DRAINAGE MANUAL

PREPARED BY

HYDRAULICS SECTION

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ACKNOWLEDGEMENT

In compiling all data presented herein various publications and individuals were consulted. Due to the number of sources, it is impossible to acknowledge them by name; however, the Hydraulics Section wishes to convey their appreciation to all contributors, both within the Department and otherwise, who assisted in the preparation of the material. The publications of the United States Department of Transportation, Federal Highway Administration, Soil Conservation Service, the United States Department of the Interior, Geological Survey and other state's drainage manuals were freely consulted and their contribution is gratefully acknowledged.

INTRODUCTION

This manual is intended as an operational handbook for use in hydrologic and hydraulic analysis. While it is presumed that the user of this manual is basically familiar with these methods of analysis, the text provides detailed instructions and criteria for their development in most cases. An exception to this rule is the case where another source document expounds upon the method in great detail. In this case, the user is directed to the source document and a brief synopsis of the subject is provided in this manual.

Also, the purpose of this manual is to continue uniformity in hydraulic design that will be within accepted policies of the Arkansas State Highway and Transportation Department and the Federal Highway Administration.

This manual is divided into chapters, each dealing with a major category of hydrologic or hydraulic analysis. Each chapter is further divided according to specific elements of the subject.

The Department recognizes the difficulty in accurately defining or predicting the dynamic properties of nature. There are numerous methods of analysis available and it is recommended that as many method (s) as may be appropriate be employed in the solution of a problem. Further, all hydraulic designs must give consideration to economic, aesthetic and environmental aspects of the given design.

Complete documentation of all analysis is essential and must be perpetually maintained. The rapid development of technology in the fields of hydrology and hydraulics necessitates a periodic review and, if necessary, update of all analysis prior to construction of the facility. All analysis completed more than three years before construction should be reviewed prior to construction.

To the best of their ability, the authors have insured that the information presented here is correct and that the procedures are reliable. The execution of an engineering design, however, involves the judgment of the designer to ascertain whether a technique or item of information can be applied to a given situation.

Therefore, neither the Arkansas State Highway and Transportation Department nor any contributor accepts responsiblity for any real or alleged error, loss, damage or injury resulting from the use of material contained herein.

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DESIGN POLICIES

1-100 DESIGN FREQUENCY

1-101 GENERAL

Storm frequencies for use in design are to be in general accordance with AASHTO and FHWA criteria. In determining the design frequency, consideration must be given to all significant impacts and risks involved. This determination is by necessity the responsibility of the designer and should not be considered as compliance with a rigid guideline. The complete documentation concerning the selection of design frequency is essential.

1-102 RECOMMENDED CRITERIA

No exact criteria for flood frequency of allowable backwater/ headwater values can be set which will apply to an entire project or roadway classification. The following flood frequency values relative to protection of the project from flooding or damage are recommended for design:

a. CROSS DRAINS

Interstate Projects: 50 year
Primary Projects: 50 year
Federal Aid Urban Projects: 25 year
Secondary Projects: 25 year
Non-Federal Aid Projects: 10 year*

*Drainage area less than two square miles; ADT less than 750. If either is exceeded, use 25 year flood frequency.

b. STORM SEWERS

Interstate and/or Interstate Type Projects: 50 year
Other Federal Aid Projects: 10 year
Non-Federal Aid Projects: 2 year

NOTE: If local drainage facilities and practices have provided drains of a lesser standard to which the highway system must connect, special consideration should be given to whether it is realistic to design the highway system to a higher standard than available outlets.

1-200 PONDING

The flow of water in the gutter should be restricted to a depth, and corresponding width, which will not severely obstruct or cause a hazard to traffic. This flow is a function of the quantity of water, gutter gradient, roughness of pavement where the flow is contained, and cross section shape of the flow area.

The following limiting widths are recommended for use at design flood stage:

a. FOR INTERSTATE AND FULLY CONTROLLED ACCESS PROJECTS:
Limit ponding to one-half the width of the outer lane - 50-year flood frequency.

b. OTHER FEDERAL AID PROJECTS:
Limit ponding to the width of the outer lane - 10-year flood frequency.

c. NON-FEDERAL AID PROJECTS:
Limit ponding to the width of the outer lane - 2-year flood frequency.

d. FOR MINOR TWO-LANE HIGHWAYS AND STREETS:
Limit ponding to a width and depth, which will allow passage of one lane of traffic with safety.

1-300 FEDERAL HIGHWAY ADMINISTRATION CRITERIA

1-301 GENERAL

This section incorporates FHWA Transmittal 315, "Location and Hydraulic Design of Encroachments on Floodplains", of November 15, 1979 and the Department's instructions relating to hydrologic and hydraulic analysis, design criteria and documentation requirements.

U. S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

FEDERAL-AID HIGHWAY PROGRAM MANUAL

VOLUME	6	ENGINEERING AND TRAFFIC OPERATIONS
CHAPTER	7	BRIDGES, STRUCTURES AND HYDRAULICS
SECTION	3	HYDRAULICS, EROSION CONTROL AND WATER QUALITY
SUBSECTION	2	LOCATION AND HYDRAULIC DESIGN OF ENCROACHMENTS ON FLOOD PLAINS

Transmittal 315
November 15, 1979
HNG-31

1. PURPOSE.* *To prescribe Federal Highway Administration (FHWA) policies and procedures for the location and hydraulic design of highway encroachments on flood plains, including direct Federal highway projects administered by the FHWA.*

2. AUTHORITY. *23 U.S.C. 109(a) and 315, 23 CFR 1.32, 49 CFR 1.48(b), Executive Order 11988 - Floodplain Management, May 24, 1977 (42 FR 26951), and Department of Transportation Order 5650.2, April 26, 1979 (44 FR 24678).*

3. POLICY. *It is the policy of the FHWA:*

   a. to encourage a broad and unified effort to prevent uneconomic, hazardous or incompatible use and development of the Nation's flood plains,

   b. to avoid longitudinal encroachments, where practicable,

   c. to avoid significant encroachments, where practicable,

* Regulatory material is italicized and is published in 23 CFR 650.

d. *to minimize impacts of highway agency actions which adversely affect base flood plains,*

e. *to restore and preserve the natural and beneficial flood-plain values that are adversely impacted by highway agency actions,*

f. *to avoid support of incompatible flood-plain development,*

g. *to be consistent with the intent of the Standards and Criteria of the National Flood Insurance Program, where appropriate, and*

h. *to incorporate "A Unified National Program for Floodplain Management" of the Water Resources Council into FHWA procedures.*

4. DEFINITIONS

a. *Action - any highway construction, reconstruction, rehabilitation, repair, or improvement undertaken with Federal or Federal-aid highway funds or FHWA approval.*

b. *Base Flood - the flood or tide having a 1-percent chance of being exceeded in any given year.*

c. *Base Flood Plain - the area subject to flooding by the base flood.*

d. *Design Flood - the peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.*

e. *Encroachment - an action within the limits of the base flood plain.*

f. *Floodproof - to design and construct individual buildings, facilities, and their sites to protect against structural failure, to keep water out or to reduce the effects of water entry.*

g. *Freeboard - the vertical clearance of the lowest structural member of the bridge superstructure above the water surface elevation of the overtopping flood.*

h. *Minimize - to reduce to the smallest practicable amount or degree.*

i. *Natural and Beneficial Flood-plain Values - include but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.*

j. *Overtopping Flood - the flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.*

k. *Practicable - capable of being done within reasonable natural, social, or economic constraints.*

l. *Preserve - to avoid modification to the functions of the natural flood-plain environment or to maintain it as closely as practicable in its natural state.*

m. *Regulatory Floodway - the flood-plain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program).*

n. *Restore - to reestablish a setting or environment in which the functions of the natural and beneficial flood-plain values adversely impacted by the highway agency action can again operate.*

o. *Risk - the consequences associated with the probability of flooding attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the highway.*

p. *Risk Analysis - an economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least total expected cost to the public. It shall include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway-aggravated flood damage to other property, and for additional or interrupted highway travel.*

q. *Significant Encroachment - a highway encroachment and any direct support of likely base flood-plain development that would involve one or more of the following construction- or flood-related impacts:*

(1) *a significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route,*

(2) *a significant risk, or*

(3) *a significant adverse impact on natural and beneficial flood-plain values.*

r. *Support Base Flood-Plain Development - to encourage, allow, serve, or otherwise facilitate additional base flood-plain development. Direct support results from an encroachment, while indirect support results from an action out of the base flood plain.*

5. APPLICABILITY

a. *The provisions of this directive shall apply to all encroachments and to all actions which affect base flood plains, except for repairs made with emergency funds (FHPM 6-9-16-1 and 2, 23 CFR 668) during or immediately following a disaster.*

b. *The provisions of this directive shall not apply to or alter approvals or authorizations which were given by FHWA pursuant to directives in effect before the effective date of this directive.*

6. PUBLIC INVOLVEMENT. *Procedures which have been established to meet the public involvement requirements of FHPM 7-7-2 (23 CFR 771) and FHPM 7-7-1 (23 CFR 795) or FHPM 7-7-5 (23 CFR 790) shall be used to provide opportunity for early public review and comment on alternatives which contain encroachments.*

   a. *Public notices issued in accordance with the above procedures shall make reference to significant encroachments which are contained in alternatives under consideration.*

   b. *Public hearing presentations shall include identification of encroachments.*

7. LOCATION HYDRAULIC STUDIES

   a. *National Flood Insurance Program (NFIP) maps or information developed by the highway agency, if NFIP maps are not available, shall be used to determine whether a highway location alternative will include an encroachment.*

   b. *Location studies shall include evaluation and discussion of the practicability of alternatives to any longitudinal encroachments.*

   c. *Location studies shall include discussion of the following items, commensurate with the significance of the risk or environmental impact, for all alternatives containing encroachments and for those actions which would support base flood-plain development:*

      (1) *the risks associated with implementation of the action,*

      (2) *the impacts on natural and beneficial flood-plain values,*

      (3) *the support of probable incompatible flood-plain development,*

      (4) *the measures to minimize flood-plain impacts associated with the action, and*

(5) *the measures to restore and preserve the natural and beneficial flood-plain values impacted by the action.*

d. *Location studies shall include evaluation and discussion of the practicability of alternatives to any significant encroachments or any support of incompatible flood-plain development.*

e. *The studies required by paragraphs 7c and d shall be summarized in environmental review documents prepared pursuant to FHPM 7-7-2 (23 CFR 771).*

f. *Local, State, and Federal water resources and flood-plain management agencies should be consulted to determine if the proposed highway action is consistent with existing watershed and flood-plain management programs and to obtain current information on development and proposed actions in the affected watersheds.*

8. ONLY PRACTICABLE ALTERNATIVE FINDING

a. *A proposed action which includes a significant encroachment shall not be approved unless the FHWA finds that the proposed significant encroachment is the only practicable alternative. This finding shall be included in the final environmental document (final environmental impact statement or finding of no significant impact) and shall be supported by the following information:*

(1) *the reasons why the proposed action must be located in the flood plain,*

(2) *the alternatives considered and why they were not practicable, and*

(3) *a statement indicating whether the action conforms to applicable State or local flood plain protection standards.*

b. *A copy of the finding shall be made available to appropriate State and areawide clearinghouses following procedures established in accordance with FHPM 4-1-4 (23 CFR 420).*

9. DESIGN STANDARDS

a. *The design selected for an encroachment shall be supported by analyses of design alternatives with consideration given to capital costs and risks, and to other economic, engineering, social, and environmental concerns.*

(1) *Consideration of capital costs and risks shall include, as appropriate, a risk analysis or assessment which includes:*

(a) *the overtopping flood or the base flood, whichever is greater, or*

(b) *the greatest flood which must flow through the highway drainage structure(s), where overtopping is not practicable. The greatest flood used in the analysis is subject to state-of-the-art capability to estimate the exceedance probability.*

(2) *The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2-percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways.*

(3) *Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.*

(4) *The effect of existing flood control channels, levees, and reservoirs shall be considered in estimating the peak discharge and stage for all floods considered in the design.*

(5) *The design of encroachments shall be consistent with standards established by the FEMA, State, and local governmental agencies for the administration of the National Flood Insurance Program for:*

(a) *all direct Federal highway actions, unless the standards are demonstrably inappropriate, and*

(b) *Federal-aid highway actions where a regulatory floodway has been designated or where studies are underway to establish a regulatory floodway.*

b. *Rest area buildings and related water supply and waste treatment facilities shall be located outside the base flood plain, where practicable. Rest area buildings which are located on the base flood plain shall be floodproofed against damage from the base flood.*

c. *Where highway fills are to be used as dams to permanently impound water more than 50 acre-feet ($6.17 x 10^4$ cubic metres) in volume or 25 feet (7.6 metres) deep, the hydrologic, hydraulic, and structural design of the fill and appurtenant spillways shall have the approval of the State or Federal agency responsible for the safety of dams or like structures within the State, prior to authorization by the Division Administrator to advertise for bids for construction.*

10. CONTENT OF DESIGN STUDIES

a. *The detail of studies shall be commensurate with the risk associated with the encroachment and with other economic, engineering, social, or environmental concerns.*

b. *Studies by highway agencies shall contain:*

(1) *the hydrologic and hydraulic data and design computations,*

(2) *the analysis required by paragraph 9a, and*

(3) *for proposed direct Federal highway actions, the reasons, when applicable, why FEMA criteria (44 CFR 60.3, formerly 24 CFR 1910.3) are demonstrably inappropriate.*

c. *For encroachment locations, project plans shall show:*

(1) *the magnitude, approximate probability of exceedance and, at appropriate locations, the water surface elevation associated with the overtopping flood or the flood of paragraph 9a(1)(b), and*

(2) *the magnitude and water surface elevation of the base flood, if larger than the overtopping flood.*

1-303 INSTRUCTIONS AND GUIDELINES RELATING TO FHWA TRANSMITTAL 315

1-303.1 INTRODUCTION

The purpose of the requirements outlined in Transmittal 315, FHPM 6-7-3-2, "Location and Hydraulic Design of Encroachments on Floodplains", is to assure that a Flood Hazard Study will be accomplished at all points where Federal Aid highways cross or encroach on floodplains.

When a roadway or appurtenant feature crosses or is located adjacent to a live stream or a watercourse with bed and banks (flowing or dry) and lies below the 100-year flood elevation of said stream or watercourse, it will fall within the requirements of this policy.

With the passage of the Flood Disaster Protection Act of 1973, flood insurance is now available to property owners in flood hazard areas. This protection is only available where local governments meet certain conditions in land development, building codes and floodplain management. Detailed hydraulic studies (Federal Flood Insurance Studies) are currently available or underway for many areas of Arkansas that specify the "official" 100-year flood elevation and define floodway boundaries (refer to Figure 1-1, page 1-21 for pictorial definition of floodway).

Some of the requirements that must be met by highway projects are that no development or encroachment will be allowed that:

a. Causes an increase in the 100-year flood elevation within a regulatory floodway to exceed a predetermined amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program).

b. Obstructs the stream flow to a degree that a 100-year flood level is increased by more than an amount determined by a risk assessment in areas where no regulatory floodway exists.

c. Places new building construction with floor elevation (including basement) below the 100-year flood unless flood proofed to the 100-year flood elevation.

The designer should endeavor to review the content of any Federal Flood Insurance Study (furnished by Hydraulics Section) completed or underway in the highway project areas and utilize information found in these studies in hydraulic design of the project.

1-303.2 IMPLEMENTATION

Project Review - A review of all projects shall be made at the corridor study stage to consider potential Flood Hazard areas. This will be accomplished through review coordination by Surveys and Design Divisions, and Hydraulics Section.

Flood Hazard Area - Where a Flood Hazard area has been previously designated or there is potential for such designation, a determination will be made of the type and extent of survey to be required in order that a proper evaluation or risk assessment can be made.

Study Coordination - In those areas where the 100-year flood has been designated or where the study is in progress, it will be necessary for Hydraulics Section to coordinate the study with the design sections. In those areas where no agency has yet studied and delineated the 100-year flood elevation, it will be possible to handle the design as usual but with proper documentation in accordance with FHPM 6-7-3-2.

Floodplain Survey - The survey party will obtain a floodplain survey at those locations where it is determined to be warranted. The survey will include ground level photos of the site when practical and elevations of pertinent structures within the base floodplain a sufficient distance upstream to evaluate backwater damages.

a. Class I Site Survey

Standard Bridge Situation Survey - For all bridge sites and major culvert installations.

b. Class II Site Survey. (When requested by design sections) Longitudinal Encroachments - Cross sections for full width of floodplain at 200' ± intervals from 500' upstream of anticipated encroachment to 500 feet downstream of anticipated encroachments. Also, continuous channel-bed profile within the cross-section limits.

1-303.3 GENERAL GUIDELINES FOR HYDRAULIC DESIGN

The interpretation of design criteria and degree of documentation to be applied to any given structure design within a project must depend upon the designers evaluation of that individual structure and site characteristics.

No exact criteria for flood frequency or allowable backwater/headwater values can be set which will apply to an entire project or roadway classification.

Locations involving a bridge or a large culvert (span ≥ 20 feet) will require a detailed hydraulic analysis. These are normally handled by the respective Design Divisions. In those areas where the 100-year floodplain has been designated, it will be necessary to coordinate the study with the Hydraulics Section.

The hydraulic design of drainage structures must be such that risks to traffic, potential property damage and failure from floods is consistent with good engineering practice and economics.

Recognizing that floods cannot be predicted precisely and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage should the design flood be exceeded.

Design headwater/backwater and flood frequency criteria for a site should be based on these risk considerations:

a. Hazard to human life
b. Damage to adjacent property
c. Damage to the structure and roadway
d. Traffic interruption
e. Damage to stream and floodplain environment
f. Emergency access

Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all hydraulic structures. Therefore, the designer should endeavor to recognize the potential damage to property or adjacent property or inconveniences to owners and determine appropriate means of obviating the damage or inconveniences.

A risk assessment or risk analysis shall form a basis of judging the significance of the risk. The detail of studies in support of selected designs are to be commensurate with the risks associated with the encroachment and with other economic, engineering, social or environmental concerns. Therefore, a risk assessment would be the minimum study that would be required for the design of any encroachment. A risk assessment suffices where the risk or capital cost is insufficient to warrant a risk analysis.

Reference to backwater limitation of 1 foot (regulatory floodway) for the 100-year flood is the maximum allowable as established by FEMA. In areas where no regulatory floodway exists, hydraulic design shall conform to required criteria as determined by a risk assessment.

The amount of increase in the established flood levels that is acceptable as a result of highway construction must be consistent with the Federal and local criteria.

The results of the Location Hydraulic studies shall be included in the environmental review documents. In case of an "only practicable alternative finding", it shall be included in the final environmental document.

The significance of this policy is as follows:

a. The 100-year flood will be computed and its stage or headwater elevation established for the existing conditions. The hydraulic performance of the proposed design during the passage of the 100-year flood will be determined and compared to (measured against) the existing condition.

b. The highway grades and hydraulic structures shall be designed to permit conveyance of the 100-year flood <u>without causing significant damage</u> to the highway, stream, or adjacent property, and without raising the 100-year flood stage by more than the allowable amount.

c. The roadway and hydraulic structure may be designed for a flood frequency other than the 100-year flood, however, the documentation of what happens with the 100-year flood will be required.

1-303.4 <u>DOCUMENTATION</u>

Applicable Items "a" through "f" listed below are to be shown on the profile sheet of the roadway plans and the layout sheet of the bridge plans.

a. The design flood for bridges (discharge, stages and backwater) shall be the largest standard frequency flood (i. e., 10-year, 25-year, 50-year, 100-year, etc.,) that passes beneath the minimum low chord with a minimum of 1 foot of freeboard.

b. The design flood for culverts and other hydraulic structures (excluding bridges) for the largest standard frequency flood that does not inundate the travelway and satisfies other allowable headwater/depth criteria (discharge, stage and backwater).

c. The overtopping flood shall be the specific frequency flood (i.e., 22-year, 83-year, etc.,) that overflows the travelway or crosses a drainage divide and diverts flow into another watershed.

d. The base flood shall be the 100-year stage elevation. When the overtopping flood stage exceeds the base flood stage, the base flood stage will be used for documentation of the design.

e. The maximum known historical high water elevation will be shown for all stream crossings where such data is available. The plan designation shall read high water elevation, date of occurrence, flood frequency, if known (i.e., high water elevation 465.3', August 1940, 50-year ± Flood). The flood frequency will be determined by the appropriate design section.

f. In order to provide water level data on the highway plans that is meaningful and complementary to the Federal 404 Permit process, the ordinary high water is to be shown on stream crossings having average annual flow of 5 cfs or greater. The ordinary high water elevation will be supplied by the Surveys Division.

The Hydrologic Data Sheet, Table 1-1, page 1-23, shall be completed by the Design Divisions on completion of all projects and forwarded to Hydraulics Section along with a memorandum stating that a risk assessment or analysis has been made and the design of the project meets the requirements of FHPM 6-7-3-2. All categories of the Data Sheet shall be filled in with the appropriate information or the statement "No Data Available" as applicable.

Hydraulics Section will review the hydraulic design of the project for compliance with FHPM 6-7-3-2 and certify with a memorandum to the master files. The Hydrologic Data Sheet and risk assessment from the applicable Design Division should be attached to this memorandum.

FLOODWAY SCHEMATIC

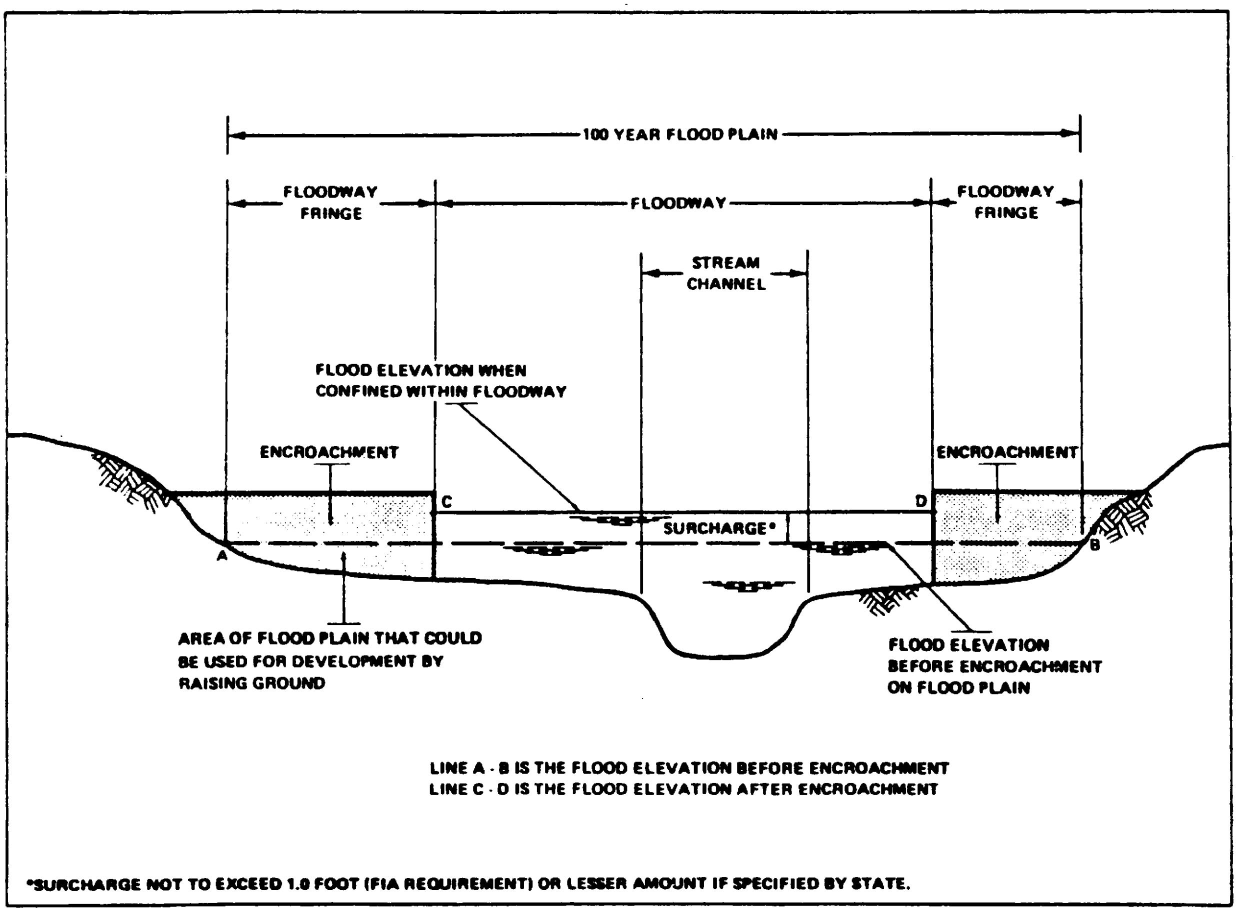

FIGURE 1-1

TABLE I-2

FORM HYD 4-I

HYDROLOGIC DATA

JOB NO. 0000

THE DATA HEREIN WAS SYSTEMATICALLY DERIVED BY EMPIRICAL METHODS AND FROM FIELD OBSERVATION. IT IS PRESENTED AS AN ESTIMATE OF THE HYDRAULIC PERFORMANCE OF THESE FACILITIES DURING THE PASSAGE OF ACTUAL FLOOD EVENTS.

FIELD INSPECTION STAGE ☐ FINAL DESIGN STAGE ☐

				BASE FLOOD *1		DESIGN FLOOD *2			OVERTOPPING FLOOD		HISTORICAL DATA *3		
STATION	STREAM NAME	DRAINAGE AREA	STRUCTURE SIZE	DISCHARGE (CFS)	STAGE ELEVATION (FT.)	DISCHARGE (CFS)	ESTIMATED EXCEEDANCE PROBABILITY%	STAGE ELEVATION (FT.)	STAGE ELEVATION (FT.)	ESTIMATED EXCEEDANCE PROBABILITY%	DATE	STAGE ELEVATION (FT.)	ESTIMATED EXCEEDANCE PROBABILITY%
16+72	Dutch	103.4	242.95	20,500	359.4	16,200	4	358.4	360.5	>4	-	357.9	<4
	Creek	Mile 2	Bridge										

REMARKS SOURCE OF INFORMATION AND OTHER RELATED DATA

Hydrology by Hydraulics Section 11-19-81

Hydraulic Analysis by Bridge Design - W.S. with backwater 360.1 (Design Flood)
*361.9 (Base Flood)

*Without overtopping (Actual overtopping frequency computation not warranted due to isolated rural location).

1. ESTIMATED 100 YEAR FREQUENCY FLOOD DATA (UNLESS OTHERWISE NOTED). THIS MAGNITUDE OF FLOODING MAY PASS THROUGH THE PROPOSED FACILITY OR IT MAY OBTAIN THE NECESSARY HYDRAULIC CONVEYENCE BY PARTIAL INUNDATION OF ROADWAYS AND/OR PARTIAL BYPASS OF THE FACILITY.

2. SPECIFIED FREQUENCY FLOOD DATA. IT IS ANTICIPATED THAT THIS MAGNITUDE OF FLOODING WILL BE CONVEYED THROUGH THE PROPOSED HYDRAULIC FACILITY UNDER ESTIMATED CONDITIONS WHICH MODIFY THE DESIGN EFFECTS APPLICABLE TO THE SITE.

3. THIS DATA WAS OBTAINED FROM OBSERVATION BY PERSONS FAMILIAR WITH THE AREA AND/OR OFFICIAL RECORDS ASSEMBLED WITH AN EVALUATION BY EMPIRICAL METHODS. THE RELIABILITY OF THIS DATA IS SUBJECT TO THE ACCURACY OF THE SOURCE. A FUTURE FLOOD OF THE SAME MAGNITUDE MAY ACHIEVE A SIGNIFICANTLY DIFFERENT STAGE ELEVATION FROM THAT SHOWN DUE TO CHANGES IN THE PHYSICAL CHARACTERISTICS OF THE WATERSHED.

FORM HYD 4-1

TABLE 1-2

HYDROLOGIC DATA

THE DATA HEREIN WAS SYSTEMATICALLY DERIVED BY EMPIRICAL METHODS AND FROM FIELD OBSERVATION. IT IS PRESENTED AS AN ESTIMATE OF THE HYDRAULIC PERFORMANCE OF THESE FACILITIES DURING THE PASSAGE OF ACTUAL FLOOD EVENTS.

