first group, Nos. 1-51, consists of separate charts for various size channels of a given shape, with all functions on each chart; the second group, Nos. 52-82, has charts covering a wide range of sizes but with only one or two functions on each chart.

The open-channel flow charts in the first group give a direct and rapid determination of normal depth and normal velocity of flow in a channel of given cross section, roughness, and slope, carrying a known discharge. Values can be read to two significant figures, which is sufficiently accurate for ordinary design purposes. While the open-channel flow charts were drawn for a specific value of "n", they can also be used for any other value of "n" by following the instructions given. For circular sections, two other "n" values are provided by additional scales.

The second group of charts, Nos. 52-82, requires only five charts to cover the hydraulic functions of a wide range of sizes of channels of a given shape and roughness. They have some small disadvantage in that normal depth must be determined by three steps, involving two charts and a simple calculation. Determination of friction slope in part-full flow also requires three similar steps. On the other hand, critical depth, critical slope, and specific head at critical depth may be read directly from these charts, and probably more accurately, than from the open-channel flow charts. The latter actually give only critical depth, critical slope, and critical velocity but require computation of velocity head to obtain specific head at critical depth.

The designer is cautioned not to use the open-channel flow charts presented in this publication as a means of estimating the size of culvert required for a given discharge because the hydraulics of culverts is not simply uniform flow at normal depth. The head required to get flow into a culvert may be several times the head required to maintain uniform flow. Other publications proposed for the Bureau of Public Roads hydraulic design series will deal with the hydraulic design of culverts.

Charts 35-60 are designed for use in the solution of the Manning equation for circular-pipe channels which have sufficient length, on constant slope, to establish uniform flow at normal depth without backwater or pressure head. It is important to recognize that they are not suitable for use in connection with most types of culvert flow, since culvert flow is seldom uniform.

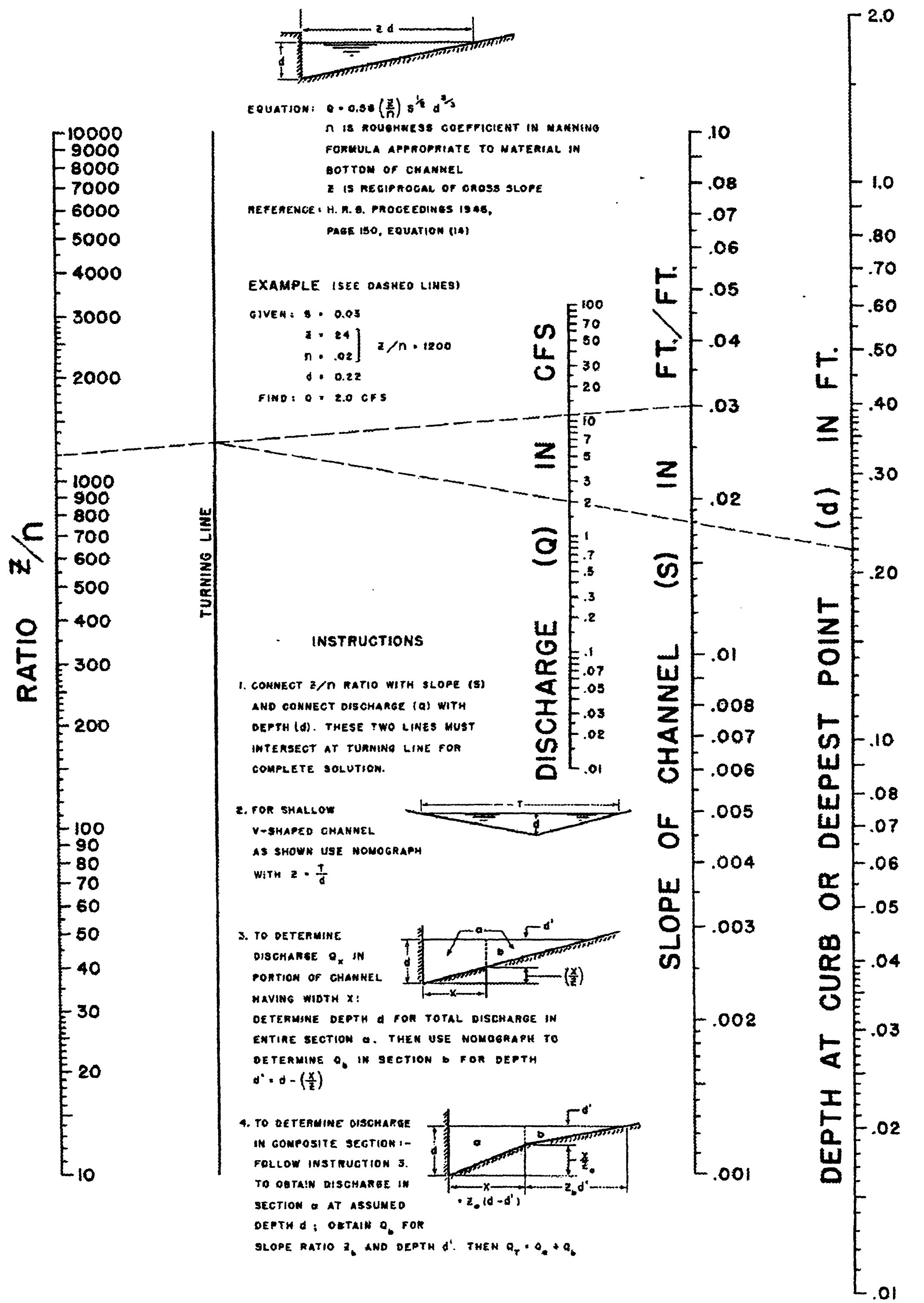
Charts 35-51 cover pipe sizes from 12 to 96 inches in diameter. It will be noted that each slope line has a hook at its right terminus. If d_n is greater than 0.82 diameter, two values of d_n will be shown by the slope line hook for a particular value of Q. In these cases, flow will occur at the lesser of the alternate depths. Interpolated slope lines follow the same pattern as those drawn on the charts. The maximum rate of uniform discharge in a circular pipe on a given slope, when not flowing under pressure, will occur with a depth of 0.94 diameter. This discharge can be determined by reading the highest Q, on the appropriate n scale, which can be read on the given slope line.

For a given discharge, slope, and pipe size, the depth and velocity of uniform flow may be read directly from the chart for that size pipe The initial step is to locate the intersection of a vertical line through the discharge (on the appropriate n scale) and the appropriate slope line. At this intersection, the depth of flow is read or interpolated from the depth lines; and the mean velocity is read opposite the intersection on the velocity scale for the n value of the pipe. The procedure is reversed to determine the discharge at a given depth of flow. If the discharge line passes to the right of the appropriate slope line, the pipe will flow full.

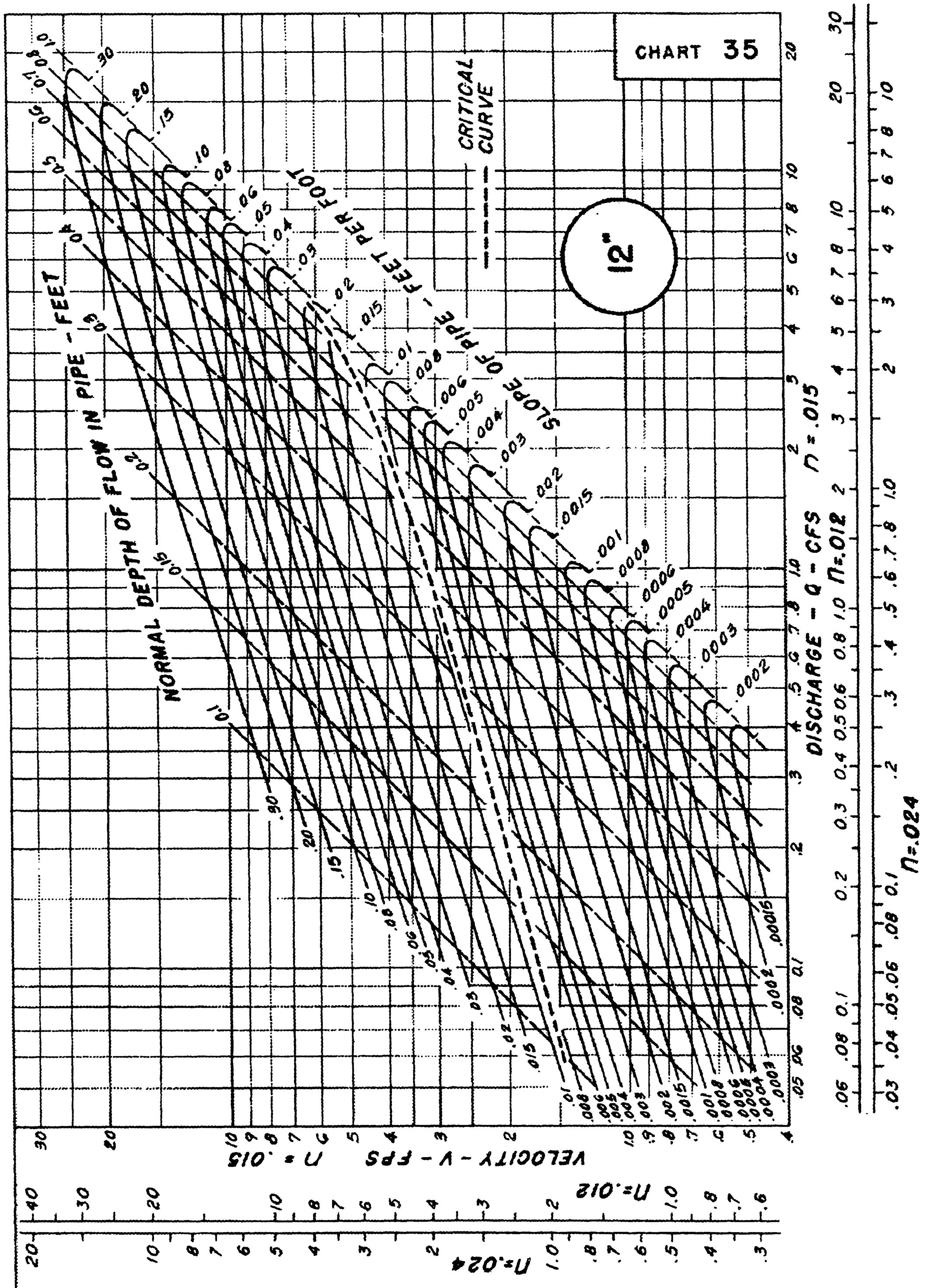
Critical depth and critical velocity are independent of the value of "n". They are read at the point where a vertical line through Q, on the scale $n = 0.015$, intersects the critical curve. Critical slope for $n = 0.015$ is also read or interpolated from the slope line at the same intersection. For "n" values of 0.012 and 0.024, critical slope is determined by first finding critical depth, using Q on the scale $n = 0.015$. Critical slope is then read or interpolated from the slope lines at the intersection of critical depth and the vertical line through Q on the appropriate n scale. Critical depths falling between the last two normal depth lines have little significance, since wave action may intermittently fill the pipe.