FIELD INSPECTION STAGE ☐ FINAL DESIGN STAGE ☐

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REMARKS SOURCE OF INFORMATION AND OTHER RELATED DATA

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REFERENCES

Chapter 1

DESIGN POLICIES

DRAINAGE MANUAL, Virginia Department of Highways and Transportation, (Richmond, Virginia, 1980).

HIGHWAY DRAINAGE GUIDELINES, American Association of State Highway and Transportation Officials, Washington, D. C., 1979

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Chapter 2

SURVEYS AND FIELD INFORMATION

2-100 FIELD SURVEYS - GENERAL

Sufficient data is necessary on all stream crossings, whether it is a small drainage ditch or a river crossing, to provide the hydraulics designer enough information to size the structure necessary for a particular situation. The amount of survey data collected and the detail of the data should be commensurate with the complexity of the hydraulics and should be coordinated with the Hydraulics Section for guidance in unusual circumstances.

The following is the recommended field information that should be provided the designer by the Surveys Division.

A. Cross Drains

1. Roadway profile extending to the extremities of the floodplain.
2. M-line profile for each drainage structure or drainage ditch at least 300 feet upstream and downstream or the centerline crossing.
3. A valley cross section referenced to the centerline approximately 300 feet upstream and downstream and normal to the floodplain. The valley section should extend above high water and the soil cover noted.
4. Cross section at each pipe or box that acts as an equalizer.
5. High water data at each drainage structure or data that will allow the designer to assume an allowable high water.
6. In an urban area a city map should be furnished showing the drainage area, the direction of flow along each street within the drainage area, and the approximate location of existing drop inlets.
7. All pipe sizes, flow line elevations, direction of flow and the location of all storm sewer pipes in the area should be furnished.

B. Bridges

1. Survey of roadway approaches should extend beyond the extremities of the floodplain and/or above the reaches of historical high water.

2. All siginificant physical features that have a potential to be adversely affected should be located. Features such as residences, commercial and industrial establishments, crop lands, wetlands, roadways, railroads, utilities, wells and other facilities can influence design and their locations and elevations (floor elevations especially) should be established by the survey.

3. Profile of channel extending approximately 2,000 feet upstream and downstream of the centerline crossing.

4. Cross section of channel upstream and downstream of the existing bridge.

5. Cross section of waterway opening for each existing bridge.

6. All major overflow and meandering channels should be located with cross sections.

7. Alignment and cross sections of existing roadway and bridge where proposed bridge crossing is on new location.

8. Sketch and description of bridge, number of spans, length, type, number and types of bents and pier shapes and sizes. Normally this information may be obtained by the designer from existing plans or Bridge Inspection Reports.

9. Historical high water elevation. Several dependable high water elevations are needed to adquately analyze flood discharges. This information should be obtained

from observing seed and mud lines on tree trunks and bridge abutments, wash-lines and fine-debris lines on banks and bridge approach fills, traces of grass or hay lodged in tree limbs and fences, and evidence of erosion and scour. Interviews with residents, commercial and school bus drivers, mail carriers, law enforcement officers, Department, county and railroad maintenance personnel, and others who might have an opportunity to observe unusual floods will yield additional information. The date of the flood occurrence, the name and address of the observer and the stage and location of the observation should be recorded. The observed frequency of occurrence should be noted since reliable information that a stream reaches a certain elevation every 2 or 3 years provides important frequency information for the designer. A few hours spent in interviewing several people who are familiar with the flood history of a stream can result in substantial savings in construction, liability or future maintenance improvements in the design.

10. Ordinary high water elevation. This information should be obtained by observing the line on the stream banks established by the fluctuations of water and indicated by physical characteristics such as a clear natural line impressed on the bank; shelving; changes in the character of soil; destruction of terrestrial vegetation; the presence of litter and debris; or other appropriate means that indicate the characteristics of the surrounding areas.

11. Water surface elevations approximately 2,000 feet (minimum 300 feet) upstream and downstream.

12. Valley Sections

    a. A valley cross section one bridge length upstream and one bridge length downstream representative of the flood plain. Additional sections may be required depending upon flow conditions at the site. Guidance should be sought from Hydraulics Section where unusual circumstances are encountered.

    b. The length of the sections should include the extremities of the flood plain.

    c. Data on land use, vegetal cover and streambed material should be furnished for assessing roughness characteristics in conveyance computations.

    d. Photographs of the existing bridge, stream, flood plain and any other feature that will aid in the design of the proposed bridge.

13. A 1 inch = 50 feet topography map with two (2) foot contour intervals should be plotted and furnished.

14. If the drainage ditch is in an active drainage district, the name of the district, name and address of the chairman, and the attorney should be furnished.

2-200 DRAINAGE BASIN CHARACTERISTICS

Drainage basin characteristics, which should either be estimated or measured in the field or from available mapping or from aerial photographs, include a number of parameters. These parameters are generally used in one form or another in any simulation model or calculation technique. These parameters include the following:

2-201 DRAINAGE BASIN AREA

This information is usually determined from available USGS topographic mapping and aerial photographs. A field inspection of the drainage area is desirable since USGS topographic maps are sometimes not up-to-date. Although 10 foot or 20 foot contour maps may show many areas as contributing to the basin runoff, a field inspection and/or aerial photographs may show natural or manmade depressions such as old gravel pits, which will intercept and contain all of the runoff from a portion of the drainage area. The basin should also be inspected to determine whether or not dams or detention areas have been constructed which will reduce predicted discharges. These may include highway or railroad embankments which, with corresponding culverts or bridges, may act as detention structures. Once the boundaries of the contributing areas have been established, they should be delineated on a base map and the areas determined using a planimeter.

2-202 LENGTH OF STREAM CHANNEL

The length of the stream channel has a direct impact on the travel time from the upstream limits of the drainage area to the design point. Shorter travel time usually results in a higher discharge. Longer travel time tends to reduce the discharge at the design point. This parameter also helps define the overall shape of a drainage basin. Long narrow basins will generally produce lower discharges than shorter, more compact drainage basins. Development also affects basin or travel length. If development has occurred, a field inspection of the basin should be made to determine the basin length or travel distance. This is especially true for small drainage basins. Man-made channels may have straightened out the natural sinuous course of a stream or the pattern of streets may cause the flow path to become a stairstep pattern rather than the natural straight course. This information is normally estimated by the designer from topographic maps or aerial photographs except in unusual circumstances where assistance may be required of the survey party.

2-203 NATURE OF TERRAIN

Stream slopes are usually estimated by the designer from contours on available topographic maps. This parameter is important because steeper stream beds tend to result in a quicker response time whereas flat stream beds decrease response time. Long response time will tend to lower discharges while a short response time tends to increase the discharge.

2-204 LAND USE

This parameter reflects the area under which natural infiltration and/or evaportion/transpiration will not occur or the area where the runoff is affected only slightly by infiltration or depression/detention storage losses. A review of aerial photographs furnished by Surveys Division and/or a field inspection of the drainage basin should be made to determine the areas of development and the types of development to determine the portion of the total drainage area which should be considered impervious. The most reliable method for calculation percent imperviousness is to measure the actual area of impervious areas from an aerial photo. An urban area can usually be broken into typical areas, each with similar characteristics as far as impervious percentage is concerned. A small area can then be selected from each typical area and the percent of impervious area, such as rooftops, driveways, parking lots, and streets, can be measured.

2-205 SOIL CLASSIFICATION

The typical infiltration rates of soils are usually available from soil surveys compiled by the Soil Conservation Service (these surveys are on file in Hydraulics Section). Infiltration reflects the ability of the soil to absorb moisture. This parameter is usually given as inches per hour. The infiltration rate may also be expressed as a decay equation with infiltration rates being higher at the beginning of a storm and decreasing as the storm continues and the soil becomes saturated.

2-300 STREAM COURSE DATA

2-301 DEPRESSION STORAGE

This parameter reflects those losses caused by natural or man-made depressions which decrease runoff. Again, it is usually necessary to estimate these depths either through the observation of previous runoff events or by making detailed field measurements. It is usually impractical to make detailed field measurements of these parameters. Depression storage consists of those areas which are not drained other than infiltration or slow release. Included would be mud puddles, undrained street depressions,depressions caused by ungraded fill areas, plugged storm drains, depressions in grassed areas and other areas flooded by poor drainage even during minor storms.

2-302 CHANNEL STORAGE

In the case of open channels, the channel cross-section can have a tremendous effect on the flood discharges. Channel storage, especially in channels with extremely wide floodplains, can be very significant and can reduce discharges considerably. Runoff models and hydrograph computations, however, do not usually include the effects of channel storage. It is, therefore, very important that a field inspection of the channel be made to insure that if channel storage should be considered, it will be considered. If channel storage is a significant factor, affecting predicted discharges, it may be necessary to compute the effects of this storage by combining the analysis of both the basin hydrology and the stream hydraulics. The Flood Hydrograph Package, HEC-1, and the Water Surface Profiles Computer Program HEC-II, developed by the U. S. Army Corps of Engineers have been developed for this type of analysis.

2-303 CHANNEL ROUGHNESS OR CONDUIT ROUGHNESS

Channel and conduit roughness usually have little effect on the predicted flood discharge unless the channel or conduit roughness is very high. This condition may exist in channels grown over with heavy trees and brush or in conduits considered to have a more significant effect on stream hydraulics where water surface elevations are important. Channel and conduit roughness does, however, have some effect on basin response time and should be considered in any hydrologic analysis. Reasonable estimates of channel roughness can be made by visual inspection of the channel or conduit being studied and compared to published roughness factor tables (Refer to Chapter 6 - Open Channels). An excellent source for determining natural channel roughness factor is Roughness Characteristics of Natural Channels (Geological Survey Water-Supply Paper 1849), which includes colored pictures of channel reaches, cross-sections of the reaches, and Manning's "n" values for the reach. Caution should be exercised to relate depth of flow to roughness when estimating "n" values.

2-400 FIELD INSPECTION CHECKLIST

A straight forward "Field Inspection Checklist", pages 2-11 to 2-12 is designed as a "reminder" to insure that the designer has proper equipment and information to adequately field analyze the drainage characteristics of the project. The form should be used only as a guide.

FIELD INSPECTION CHECKLIST

Job No. ____________________ Date ____________________

Job Name ____

Inspection by ____

1. Equipment
   - ☐ a. Measuring tape
   - ☐ b. 6 foot engineering rule
   - ☐ c. Engineer's scale
   - ☐ d. Hand level
   - ☐ e. Camera

2. Engineering Data
   - ☐ a. Quadrangle map
   - ☐ b. County and/or city maps
   - ☐ c. Aerial photographs
   - ☐ d. Construction plans
   - ☐ e. Information from other agencies
   - ☐ f. Regulatory floodway

3. Site Inspection
   - a. Briefly describe existing structures in floodplain (house, etc.)
     ____
     ____
   - ☐ b. Pictures
   - ☐ c. Review drainage area
   - ☐ d. Describe floodplain
     ____
     ____
   - ☐ e. Valley section
   - ☐ f. "n" values
     ____
     ____
   - ☐ g. Obstructions of floodflows (levees, flood control lakes and etc.)
     ____
     ____
   - ☐ h. Split flow or sheet flow
     ____
     ____

4. ☐ Interview local residents for historical high water
   ____

5. ☐ Complete encroachment decision model form

6. ☐ Meet with local officials

JOB NO. ____________ DATE ____________

LOCATION ________________________

STREAM EVALUATOR ________________

ENCROACHMENT DECISION MODEL

If the proposed project is one of the following: (New location, widening, reconstruction, bridges and approaches or overlay which is greater than 50% of replacement cost), circle the appropriate category and complete this form.

1. HAZARD TO PEOPLE:	Low	Moderate	Significant
a. ADT	100	100-750	greater than 750
b. Homes in base floodplain: upstream	0	1-5	greater than 5
downstream	0	1-5	greater than 5
c. Overtopping flood	less than 2%	2% to 10%	greater than 10%
d. Ponded depth before emergency evacuation required		less than 2.5'	greater than 2.5'
CIRCLE ASSESSMENT	Low	Moderate	Significant

2. HAZARD TO PROPERTY	Low	Moderate	Significant
a. Height of fill	less than 20'	20-50'	greater than 50'
b. Structure cost	less than $50K	$50-100K	greater than $100K
c. Adjacent property value	low	medium	high
d. Backwater: urban	less than 0.5'	0.5-1'	greater than 1'
rural	less than 2'	2'-2.5'	greater than 2.5'
CIRCLE ASSESSMENT	Low	Moderate	Significant

3. OTHER FACTORS		
a. Does either 1 or 2 contain a significant alternate?	No	Yes
b. Needed for emergency supply and evacuation route	No	Yes
c. Needed for emergency vehicle access	No	Yes
d. Lacks practicable detour	No	Yes

If any of the above is yes, make a detailed risk analysis, or justify why such an analysis is not required. The risk analysis should emphasis those variables indicated as significant.

Note: After example encroachment Risk Assessment furnished by Dan O'Conner, FHWA Region 6 Hydraulics Engineer 4-23-80.

2-500 DOCUMENTATION OF "UNUSUAL" HIGH WATER

This guide has been prepared for use by the Department in recording high water data that will form a permanent hydraulic inventory data base. By having a complete and thorough high water inventory, an accurate report can be compiled and arranged in a matter that would best suit the needs of the Department for maintenance and design of our highways.

Each District and/or Surveys Division survey party should assign experienced personnel to identify and evaluate high water marks for the Department's permanent records. This information is to be obtained in time of "UNUSUAL" high water, or at any time available from a reliable source. The elevations can be recorded by level reading, by measurement from bridge floor, in relation to a house, above roadbed or any other dependable and clear manner that fits the situation. The highest order possible should be used.

Form HYD 2-3, page 2-17, is to be used to record high water information. The back page of the form should be used to pictorially show pertinent information, i.e., sketch of roadway, structures, buildings, location of high water marks, and etc. This form should be forwarded to the Hydraulics Section upon completion for their permanent records. The form will provide historical information for the structures and roadway within the area of interest for any future use by the Department.

Reliable high water data can be invaluable information for establishing the stage and discharge of past floods, for locating existing hydraulic controls, and for establishing highway profiles.

It is extremely important that experienced personnel be used in identifying and evaluating high water marks. High water marks should be flagged and surveyed as soon as possible after a flood because they may disappear within weeks in heavy vegetated areas or additional rainfall may wash away the evidence.

It is also important to obtain and record high water information at culvert sites with spans 20 feet or greater. Also, high water marks should be recorded for inundated roadways that are isolated from hydraulic structures.

Information on high water elevations can usually be obtained by observing seed and mud lines on tree trunks and bridge abutments, wash-lines, and fine-debris lines on tree trunks and bridge approach fills, grass or hay lodged in tree limbs and fences, and evidence of erosion and scour. Interviews with residents, commercial and school bus drivers, mail carriers, law enforcement officers, railroad maintenance personnel and others who might have opportunity to observe the floods can also yield additional information.

2-501 INSTRUCTIONS FOR COMPLETING "FLOOD INFORMATION FORM"

Following are step-by-step instructions for completing the "Flood Information Form."

Items 1 to 7:	These items are self-explanatory.
Item 8:	Enter date of flood or unusual high water.
Item 9:	High water marks tend to disappear rapidly after the flood peak. For this reason, the work of surveying should begin as soon as possible. If a survey party is not available, locating the high water marks at the site before surveys is a necessity. Identify the marks by means of stakes, cloth tags, flagging, paint, paint sticks, nails, or crayon. Make field sketches showing the approximate locations of these marks for the benefit of the survey party. Enter the date and name of person who flagged the marks.
Item 10:	Enter the date and name of party chief who surveyed the marks. Because it is difficult to flag sufficient marks as described under Item 9, the field party should attempt to survey all additional marks necessary to define the profiles well. An assumed level datum referenced to the bridge type structure may be used if necessary.
Item 11:	If the bridge, bridge length culvert and/or roadway was overtopped (inundated), enter maximum stage elevation. If overtopping did not occur, enter N/A and proceed to Items 12 and 13.
Item 12:	Enter the upstream water surface level resulting from the contraction of the stream by bridge type structures. This elevation should be defined by high water marks along each bank of the stream or the upstream face of the embankment.
Item 13:	Enter the downstream water surface elevation. This is obtained on the downstream side of the embankment adjacent to the bridge type structure.
Item 14:	Enter the elevation of the lowest point of the bridge deck. Normally, this point should be surveyed at the center of bridge except in superelevated sections. Then the lowest point next to the bridge curb should be used.

Item 15: Briefly describe location of point surveyed and include in sketch on back of form.

Item 16: Enter estimated time of overtopping flood. This information may be obtained from a local resident and it will be helpful to rate stated information as "excellent," "good," "fair," or "poor."

Item 17: Enter the approximate 24-hour rainfall received during the day of "unusual" high water. This information is not necessary, but if an estimate is available from local people or news sources, please enter.

Item 18: This item is self-explanatory.

Item 19: Describe briefly the location of marks used for setting elevations for Items 11, 12, and 13.

Items 20 to 25: These items are self-explanatory.

Item 26: Enter any information here that will make the recorded information more clear. Describe any circumstance (in your opinion) that contributed to the flooding other than intense rainfall. You cannot make too many remarks.

Items 27 to 31: To be completed by the Hydraulics Section.

List the name of the person completing the form along with the date. On the back of the form please sketch the location of the stream in relation to the highway, bridge type structures, houses and buildings in the vicinity of the floodplain and sketch the location of survey marks as described under Items 11, 12, 13, and 15.

Upon completion, the form should be forwarded to:

Hydraulics Section
Room 603
P. O. Box 2261
Little Rock, Arkansas 72203

NOTE: Form HYD 2-3, page 2-19, has been hypothetically completed and is to be used as a guide.

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

FLOOD INFORMATION FORM

IDENTIFICATION

1) District _______ 2) County ______________ 3) Route _____ 4) Section _____
5) Log Mile _______ 6) Bridge Number _______ 7) Name of Stream ____________

PERTINENT DATA

8) Date flood occurred ________________
9) Date high water flagged ________________ By ________________
10) Date high water surveyed ________________ By ________________
11) Overtopping high water elevation ________
12) Upstream high water ____________ 13) Downstream high water __________
14) Bridge deck elevation (lowest point) ______________
15) Describe point surveyed under Item 14 ________________
____
16) Approximate duration of overtopping flood ________________
17) Approximate 24 hour rainfall ________________

GENERAL

18) High water mark obtained from ☐ actual water ☐ drift ☐ local resident
☐ Government Agency - Name ________________ ☐ other
19) Location of Marks (Describe briefly here and sketch location on back of this form.
19a) For Item 11 ________________
19b) For Item 12 ________________
19c) For Item 13 ________________
20) Description of marks, i.e., seed and mud lines on tree trunks and bridge abutments, wash lines and fine-debris lines on banks and bridge approach fills, whisps of grass or hay lodged in tree limbs and fences, evidence of erosion and scour, etc.
20a) For Item 11 ________________
20b) For Item 12 ________________
20c) For Item 13 ________________

STATED INFORMATION

21) Name of Observer ________________
22) Address ________________
23) Stage of observation (distance above low point in highway, distance below bridge deck, distance above floor elevation of house, etc.
____
24) Location of observer ________________
25) Observed frequency of occurrence (stream reaches this elevation, say, every two or three years or some estimated interval) ____________

REMARKS

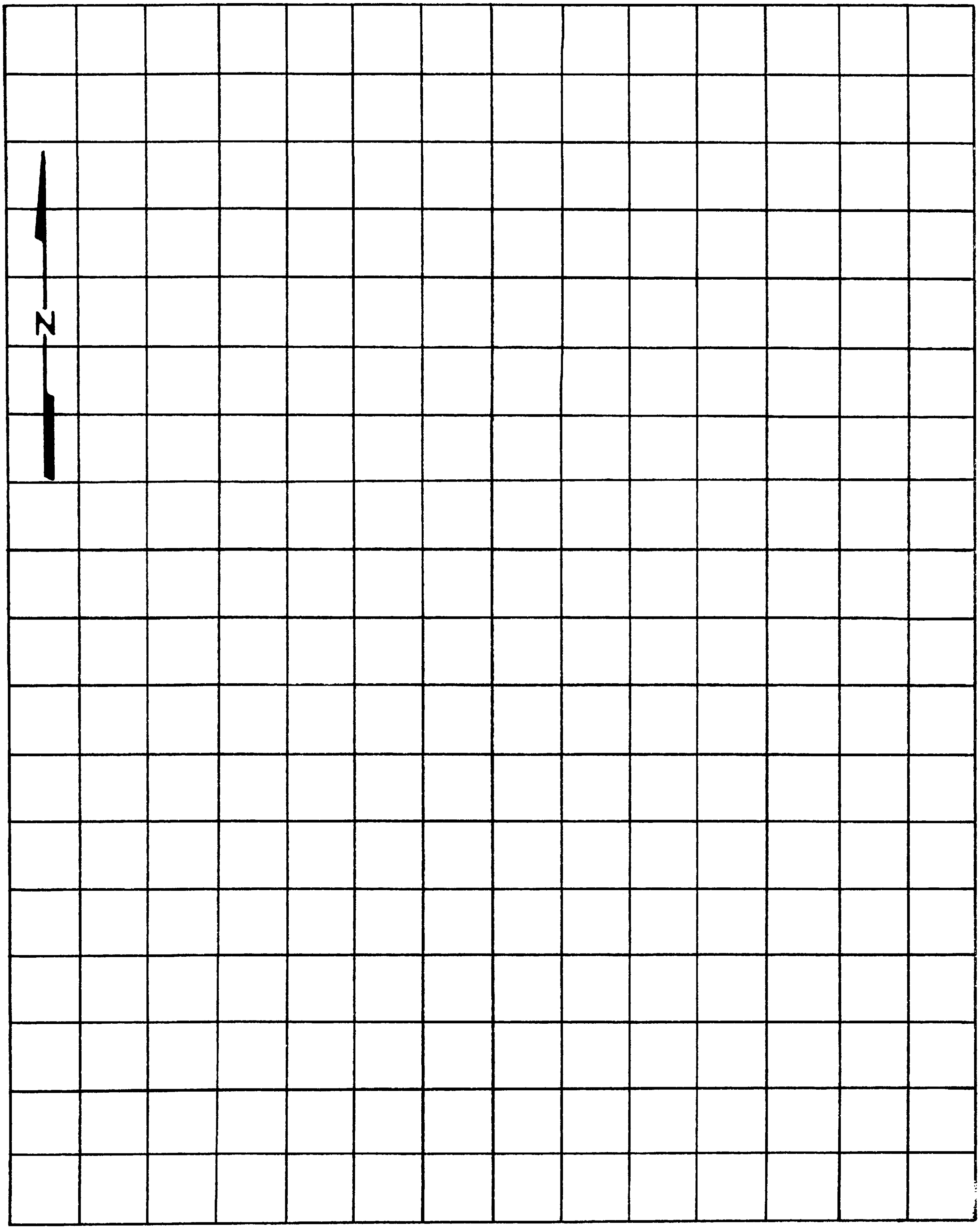
26) Include brief description of flood damage, unusual circumstances that contributed to flood, and any other remarks relative to flood action:
____
____
____
____

COMPLETED BY HYDRAULICS SECTION

27) Rainfall ____________
28) Information from rainfall recording station located at ____________
29) Rainfall frequency ____________
30) Stage from USGS gauging station ________________
31) Discharge ________________ cfs

FORM COMPLETED BY ________________ DATE ____________

(Locations referenced above to be sketched on back of this form)


N

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

FLOOD INFORMATION FORM

IDENTIFICATION

1) District 6 2) County GARLAND 3) Route 227 4) Section 0
5) Log Mile 6.90 6) Bridge Number M2413 7) Name of Stream HALLMAN'S CR.

PERTINENT DATA

8) Date flood occurred 12-3-82
9) Date high water flagged 12-5-82 By C.E. COMBS
10) Date high water surveyed 1-10-83 By K.H. LITTLE
11) Overtopping high water elevation 522.7
12) Upstream high water NA 13) Downstream high water NA
14) Bridge deck elevation (lowest point) 520.4
15) Describe point surveyed under Item 14 ℄ of deck @ north end of bridge
16) Approximate duration of overtopping flood 3 1/2 hours
17) Approximate 24 hour rainfall 5"

GENERAL

18) High water mark obtained from ☐ actual water ☒ drift ☐ local resident ☐ Government Agency - Name ______ ☐ other
19) Location of Marks (Describe briefly here and sketch location on back of this form.
19a) For Item 11 North end of bridge
19b) For Item 12
19c) For Item 13
20) Description of marks, i.e., seed and mud lines on tree trunks and bridge abutments, wash lines and fine-debris lines on banks and bridge approach fills, whisps of grass or hay lodged in tree limbs and fences, evidence of erosion and scour, etc.
20a) For Item 11 Fine seed line on bridge rail 2.3' above ℄ of deck
20b) For Item 12
20c) For Item 13

STATED INFORMATION

21) Name of Observer Otto Jones
22) Address Rt. 1, 92 Oak Rd., Hot Springs, Ar
23) Stage of observation (distance above low point in highway, distance below bridge deck, distance above floor elevation of house, etc.
6" above low point in highway
24) Location of observer Porch of residence 200' SE of bridge
25) Observed frequency of occurrence (stream reaches this elevation, say, every two or three years or some estimated interval) annually

REMARKS

26) Include brief description of flood damage, unusual circumstances that contributed to flood, and any other remarks relative to flood action:
Approx. 15' scour @ northern most pier

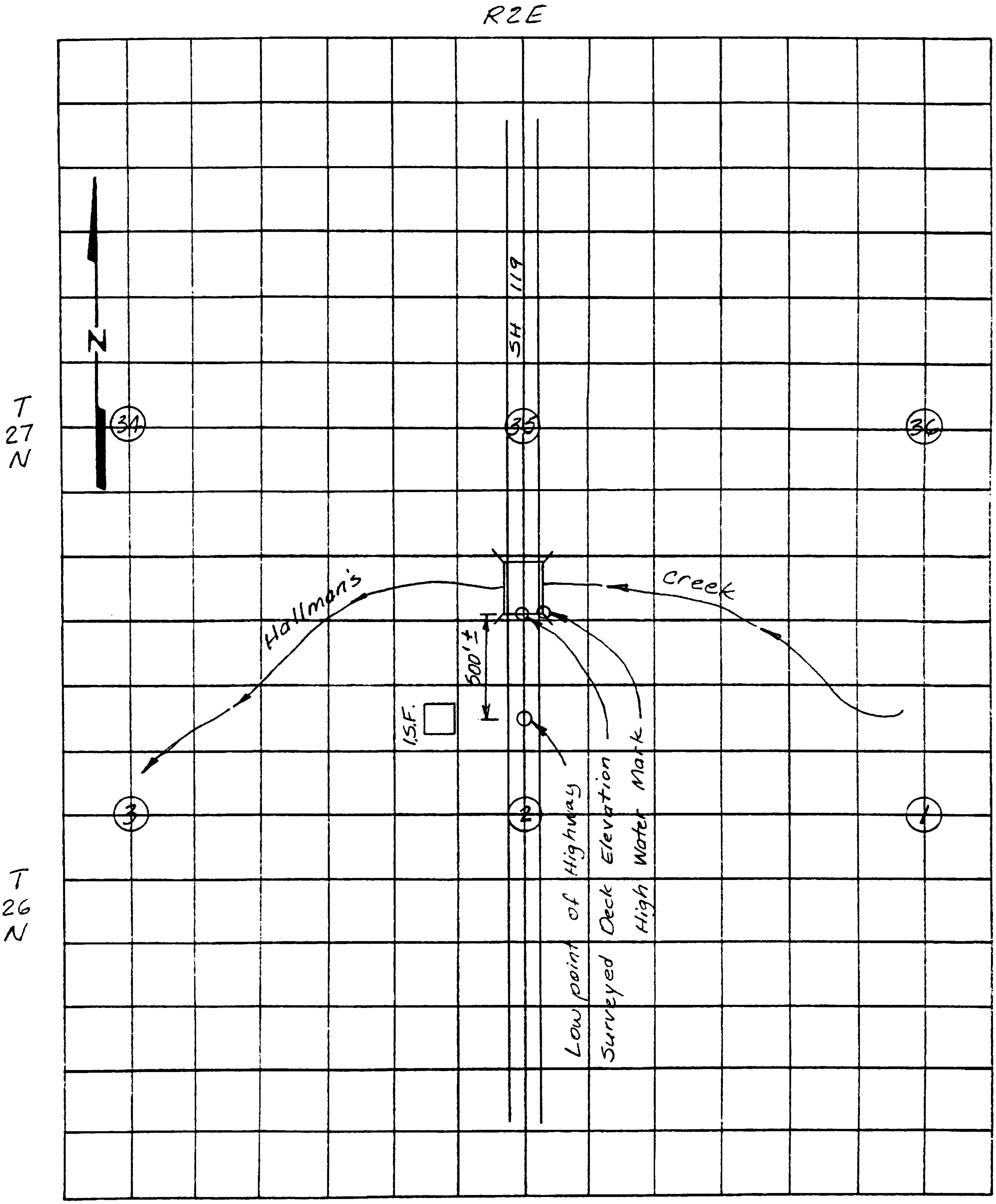
Mr. Jones stated the 6" overtopping under Item 23 occurred 12-25-82

COMPLETED BY HYDRAULICS SECTION

27) Rainfall
28) Information from rainfall recording station located at
29) Rainfall frequency
30) Stage from USGS gauging station
31) Discharge ______ cfs

FORM COMPLETED BY K. H. Little DATE 1/21/83

(Locations referenced above to be sketched on back of this form)


R2E
T
27
N
T
26
N
N
SH 119
34
35
36
3
2
1
Hollman's
Creek
500'±
I.S.F.
Low point of Highway
Surveyed Deck Elevation
High Water Mark

REFERENCES

Chapter 2

SURVEYS AND FIELD INFORMATION

2-1 HYDROLOGY FOR TRANSPORTATION ENGINEERS, Federal Highway Administration, Offices of Research and Development Washington, D.C., 1980

2-2 MANUAL FOR PRELIMINARY ENGINEERING SURVEYS, Arkansas State Highway and Transportation Department, Little Rock, Arkansas, 1978.