Chart Number	Type of Chart	Size or Shape
29	Flow Nomograph	Triangular Channels
35	Pipe Flow Chart	12-inch Diameter
36	Pipe Flow Chart	15-inch Diameter
37	Pipe Flow Chart	18-inch Diameter
38	Pipe Flow Chart	21-inch Diameter
39	Pipe Flow Chart	24-inch Diameter
40	Pipe Flow Chart	27-inch Diameter
41	Pipe Flow Chart	30-inch Diameter
42	Pipe Flow Chart	33-inch Diameter
43	Pipe Flow Chart	36-inch Diameter
44	Pipe Flow Chart	42-inch Diameter
45	Pipe Flow Chart	48-inch Diameter
46	Pipe Flow Chart	54-inch Diameter
47	Pipe Flow Chart	60-inch Diameter
48	Pipe Flow Chart	66-inch Diameter
49	Pipe Flow Chart	72-inch Diameter
50	Pipe Flow Chart	84-inch Diameter
51	Pipe Flow Chart	96-inch Diameter
55	Part Full Flow	Circular Pipe
56	Critical Depth	Circular Pipe
83	Manning's Equation	

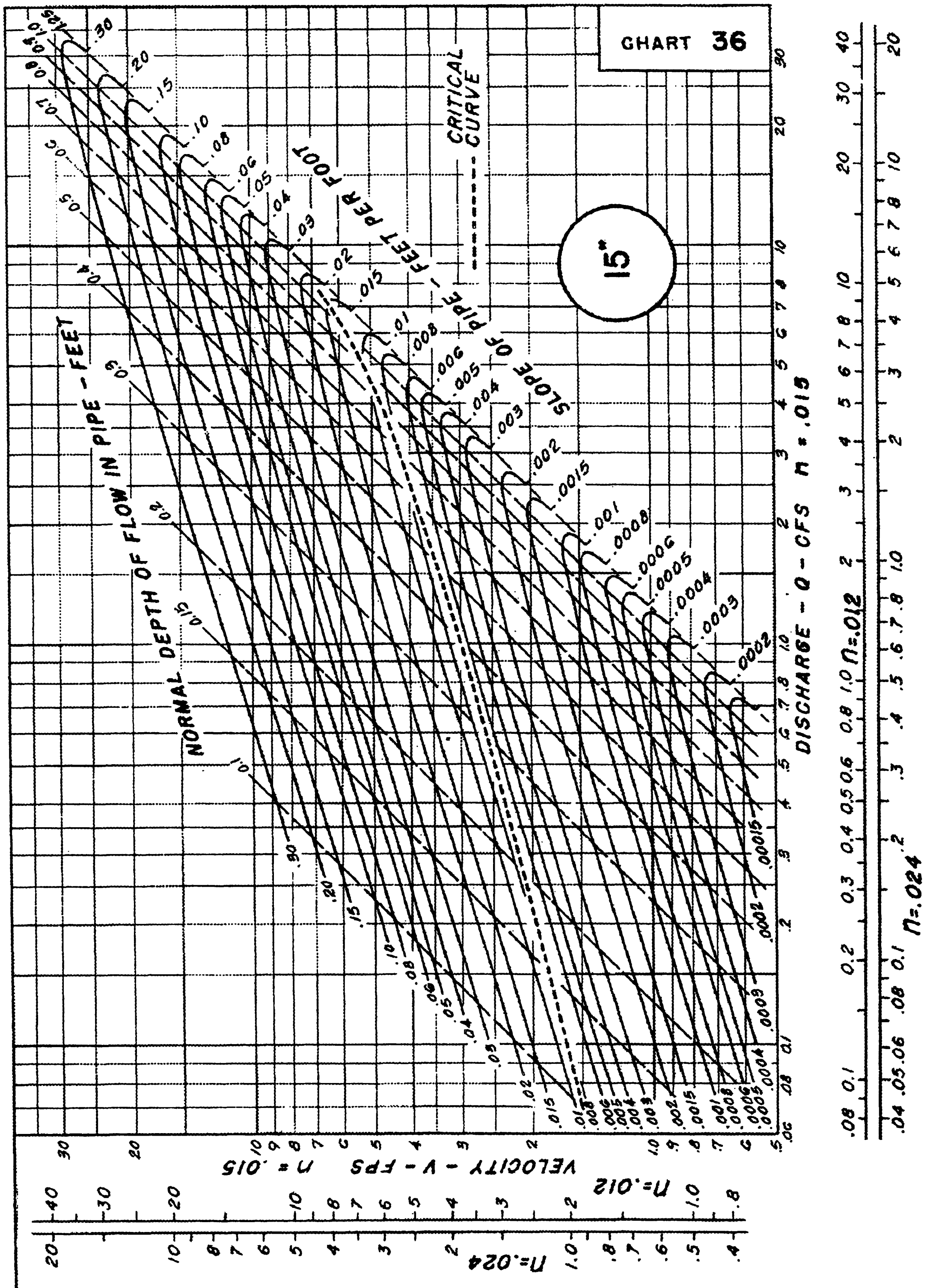
CHART 29



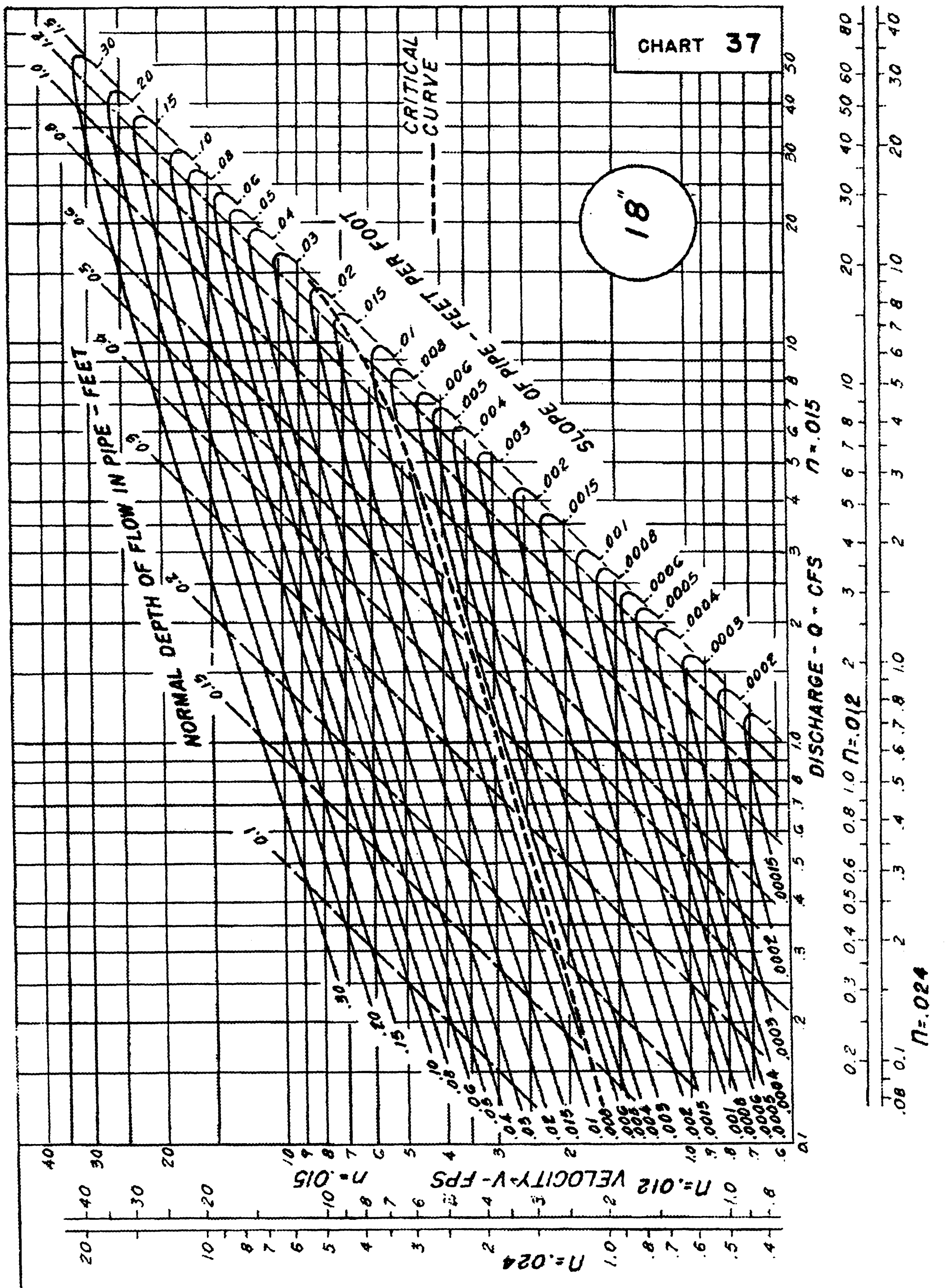
NOMOGRAPH FOR FLOW
IN TRIANGULAR CHANNELS



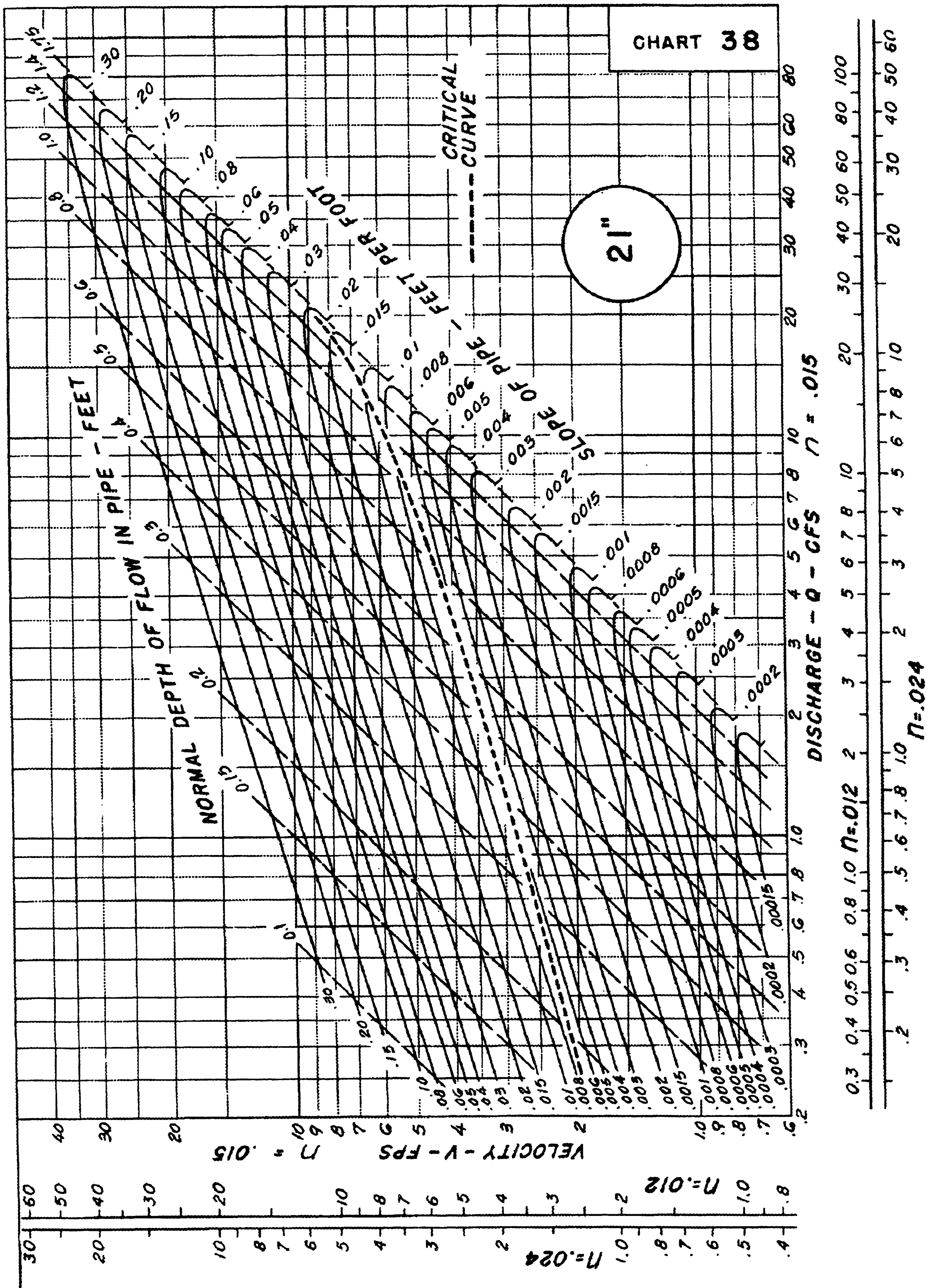
PIPE FLOW CHART
12-INCH DIAMETER



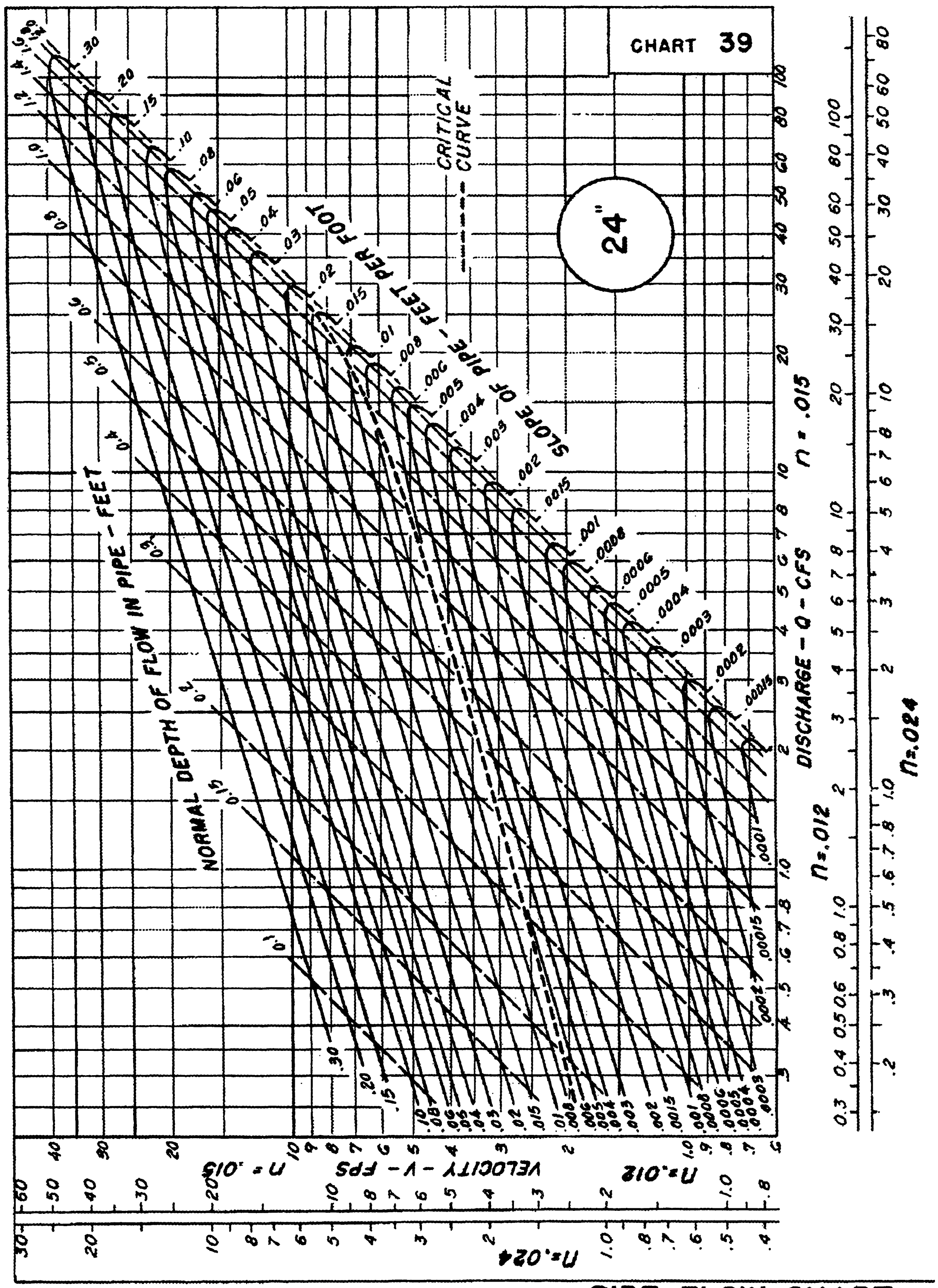
PIPE FLOW CHART
15-INCH DIAMETER



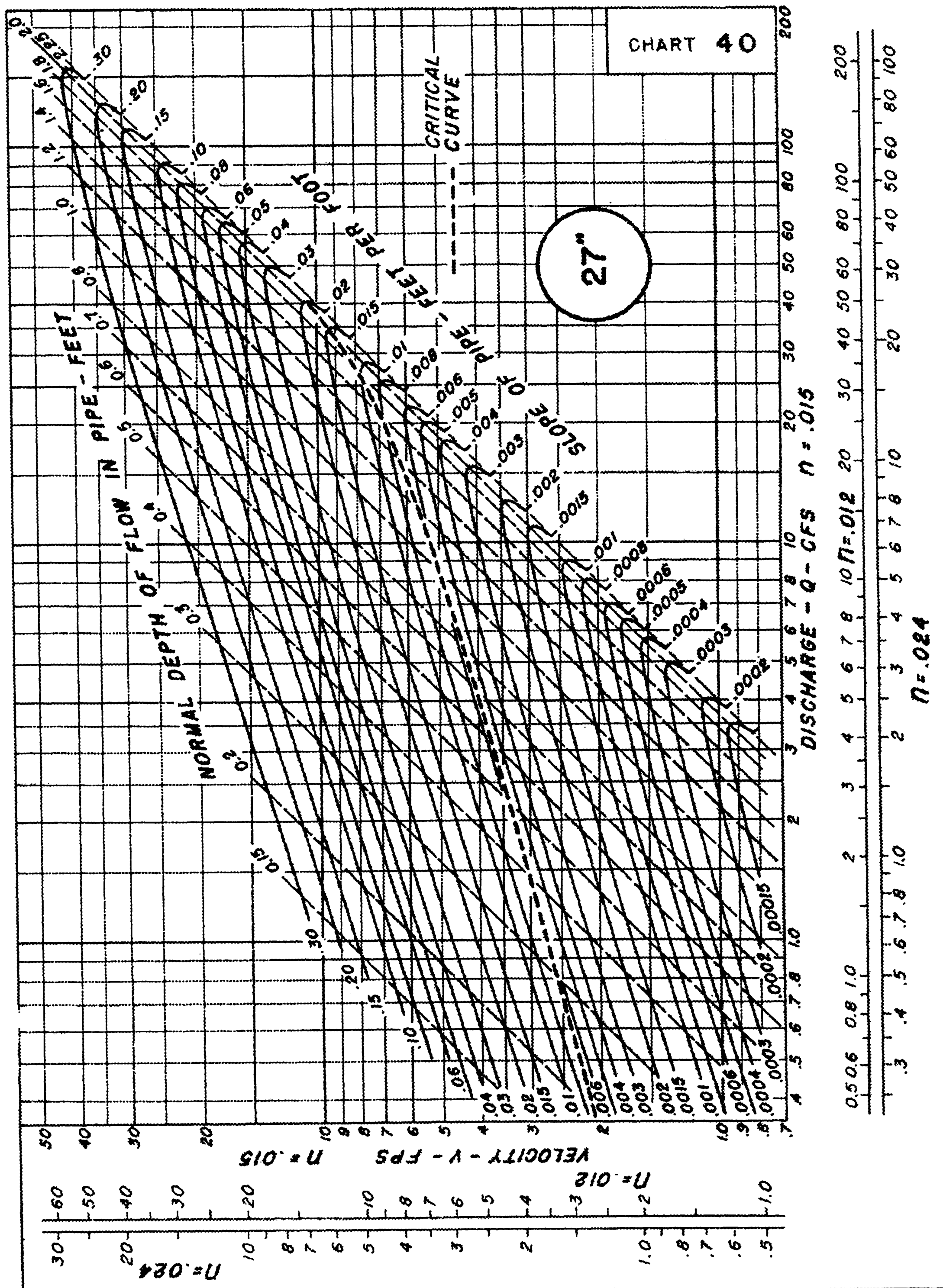
PIPE FLOW CHART
18-INCH DIAMETER



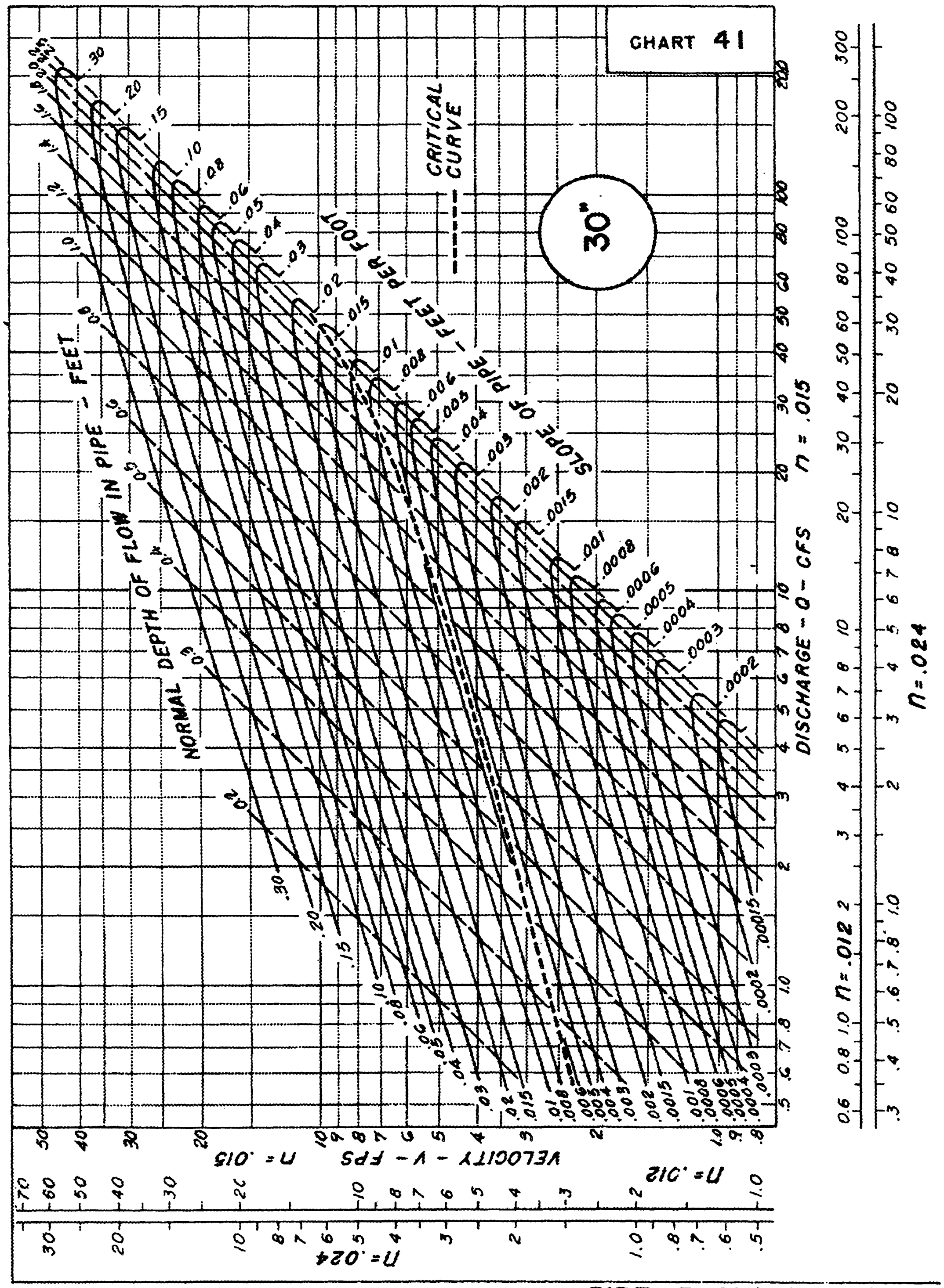
PIPE FLOW CHART
21-INCH DIAMETER



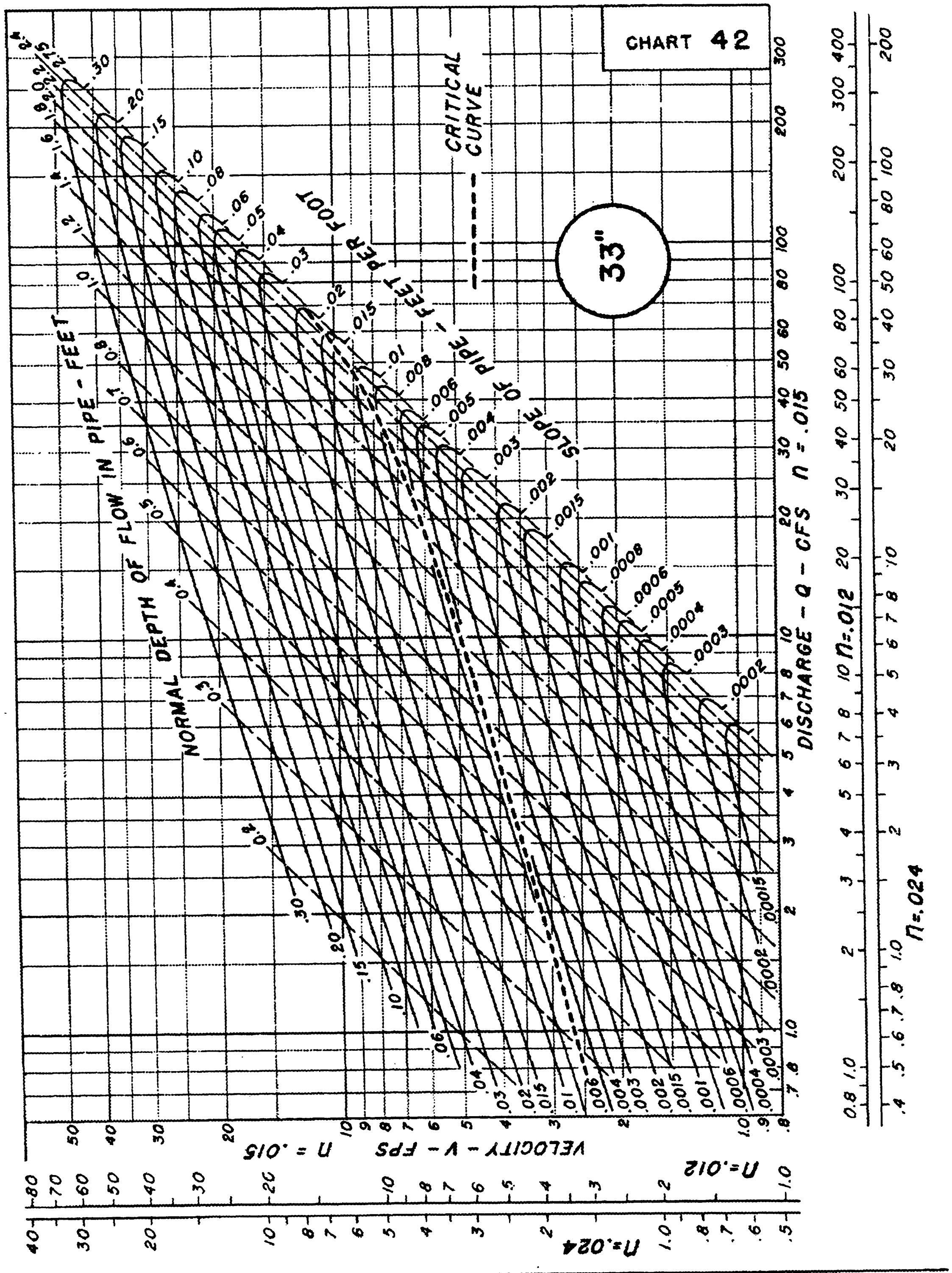
PIPE FLOW CHART
24-INCH DIAMETER



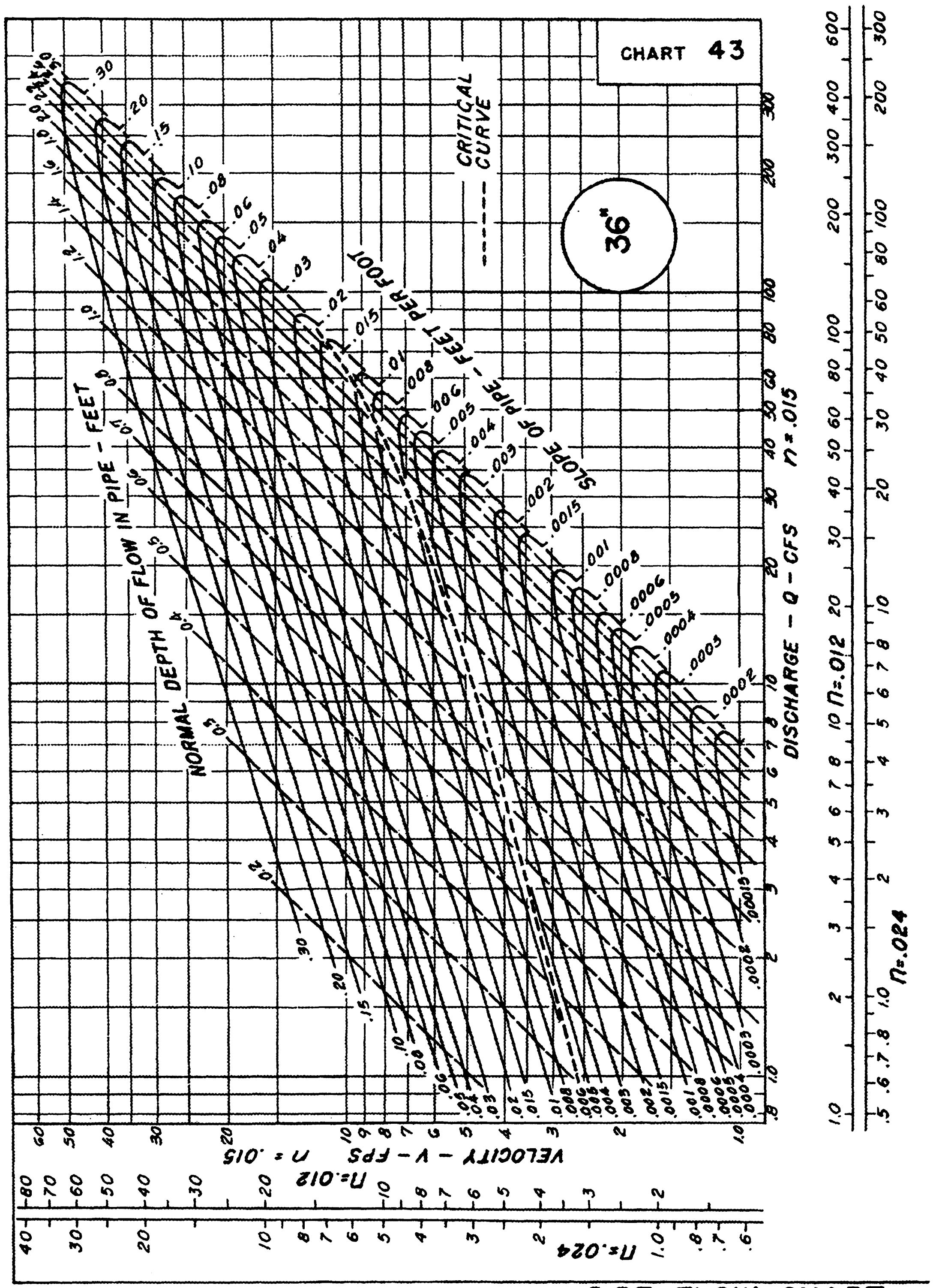
PIPE FLOW CHART
27-INCH DIAMETER



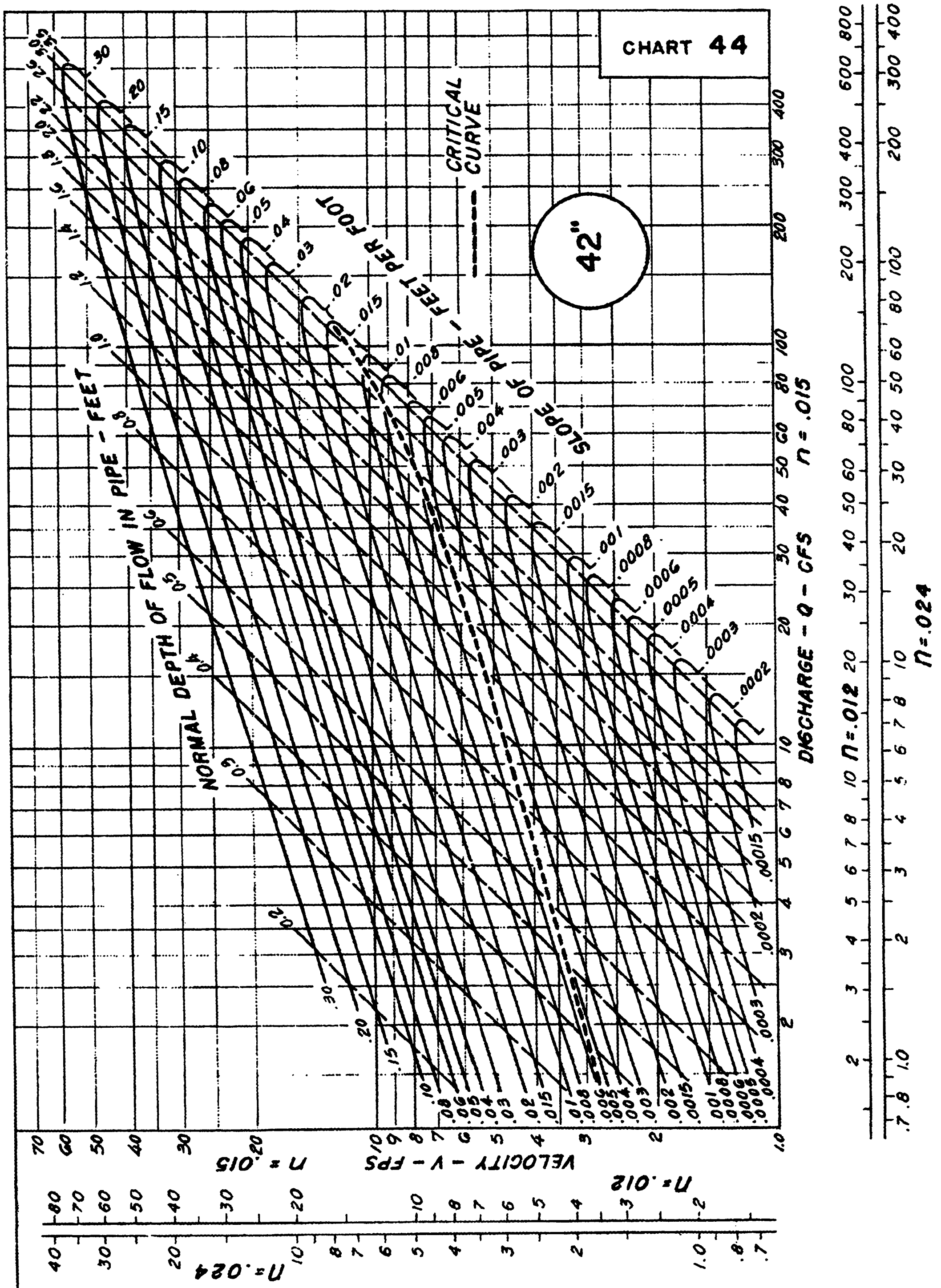
PIPE FLOW CHART
30-INCH DIAMETER



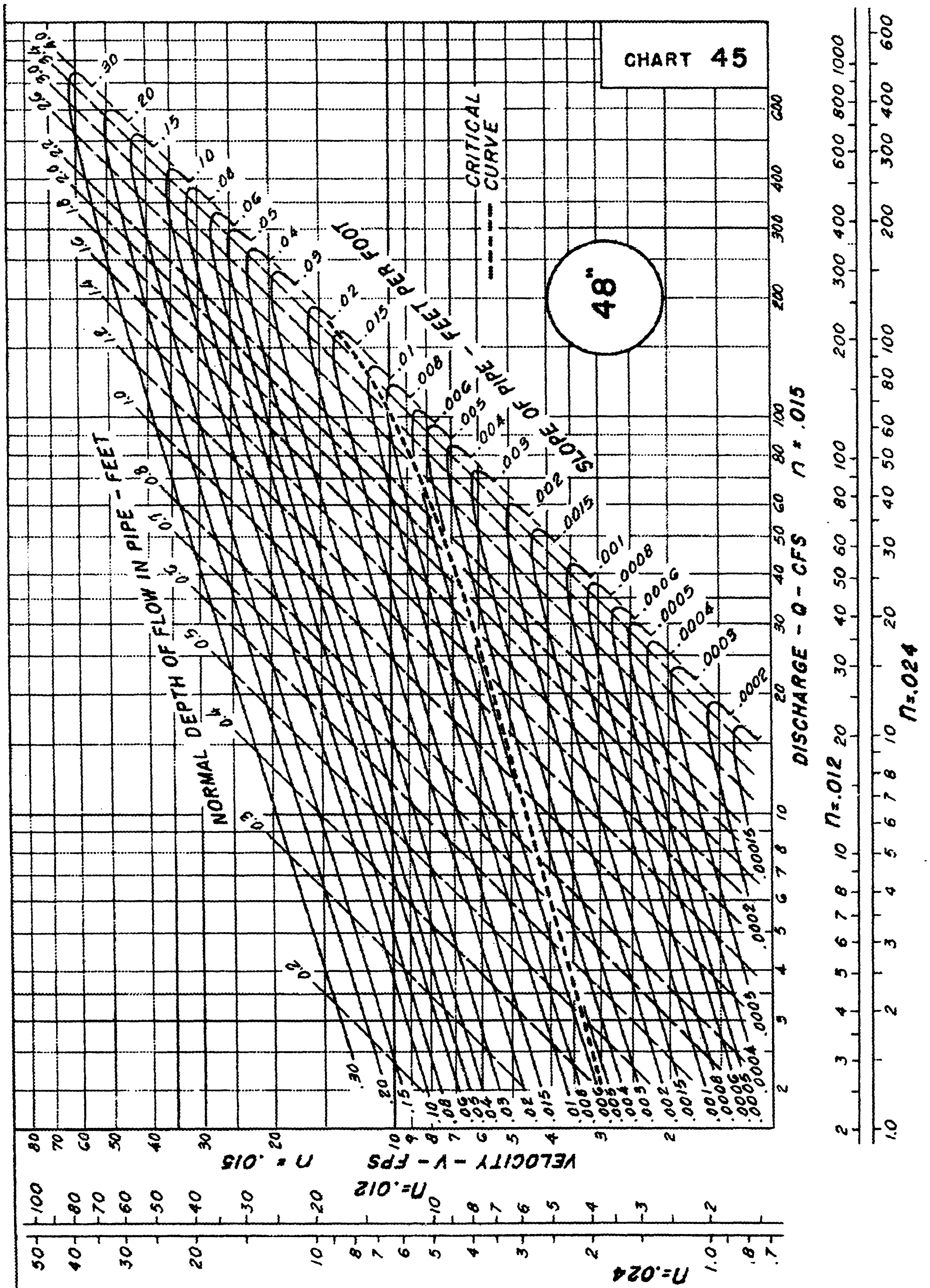
PIPE FLOW CHART
33-INCH DIAMETER



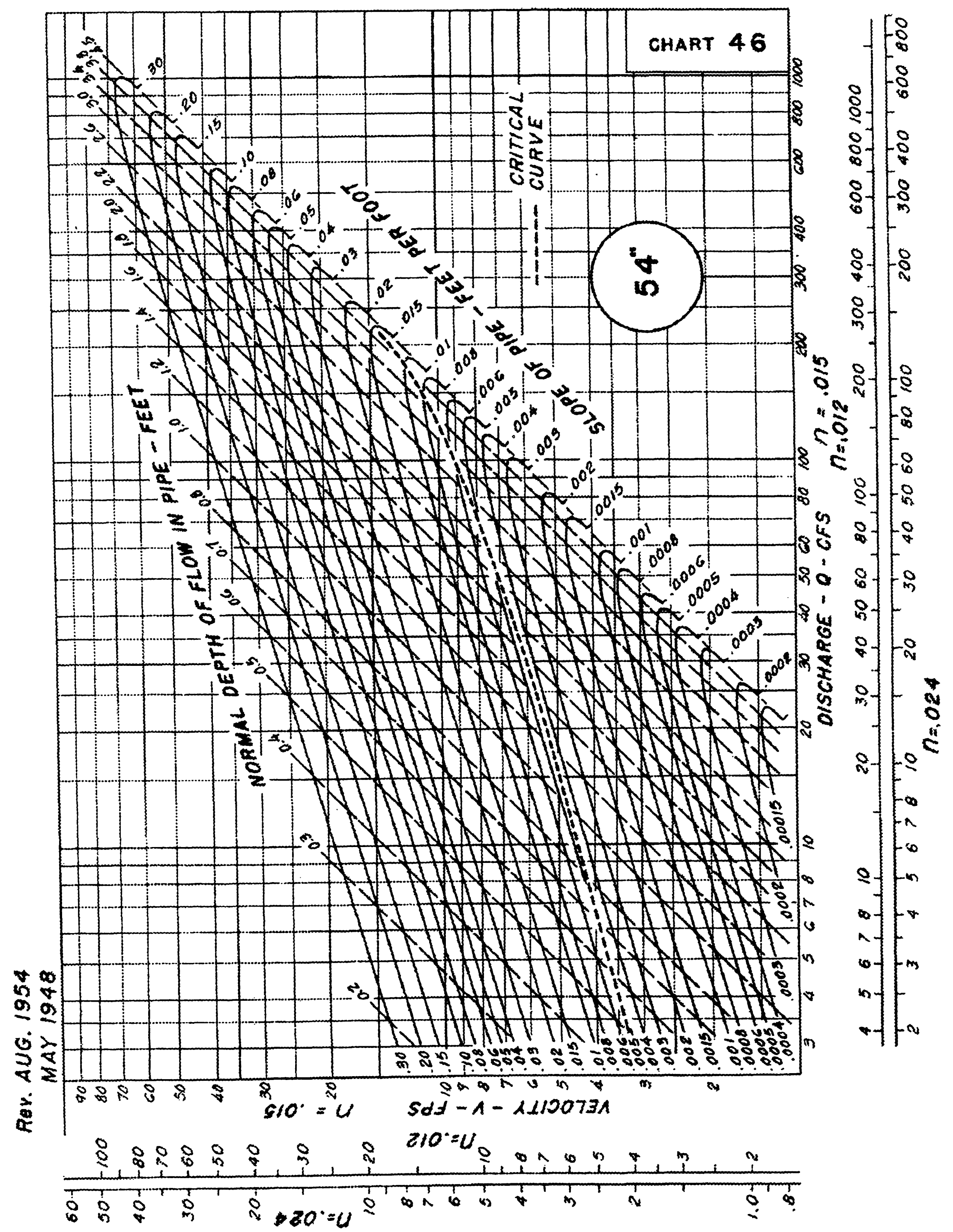
PIPE FLOW CHART
36-INCH DIAMETER



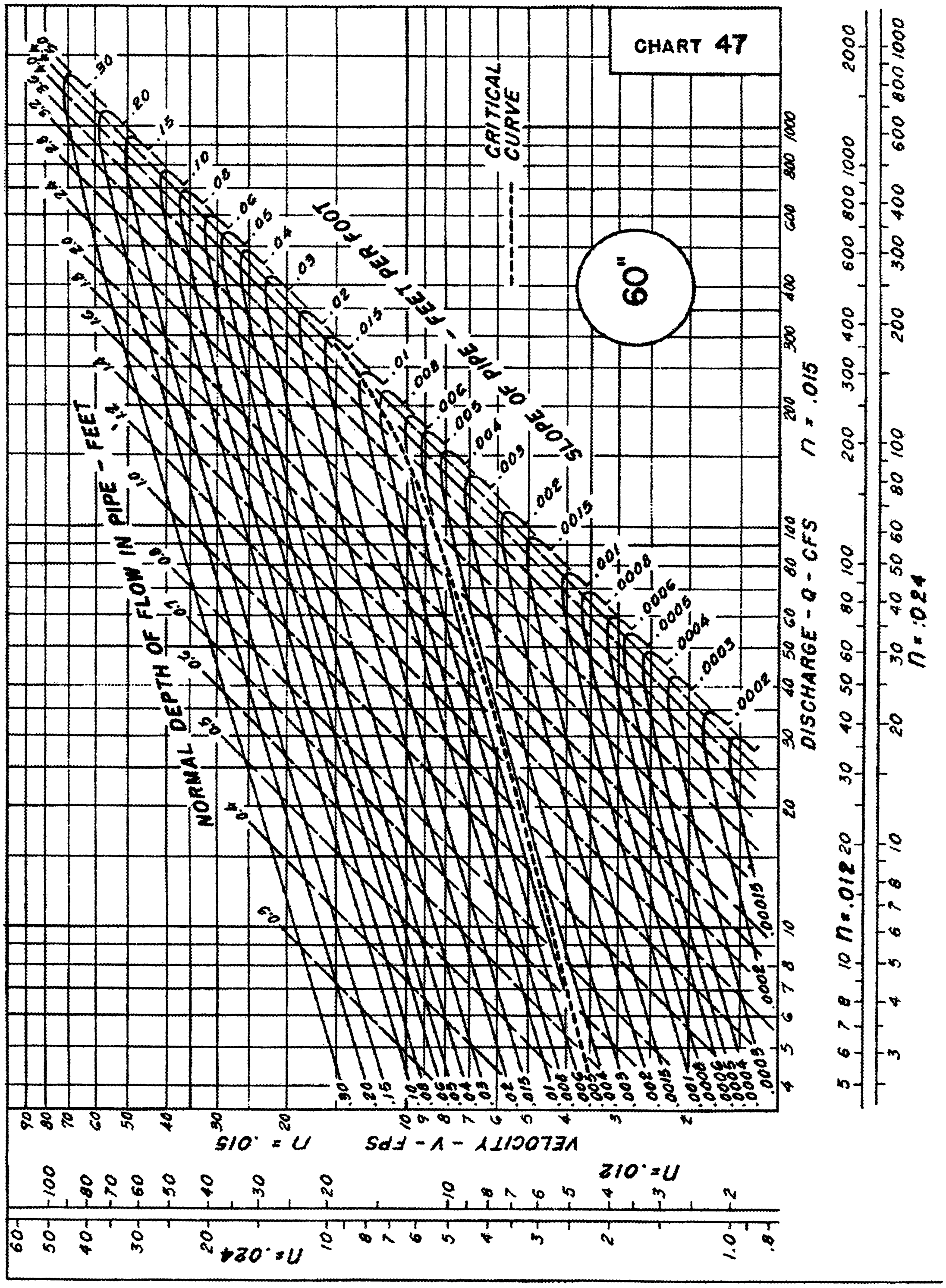