2-3 ARKANSAS HIGHWAY DEPARTMENT DRAINAGE MANUAL, Arkansas State Highway and Transportation Department, Little Rock, Arkansas, 1967.

2-4 HANDBOOK OF HYDRAULICS, by Horace Williams King and Ernest F. Brater, McGraw-Hill Book Company, Fifth Addition, 1963.

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Chapter 3

HYDROLOGY

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3-100 INTRODUCTION

Hydrology is that branch of the applied earth sciences that deals with water on and under the earth and in the atmosphere. For the purpose of this Manual, hydrology will deal with estimating storm water runoff for different design frequencies as the result of rainfall. Various hydrologic methods can be used to determine storm water runoff, which is usually considered as a peak runoff or discharge in cubic feet per second (cfs).

3-200 FACTORS AFFECTING RUNOFF

Factors affecting the hydrological analysis in determining storm water runoff have many variables that need to be closely scrutinized on a site by site basis.

Some of these factors are:

a) Climatic Factors

1. Precipitation: From rain, snow, frost, etc., type, intensity, duration, time distribution, aerial distribution, frequency of occurrence and antecedent precipitation.

2. Interception: Vegetation species, composition, age, season and density of stand.

3. Evaporation: Temperature, wind and shape of evaporative surface.

4. Transportation: Temperature, solar radiation, wind, humidity, soil moisture and kinds of vegetation.

b) Physiographic Factors

1. Basin characteristics
   a. Geometric factors: Size, shape, slope, elevation.

b. Physical factors: Land use and cover, surface infiltration condition, soil type, presence of lakes, swamps and artificial drainage.

2. Channel characteristics

a. Carrying capacity: Size and shape of cross sections, slope, roughness, length and tributaries.

b. Storage capacity: Backwater effect.

The distinction between large and small drainage areas should not always be measured in area, but also, on the basis of hydrologic behaviors and the effects of certain dominating factors. One drainage basin may show prominent channel storage effects, like most large basins, while the other may manifest a strong influence of land use, like most small basins. A distinct characteristic of small basins is that the effect of overland flow rather than the effect of channel flow is a dominant factor affecting peak runoff. Also, small basins are very sensitive both to high-intensity rainfalls of short duration and to land use. In large basins, the effect of channel storage is so pronounced that such sensitivities are greatly suppressed. Therefore, a small drainage basin may be defined as one that is so small that its sensitivity to high-intensity rainfalls of short durations and to land use is not suppressed by channel characteristics.

3-300 DESIGN FREQUENCIES AND GOVERNING FACTORS

The extent of damage and the hazard and inconvience to the highway user due to the hydraulic design of a structure are difficult to predict and measure. They vary with the amount by which capacity of the structure is exceeded and the frequency with which such events occur. Since high runoff, like excessive rainfall, occurs at random or haphazard intervals, it is desirable to consider floods or peak runoff on a frequency basis.

The Department has established recommended storm design frequency criteria, as shown in Chapter 1, based on roadway system classification and type of structure.

It is important to realize what is meant by a 10, 25, 50 or 100-year frequency storm. The storm frequency (recurrence interval or return period) may be defined as the average interval of time within which the given flood will be equaled or exceeded once.

A 100-year storm has a 1 to 100 chance, or a 1 percent chance, of occurring at any particular time. A 50-year storm has a 2 percent chance of occurring at any particular time. Hence, a 25- or 10-year storm has a 4 percent or 10 percent chance, respectively, of occurring at any particular time.

The intensity of a storm is measured in inches per hour. The intensity of a 50-year storm will be much greater than the intensity of a 10-year storm in the same location. The intensity of a storm will vary with its duration, other factors being similar. The average rate of rainfall for a storm 10 minutes long will be much greater than the average rate of rainfall for a storm one hour long of comparable frequency, but the total amount of rain that falls will not vary in the same proportion.

In determining the design frequency, the recommended storm design frequency criteria table in Chapter 1 should not be considered a rigid guideline. Consideration should also be given to all significant impacts and risks involved.

Recognizing that floods cannot be predicted and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage should the design flood be exceeded. Design headwater or backwater and flood frequency criteria should be based upon these and other considerations:

1. Damage to adjacent property.

2. Damage to the structure and roadway
3. Traffic Interruption
4. Hazard to human life.
5. Damage to stream and flood plain environment as described in Volume 6, Chapter 7, Section 3, Subsection 2, of the Federal Highway Administration's Highway Program Manual (For instructional guidelines relating to design criteria and documentation of FHWA transmittal 315, FHPM 6-7-3-2, see Chapter I and VIII).

Potential damage to adjacent property, structures, roadways, and inconvenience to others, should be of major concern in the design of all hydraulic structures.

3-400 CALCULATING DESIGN DISCHARGE

Of all the phases of drainage design, one of the most difficult is that of determining design discharges. There are several methods for determining discharge and it is not unusual to obtain significantly different results from two or more different methods. It must be recognized that hydrology is not an exact science and that in determining a design discharge, sound judgment coupled with experience is a most useful tool. There is no single method for determining peak discharge that is applicable to all watersheds. It is the designer's responsibility to examine all methods that can apply to a particular site and to make the decision as to which discharge is the most applicable. Consequently the designer must be familiar with general sources of the various methods and their applications and limitations. Table 3-1, page 3-10 summarizes the characteristics and applications of some of the methods available. It can be concluded that the method chosen is a function of the size of the drainage area, availability of data and the degree of accuracy desired. It is not the intent of this manual to serve as a text for all the various methods of determining discharge.

After reviewing practices in different states, it became clear that two general procedures were in use across the country for calculating runoff or discharge from small watersheds.

1. Rational Method (to be used primarily for drainage areas less than 200 acres.)

2. Soil Conservation Service Method (using Technical Release No. 55 and SCS-TP-146). This method is appropriate for watersheds of 2000 acres or less subject to the limitation noted in the publications. It is particularly useful in determining the affects of urbanization in a watershed.

And one general procedure was in use for large watersheds.

1. Multiple Regression Analysis. This approach uses the U.S. Geological Survey's "Floods in Arkansas, Magnitube and Frequency" by James L. Patterson. James L. Patterson.

The before mentioned methods are recommended and are included in summary form to depict a sampling of the various methods available. It should be noted that in all cases "Floods in Arkansas, Magnitude and Frequency" (Ref. 1) should be consulted for locations of gaging stations in the vicinity of the area in question. Also, drainage areas of most streams in Arkansas can be obtained using the USGS publications of the different river basins in the state.

<u>Arkansas River Basins</u>

1. Ouachita River Basin
2. Arkansas River Basin
3. Red River Basin
4. White River Basin
5. St. Francis River Basin

TABLE 3-1

RUNOFF ESTIMATION METHODS

Method	Size of Drainage Area	Required Information	Variables	Output	Applications
Rational* Method	200 acres or less	Land cover T_c Area	Runoff Coefficient C	Peak flow	Minor structure design that does not require great accuracy
SCS* Graphical Method	2000 acres or less	Soil Type Cumulative Rainfall	Runoff Curve No. (CN) Accounts for Hydrological Abstraction	Peak flow	Minor structure design that does not require great accuracy
Multiple* Regression Analysis	3000 square mile or less	Area Slope	Equation Exponents	Peak flow	Minor and major structures rural areas
Unit Hydrograph	1000 square mile or less (only daily rainfall and average daily discharge) up to 5000 square mile (extensive records required	Rainfall and streamflow records		Hydrograph	Flood flows major systems storage volume
SCS Unit Hydro-Graph	20 square mile or less	Soil Type Rainfall T_c	Runoff Curve No. (CN)	Hydrograph	Flood flows major systems storage volume
Cook Method	200 acres or less	Land cover Annual precipitation	Rainfall distribution factor, watershed characteristics	Peak flow	Minor structure design, that does not require great accuracy

* Methods covered in this chapter.

3-401 - THE RATIONAL METHOD

The most common means of determining runoff for minor hydraulic structures is the rational formula:

$$Q = CIA$$

where

Q = peak discharge, cfs

C = a coefficient representing the ratio of runoff to rainfall (related to impervious area i.e., 1.0 = 100% runoff).

I = rainfall intensity, in./hr.

A = drainage area in acres

The assumptions inherent in the rational formula are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during a period of time equal to the time of concentration.

2. The maximum rate of rainfall occurs during the time of concentration and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow, which is defined as the time of concentration (T_c).

Since these assumptions apply reasonably well for urbanized areas with drainage facilities of fixed dimensions and hydraulic characteristics, the rational formula has gained widespread use in the design of drainage systems for these areas. Its simplicity and ease of application have resulted in its being used in rural areas where these assumptions are not as applicable.

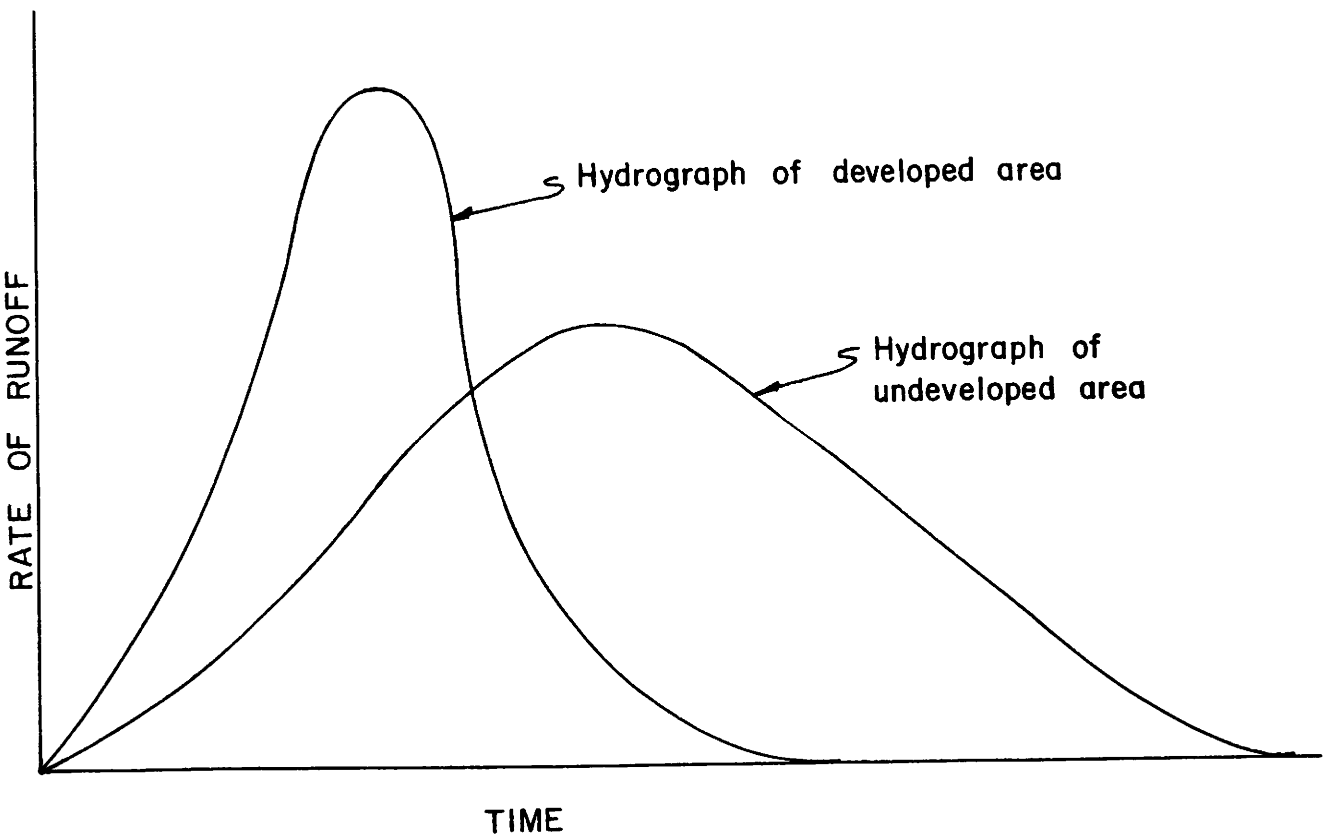
The rational formula has certain limitations, the main one being that is most applicable to areas under 200 acres. It is criticized for expressing runoff as a fraction of rainfall rather than as rainfall minus losses and for combining all complex factors that affect runoff into a single coefficient.

While these and similar criticisms are valid, use of a more complicated formula for small drainage areas is not justified, because the time and expense to obtain the necessary data is not usually warranted. This is a judgment the engineer has to make when extenuating circumstances and risks are involved.

3-401.1 EFFECTS OF URBANIZING A WATERSHED

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall. The hydraulic efficiency of a drainage area is generally improved by urbanization which in effect reduces that storage capacity of a watershed. This reduction of a watershed's storage capacity is a direct result of the elimination of porous surfaces, small ponds and holding areas. This comes about by the grading and paving of building sites, streets, drives, parking lots and sidewalks and by constructing buildings and other facilities characteristic of urban development. The result of improved hydraulic efficiency is illustrated graphically in Figure, 3-1, page 3-13, which is a plot of the runoff rate versus time for the same storm with two different stages of watershed development.

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



EFFECTS OF WATERSHED DEVELOPMENT ON STORM HYDROGRAPH

FIGURE 3-1

3-401.2 DISCUSSION AND EXAMPLES ON THE USE OF THE RATIONAL METHOD

The runoff coefficeint "C" should be determined by field observations of the terrain ground cover and soils, and by the use of Table 3-2, page 3-18.

Where the drainage area is composed of several types of runoff surfaces the runoff coefficeint should be weighted according to the area of each type of runoff surface present.

The following example will indicate the procedure in computing a weighted runoff coefficient:

AREA (acres)	TYPE OF SURFACE	C	CA
7.5	Flat pasture, pervious	.20	1.50
15.5	Rolling woodland, pervious	.15	2.32
9.5	Paved	.90	8.55
32.5			12.37

$$\text{Weighted } C = \frac{CA}{A} = \frac{12.37}{32.5} = .38$$

A thorough study by the University of Maryland (Ref. 6), found that the soundest, most realistic formula for overland flow time of concentration T_c was the following kinematic wave equation:

$$T_c = \frac{K L_o^{0.6} n^{0.6}}{i^{0.4} S_o^{0.3}}$$

where

T_c = time of concentration in seconds

L_o = the overland flow length in feet

n = the Manning roughness coefficient

i = The rainfall rate in inches per hour

S_o = the overland flow slope in feet per foot

K = 56 for English units

The kinematic wave theory nomograph, Figure 3-2, page 3-19, is consistent with the latest concepts of fluid mechanics and considers all those parameters found important in overland flow when the flow is turbulent (where the product of the rainfall intensity and length of the slope is in excess of 500).

In using the nomograph the designer has two unknowns as the time of concentration and the associated rainfall computations are started. The problem is one of iteration of trial and error. A value for i must be assumed and Fig. 3-2, will give a related time of concentration. The assumed rainfall intensity must then be checked against the rainfall-intensity-duration curve for the frequency of recurrence interval chosen for the particular design problem. If the assumed intensity and that imposed by the frequency curve do not compare favorably, a new rainfall intensity must be assumed and the process repeated. Perhaps this procedure is best illustrated by the example on page 3-16.

Rainfall intensity-duration-frequency (I-D-F) curves for different areas of the state are provided beginning with Figure 3-4 through Figure 3-7, page 3-21. These curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a certain return frequency from a series of storms. These curves are then said to represent storms of a specific return frequency. Intensity, or rate or rainfall is usually expressed in depth per unit of time with the highest intensities occuring over short time intervals and progressively decreasing as the time intervals increase. The greater the intensity of the storm the lesser its recurrence frequency. Thus, the highest intensity for a specific duration of n years of records, is called the n year storm, with a frequency of once in n years. It should be noted that the I.D.F. curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of n frequency. Rainfall intensity data is taken from the U.S. Weather Bureau Atlas, Technical Paper 40.

PROCEDURE FOR DETERMINING "Q" (discharge in cfs)

The drainage area "A" can be determined in the field or from topographic maps. "Q" (discharge in cfs) can then be determined by multiplying "C" x "I" x "A", which is used to size structures.

Example 1

A development located in Area I Fig. 3-3 , page 3-20 has the following characteristics, L_o = 800 feet, S_o = .015 and n = 0.06. Find i (Rainfall intensity) and T_c (time of concentration) for a 25-year frequency storm.

Assume i is 5.0 iph: From Fig. 3-2 T_c = 17.6
From Fig. 3-4 T_c = 23.0
Assume i is 6.5 iph: From Fig. 3-2 T_c = 15.8
From Fig. 3-4 T_c = 12.0

Assume i is 5.6 iph: From Fig. 3-2 T_c = 16.8
From Fig. 3-4 T_c = 17.0

Use T_c = 16.8 Min. and I = 5.6 iph.

Example 2

Find peak discharge (Q) for 10, 25 and 100-year frequencies.

Location

Fort Smith, Arkansas (Area IV, Fig. 3-7)

Drainage Area	Area Acres	Runoff Coef. "C"	Area x C
Commercial	2	.95	1.9
Office/Mics., Commercial	17	.85	14.5
Residential	53	.45	23.9
Park	62	.25	15.5
	134		55.8

Average "C" value: $\frac{55.8}{134} = .47$

Length of Watershed = 820 feet

Elevation Difference 1164-1148 = 16 feet

Predominantly Open Grassed Channel n = .06

Slope = .0195 ft./ft.

Runoff (Rational Method

Time of concentration (T_c) is determined by trial and error, using Fig. 3-2, or the following equation and the procedure previously described.

Discharge Q = CIA

$$T_c = \frac{56\ L_o^{.6} n^{.6}}{i^{.4} S_o^{.3}}$$

$$T_c = \frac{56\ (820)^{.6}\ (.06)^{.6}}{60\ (i)^{.4}\ (.0195)^{.3}} = \frac{31.47}{(i)^{.4}}$$

	Frequency (Years)			
	10	25	50	100
T_c, Minutes	15.8	15.3	14.4	13.7
i, In./Hr.	5.3	6.0	7.0	7.9
C	0.41	0.41	.41	.41
A, Acres	134	134	134	134
Q, cfs	302	329	385	434

TABLE 3-2

RUNOFF COEFFICIENT VALUES (C)

for formula Q = C x I x A in

The Rational Method

(Drainage Areas 200 Acres or Less)

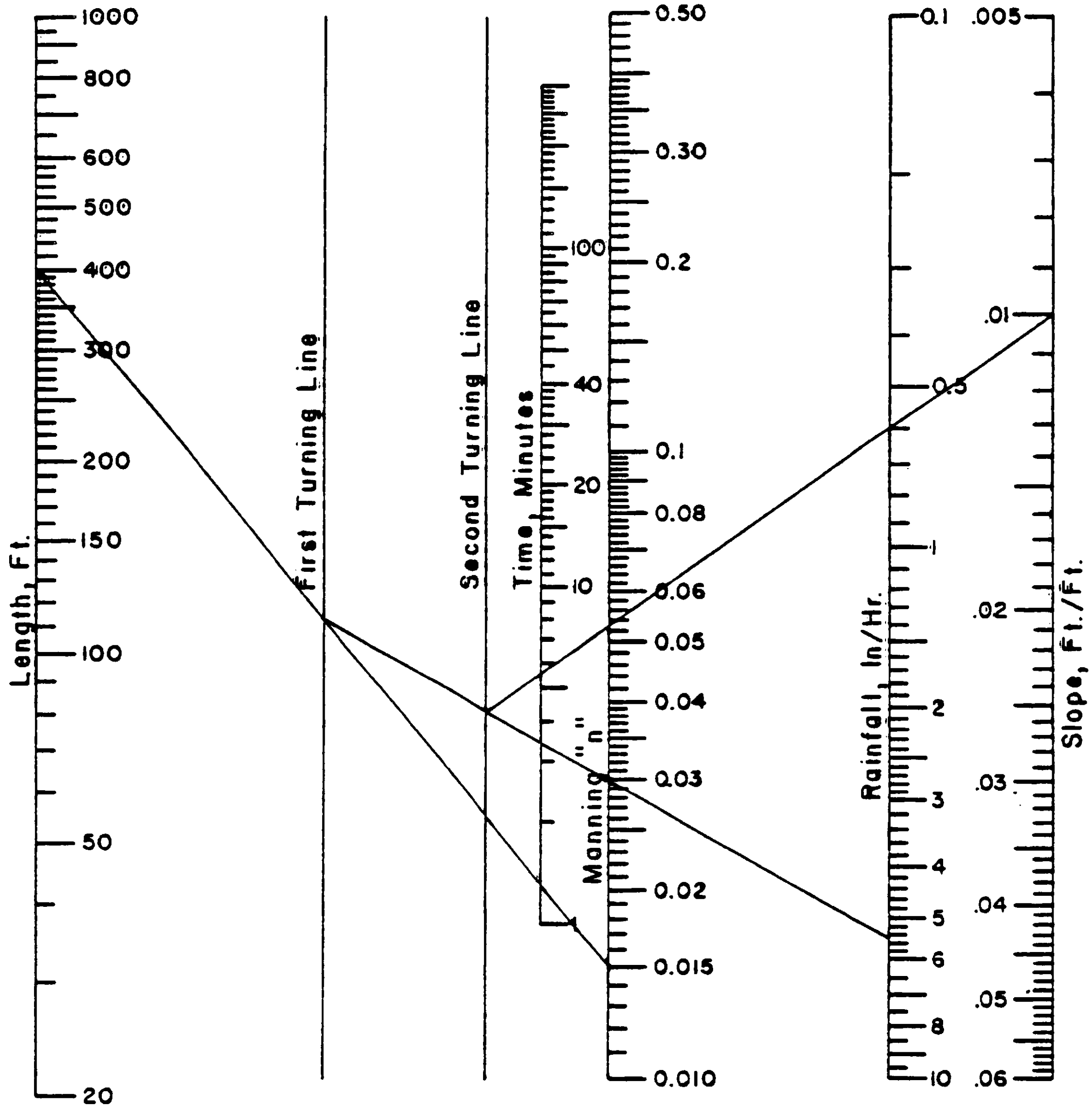
Slope	Land Use	Soil Classification: Sand or Sandy Loam Soils (Pervious) Min.	Sand or Sandy Loam Soils (Pervious) Max.	High Clay Soils (Impervious) Min.	High Clay Soils (Impervious) Max.
Flat (0% - 1%)	Cultivated	0.25	0.35	0.30	0.40
	Woodlands	0.15	0.20		
	Pasture	0.20	0.25		
	Paved	0.90		0.90	
	Residential	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90
Rolling (1% - 3.5%)	Cultivated	0.45	0.65	0.50	0.70
	Woodlands	0.15	0.20	0.18	0.25
	Pasture	0.30	0.40	0.35	0.45
	Paved	0.90		0.90	
	Residential	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90
Hilly (3.5% - 5.5%)	Cultivated	0.60	0.75	0-70	0.85
	Woodlands	0.20	0.25	0.25	0.30
	Pasture	0.35	0.45	0.45	0.55
	Paved	0.90		0.90	
	Residential	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90
Mountainous (over 5.5%)	Woodlands			0.70	0.80
	Bare			0.80	0.90
Grassed ROW Slopes		0.70		0.70	

Revised 5-16-83

FIG. 3-2

Equation solved by nomograph:

$$t_c\ (\text{Sec}) = 56\,\frac{L_o^{.6}\ n^{.6}}{i^{.4}\ S_o^{.3}}$$



The initially assumed value of i and the nomograph value of t must be checked against the applicable intensity-duration-frequency curve by trial and error.

Example:

L_o = 400 ft.
n = 0.015
i = 5.5 in./hr.
S_o = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Recommended Manning "n" values:

Paved Surface	0.013-0.016
Turf	0.030-0.050

For other values, refer to Table 6-1, page 6-15.

Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. *(After Ragan.)*

RAINFALL INTENSITY AREAS

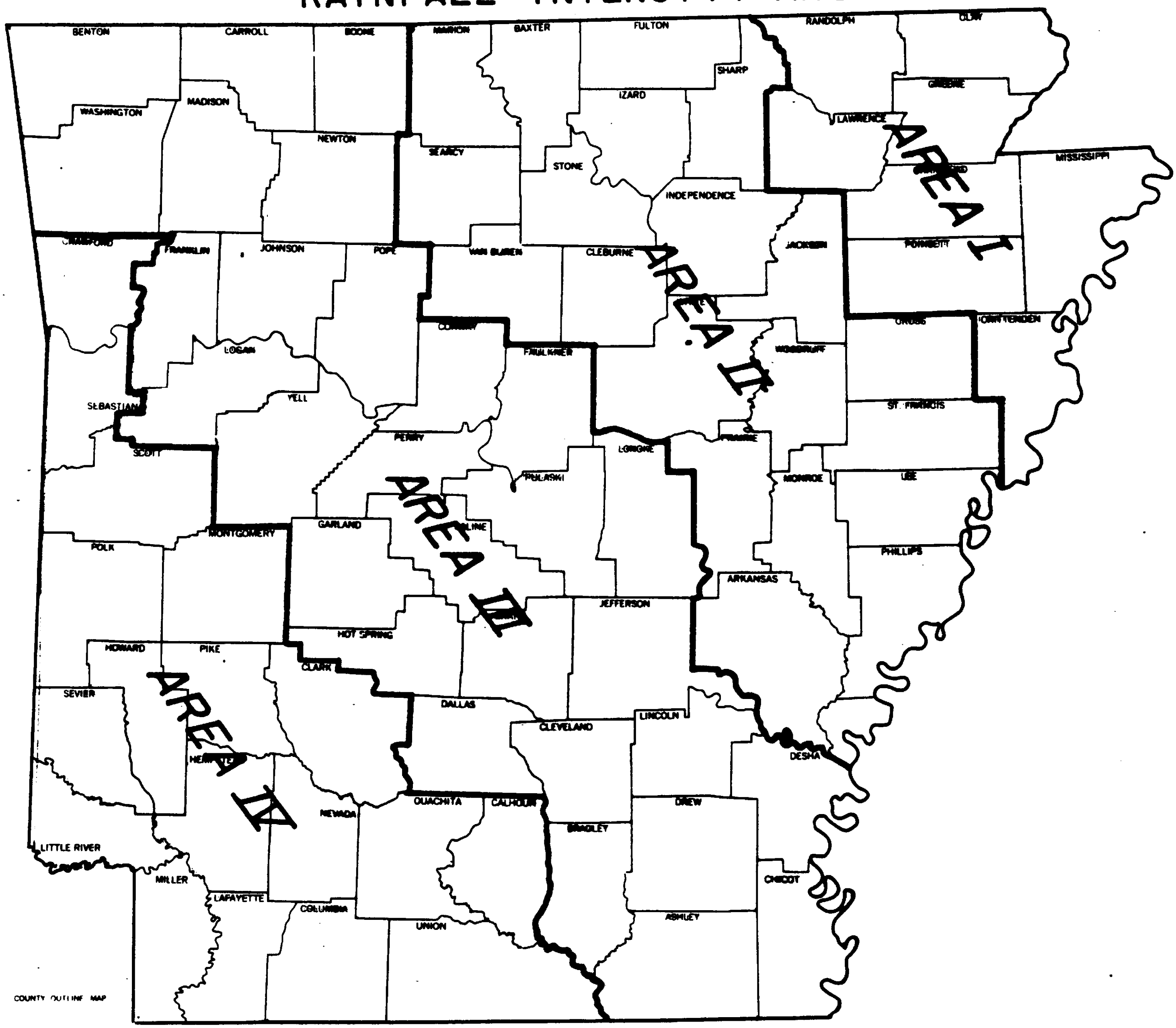


Fig. 3-4

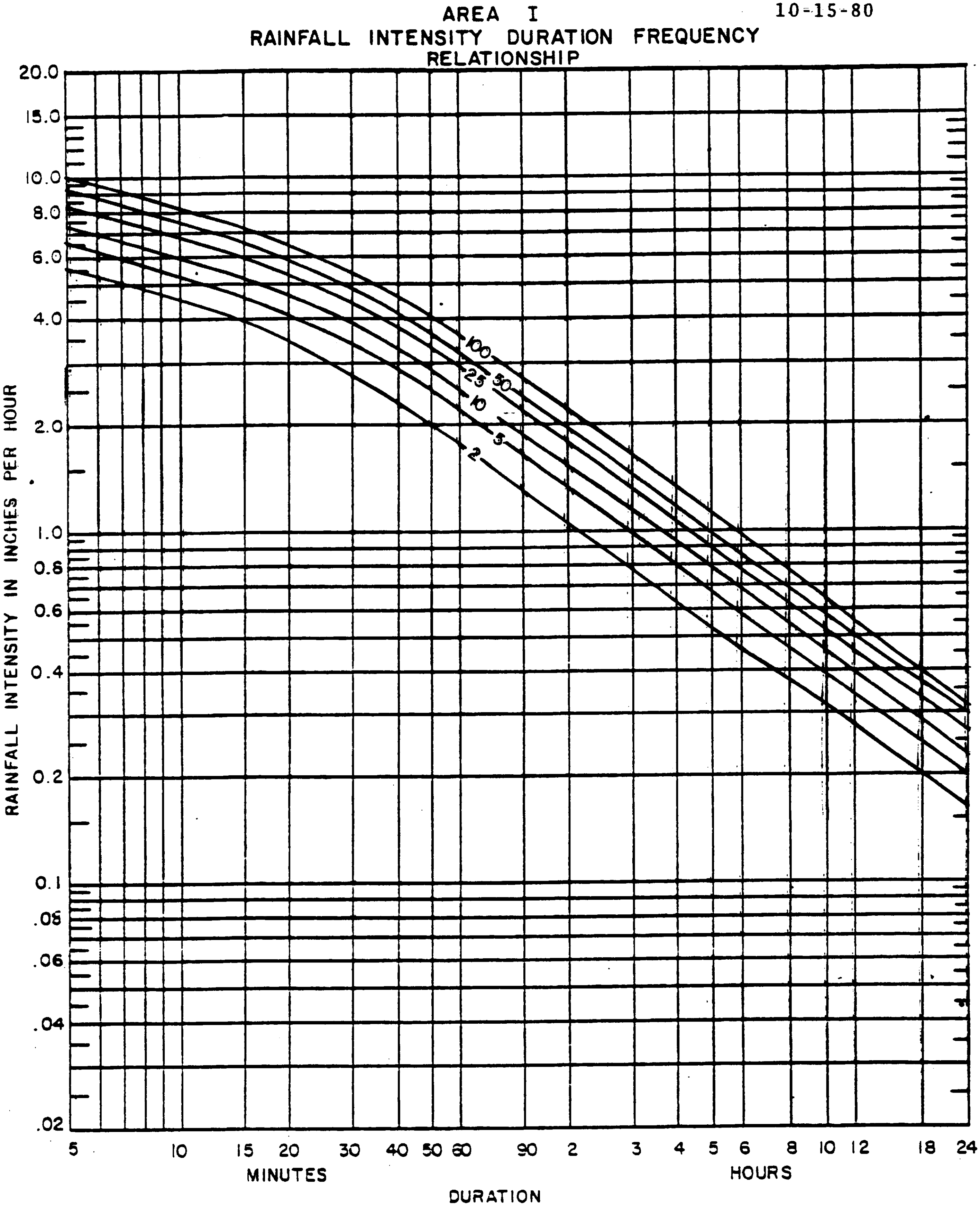

AREA I
10-15-80
RAINFALL INTENSITY DURATION FREQUENCY RELATIONSHIP
RAINFALL INTENSITY IN INCHES PER HOUR
20.0
15.0
10.0
8.0
6.0
4.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
.08
.06
.04
.02
100
50
25
10
5
2
5
10
15
20
30
40
50
60
90
2
3
4
5
6
8
10
12
18
24
MINUTES
HOURS
DURATION

Fig. 3-5

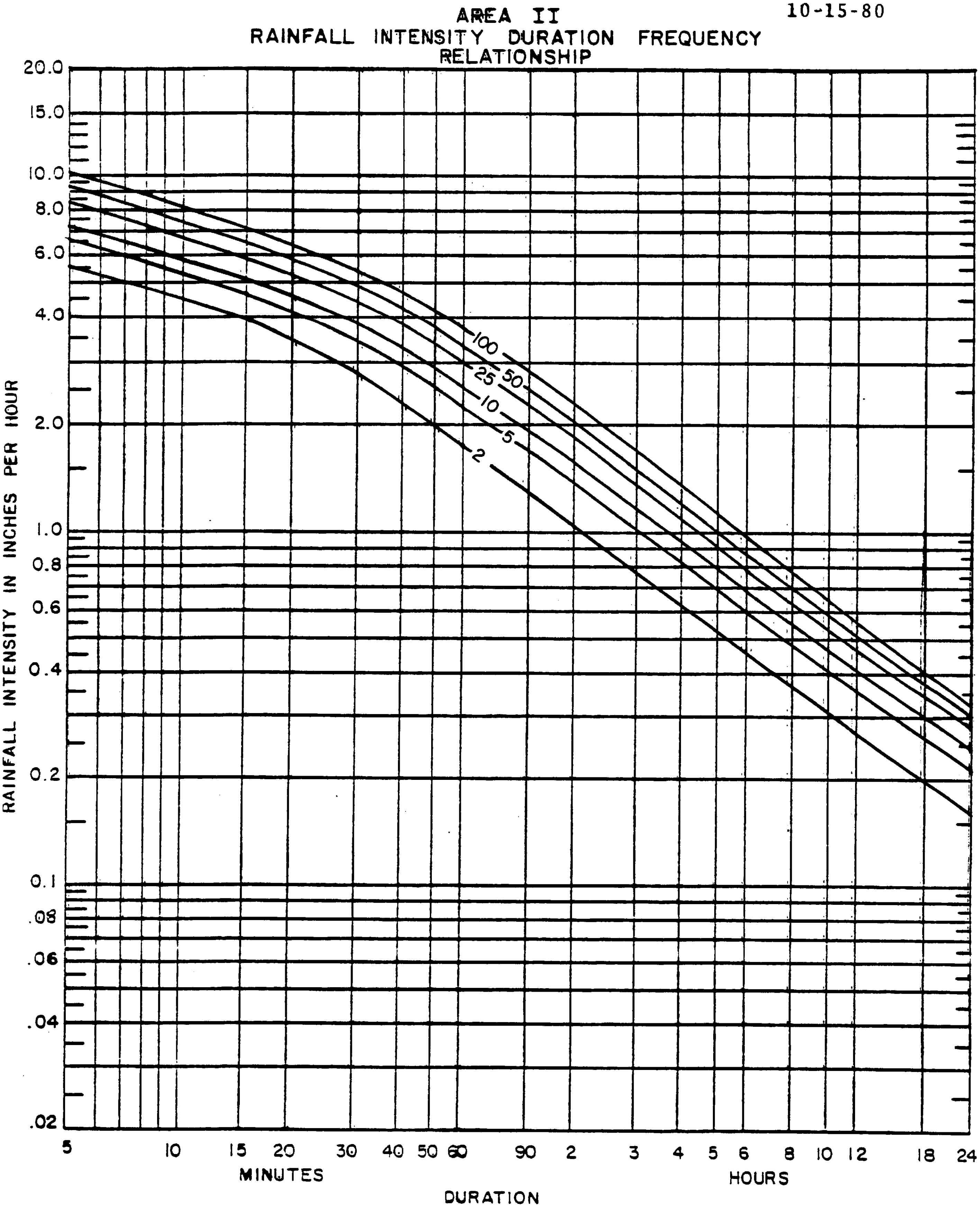
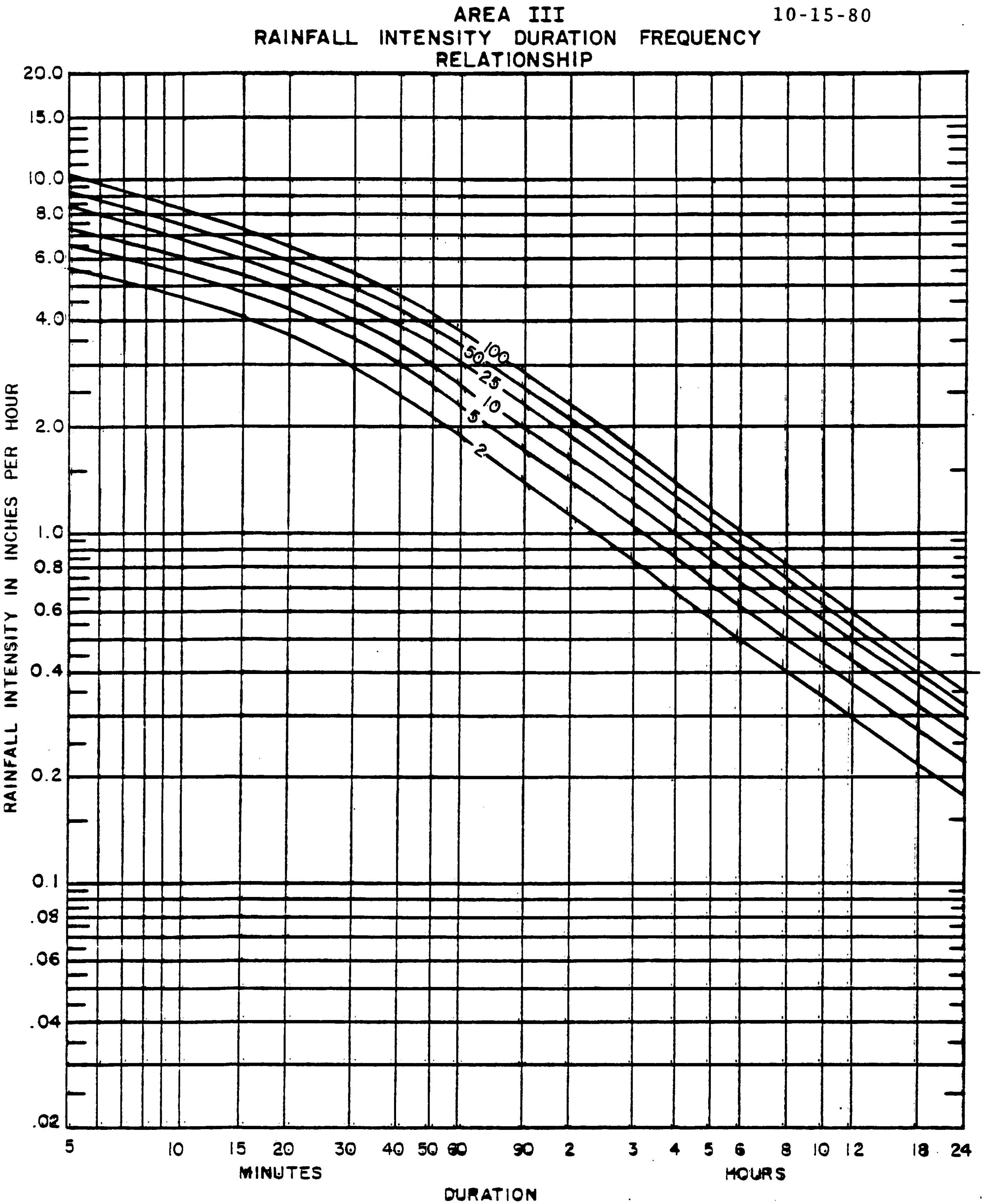
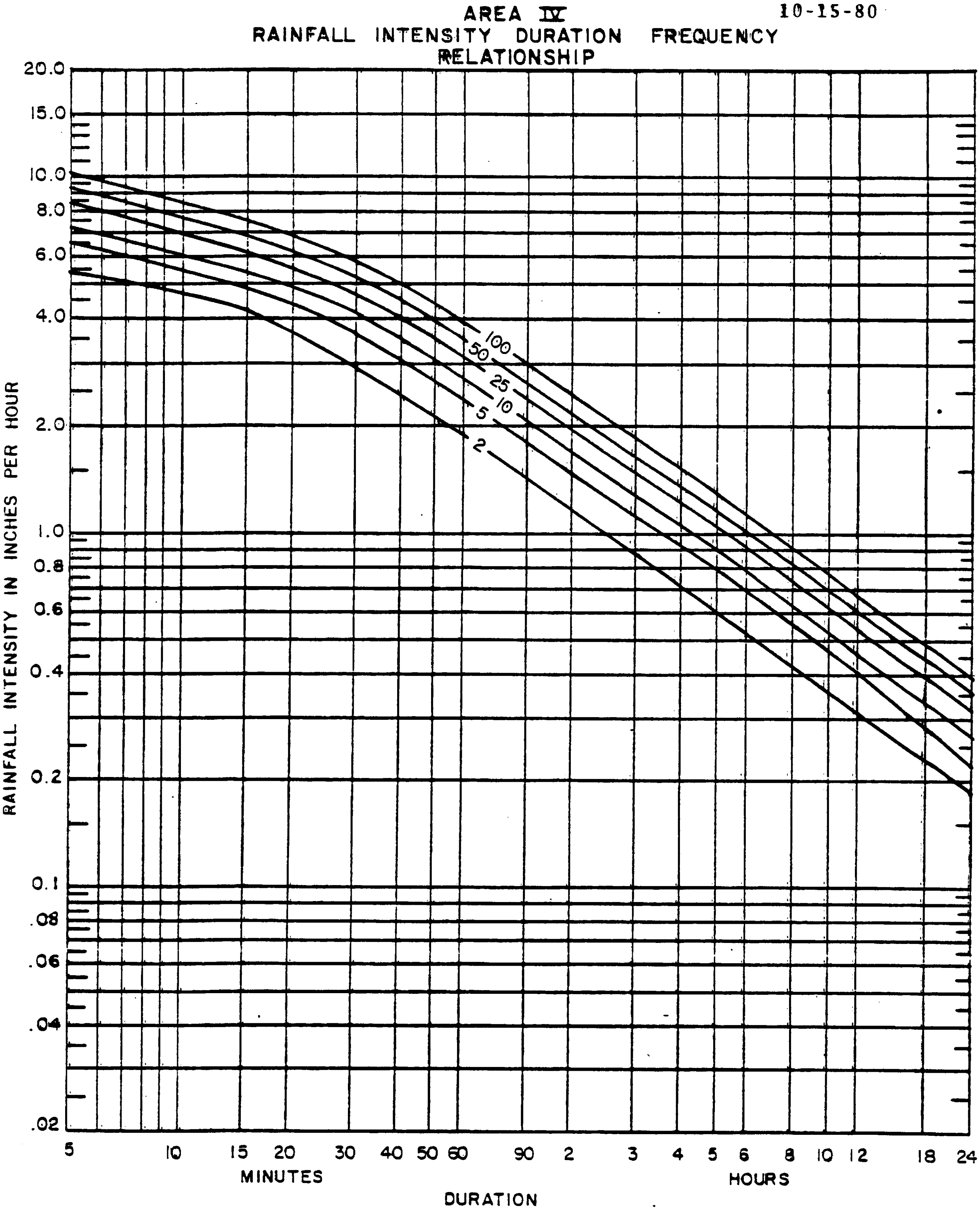

AREA II
10-15-80
RAINFALL INTENSITY DURATION FREQUENCY RELATIONSHIP
RAINFALL INTENSITY IN INCHES PER HOUR
20.0
15.0
10.0
8.0
6.0
4.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
.08
.06
.04
.02
100
50
25
10
5
2
5
10
15
20
30
40
50
60
90
2
3
4
5
6
8
10
12
18
24
MINUTES
HOURS
DURATION

Fig. 3-6



AREA III
10-15-80
RAINFALL INTENSITY DURATION FREQUENCY RELATIONSHIP
RAINFALL INTENSITY IN INCHES PER HOUR
20.0
15.0
10.0
8.0
6.0
4.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
.08
.06
.04
.02
100
50
25
10
5
2
5
10
15
20
30
40
50
60
90
2
3
4
5
6
8
10
12
18
24
MINUTES
HOURS
DURATION

Fig. 3-7


AREA IV
10-15-80
RAINFALL INTENSITY DURATION FREQUENCY RELATIONSHIP
RAINFALL INTENSITY IN INCHES PER HOUR
20.0
15.0
10.0
8.0
6.0
4.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
.08
.06
.04
.02
100
50
25
10
5
2
5
10
15
20
30
40
50
60
90
2
3
4
5
6
8
10
12
18
24
MINUTES
HOURS
DURATION

3-402 SOIL CONSERVATION SERVICE METHOD

The following discussion of the SCS method established procedures for estimation peak rates of runoff from small watersheds for use in designing highway drainage structures when there is no stream flow data available. The procedures are applicable for rural and urban area of less than 2,000 acres and for estimating the effects of land use changes on hydraulic and hydrologic parameters, runoff volume and peak rates of discharge. The results should be compared with other available methods and engineering judgment should be used in arriving at a final estimate. For example, the rational method can be used and compared with the SCS method up to 200 acres and the SCS method can be used and compared with the Multiple Regression Approach up to 2,000 acres. (The Multiple Regression Approach is discussed later in this chapter).

The SCS method uses three main variables to estimate rainfall runoff during a given event. These variables are rainfall, antecedent moisture condition and hydraulic soil cover complex. There are also other variables that can be applied if a largely urban area is being studied or planned.

The runoff equation used by the SCS is a relationship between accumulated rainfall and accumulated runoff and was derived from experimental plots for numerous soils and vegatative cover conditions.

The equation is:

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$

where

Q = accumulated direct runoff

P = accumulated rainfall (potential maximum runoff)

Ia = initial abstraction including surface storage, interception and infiltration prior to runoff.

S = potential maximum retention

The relationship between Ia and S was developed from experimental watershed data. The empirical relationship used in the SCS runoff equation is:

$$Ia = 0.2S$$

Substituting 0.2S for Ia in the runoff equation:

$$Q = \frac{(P-0.2S)^2}{P + .8S}$$

The Soil Conservation Service has made extensive experiments and analyses of water data to determine the best way to relate the variable S to the soil water storage and the infiltration rates of a watershed. The method adopted is the curve-number (CN) technique. This is simply a method of combining the properties of soil groups in the watershed with both the land use and treatment classes, and antecedent moisture conditions.

The variable S is related to the CN by the following relationship:

$$S = \frac{1000 - 10\ CN}{CN}$$

The SCS technique is a useful and reliable method of representing infiltration characteristics of a watershed. Once the CN is obtained, Figure 3-8, page 3-36, can be used to determine rainfall excess in inches. The first step in arriving at the curve number is to determine the SCS soils group.

The reader is encouraged to review related references for details of the SCS method (Referenced 2, 3 and 4).

Determination of the Hydrologic Soil Group. The major soil groups are defined for the estimated watershed soil conditions. The groups, as defined by SCS, are:

Group A. (Low runoff potential.) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well-to excessively-drained sands or gravels. These soils have a high rate of water transmission, in that water readily passes through them.

Group B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well-drained to well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

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

Group D. (High runoff potential.) Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

The SCS has published county soil survey books for most counties in the State which contain the hydrologic soil group. These counties are shown in Figure 3-9, page 3-37. In area where there are no soil survey books, the soil group can be determined by using general soil maps, engineering judgment and field observations. The Curve Number (CN) can be derived by using Table 3-3 or 3-4. Table 3-7 contains the names of different soils in Arkansas and their Hydrologic Soil Group.

In the SCS method, the change is S (actually in CN) is based on an antecedent moisture condition (AMC) determined by total rainfall in the 5-day period preceding a storm. Three levels of AMC are used: AMC-I is the lower limit of moisture or the upper limit of S, AMC-II is the average and AMC-III is the upper limit of moisture or the lower limit of S. Comparisons of computed and actual runoffs show that for most problems the extreme AMC can be ignored and the average AMC (AMC-II) should be used. Table 3-5, page 3-33 shows comparisons of curve numbers for conditions I, II and III.

In general, for rural areas the Curve Number charts, Figure 3-12 through Figure 3-32, pages 3-43 to 3-63 can be used to compute the peak discharge. The charts are set up for three slope catagories, FLAT (1 percent), MODERATE (4 percent) and STEEP (16 percent). For other slopes see Table 3-6, page 34 for slope adjustment factors. When using the charts, it should be noted that they are designed to be used only with 24-hour rainfall totals. It should also be noted that the Curver Number charts were based in part on a relationship between the hydraulic length (greatest flow length) and the watershed area from Agricultural Research Service's small experimental watersheds. Figure 3-11 shows the best fit line relating hydraulics length to drainage area. The equation of the line is $L = 209a^{0.6}$. There are watersheds that deviate considerably from the drainage area to length of watershed relationship. The peak discharge can be modified for these deviations. The procedure is as follows:

1. Determine the hydraulic length of the watershed and compute an "equivalent" drainage area using $L = 209a^{0.6}$ or Figure 3-11.

2. Determine the "equivalent" peak flow from the curve number charts.

3. Compute the "actual" peak discharge for the watershed by multiplying the equivalent peak discharge by the ratio of actual drainage area to the equivalent drainage area.

Example 1 (page 40), gives a step-by-step description of this procedure.

3-402.1 COMPUTING PEAK RATES OF DISCHARGES FOR URBAN AREAS

As population density and land values increase, the effects of uncontrolled runoff become an ecomonic burden and serious threat to the health and well being of a community and its citizens. Emphasis must be placed on providing solutions to the water problems caused by radical changes in land use (urbanization).

An urban or urbanizing watershed can be defined as an area in which all or part of the watershed will be covered by impervious structures, such as roads, sidewalks, parking lots, and houses. Urban stream channels may also be supplemented by some form of artificial drainage system, such as paved gutters and storm sewers. Urbanization of a watershed changes its response to precipitation. The most common effects are reduced infiltration and decreased travel time, which result in significantly higher peak rates or runoff.

The method for estimating peak discharges which has been described in this section was developed primarily for use with rural lands; in order to modify the precedure for use in urban situations, the following procedures should be followed:

1. Select a Curve Number from Table 3-4, page 3-32 based upon future land use.

2. Make an estimate of the total percentage of the area which is likely to be impervious and select a "Peak Factor' from Figure 3-10a, page 3-38 $(P.F.)_1$

3. Make an estimate of the percentage of the Hydraulic length of the watershed which is likely to be (or has been) modified as a result of urbanization (channel improvements, storm sewers, etc.) and select a second Peak Factor from Figure 3-10b, page 3-38 $(P.F.)_2$

4. Using the Curve Number determined in Step 1, and the methods described previously in this section, determine the basic discharge, Q_b. The adjusted discharge, $Q_{adj.}$ is the product of Q_b and the two Peak Factors determined in steps 2 and 3.

$$Q_{adj} = (Q_b)\ (P.F.)_1\ (P.F.)_2$$

Example 2 (page 3-41), gives a step-by-step description of this procedure:

Table 3-3---Runoff Curve Numbers for hydrologic soil-cover complete (Antecedent moisture condition II, and I_a =) (.2S)

Land use and treatment or practice	Hydrologic condition	Hydrologic soil group			
		A	B	C	D
Fallow					
Straight row..............	----	77	86	91	94
Row crops					
Straight row..............	Poor	72	81	88	91
Straight row..............	Good	67	78	85	89
Contoured.................	Poor	70	79	84	88
Contoured.................	Good	65	75	82	86
Contoured and terraced.....	Poor	66	74	80	82
Contoured and terraced.....	Good	62	71	78	81
Small grain					
Straight row..............	Poor	65	76	84	88
Straight row..............	Good	63	75	83	87
Contoured.................	Poor	63	74	82	85
Contoured.................	Good	61	73	81	84
Contoured and terraced.....	Poor	61	72	79	82
Contoured and terraced.....	Good	59	70	78	81
Close-seeded legumes or rotation meadow					
Straight row..............	Poor	66	77	85	89
Straight row..............	Good	58	72	81	85
Contoured.................	Poor	64	75	83	85
Contoured.................	Good	55	69	78	83
Contoured and terraced.....	Poor	63	73	80	83
Contoured and terraced.....	Good	51	67	76	80
Pasture or range					
No mechanical treatmemt....	Poor	68	79	86	89
No mechanical treatment....	Fair	49	69	79	84
No mechanical treatment....	Good	39	61	74	80
Contoured.................	Poor	47	67	81	88
Contoured.................	Fair	25	59	75	83
Contoured.................	Good	6	35	70	79
Meadow.....................	Good	30	58	71	78
Woods......................	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	25	55	70	77
Farmsteads..................	----	59	74	82	86
Roads					
Dirt......................	----	72	82	87	89
Hard surfaces..............	----	74	84	90	92

Table 3-4---Runoff Curve Numbers for selected agricultural, suburban, and urban land use. (Antecedent moisture condition II, and $I_a = 0.2S$)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
: good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
: good cover	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential:				
Average lot size — Average % Impervious				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

Table 3-5

CURVE NUMBERS (CN) AND CONSTANTS FOR THE CASE $I_a = 0.2\ S$

1	2	3	4	5**	1	2	3	4	5**
CN for Condition II	CN for Conditions I	III	S Values*	Curve Starts Where P =	CN for Condition II	CN for Conditions I	III	S Values*	Curve Starts Where P =
			(inches)	(inches)				(inches)	(inches)
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	0.101	0.02	59	39	77	6.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	59	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.76
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	91	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.72
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

* For CN in Column 1
** Refer to Figure 3-8

Table 3-6---Slope Adjustment Factors by Drainage Areas

Slope (percent)	10 acres	20 acres	50 acres	100 acres	200 acres	500 acres	1,000 acres	2,000 acres
				FLAT SLOPES				
0.1	0.49	0.47	0.44	0.43	0.42	0.41	0.41	0.40
0.2	.61	.59	.56	.55	.54	.54	.53	.52
0.3	.69	.67	.65	.64	.63	.62	.62	.61
0.4	.76	.74	.72	.71	.70	.69	.69	.69
0.5	.82	.80	.78	.77	.77	.76	.76	.76
0.7	.90	.89	.88	.87	.87	.87	.87	.87
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.13	1.14	1.14	1.15	1.16	1.17	1.17	1.17
2.0	1.21	1.24	1.26	1.28	1.29	1.30	1.31	1.31
				MODERATE SLOPES				
3	.93	.92	.91	.90	.90	.90	.89	.89
4	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
5	1.04	1.05	1.07	1.08	1.08	1.08	1.09	1.09
6	1.07	1.10	1.12	1.14	1.15	1.16	1.17	1.17
7	1.09	1.13	1.18	1.21	1.22	1.23	1.23	1.24
				STEEP SLOPES				
8	.92	.88	.84	.81	.80	.78	.78	.77
9	.94	.90	.86	.84	.83	.82	.81	.81
10	.96	.92	.88	.87	.86	.85	.84	.84
11	.96	.94	.91	.90	.89	.88	.87	.87
12	.97	.95	.93	.92	.91	.90	.90	.90
13	.97	.97	.95	.94	.94	.93	.93	.92
14	.98	.98	.97	.96	.96	.96	.95	.95
15	.99	.99	.99	.98	.98	.98	.98	.98
16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10
25	1.06	1.08	1.12	1.14	1.15	1.16	1.17	1.19
30	1.09	1.11	1.14	1.17	1.20	1.22	1.23	1.24
40	1.12	1.16	1.20	1.24	1.29	1.31	1.33	1.35
50	1.17	1.21	1.25	1.29	1.34	1.37	1.40	1.43