PIPE FLOW CHART
42-INCH DIAMETER



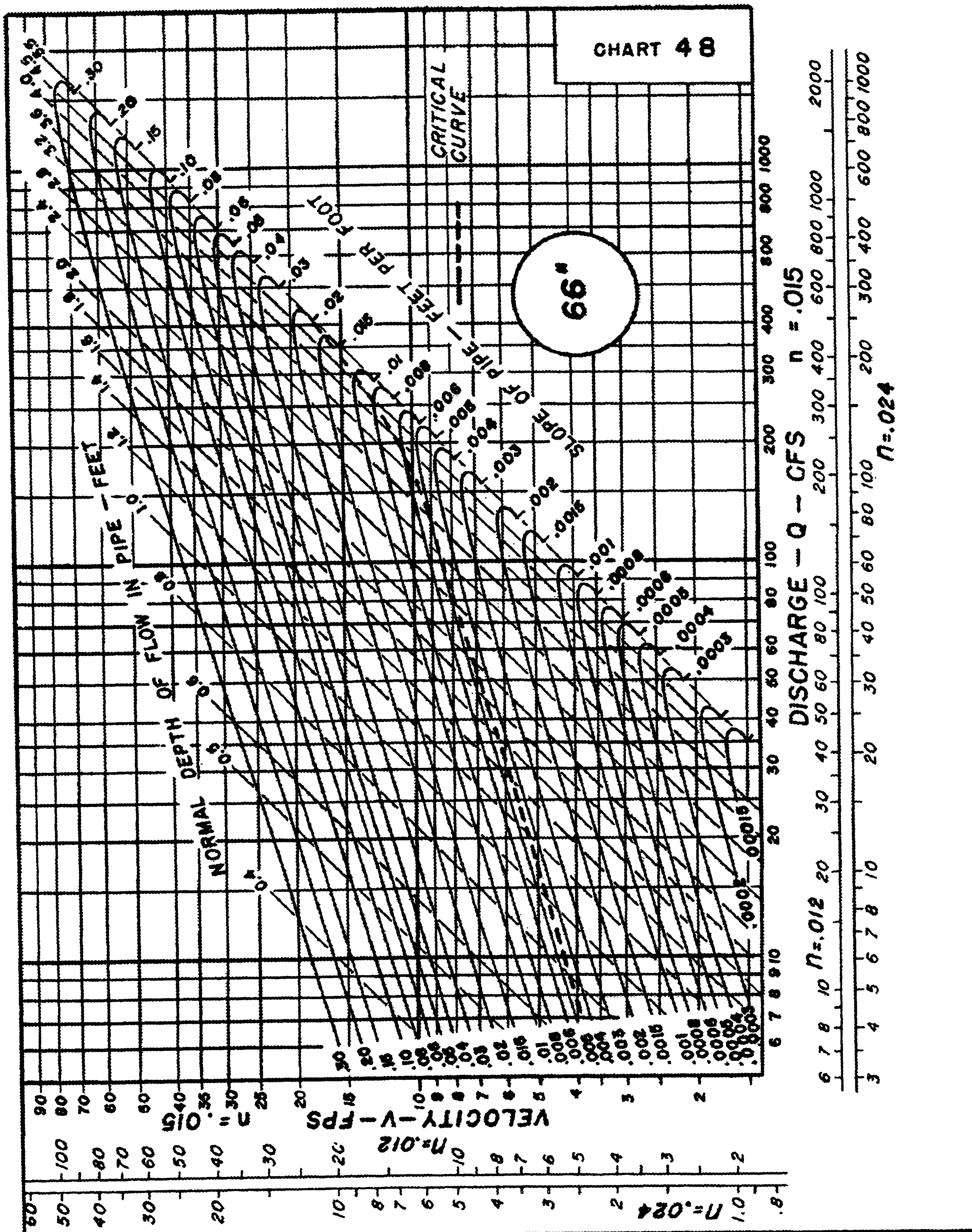
PIPE FLOW CHART
48-INCH DIAMETER



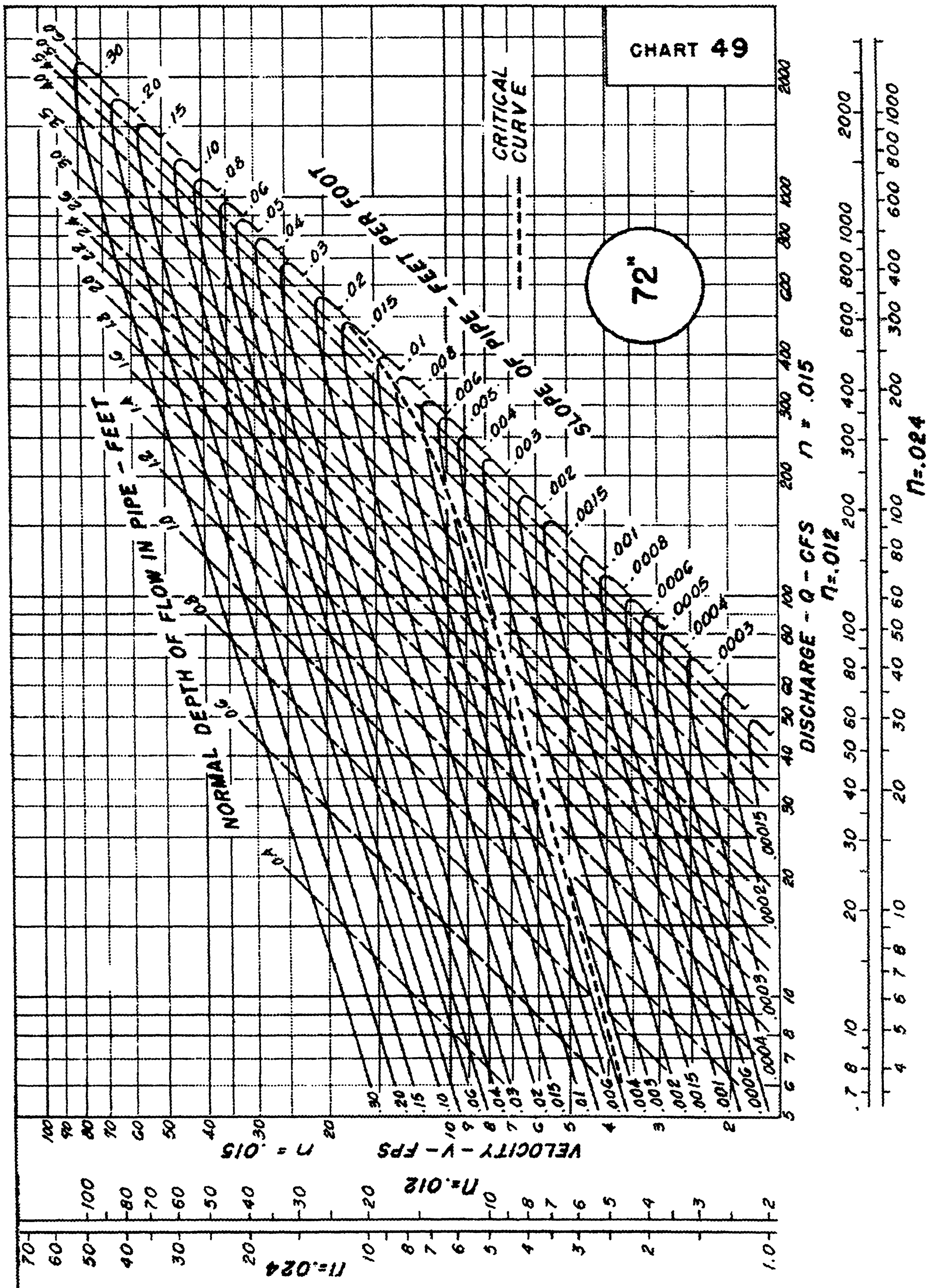
PIPE FLOW CHART
54-INCH DIAMETER



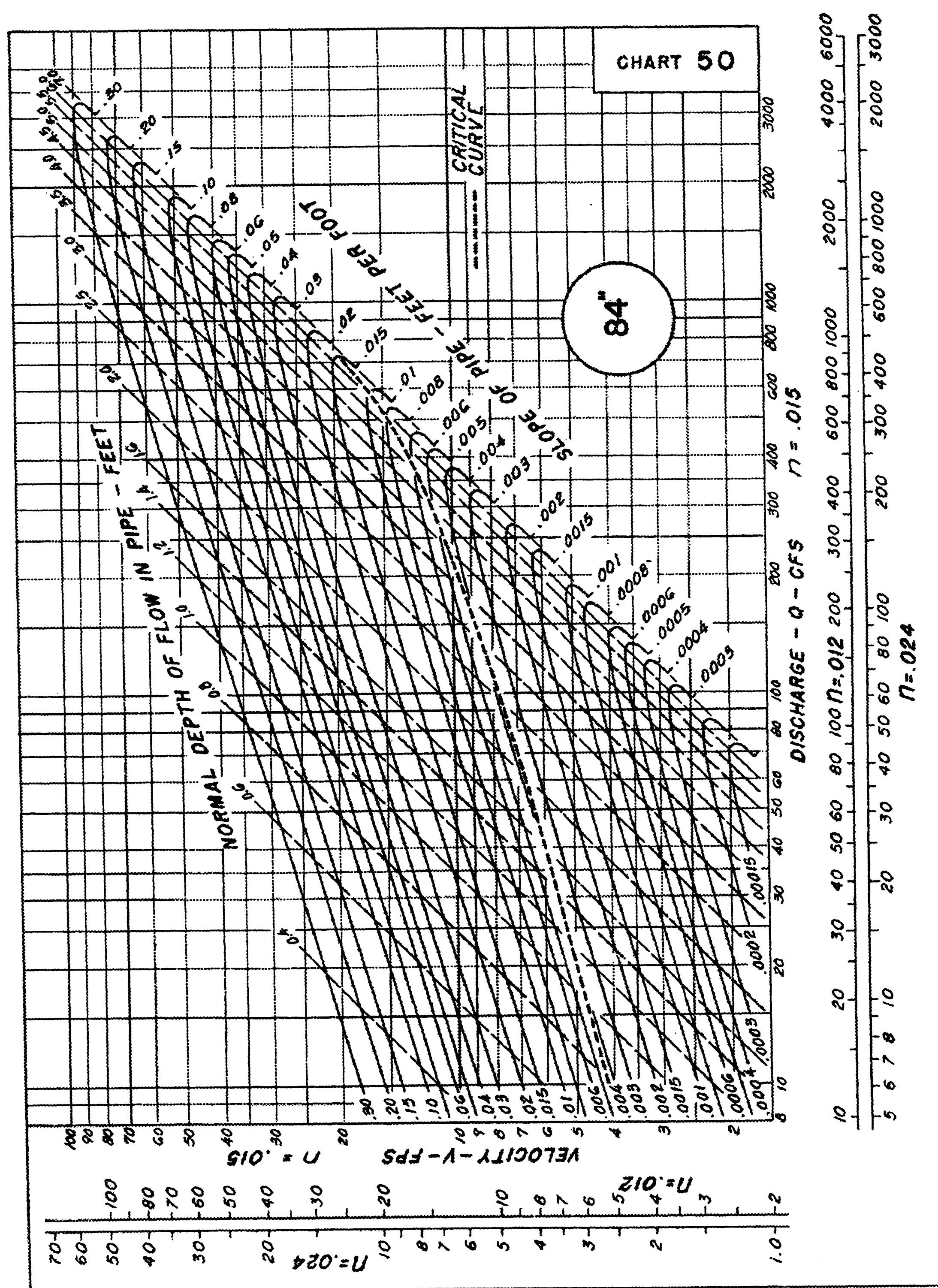
PIPE FLOW CHART
60-INCH DIAMETER



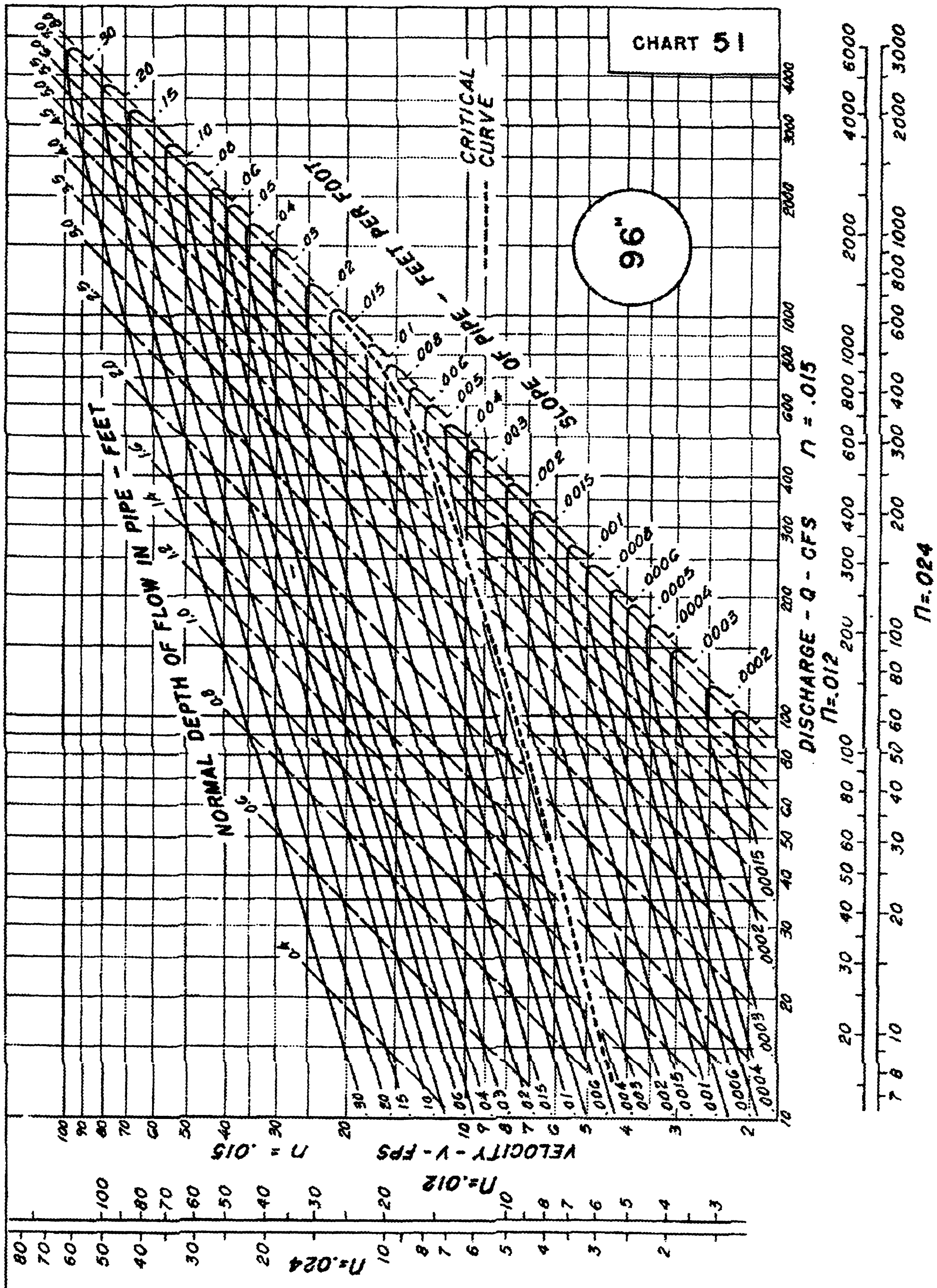
PIPE FLOW CHART
66-INCH DIAMETER



PIPE FLOW CHART
72-INCH DIAMETER

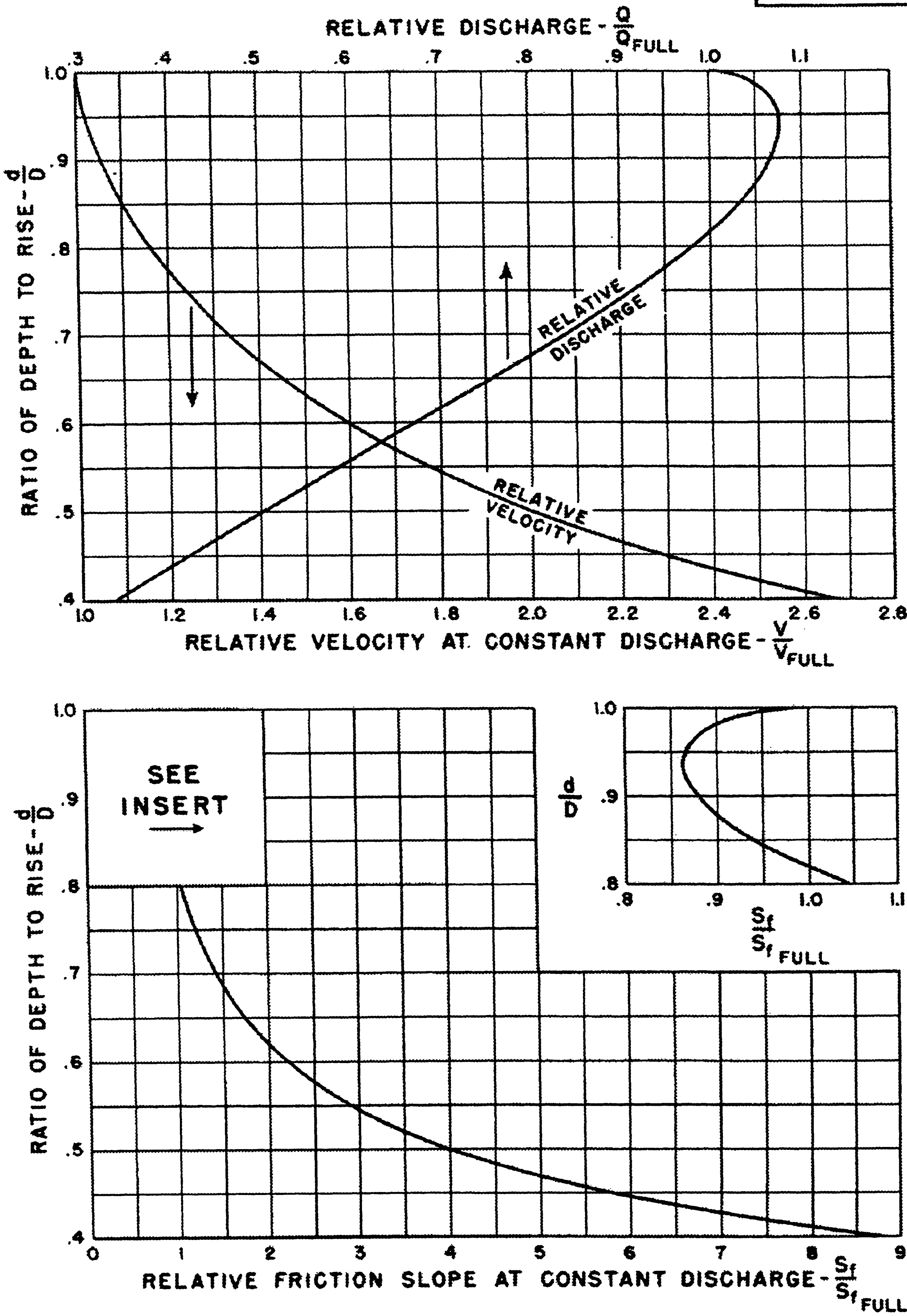


PIPE FLOW CHART
84-INCH DIAMETER

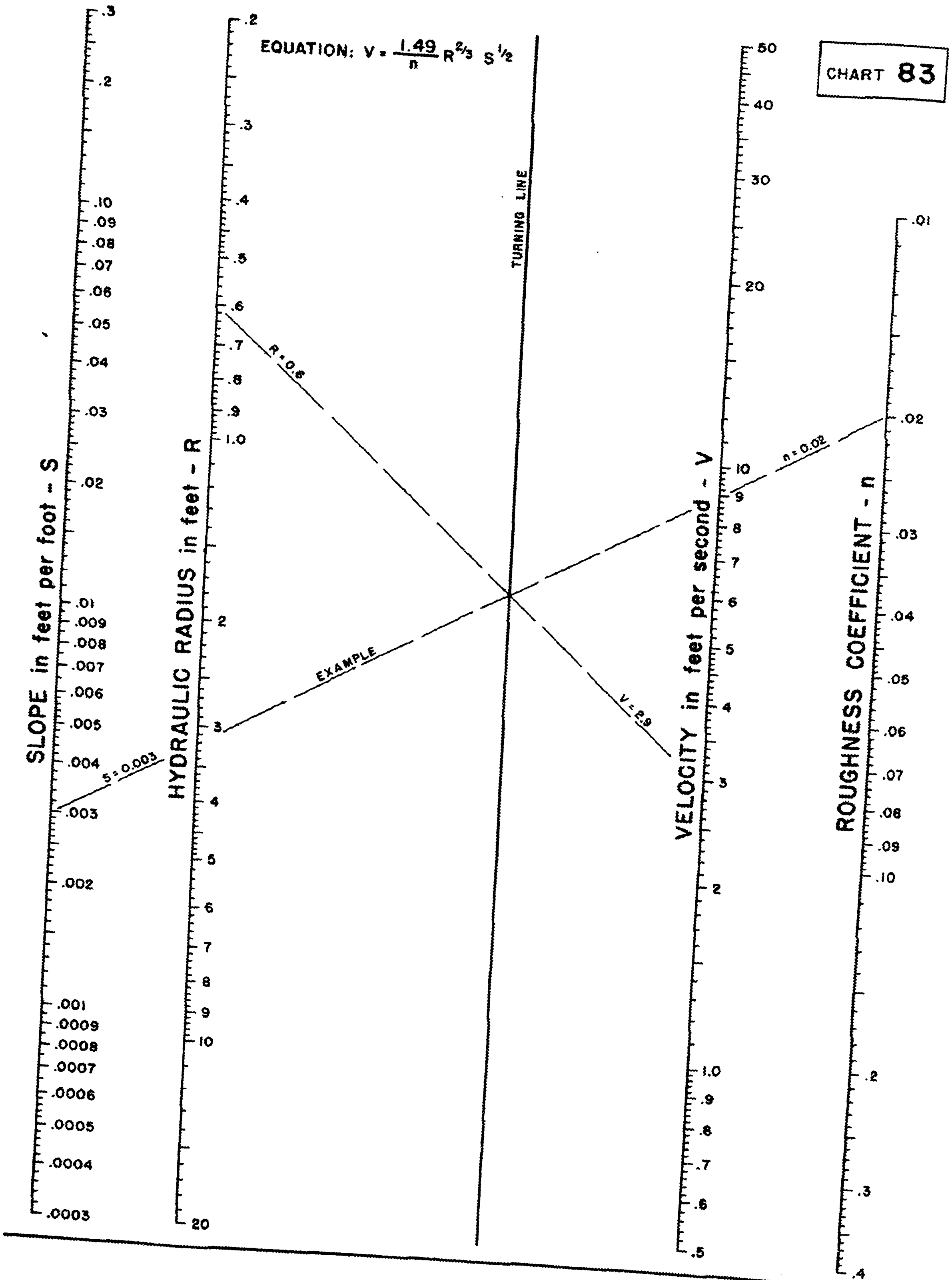


PIPE FLOW CHART
96-INCH DIAMETER

CHART 55



CIRCULAR PIPE
PART FULL FLOW



NOMOGRAPH FOR SOLUTION OF MANNING EQUATION

Appendix E: COMPUTER PROGRAMS AND REFERENCE MATERIALS

E.1 Computer Programs

Numerous software packages are available which provide quick and precise hydrologic and hydraulic analysis of drainage components. This is