Table 3-7

Soil Names and Hydrological Classification in Arkansas

Acadia D
Adaton D
Agnos B
Alaga A
Allen B
Alligator D
Amagon D
Amy D
Angie C
Apison B
Arkabutla C
Ashton B
Ashwood C
Askew C
Avilla B

Barling C
Baxter B
Beasley C
Beulah B
Bibb D
Billyhaw D
Blevins B
Boden C
Bodine B
Bonn D
Bosket B
Boswell D
Bowdre C
Bowie B
Brandon B
Briley B
Britwater B
Brocket C
Broseley B
Bruno A
BudeC
Buxin D
Brockwell B

Caddo D
Cahaba B
Calhoun D
Calloway C
Cane C
Captina C
Carnasaw C
Carytown D
Cascilla B
Caspiana B
Catalpa C
Ceda B
Chastain D
Chennely C
Cherokee D
Christian C
Clarksville B
Clebit D
Cleora B
Collins C
Commerce C
Conasauga C
Convent C
Corydon D
Coushatta B
Crevassee A
Crowley D
Cuthbert C

Darco A
Dardanelle B
Demopolis C
Desha D
Dexter B
Doniphan B
Dubbs B
Dundee C

Dunning D

Earle D
Egam C
Elsah B
Emory B
Enders C
Ennis B
Estate C
Etowah B
Eutaw D
Eylau C

Falaya C
Falkner C
Fatima B
Fayetteville B
Felker D
Foley D
Forrestdale D
Fountain D

Gallion B
Gasconade D
Gassville C
Geep B
Gladwater D
Goldsboro C
Goldston C
Gore D
Grenda C
Grubbs D
Guin A
Guthrie D
Guyton D

Harleston C
Hartsells B
Hatchie C
Hayti D
Healing B
Hebert C
Hector D
Henry D
Hillemann C
Hollywood D
Holston B
Houlka D
Houston D
Huntington B

Iberia D
Iuka C
Izagora C

Jackport D
Jay C
Jeanerette D
Johnsburg D

Kalmia B
Kamie B
Karma B
Kaufman D
Keo B
Kiematia A
Kipling D
Kirvin C
Kobel D

Lafe D
Lagrange D
Latanier D
Latonia B
Leadvale C
Leaf D

Leeper D
Leesburg B
Lexington B
Lindside C
Lily B
Linker B
Lobelville C
Locust C
Lonoke B
Loring C
Lucy A
Luverne C

Mantachie C
Marietta C
Marvell B
Mashulaville D
Mayes D
Mayhew D
McCrory D
McGehee C
McKamic D
McLaurin B
Melvin D
Memphis B
Mhoon D
Miller D
Millwood D
Moko C
Monongahela C
Montevallo D
Moreland D
Morganfield B
Morse D
Mountainburg D
Muldrow D
Muskogee C
Myatt D

Nacogdoches B
Natchez B
Neila B
Newardk C
Newellton D
Newtonia B
Nixa C
Noark B
Norfolk B
Norwood B
Nugent A

Oaklimeter C
Oklared B
Ochlockonee B
Oktibbeha B
Ora C
Orangeburg B
Ouachita C
Ozan D

Patterson C
Pembroke B
Peridge B
Perry D
Pheba D
Philo C
Pickens D
Pickwick B
Pikeville B
Pirum B
Pipe B
Protia C
Portland D
Prentiss C
Providence C

Ramsey D
Razort B
Rexor A
Rilla B
Roanoke D
Robinsville B
Roellen D
Routon D
Ruston B

Sacul D
Saffell B
Sallisaw B
Samba D
Sardis C
Savannah C
Sawyer C
Secesh B
Sequatchie B
Sessum D
Sharkey D
Sherwood B
Shubuta C
Sidon C
Sloan D
Smithdale B
Smithton D
Sogn D
Spadra B
Stanser B
State B
Steele B
Steprock B
Sterlington B
Stough C
Sturkie B
Stuttgart D
Summit C
Sumter C
Susquehanna D

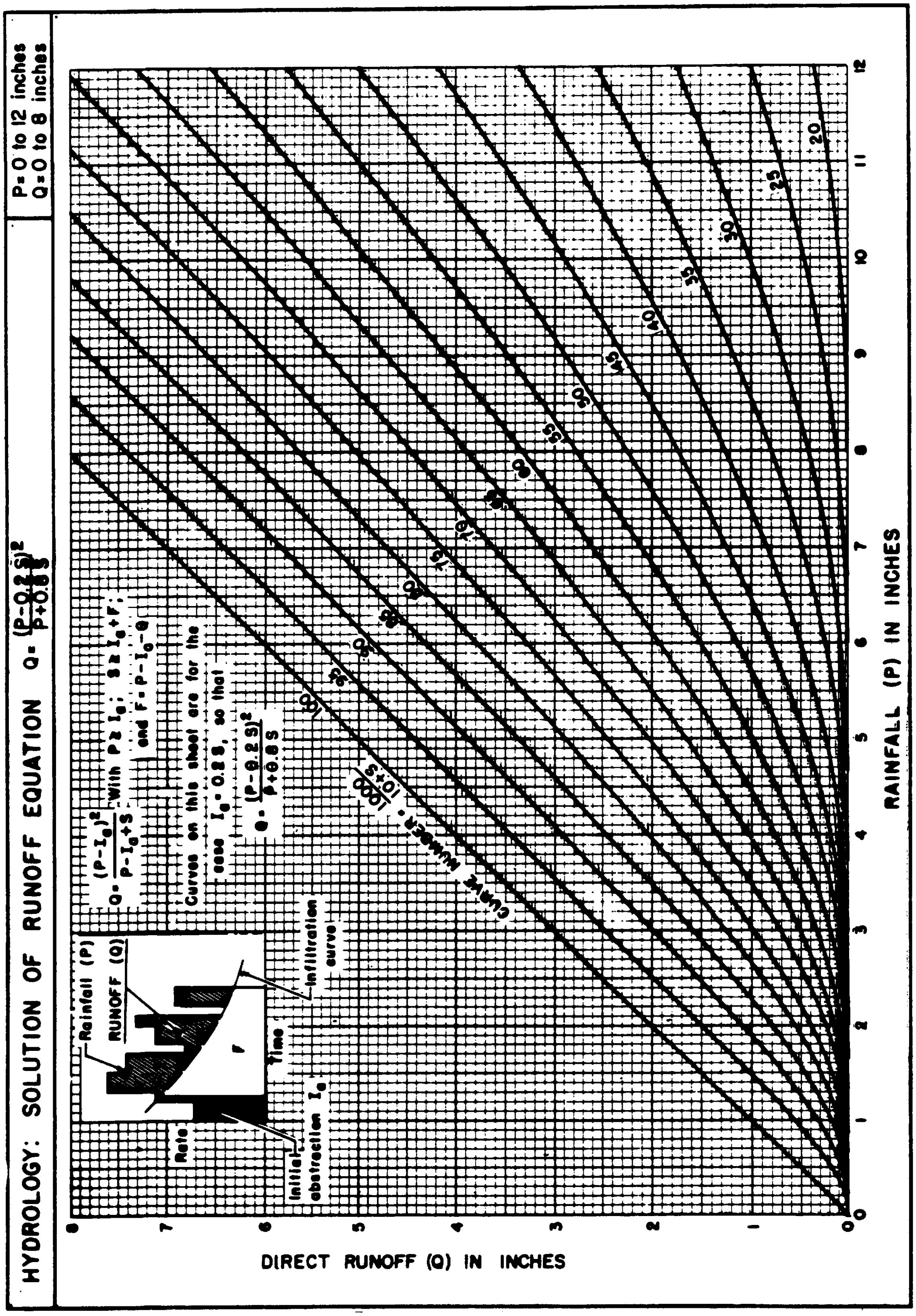
Taft C
Talbott C
Taloka D
Terouge D
Tiak C
Tichnor D
Tippah C
Tiptonville B
Toine B
Townley C
Trebloc D
Trinity D
Troup A
Tuckerman D
Tunica D
Tuscumbia D
Tutwiler B

Una D

Vaiden D
Ventris D

Wabbaseka D
Wabeh B
Wardell C
Waverly D
Waynesboro B
Weston D
Wickham B
Wideman A
Wilcox D
Wilson S

Fig. 3-8

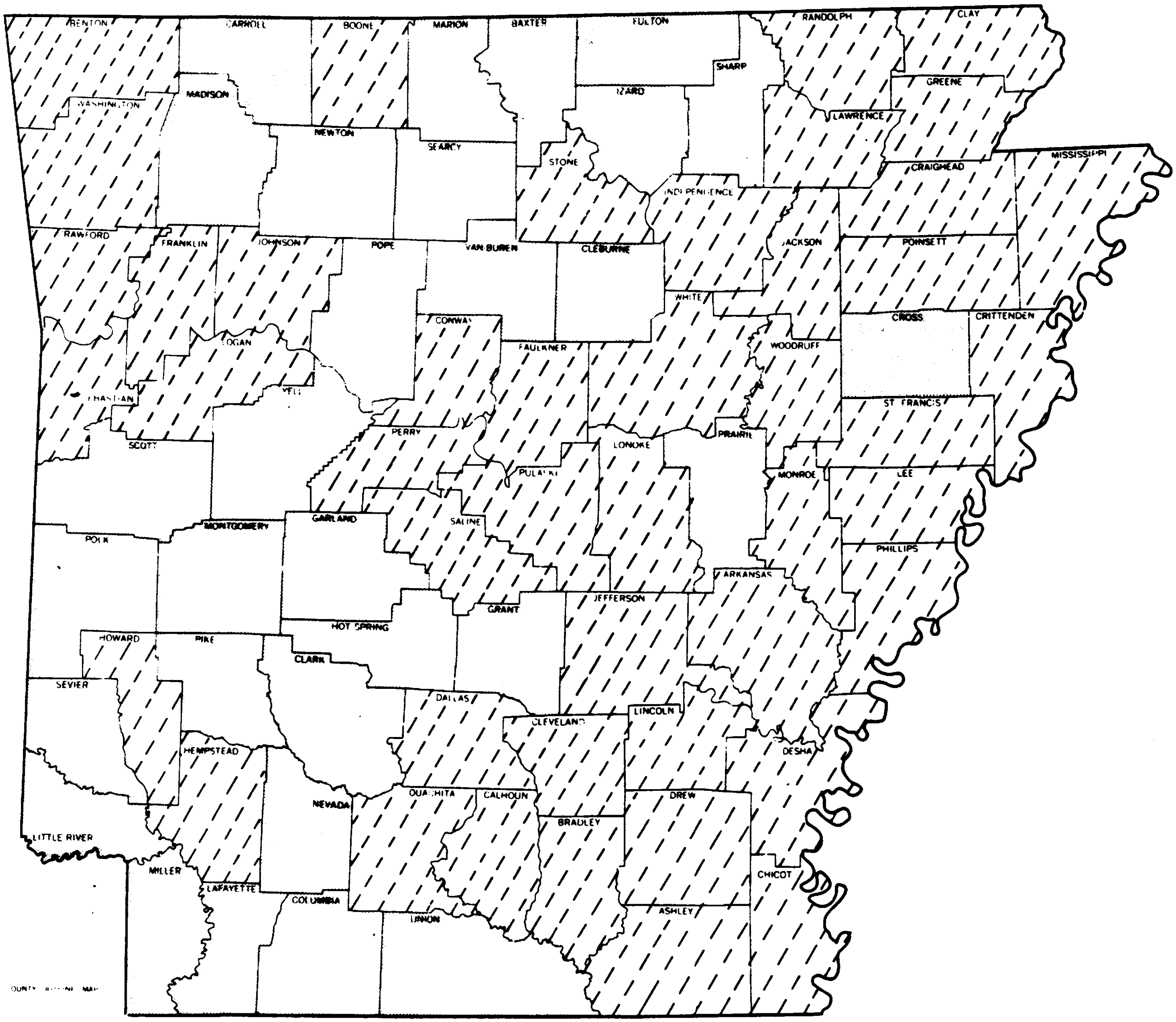


HYDROLOGY: SOLUTION OF THE SCS RUNOFF EQUATION

Revised 5-16-83

FIGURE 3-9

STATUS OF SOIL SURVEYS
ARKANSAS



LEGEND

☐ General Soil Map Available

▨ Comprehensive Soil Survey Manual Available

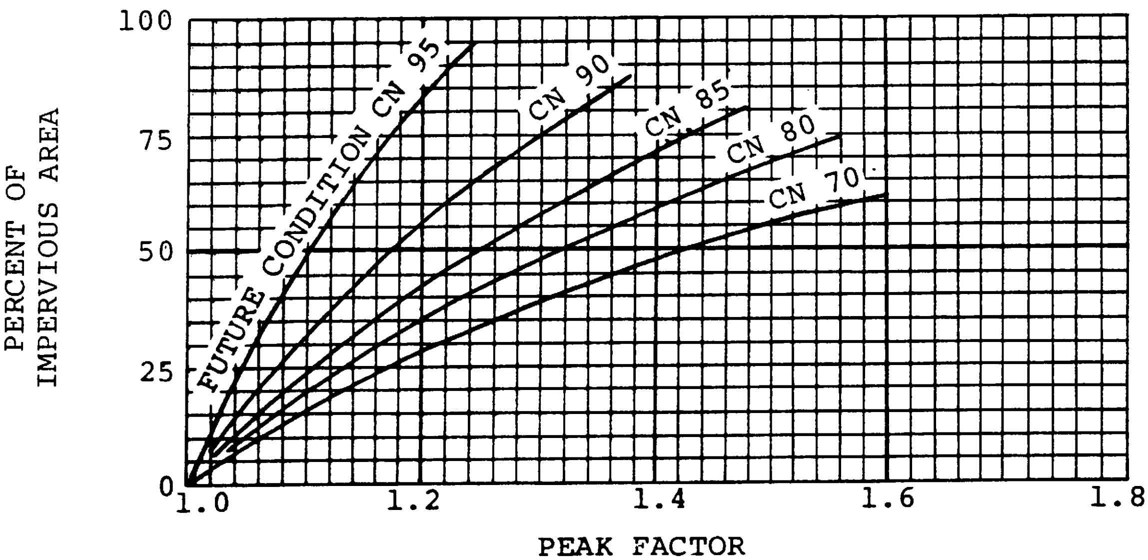


Figure 3-10a--Factors for adjusting peak discharges for a future-condition runoff curve number based on the percentage of impervious area in the watershed.

NOTE: **It is recommended that Figure 3-10a be used only when the future curve number (CN) is not known. It should not be applied when using computed future CN.**

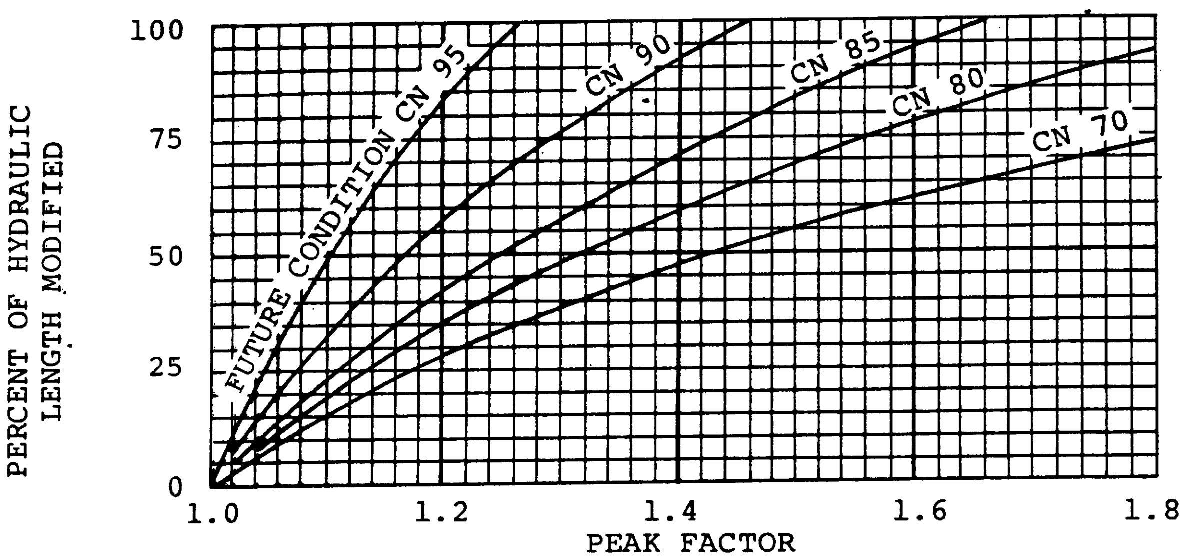


Figure 3-10b--Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of hydraulic length modified.

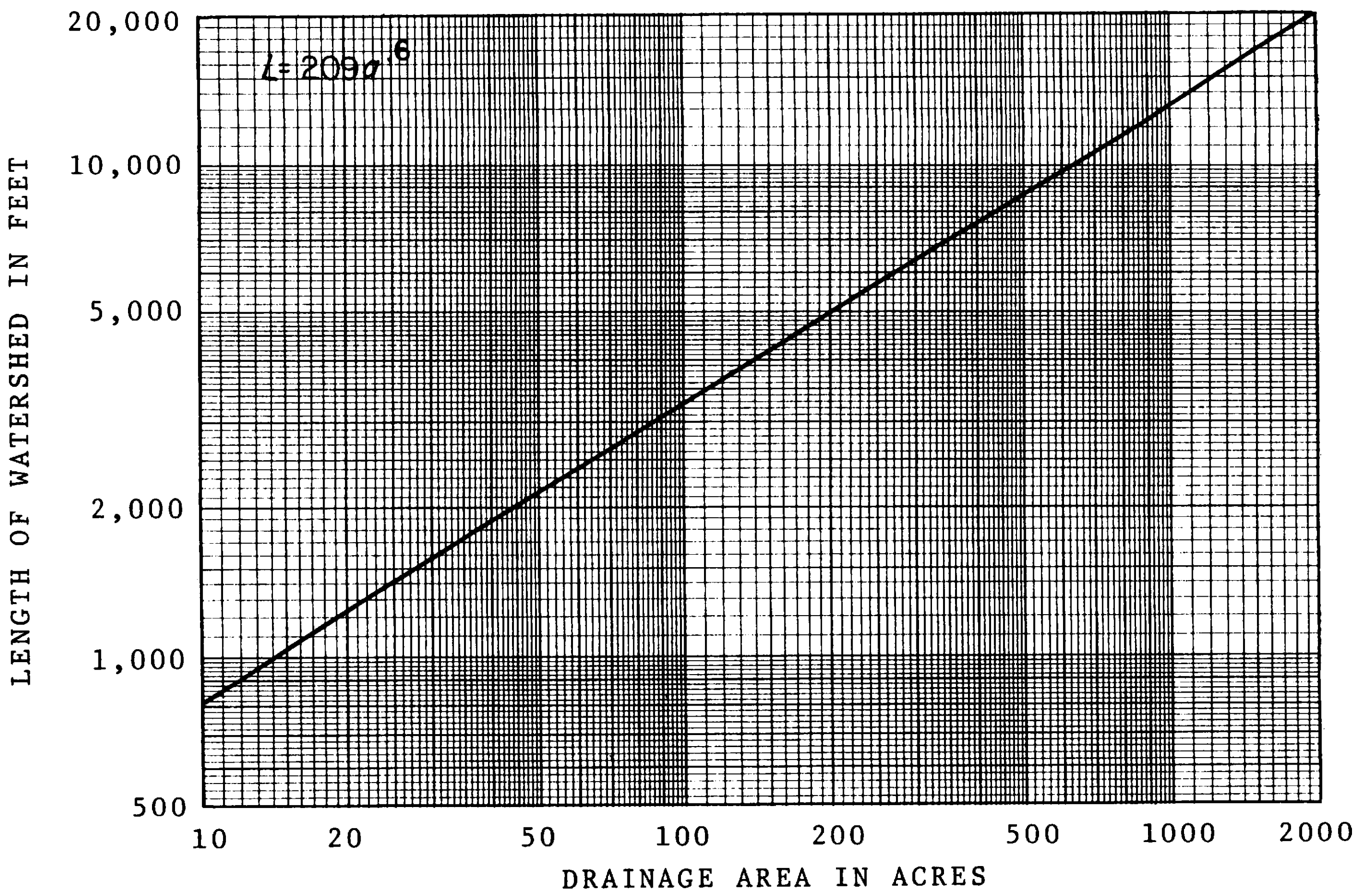


Figure 3-11,--Hydraulic length and drainage area relationship.

EXAMPLE 1

A 500 acre watershed is 60 percent agricultural and 40 percent urban land. The agricultural land is 50 percent pasture in good condition and 50 percent forest land with good cover. The urban area is residential: 20 percent is gravel roads and 80 percent is 1/2 acre lots. The watershed is located in Area III of Figure 3-3 and is 6000 feet long with an average watershed slope of 3 percent. The entire watershed is in a C hydrologic soil group. Compute the peak discharge for a 25-year storm.

(a) Compute weighted composite runoff curve number using curve numbers from Table 3-4, page 3-32.

LAND USE	ACRES	CN	PRODUCT
Agricultural	(300)		
Pasture (good condition)	150	74	11100
Forest (good cover)	150	70	10500
Urban	(200)		
Gravel Road	40	89	3560
1/2 - acre lots	160	80	12800
	500		37960

Weighted CN $= \frac{37960}{500} = 75.92$ use 76.

(b) Find Rainfall

*From Figure 3-6, page 3-23 a 24-hour rainfall for a 25-year storm = 24 (.3) = 7.2"

*Can also be obtained from Reference 7.

(c) Adjust discharge (Q) for 3% slope and watershed length. From Table 3-6, page 3-34 slope adjustment factor = .90.

From Figure 3-11, a hydraulic length of 6000 feet gives an equivalent drainage area of 270 acres.

Then using 270 acres and the Curve Number Charts for a moderate slope, (Figure 3-22 and Figure 3-23, page 3-53, 3-54.

CN 75 = 500 cfs

CN 80 = 630 cfs

Interpolating CN 76 = 526.

$$Q = 526 \frac{(500)}{270} (.9) = 876.7 \text{ cfs}$$

EXAMPLE 2

A 1200 acre watershed is to be developed. The composite curve number for the proposed development is computed to be 80. Approximately 50 percent of the hydraulic length will be modified by the installation of gutters and storm drains to the watershed outlet. Also, approximately 40 percent of the watershed will now be impervious. The average watershed slope is estimated to be 4 percent and the length of the watershed is 14,800 feet. The development is in area II of Figure 3-3, page 3-20. Compute the present condition and anticipated future condition peak discharge for a 25-year storm. The present CN for the development is 70.

(a) From the Curve Number Charts, (Figure 3-21, page 3-52, using a moderate slope and a CN of 70 and 1200 acres, the present discharge for a 25-year frequency storm in Area II is:

1. Rainfall from Figure 3-5, page 3-22 = .28 (24) = 6.72 inches
2. Q = 950 cfs

(b) Using a CN of 80 (Figure 3-23, page 3-54 and the same information, Q = 1600 cfs.

1. From Figure 3-10(a), page 3-38, with 40 percent impervious area and future runoff CN of 80, read Peak Factor = 1.24.

2. From Figure 3-10(b), page 3-38, with 50 percent of the hydraulic length modifed and future runoff CN of 80, read Peak Factor = 1.32.

3. Therefore, future condition peak discharge is:
   1600(1.24) (1.32) = 2619 cfs

(c) The effect of this proposed development is to increase the peak discharge from 950 cfs to 2619 cfs.

NOTE: This example show the tremendous change urbanization can cause and how drainage structures must be designed for conditions that might exist in the future.

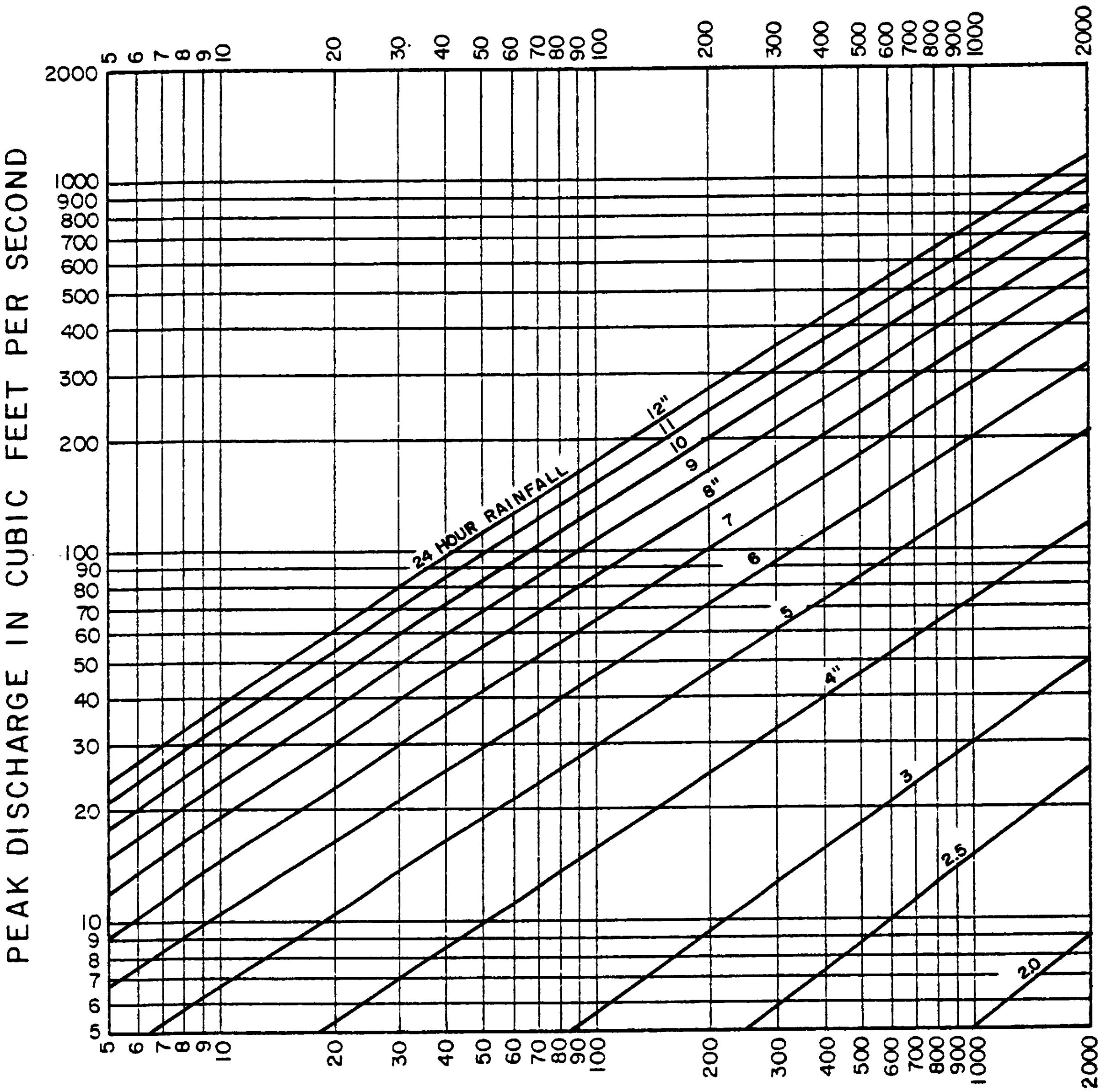
Included at the end of this section (page 3-64), is a worksheet for the SCS Method.

Fig. 3-12

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 60

24 HOUR RAINFALL U.S. WB TP-40



STANDARD DWG. NO.
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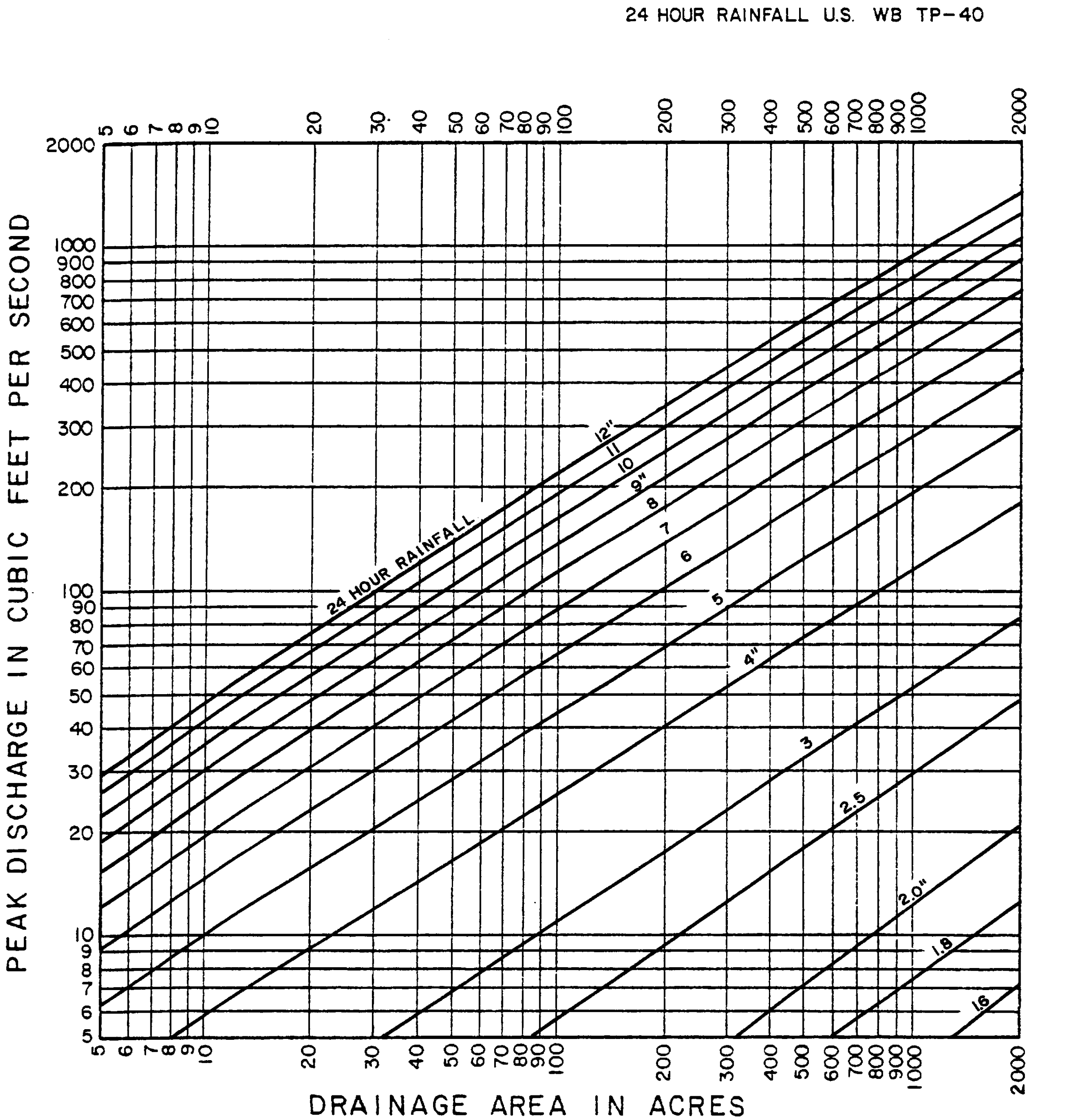
Fig. 3-13

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 65

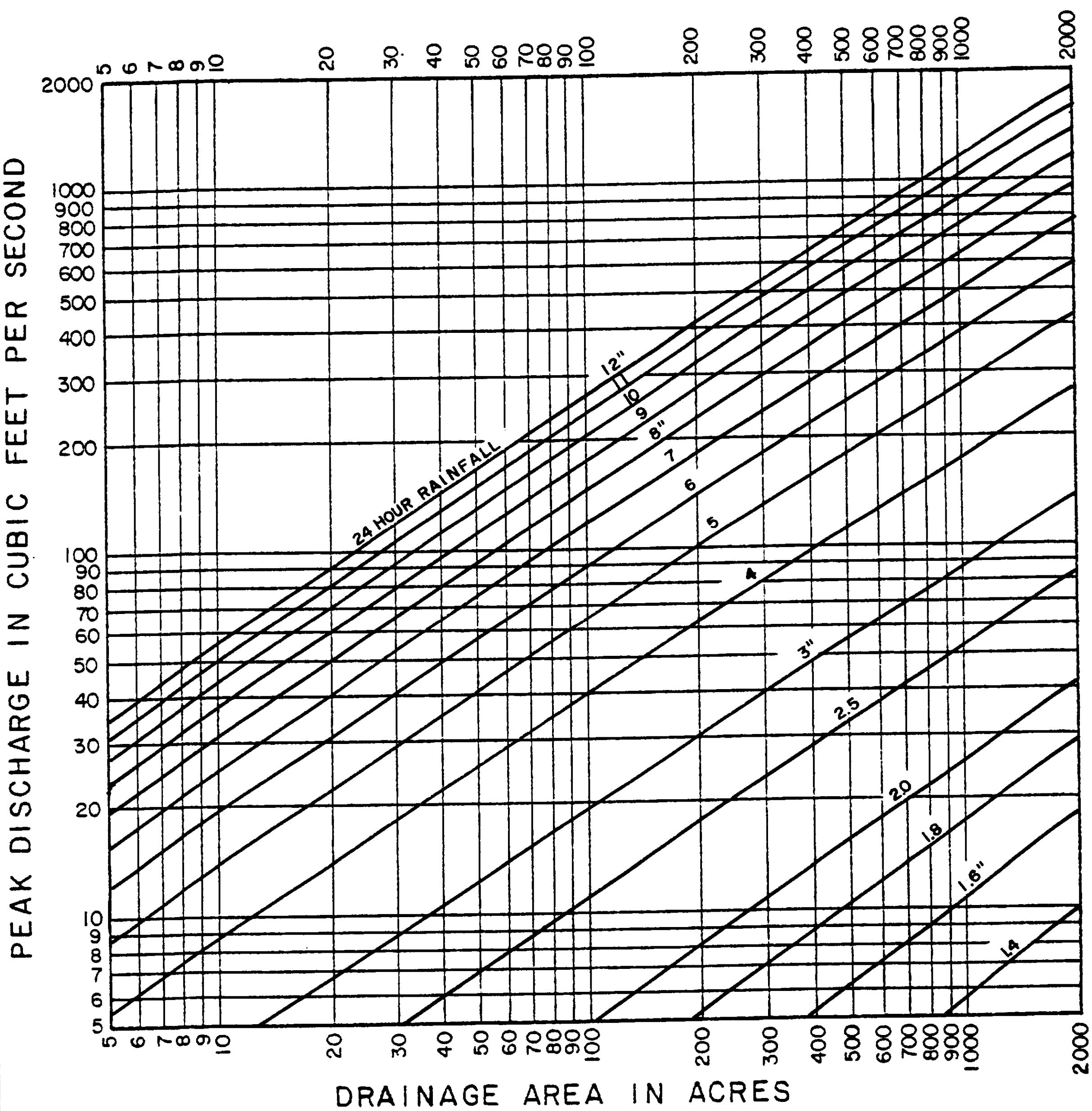


STANDARD DWG. NO.
ES-1027
SHEET 2 OF 21
DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES- FLAT
CURVE NUMBER- 70

24 HOUR RAINFALL U.S. WB TP-40



STANDARD DWG. NO.
ES-1027
SHEET 3 OF 21
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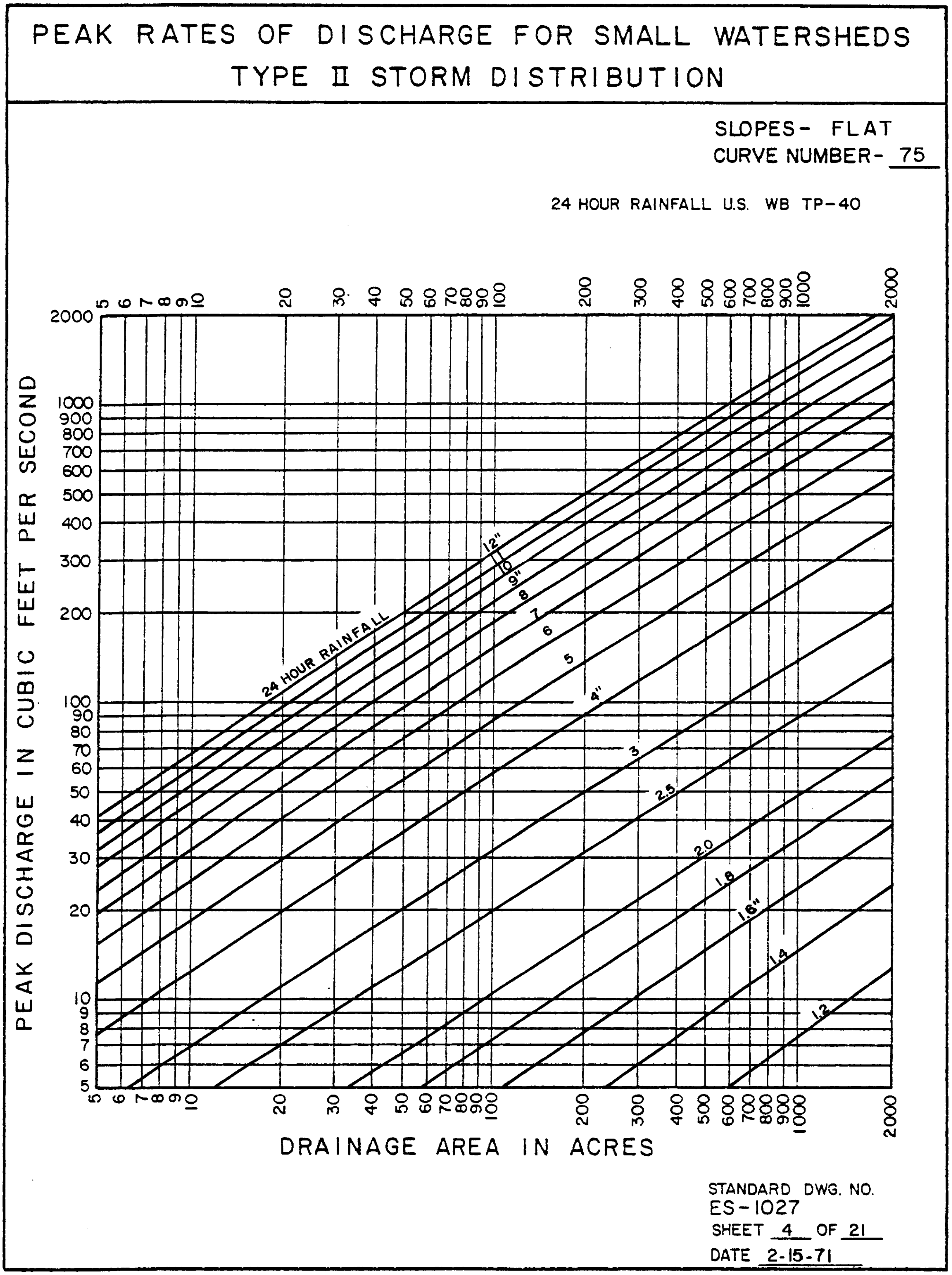
PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES- FLAT
CURVE NUMBER- 75
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
10
9"
8
7
6
5
4"
3
2.5
2.0
1.8
1.6"
1.4
1.2
PEAK DISCHARGE IN CUBIC FEET PER SECOND
DRAINAGE AREA IN ACRES
STANDARD DWG. NO.
ES-1027
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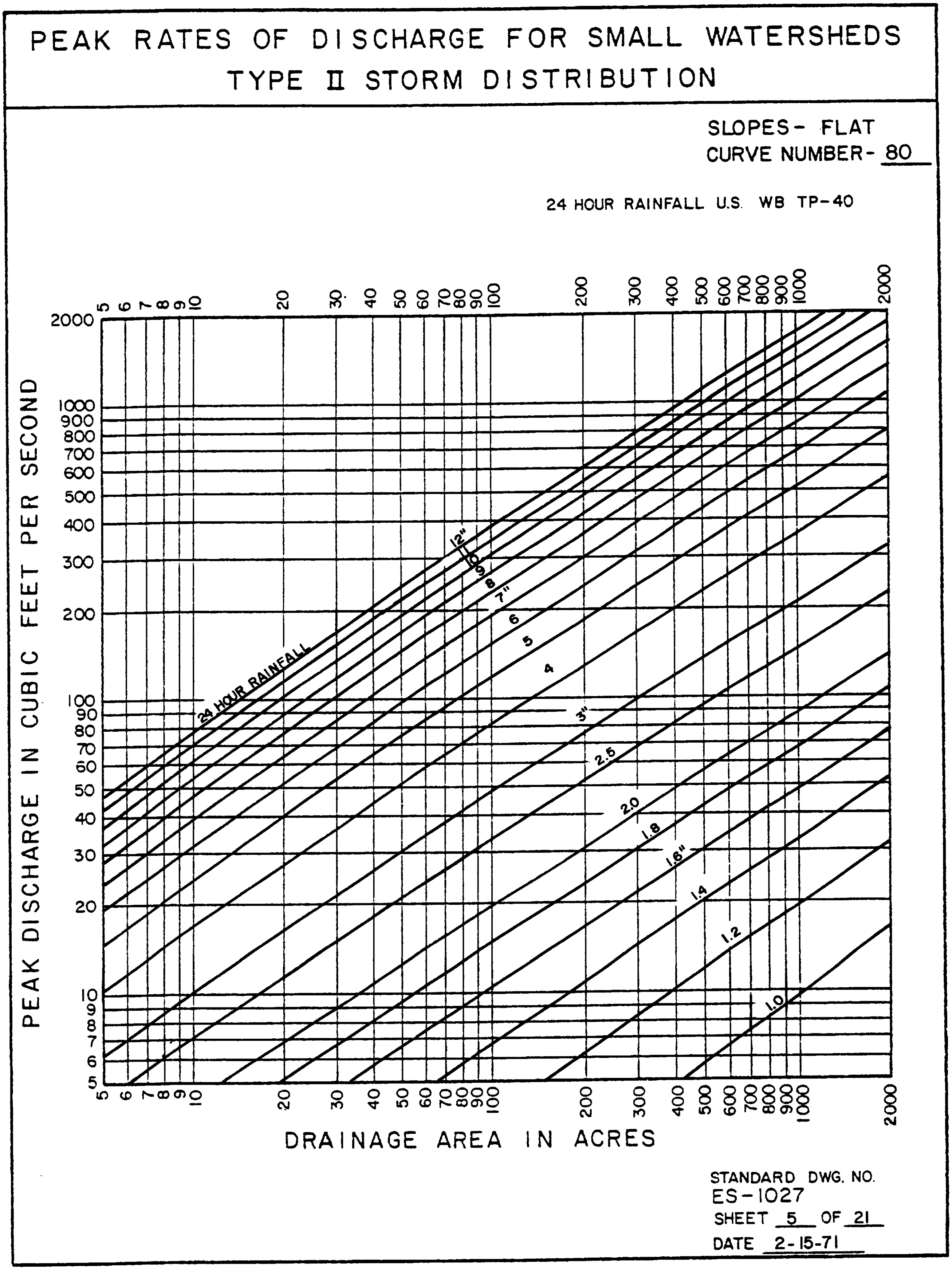
Fig. 3-16


PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES- FLAT
CURVE NUMBER- 80
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
10
9
8
7"
6
5
4
3"
2.5
2.0
1.8
1.6"
1.4
1.2
1.0
PEAK DISCHARGE IN CUBIC FEET PER SECOND
DRAINAGE AREA IN ACRES
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
700
800
900
1000
2000
STANDARD DWG. NO.
ES-1027
SHEET 5 OF 21
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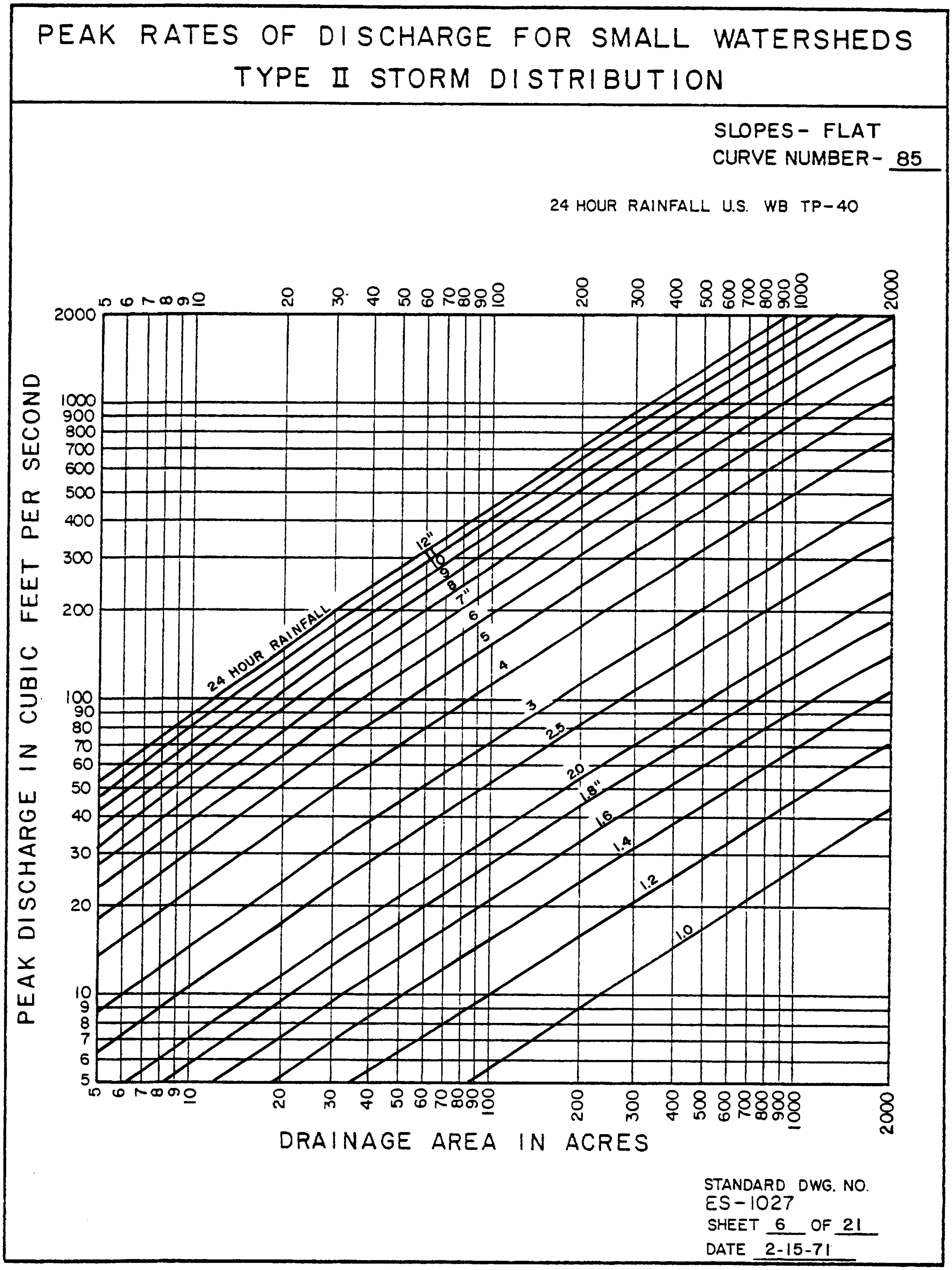
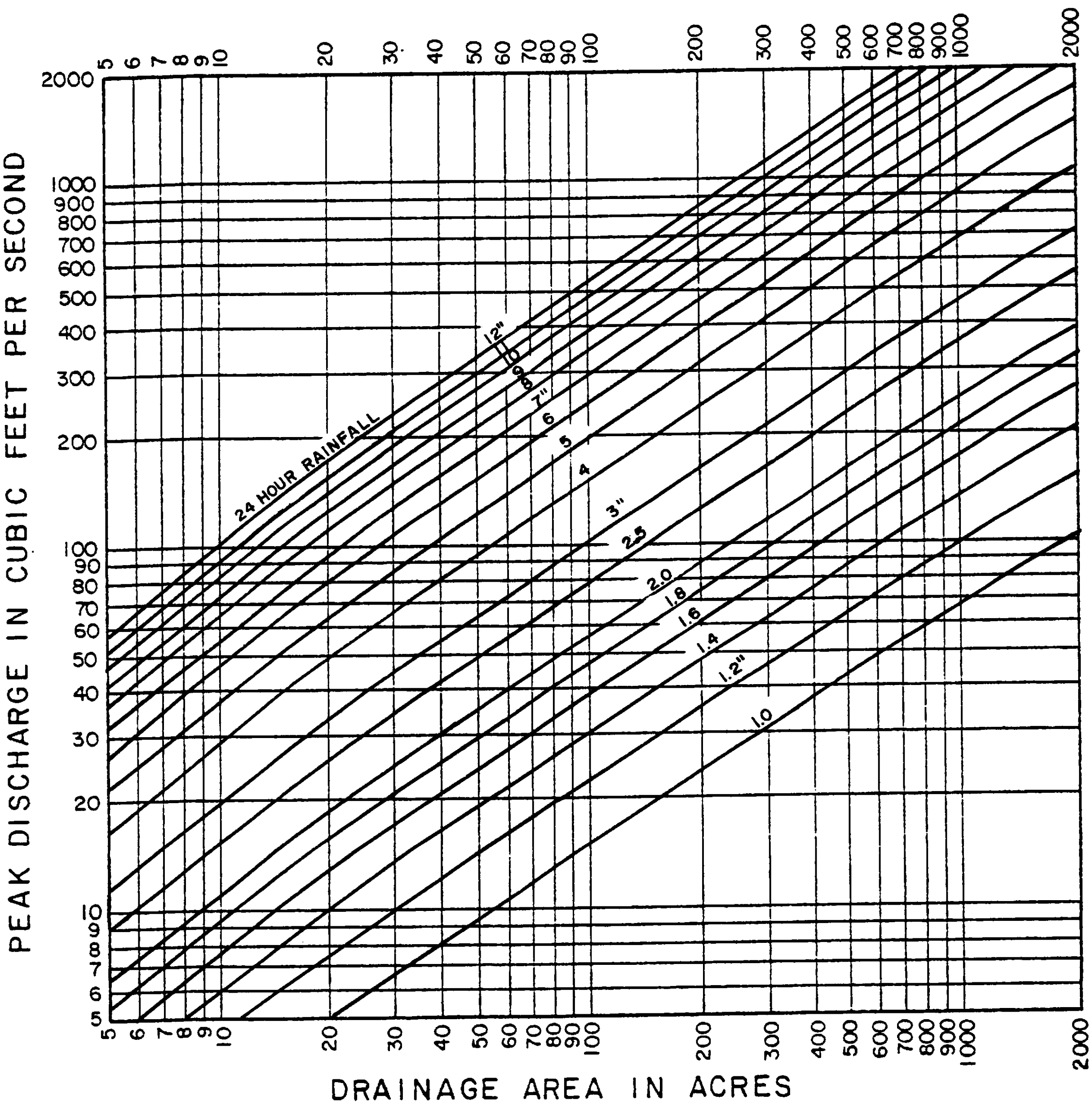

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES - FLAT
CURVE NUMBER - 85
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
10
9
8
7"
6
5
4
3
2.5
2.0
1.8"
1.6
1.4
1.2
1.0
PEAK DISCHARGE IN CUBIC FEET PER SECOND
5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000
DRAINAGE AREA IN ACRES
STANDARD DWG. NO.
ES-1027
SHEET 6 OF 21
DATE 2-15-71