not a comprehensive listing and other programs may be acceptable. The software summarized in this appendix are primarily public sector programs which incorporate many of the procedures discussed in this manual as well as selected commercial programs that are used within Mn/DOT. For all computer applications, the engineer and/or designer must be knowledgeable regarding the procedures and analysis in order to appropriately select, use and review the results of the application.

The Table E.1 shows the modeling packages being reviewed in this appendix as well as the capabilities of these software packages.

Table E.1 Summary of Related Computer Programs

	Hydrology	Roadside Channels	Water Surface Profiles	Culverts	Pond Routing	Water Quality	Pavement Drainage	Storm Drains
GEOPAK Drainage	•	•		•			•	•
HEC-1	•				•			
HEC-2			•					
HEC-HMS	•				•			
HEC-RAS			•	•				
HY-8				•				
HYDRAIN	•	•	•	•	•		•	•
HydroCAD	•				•			
SMS/ FESWMS-2DH			•					
TR-20	•				•			
TR-55	•							
Urban Drainage		•			•		•	•
WSPRO			•					
XP-SWMM	•				•	•		•

GEOPAK Drainage

GEOPAK Drainage is a commercial hydraulics design package that is integrated with the GEOPAK software that Mn/DOT uses for road design. Drainage can be used to design and analyze surface collection systems and includes functions for Rational method hydrology, inlet design and spread analysis, storm drain pipe sizing and analysis, hydraulic gradeline computation, and culvert design and analysis. GEOPAK Drainage uses MicroStation for the graphical interface. Graphics are created in the design file using MnDOT CADD standards. Options are available to produce graphics and report output.

GEOPAK Drainage is available from: GEOPAK Corporation
1190 N.E. 163rd St.
North Miami Beach, FL 33162
www.geopak.com
305-944-5151

HEC-1

HEC-1 Flood Hydrograph Package was developed by the U.S. Army Corps of Engineers to simulate the surface runoff response of a river basin to precipitation. The drainage basin can be modeled as a group of subareas for which runoff hydrographs can be simulated using options including the NRCS runoff curve number, synthetic storm, and dimensionless unit hydrograph. Computed hydrographs can be routed through stream channel reaches or reservoirs and combined to give discharge hydrographs at multiple locations within the drainage basin. The data is input via a batch file, there is an input program available to help the user assemble the correct sequence of records for the HEC-1 input file.