Fig. 3-18

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES- FLAT
CURVE NUMBER- 90

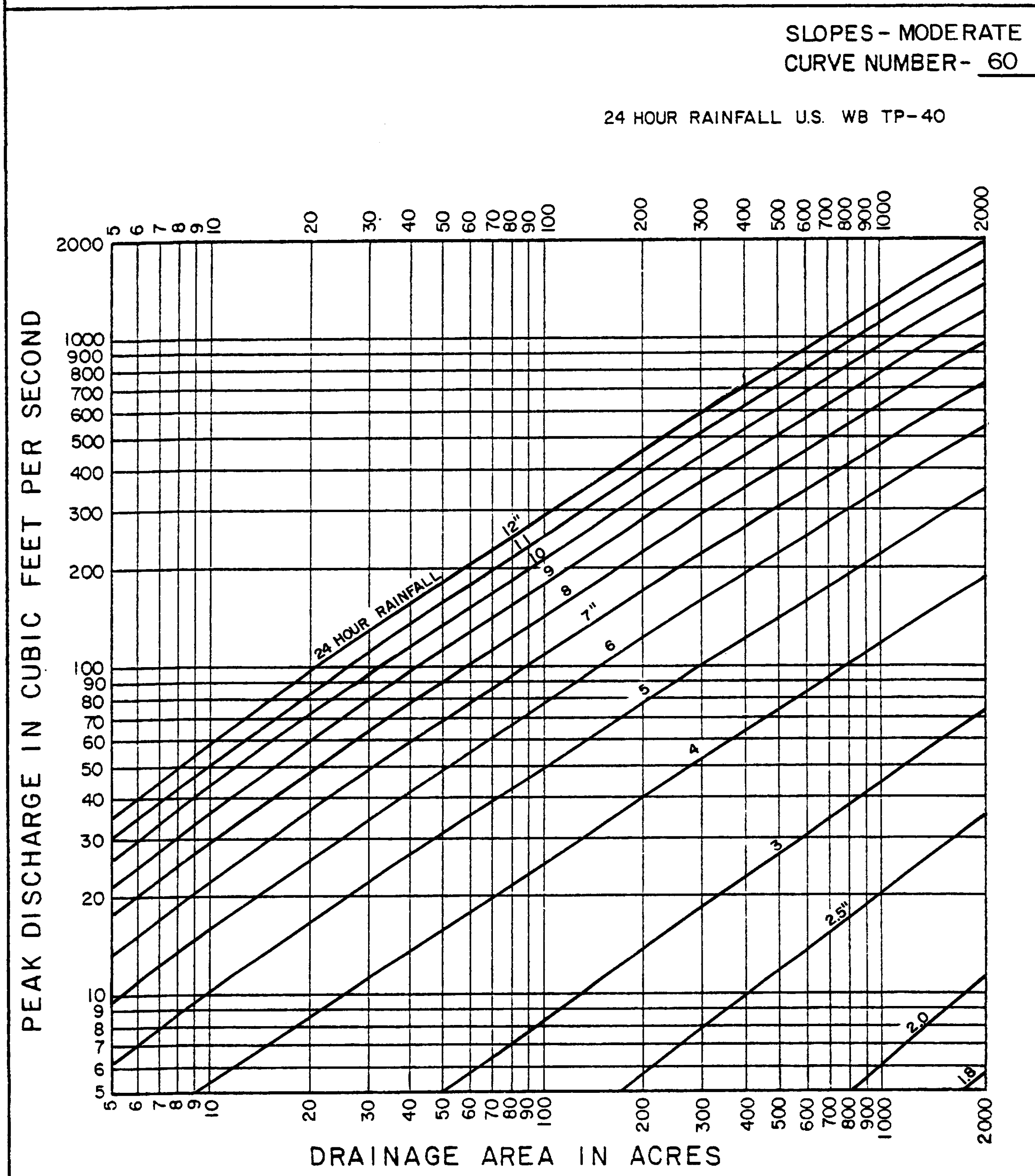
24 HOUR RAINFALL U.S. WB TP-40



STANDARD DWG. NO.
ES-1027
SHEET 7 OF 21
DATE 2-15-71

Fig. 3-19

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION



STANDARD DWG. NO.
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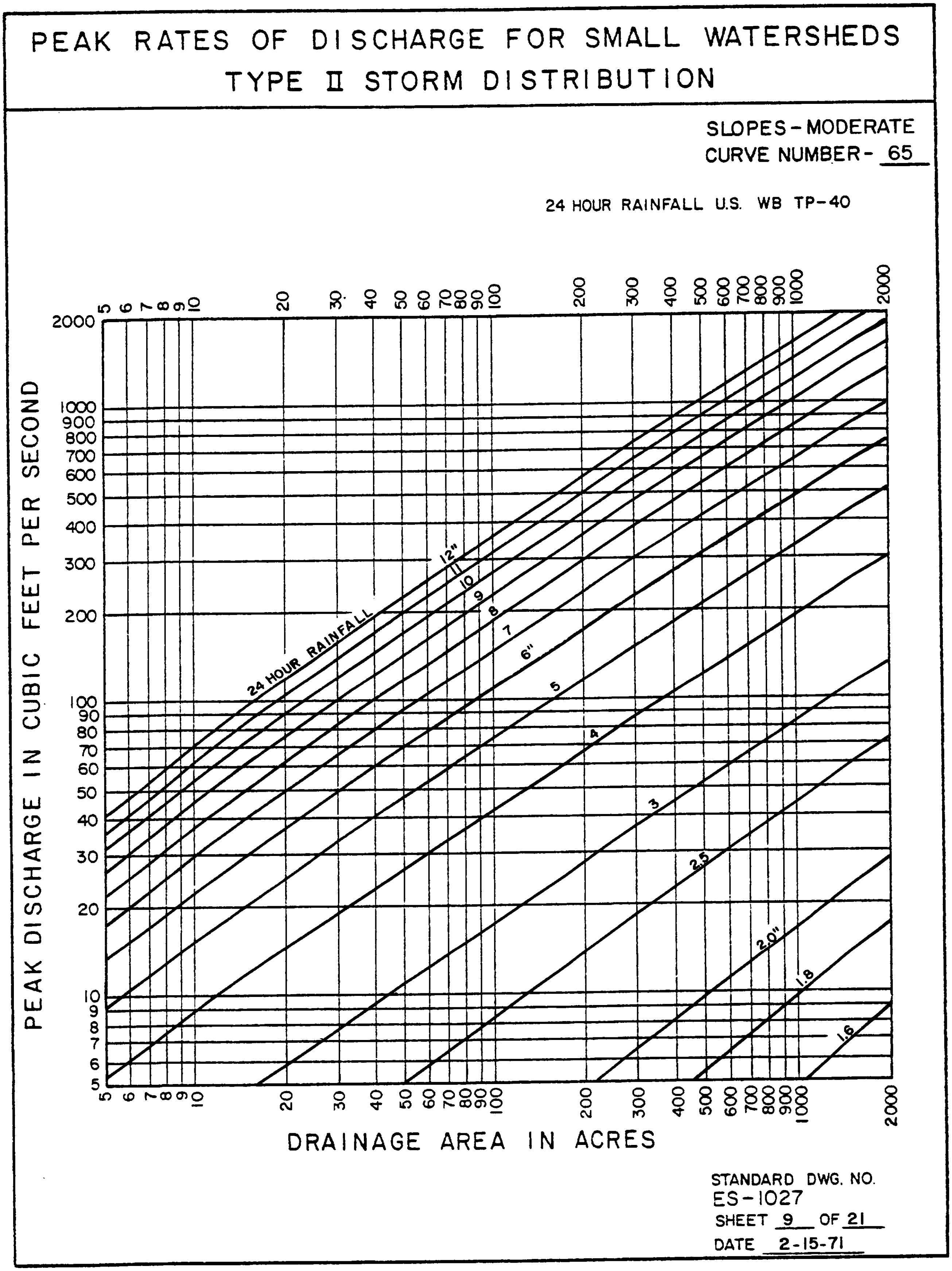
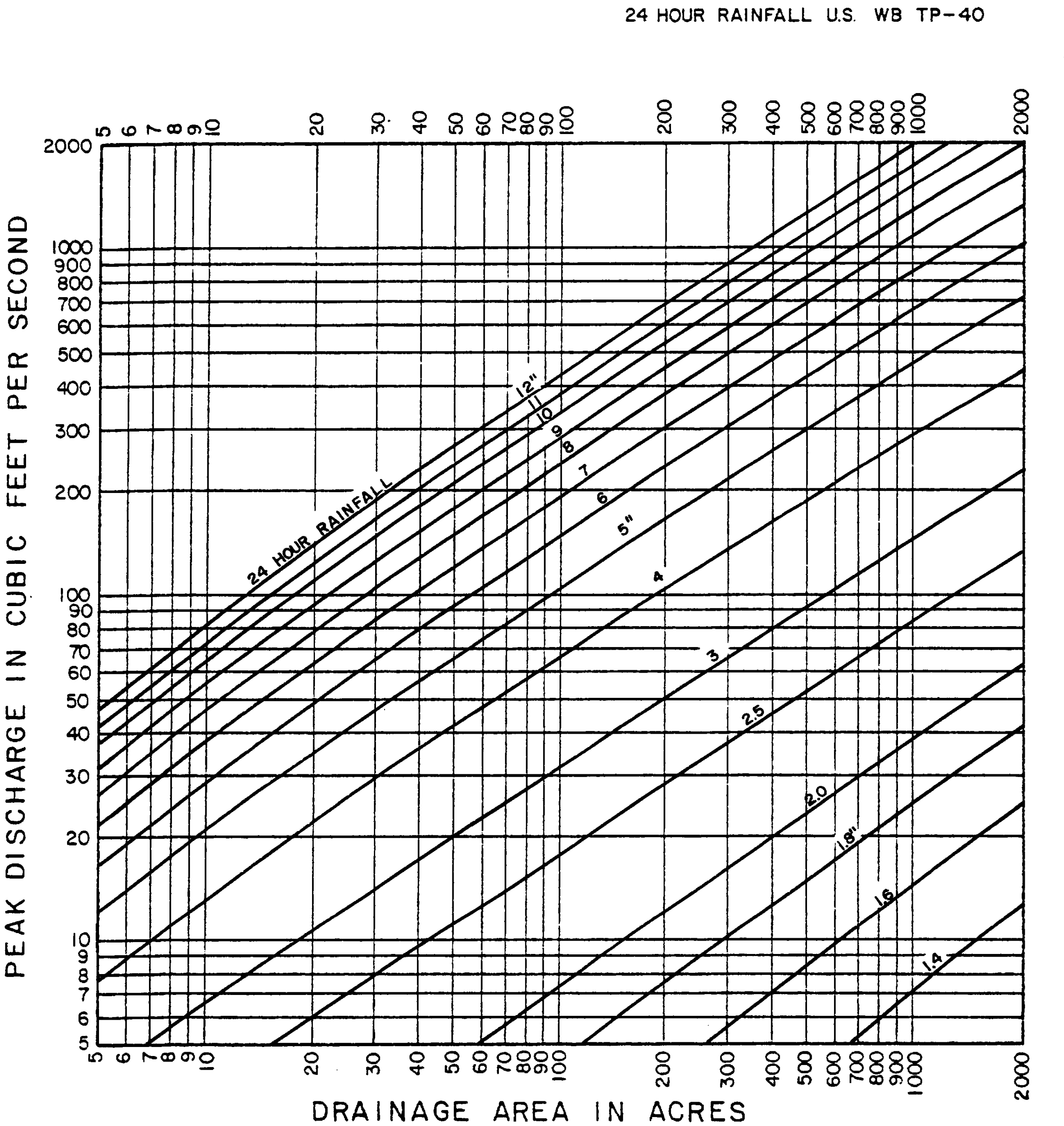
SHEET 8 OF 21
DATE 2-15-71

Fig. 3-20


PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES - MODERATE
CURVE NUMBER - 65
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
11
10
9
8
7
6"
5
4
3
2.5
2.0"
1.8
1.6
PEAK DISCHARGE IN CUBIC FEET PER SECOND
5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000
DRAINAGE AREA IN ACRES
STANDARD DWG. NO.
ES-1027
SHEET 9 OF 21
DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 70



STANDARD DWG. NO.
ES-1027
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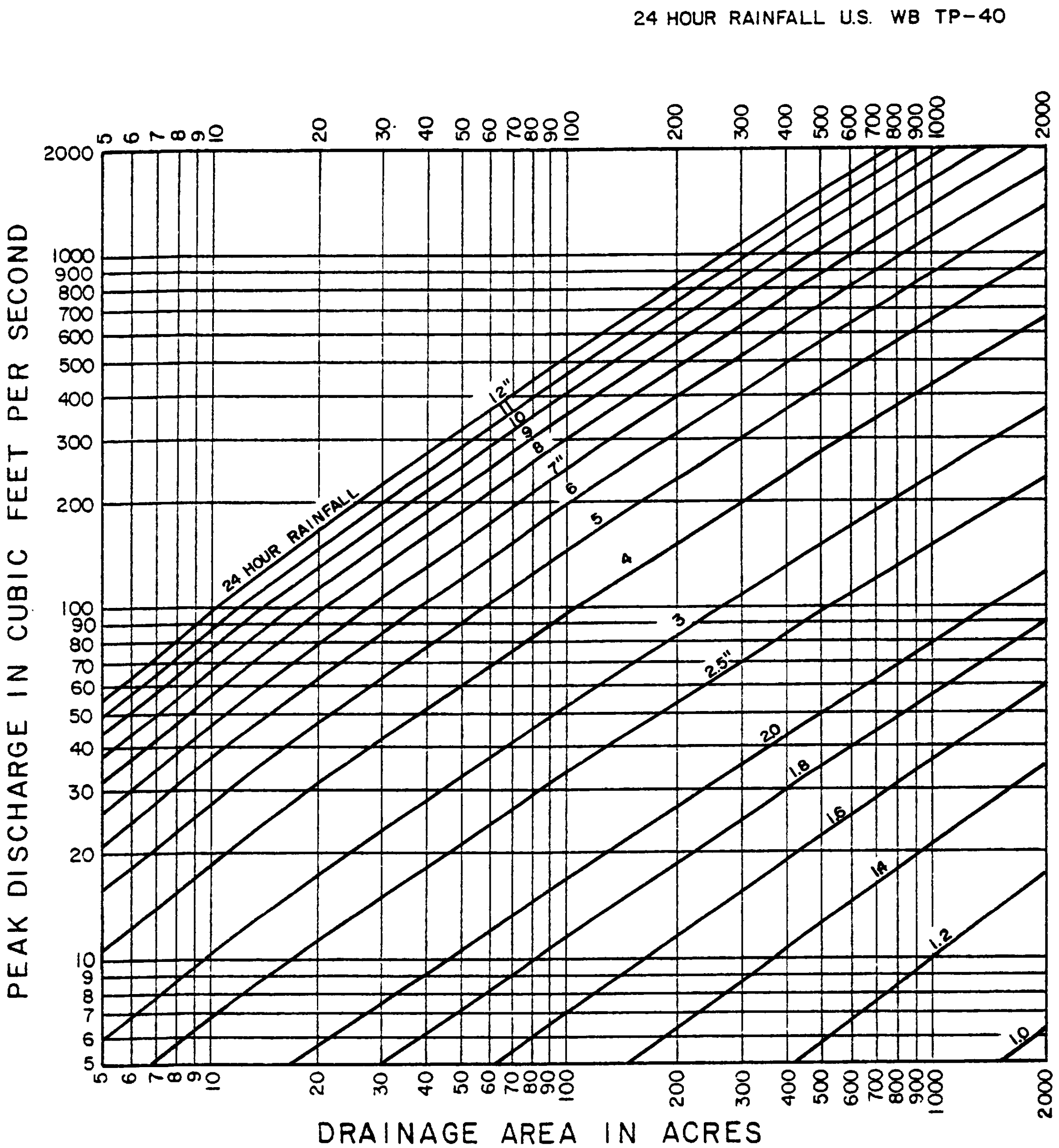
DATE 2-15-71

Fig.3-22

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

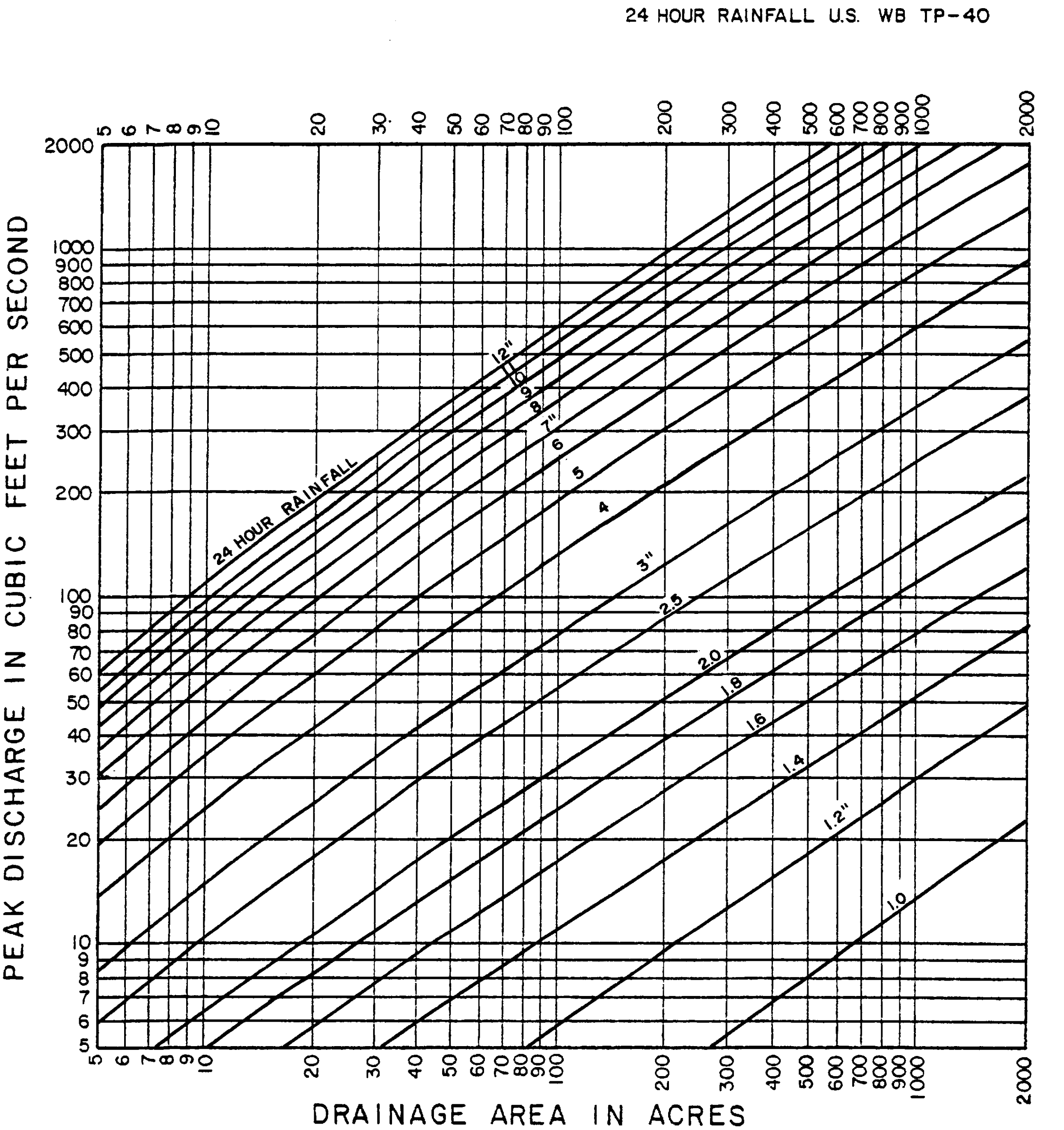
SLOPES - MODERATE
CURVE NUMBER - 75



STANDARD DWG. NO.
ES-1027
SHEET 11 OF 21
DATE 2-15-71

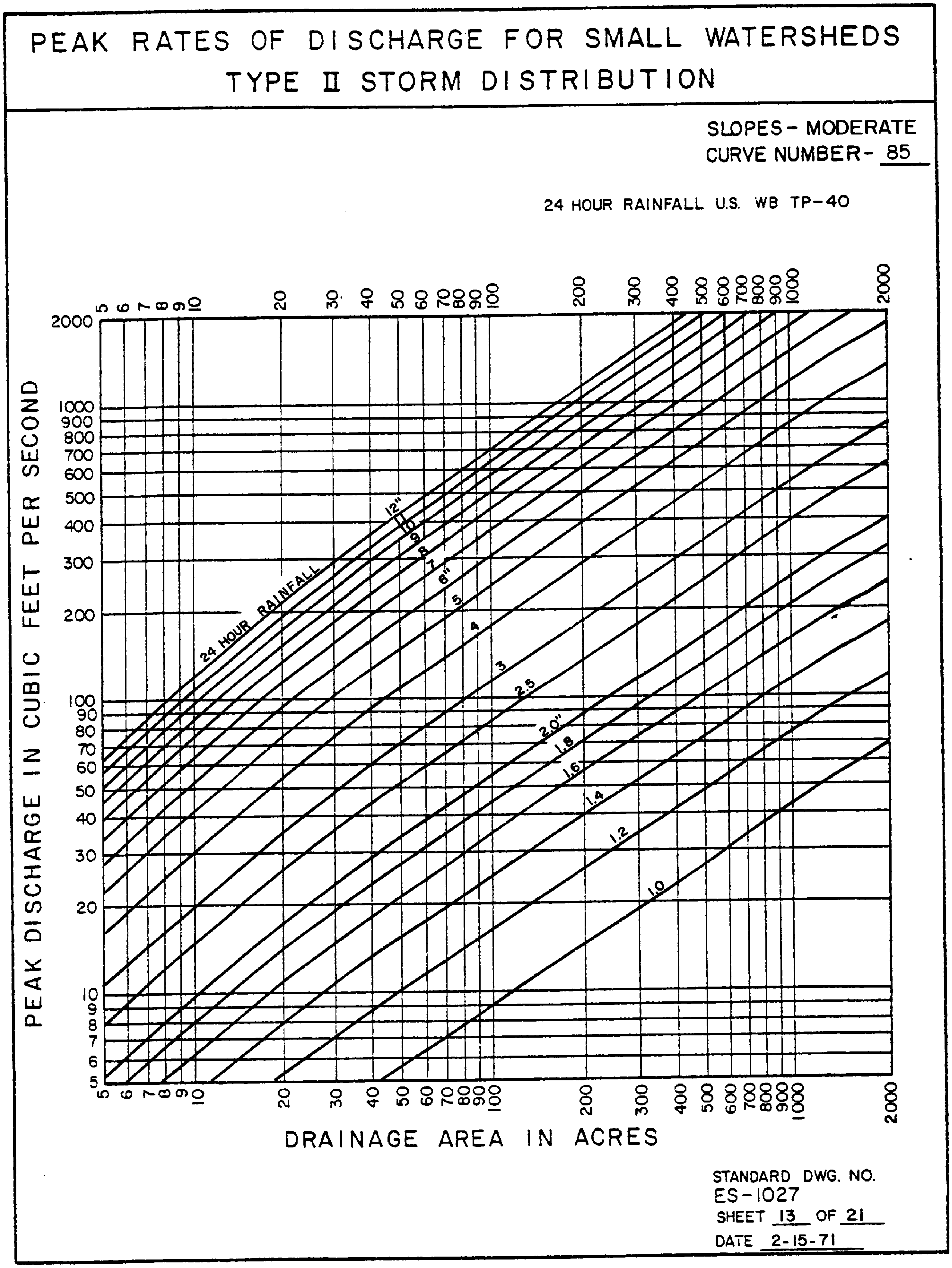
PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 80

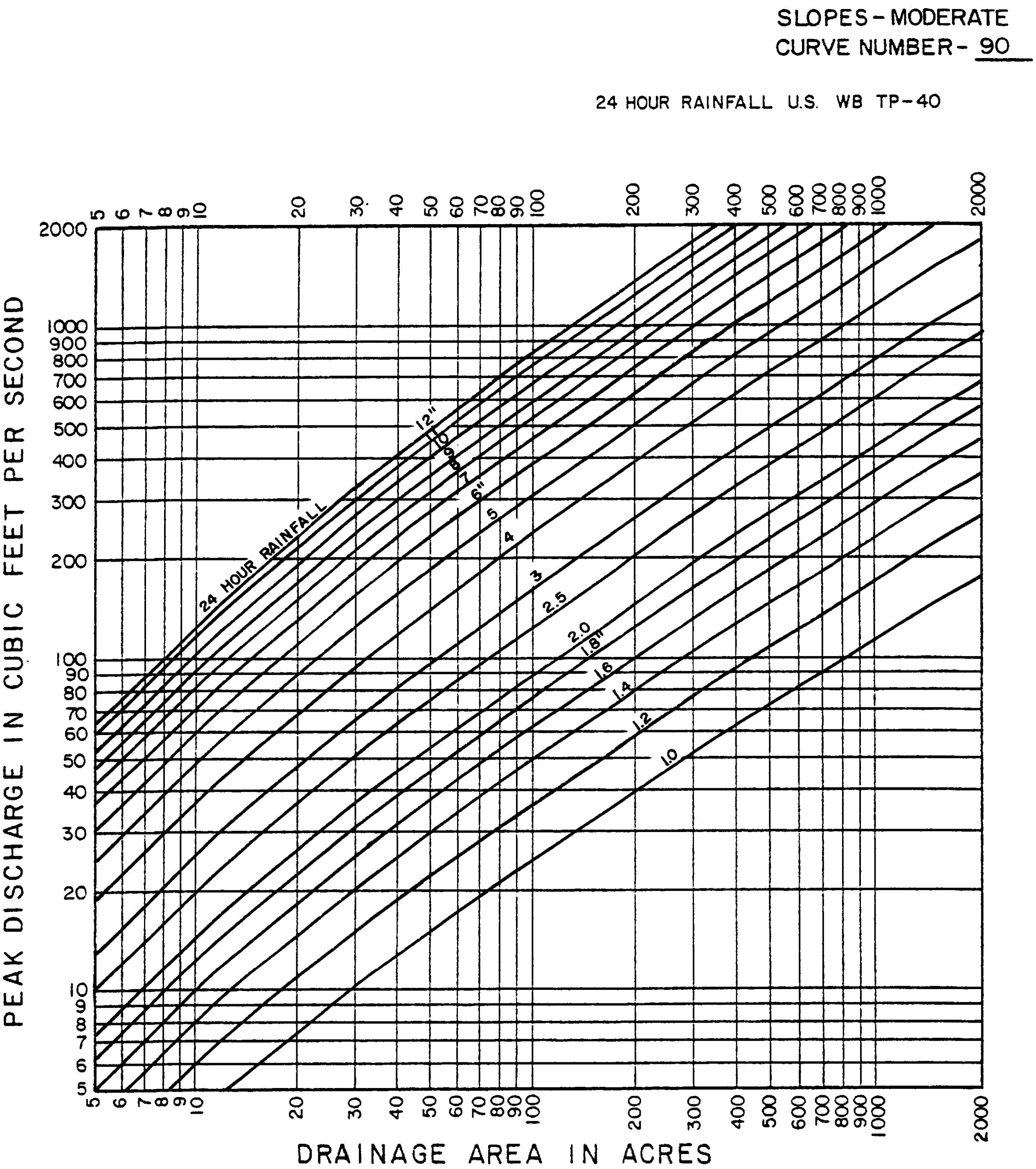


STANDARD DWG. NO.
ES-1027
SHEET 12 OF 21
DATE 2-15-71

Fig. 3-24


PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES - MODERATE
CURVE NUMBER - 85
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
10
9
8
7
6"
5
4
3
2.5
2.0"
1.8
1.6
1.4
1.2
1.0
PEAK DISCHARGE IN CUBIC FEET PER SECOND
2000
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200 300 400 500 600 700 800 900 1000 2000
DRAINAGE AREA IN ACRES
STANDARD DWG. NO.
ES-1027
SHEET 13 OF 21
DATE 2-15-71

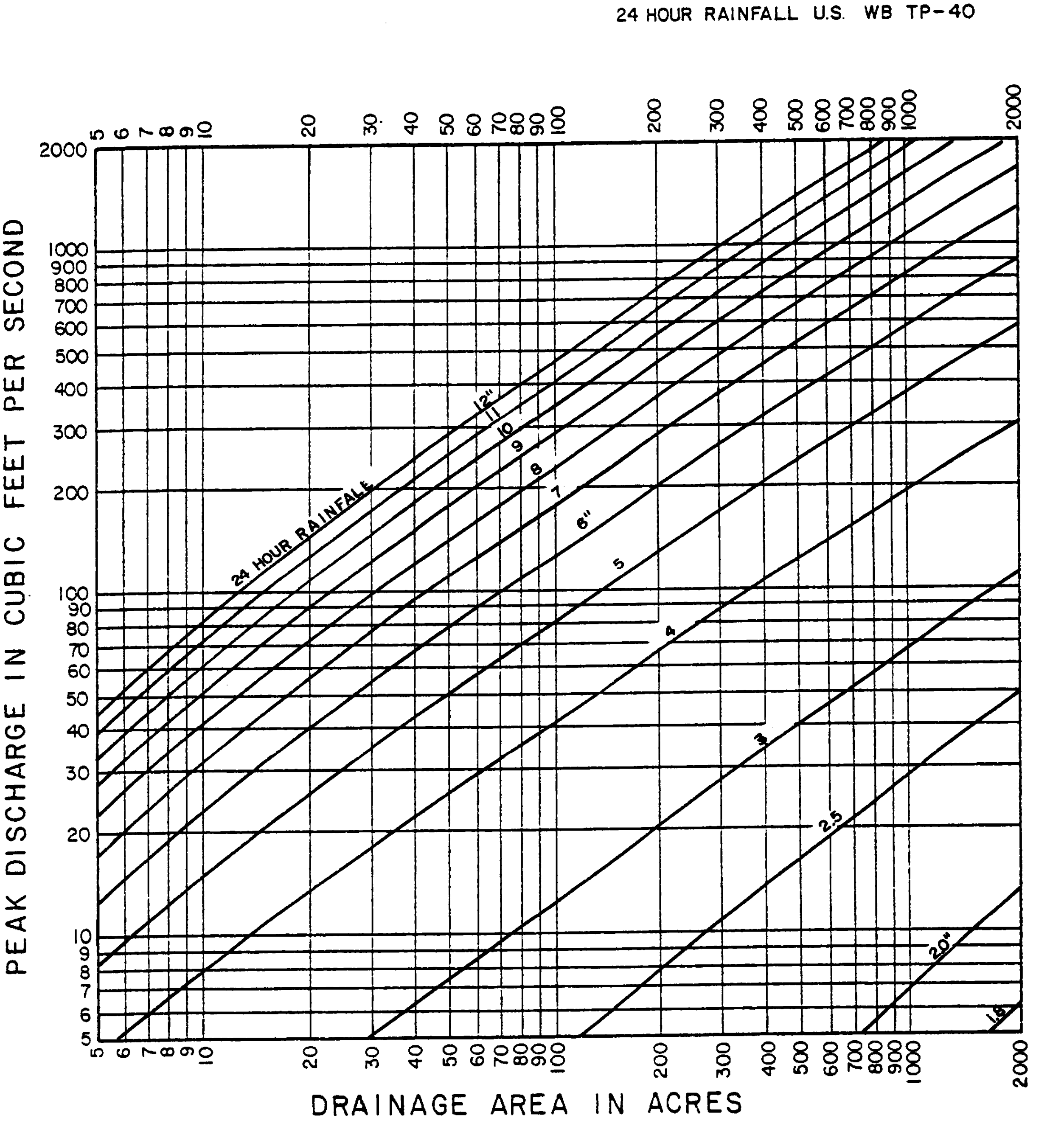
PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION



STANDARD DWG. NO.
ES-1027
SHEET 14 OF 21
DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 60