HEC-1 is available from: U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California 95616
www. hec.usace.army.mil/
530-756-1104

HEC-2

The U.S. Army Corps of Engineers HEC-2, Water Surface Profiles, computes water surface profiles and flood plain boundaries for steady, gradually varied flow in channels. The data is input via a batch file, there is an input program available to help the user assemble the correct sequence of records for the HEC-2 input file.

HEC-2 is available from: U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California 95616
www. hec.usace.army.mil/
530-756-1104

HEC-HMS

HEC-HMS, Hydrologic Modeling System, was developed by the U. S. Army Corps of Engineers to supercede HEC-1. HEC-HMS includes most of the capabilities of HEC-1 with additional capabilities to use a linear distributed-runoff transformation for gridded (such as radar) rainfall data and a moisture depletion option to simulate extended time periods. HEC-HMS has a graphical user interface and both graphics and reporting output options.

HEC-HMS is available from: U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California 95616
www. hec.usace.army.mil/
530-756-1104

HEC-RAS

HEC-RAS, River Analysis Software, was developed by the U. S. Army Corps of Engineers to calculate water surface profiles for steady gradually varied flow. The system can be comprised of a network of channels or a single river reach. Subcritical, supercritical, and mixed flow regime water surface profiles can be computed. HEC-RAS has a graphical user interface and both graphics and report output options.

HEC-RAS is available from: U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California 95616
www. hec.usace.army.mil/
530-756-1104

HY-8

HY-8, Culvert Analysis Program, was developed by FHWA to design and analyze culvert system hydraulics. HY-8 has modules for culvert analysis and design, hydrograph generation and routing, and energy dissipation. The culvert module incorporates the HDS-5 procedures to analyze single or multiple culverts with varying geometries and overtopping flow. HY-8 has an interactive data input procedure. Output is available as reports and some graphics.

HY-8 is available online from FHWA: www.fhwa.dot.gov/bridge/hyd.htm

Or: McTrans
512 Weil Hall
University of Florida
Gainesville, FL 32611-6585
www.mctrans.ce.ufl.edu/
352-392-0378

HYDRAIN

HYDRAIN is a suite of programs developed by GKY & Associates, Inc, for FHWA. The programs are embedded in a system shell to facilitate access to each module. The individual modules are:

HYDRA: Storm Drain and Sanitary Sewer Design and Analysis
HYDRA can design and analyze gravity sewer networks, using Rational Method for peak flow, or a user supplied hydrograph. Capabilities include inlet analysis, hydraulic gradeline computation, detention basin routing, and dynamic routing.

WSPRO: Open Channel Water Surface Analysis, Bridge Hydraulics, Scour (this program is also used as a stand-alone and is described below)

HYDRO: Hydrology
HYDRO has capabilities for calculating rainfall intensities, IDF curves, Rational method, log-Pearson Type III analysis, and hydrograph development using the USGS nationwide urban and semi-arid dimensionless hydrographs.

HY-8: FHWA Culvert Analysis and Design (this program is also used as a stand-alone and is discussed above)

HYCHL: Flexible and Rigid Channel Lining Design and Analysis
HYCHL can design and analyze a variety of channel lining materials using the criteria from the FHWA publications: HEC-15, "Design of Roadside Channels with Flexible Linings," and HEC-11, "Design of Riprap revetment."

NFF: USGS National Flood Frequency Program
NFF will solve the USGS Regression Equations, it includes the 1987 version of the equations for Minnesota.

HYDRA, WSPRO, and HYDRO use a batch input format. The editor provided with HYDRAIN assists the users in preparing the input files. HY-8, HYCHL, and NFF have interactive input procedures. Output is available as reports with limited graphics.

HYDRAIN is available online from FHWA: www.fhwa.dot.gov/bridge/hyd.htm

Or: McTrans
512 Weil Hall
University of Florida
Gainesville, FL 32611-6585
www.mctrans.ce.ufl.edu/
352-392-0378

HydroCAD

HydroCAD is a commercial software application that generates and routes hydrographs through ponds and channels. It incorporates the NRCS runoff curve number, synthetic storm, and dimensionless unit hydrograph procedures and is considered an alternative to TR-20 (see below). HydroCAD also has capabilities to analyze the hydraulics of outlet structures. HydroCAD has an interactive data input procedure that includes a graphical interface for laying out then drainage systems components. Output options include reports and graphs.

HydroCAD is available from:

Applied Microcomputer Systems
P.O. Box 350
Chocorua, NH 03817

www.hydrocad.net
603-323-8666

SMS/FESWMS-2DH

SMS (Surface Modeling System), a commercial product of Brigham Young University, is a graphical user interface for developing and displaying the results of 2-dimensional models (including FESWMS-2DH) of river systems. FESWMS-2DH is a numerical model developed for FHWA that solves the system of equations describing two-dimensional depth-averaged flow in a horizontal plane using the finite element method. FESWMS was developed primarily to evaluate complex hydraulic conditions at highway river crossings. FESWMS is a complicated model to develop the input data for and successfully run. SMS has a graphical user interface that facilitates the development of the necessary model input, output options include a variety of graphical products and reports.

SMS is available from:

Environmental Modeling Systems Incorporated
1890 West 719 North #38B
Provo, UT 84601

www.ems-i.com/software.html
801-373-5200

TR-20

TR-20 (Computer Program for Project Formulation Hydrology) was developed by NRCS to automate hydrographic analysis of watersheds using the runoff curve number, synthetic storm, and dimensionless unit hydrograph procedures. The program capabilities include hydrograph generation, routing through channels and reservoirs, and comparison of discharges for varying watershed parameters. TR-20 has a batch input format, there is a program that helps the user develop an input file. Output is available as text reports.

TR-20 is available online from NRCS at: www.wcc.nrcs.usda.gov/water/quality/wst.html

TR-55

TR-55, developed by NRCS, contains simplified procedures to perform hydrologic analysis. Capabilities include peak flow using NRCS graphical procedures), time of concentration and travel time, tabular hydrograph generation, and an estimated or required detention storage volumes. TR-55 has an interactive data input procedure, output is in the form of text reports.

TR-55 is available online from the NRCS at: www.wcc.nrcs.usda.gov/water/quality/wst.html

Or from:

PC-TRANS
2011 Learned Hall
Lawrence, KS 66045

kuhub.cc.ukans.edu/~pctrans/index.html
913-864-5655

Urban Drainage

The Urban Drainage Design Software (HY-22) is a collection of programs developed for FHWA that have capabilities for storm drain inlet analysis, channel flow hydraulics, and reservoir routing. These programs have an interactive data input procedure, and output data is available as a text report.

Urban Drainage (HY-22) is available online from FHWA:		www.fhwa.dot.gov/bridge/hyd.htm
Or:	McTrans 512 Weil Hall University of Florida Gainesville, FL 32611-6585	www.mctrans.ce.ufl.edu/ 352-392-0378

WSPRO

WSPRO, Water Surface Profile, was developed for FHWA and is used to analyze one-dimensional, gradually-varied, steady flow in open channels. WSPRO can also be used to analyze flow through bridges and culverts, embankment overflow, and scour at bridges. WSPRO has a batch data input format, the HYDRAIN line editor (see description of HYDRAIN above) can be used to facilitate data entry. Output is available in text report files.

WSPRO (HY-7) is available online from FHWA:		www.fhwa.dot.gov/bridge/hyd.htm
Or:	McTrans 512 Weil Hall University of Florida Gainesville, FL 32611-6585	www.mctrans.ce.ufl.edu/ 352-392-0378

XP-SWMM

XP-SWMM is a commercial application that incorporates the EPA Storm Water Management Model (SWMM) along with an enhanced user interface and additional features. SWMM is a comprehensive model for simulating runoff quantity and quality through all aspects of the hydrologic cycle. Capabilities include surface runoff, dynamic routing through the drainage system, storage, and treatment effects. SWMM is a complex model with many features. XP-Software has added the capability of using NRCS runoff and hydrograph procedures. XP-SWMM has a graphical user interface to facilitate data input, output options include text reports and a variety of graphic plots.

XP-SWMM is available from:	XP-Software, Inc. 2000 NE 42nd Ave, Suite 214 Portland, Oregon, 97213-1305	www.xpsoftware.com.au 888-554-5022

E.2 Reference Material Sources

Minnesota Department of Transportation Manuals can be obtained through:

Map and Manual Sales
M.S. 260 Room G19
Transportation Building
295 John Ireland Blvd.
St. Paul, MN 55155

Phone: (651) 296-2216

This Federal Highway Administration's Hydraulic Engineering Circular (HEC) and Hydraulic Design Series (HDS) listing is based on an April 17, 2000 update. The Hydraulics publications listed in this reference list are available through either NTIS or the FHWA Report Center. The publications are not available from the Bridge Division.

Hydraulic Design Series (HDS)

		YEAR	FHWA-#	NTIS-#
HDS 1	Hydraulics of Bridge Waterways	1978	EPD-86-101	PB86-181708
HDS 2	Highway Hydrology (SI)	1996	SA-96-067	PB97-134290
HDS 3	Design Charts for Open-Channel Flow	1961	EPD-86-102	PB86-179249
HDS 4	Introduction to Highway Hydraulics (SI)	1997	HI-97-028	PB97-186761
HDS 5	Hydraulic Design of Highway Culverts *	1985	IP-85-15	PB86-196961

Hydraulic Engineering Circulars (HEC)

		YEAR	FHWA-#	NTIS-#
HEC 9	Debris-Control Structures	1971	EPD-86-106	PB86-179801
HEC 11	Design of Riprap Revetment	1989	IP-89-016	PB89-218424
HEC 12	Drainage of Highway Pavements	1984	TS-84-202	PB84-215003
HEC 14	Hydraulic Design of Energy Dissipators for Culverts & Channels*	1983	EPD-86-110	PB86-180205
HEC 14	Hydraulic Design of Energy Diss. for Culverts & Channels (SI) (4.9 M)	1999		
HEC 15	Design of Roadside Channels with Flexible Linings *	1988	IP-87-7	PB89-122584
HEC 17	The Design of Encroachments on Flood Plains using Risk Analysis	1981	EPD-86-112	PB86-182110
HEC 18	Evaluating Scour at Bridges, Edition 3 (SI)	1995	HI-96-031	PB96-163498
HEC 20	Stream Stability at Highway Structures, Edition 2 (SI)	1995	HI-96-032	PB96-163480
HEC 21	Bridge Deck Drainage Systems	1993	SA-92-010	PB94-109584
HEC 22	Urban Drainage Design Manual (SI)	1996	SA-96-078	PB97-134308
HEC 23	Bridge Scour and Stream Instability Countermeasures (SI)	1997	HI-97-030	PB97-199491

Hydraulic Reports

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HI	Highways in the River Environment	1990	HI-90-016	PB90-252479
TS	Underground Disposal of Storm Water Runoff, Design Guidelines Manual	1980	TS-80-218	PB83-180257
TS	Guide for Selecting Manning's Roughness Coefficient For Natural Channels & Flood Plains	1984	TS-84-204	PB84-242585
IP	Culvert Inspection Manual	1986	IP-86-2	PB87-151809
IP	Structural Design Manual for Improved Inlets and Culverts *	1983	IP-83-6	PB84-153485
FLP	Best management Practices for Erosion and Sediment Control	1995	FLP-94-005	
RD	Countermeasures for Hydraulic Problems at Bridges, Vol. 1 Analysis and Assessment	1978	RD-78-162	PB-297132
RD	Countermeasures for Hydraulic Problems at Bridges, Vol. 2 Case Histories	1978	RD-78-163	PB-297685

Publications on CD-ROM **

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HDS-5	Hydraulic Design of Highway Culverts (CDROM), v. 1.00	1996	SA-96-080	N/A
	Installation and User's Guide	1996	SA-96-081	N/A

"*" Several manuals are also available from McTrans. These are marked with a "*".
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Most of the Federal Highway Administration hydraulics publications listed are available through either NTIS or the FHWA Report Center. Contact information for these offices is listed below. Contact NTIS to obtain copies of any publication that has a listed NTIS number. FHWA now has a number of manuals (including HEC-22, HDS-5, HEC-12) available electronically at http://www.fhwa.dot.gov/bridge/elibrary.htm

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