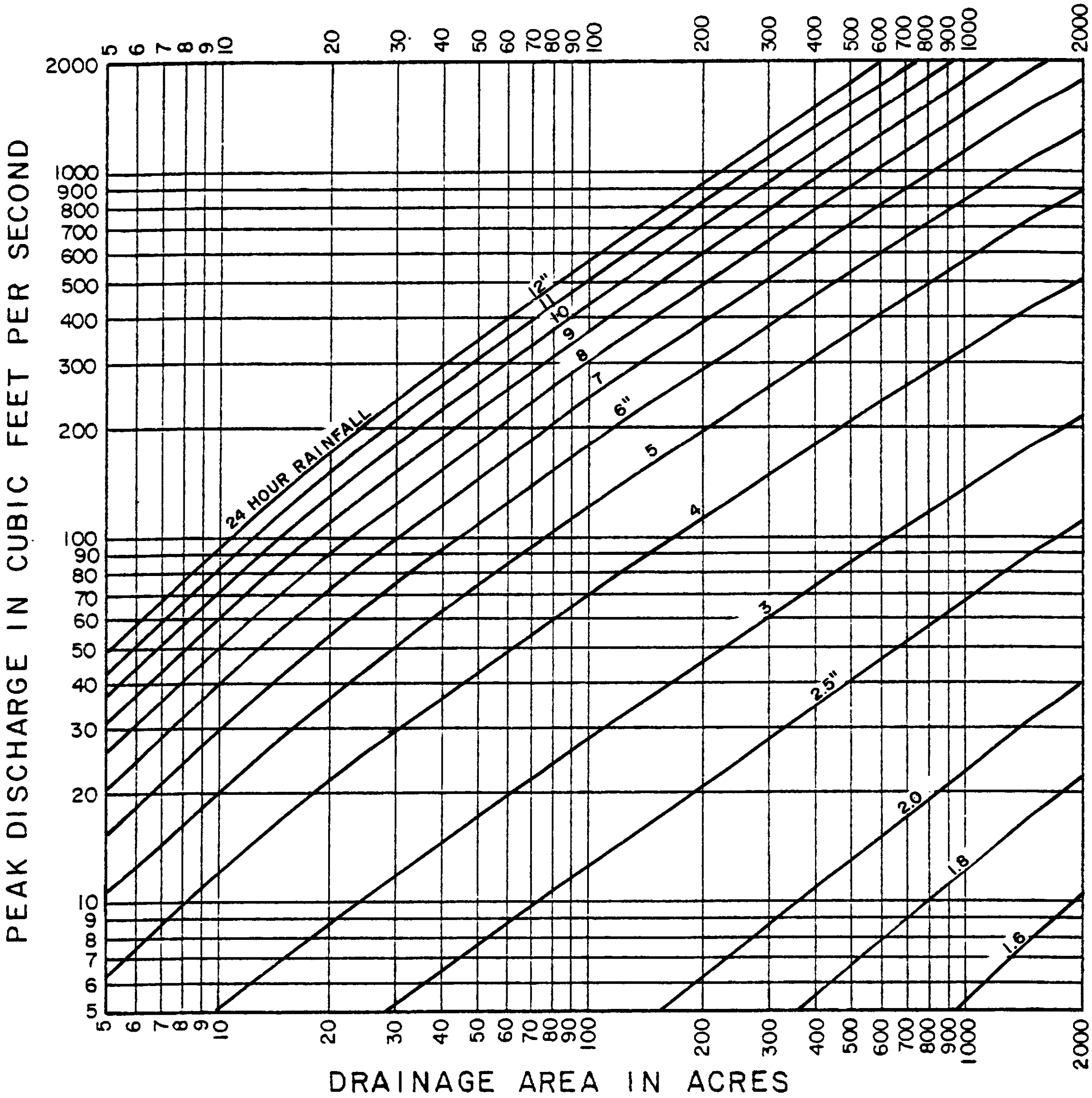
STANDARD DWG. NO.
ES-1027
SHEET 15 OF 21
DATE 2-15-71

Fig. 3-27

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES- STEEP
CURVE NUMBER- 65

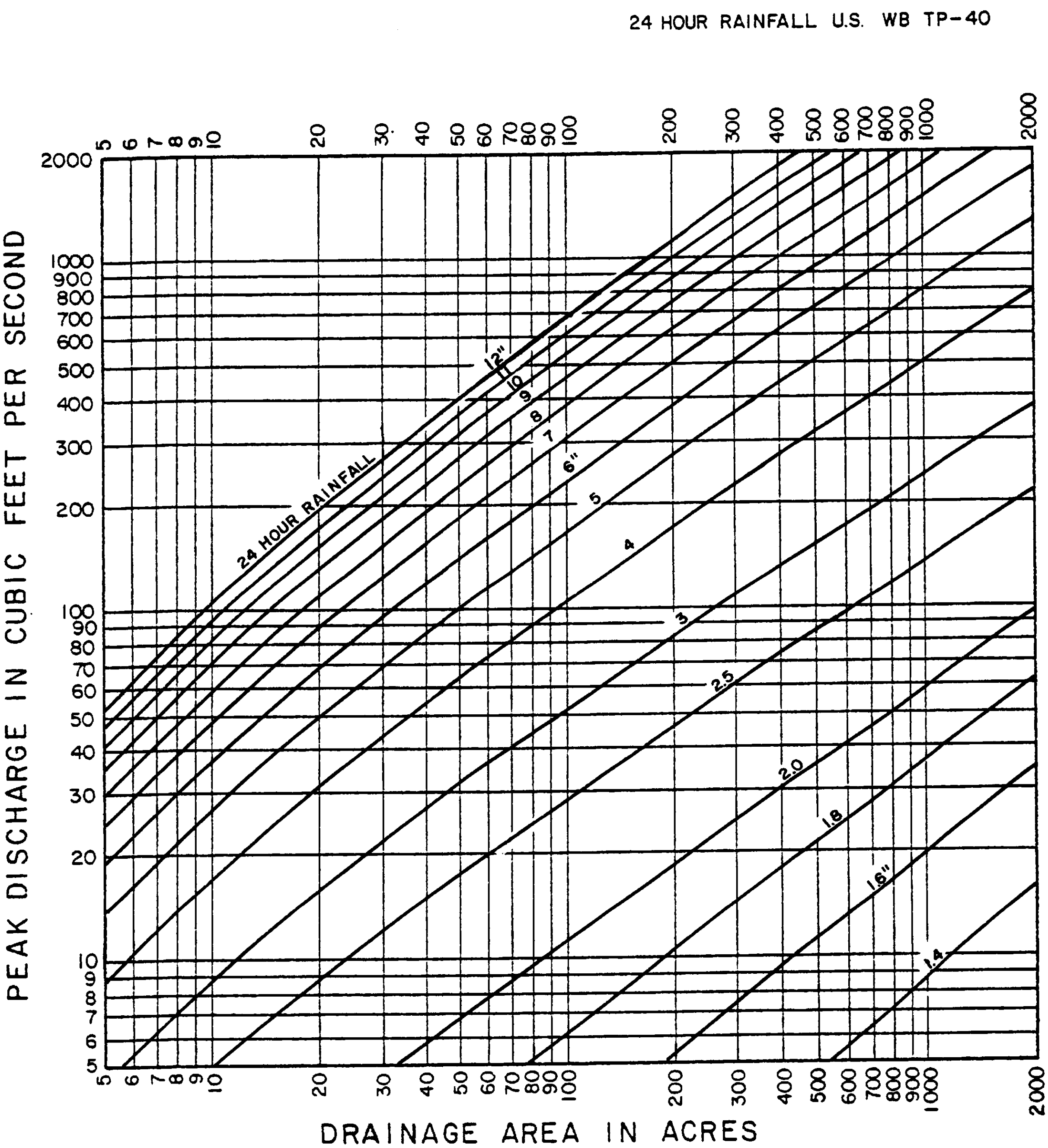
24 HOUR RAINFALL U.S. WB TP-40



STANDARD DWG. NO.
ES-1027
SHEET 16 OF 21
DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 70



STANDARD DWG. NO.
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Fig. 3-29

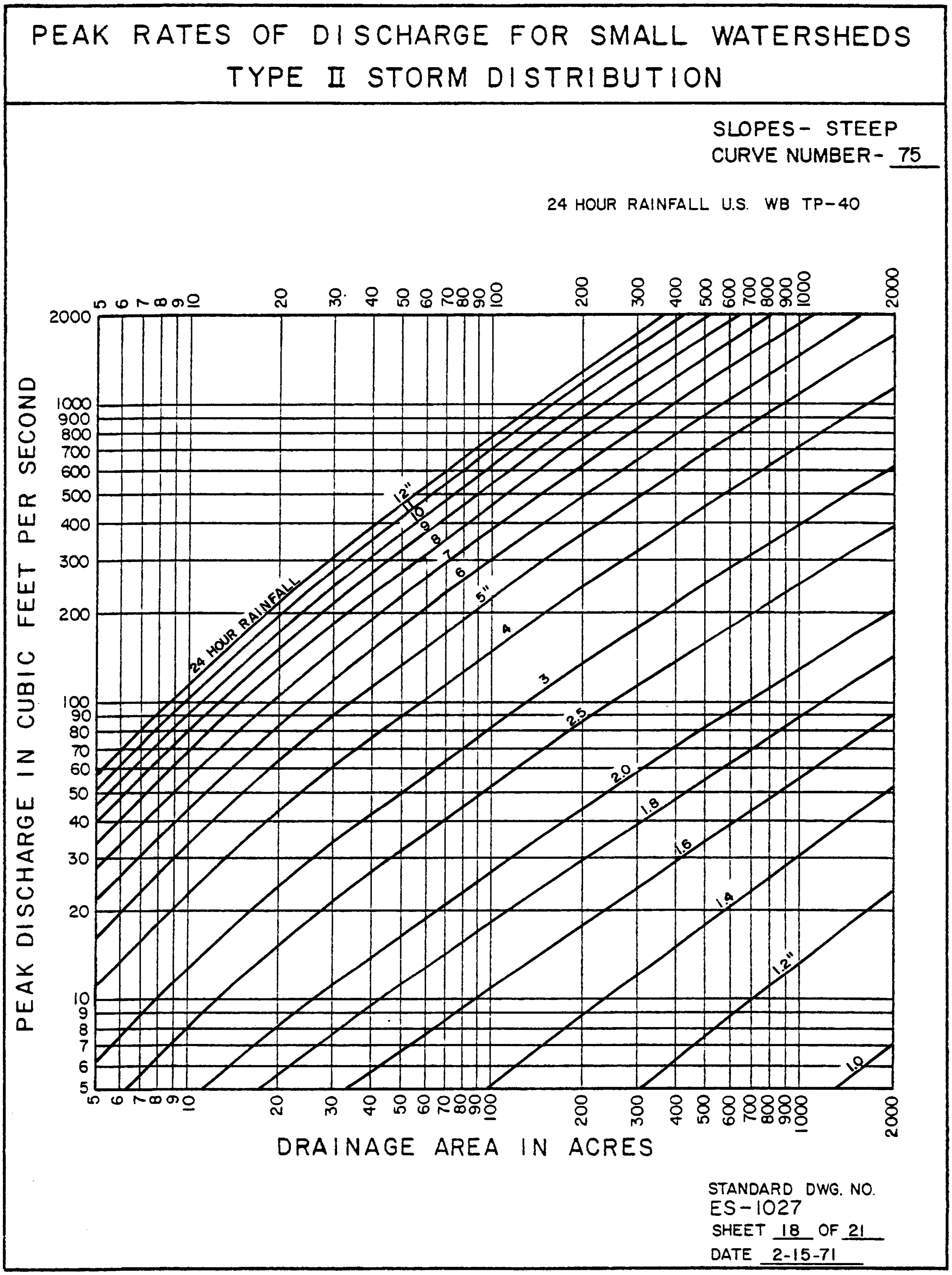
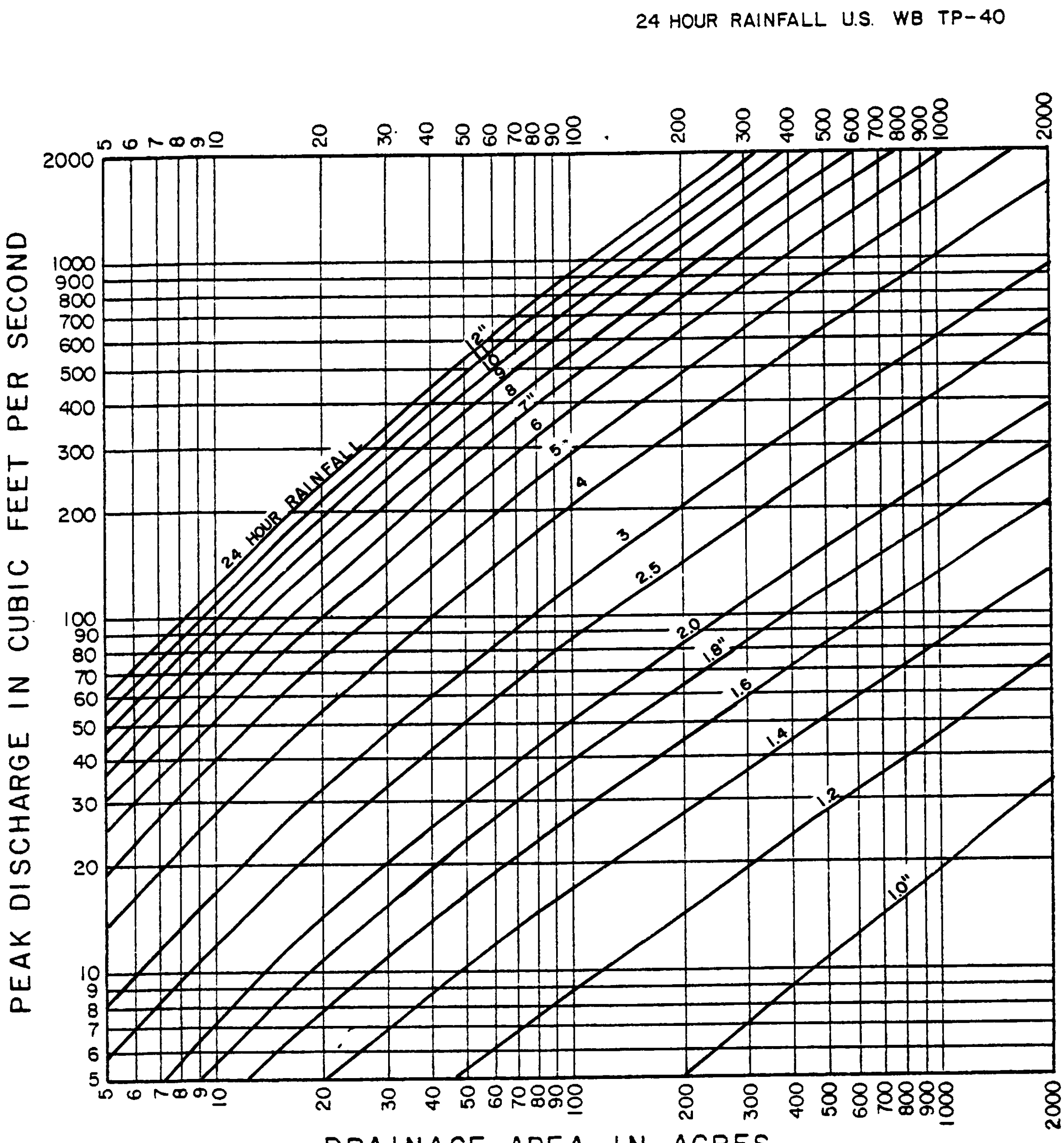

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION
SLOPES - STEEP
CURVE NUMBER - 75
24 HOUR RAINFALL U.S. WB TP-40
24 HOUR RAINFALL
12"
10
9
8
7
6
5"
4
3
2.5
2.0
1.8
1.6
1.4
1.2"
1.0
PEAK DISCHARGE IN CUBIC FEET PER SECOND
DRAINAGE AREA IN ACRES
STANDARD DWG. NO.
ES-1027
SHEET 18 OF 21
DATE 2-15-71

Fig. 3-30

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES- STEEP
CURVE NUMBER- 80



STANDARD DWG. NO.
ES-1027
SHEET 19 OF 21
DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

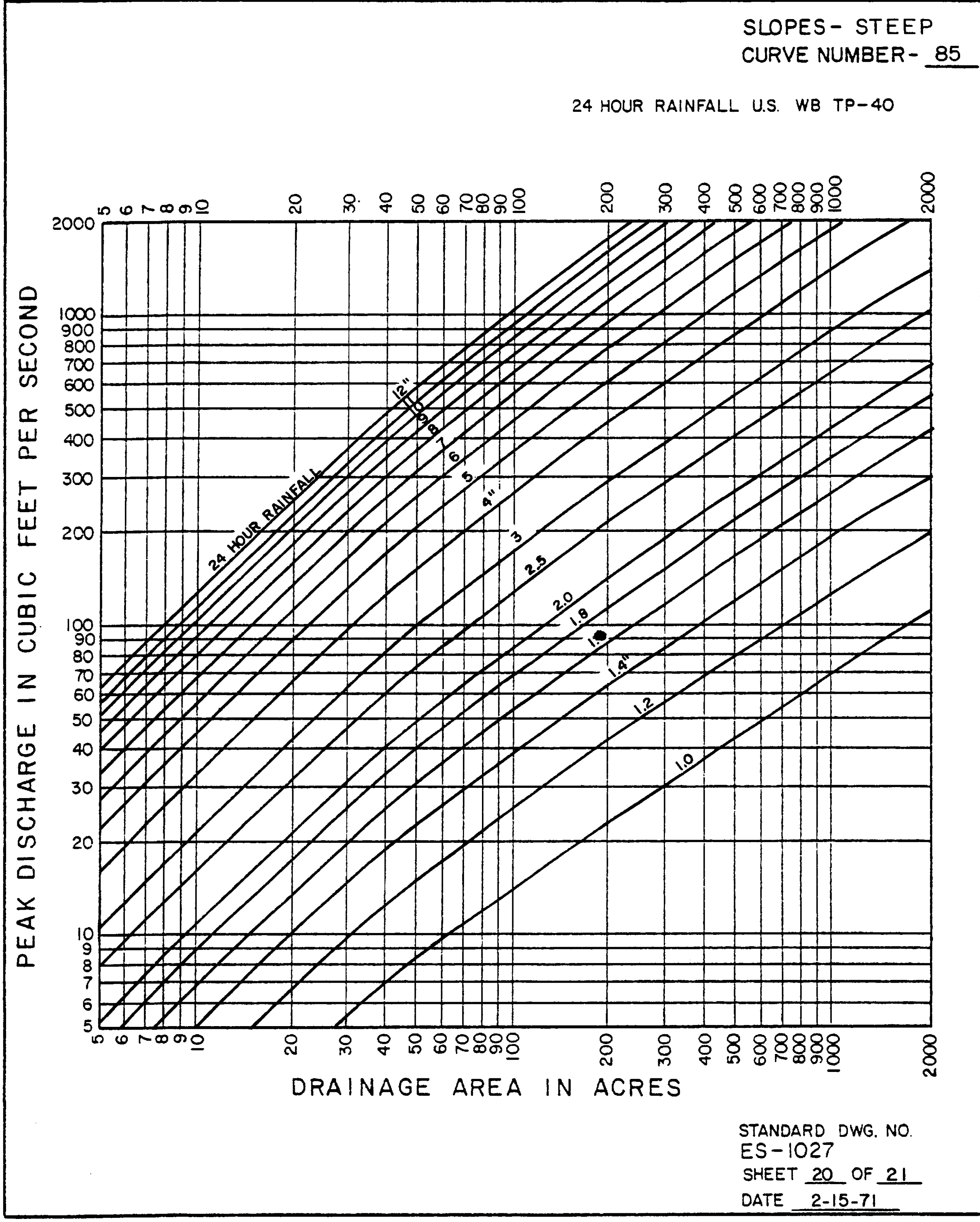
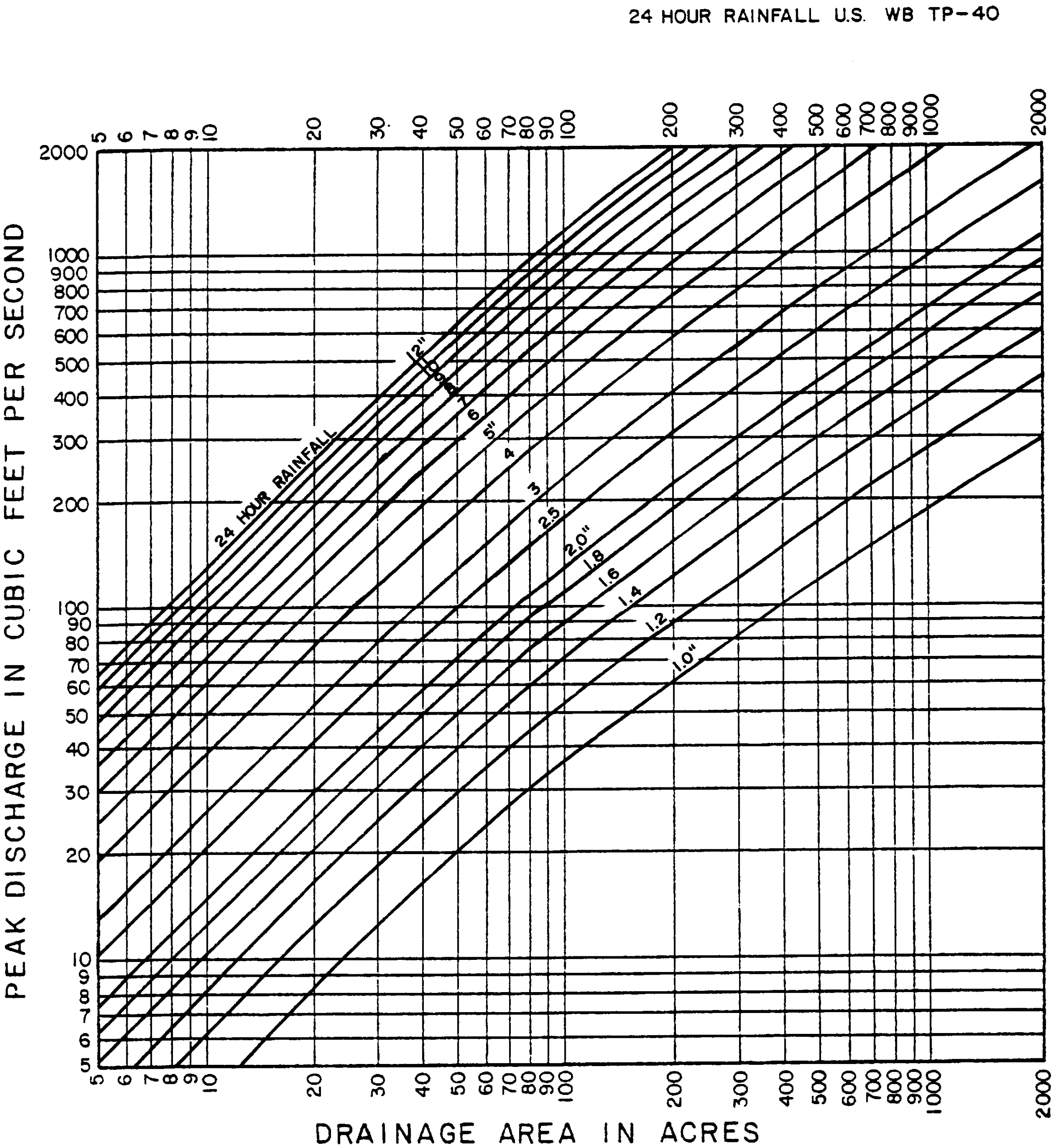


Fig. 3-32

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 90



STANDARD DWG. NO.
ES-1027
SHEET 21 OF 21
DATE 2-15-71

WORKSHEET FOR SCS METHOD

LOCATION:

Refer to "Urban Hydrology for Small Watersheds" Technical Release No. 55, SCS.

DRAINAGE AREA ______________________ ACRES

WEIGHTED CURVE NUMBER (PRESENT CONDITION) =

WEIGHTED CURVE NUMBER (FUTURE CONDITION) =

HYDRAULIC LENGTH = FT.

EQUIVALENT DRAINAGE AREA (FIGURE 3-11) = (A)

Q_2 24-HOUR RAINFALL (FIGURE 3-4 TO 3-7) = 24 ( ) = IN.*

Q_5 24-HOUR RAINFALL (FIGURE 3-4 TO 3-7) = 24 ( ) = IN.

Q_{10} 24-HOUR RAINFALL (FIGURE 3-4 TO 3-7) = 24 ( ) = IN.

Q_{25} 24-HOUR RAINFALL (FIGURE 3-4 TO 3-7) = 24 ( ) = IN.

Q_{50} 24-HOUR RAINFALL (FIGURE 3-4 TO 3-7) = 24 ( ) = IN.

*Can also be obtained from Reference 7.

ELEVATION AT BASIN DIVIDE = FT.

ELEVATION AT STRUCTURE = FT.

ΔH = FT.

SLOPE = (ΔH ÷ Hydraulic Length) (100) = (%)

SLOPE ADJUSTMENT FACTOR (Table 3-6) = (B)

PEAK FACTOR FOR IMPERVIOUS AREA (Figure 10A) = (C)

PEAK FACTOR FOR HYDRAULIC LENGTH MODIFICATION (Figure 10B) = (D)

Q_b = (Use Figure 3-12 to Figure 3-32 and equivalent drainage area).

$$Q = Q_b \frac{\text{Drainage Area}}{(A)} (B)\ (C)\ (D)$$

3-402.2 METHODS FOR CONTROLLING PEAK DISCHARGE FROM URBANIZING AREAS

As rural areas urbanize, the increase in peak discharge due to more efficient conveyance paths and increased impervious areas can have a significant adverse impact on downstream areas. There is a growing interest on the part of planners, developers, and the public in protecting downstream areas from induced flood damages that may accompany increased peaks and stages. Planning authorities are proposing local ordinances that restrict the type of development permitted and the impact development can have on the watershed. One of the primary controls being imposed is that future condition discharges cannot exceed present-condition discharge at some predetermined frequency of occurrence at specified points on the channel.

Earlier sections discussed methods of determining changes in peak discharges. This section discusses type of measure or construction techniques that can be used to control peak discharges from urbanizing area through planned runoff delay and increased infiltration.

Methods to control runoff in urbanizing areas reduce either the volume or the rate of runoff. The effectiveness of any control method depends on available storage, outflow rate, and inflow rate. Because a great variety of methods can be used to control peak flows, each method proposed should be evaluated for its effectiveness in the given area.

Table 3-8, page 3-67 lists measure for reducing and delaying urban storm runoff. Table 3-9, page 3-68 lists some advantages and disadvantages of each measure. Both tables were adapted from tables prepared at Pennsyvania State University under the direction of Gert Aron, Associate Professor of Civil Engineering. Effective measures for reducing peak rates of runoff are, of course, not limited to those listed in Table 3-8.

The direct reduction of peak flows and volume of runoff through installation of these measures is very difficult to determine. Measures that increase infiltration also reduce runoff; therefore, the runoff curve number will be lower than it would be without the measures. Measures that delay runoff also increase the time of concentration. The degree of change in curve number or time of concentration over the watershed depends on how extensively each measure is applied.

When a structure, such as a retarding dam or holding pond is installed, hydraulic routing procedures can be used to determine the effect on peak discharges. A less accurate method has been developed for quickly analyzing effects of storage reservoirs on peak discharges. The method is based on average storage and routing effects for many structures, and is covered in detail in Chapter 7, of the Soil Conservation Service Technical Release No. 55.

Table 3-8 - Measures for reducing and delaying urban storm runoff

AREA	REDUCING RUNOFF	DELAYING RUNOFF
Parking Lots	1. Porous payment a. Gravel parking lots b. Porous or punctured asphalt 2. Concrete vaults and cisterns beneath parking lots in high value areas. 3. Vegetated ponding areas around parking 4. Gravel trenches	1. Grassy strips on parking lots 2. Grassed waterways drainage parking 3. Ponding and detention measures for impervious areas a. Rippled pavement b. Depressions c. Basins
Residential	1. Gravel driveways 2. Contoured landscape 3. Ground-water recharge a. Perforated pipe b. Gravel (sand) c. Trench d. Porous pipe e. Dry walls 4. Vegetated depressions	1. Reservoir or detention basin 2. Planting a high delaying grass (high roughness) 3. Gravel driveways 4. Grassy gutters or channels 5. Increased length of travel of runoff by means of gutters, diversions, etc.
General	1. Gravel alleys 2. Porous sidewalks 3. Mulched planters	1. Gravel alleys

Table 3-9 - Advantages and disadvantages of measures for reducing and delaying runoff - Continued

MEASURE	ADVANTAGES	DISADVANTAGES
A. Porous pavement (parking lots and alleys) a. Gravel parking lot b. Holes in impervious pavements (1/4 in.) filled with sand	1. Runoff reduction (a and b) 2. Potential ground water recharge (a and b) 3. Gravel pavements may be cheaper than asphalt or concrete (a)	1. Clogging of holes gravel pores (a and b) 2. Compaction of earth below pavement or gravel decreases permeability of soil (a and b) 3. Ground-water pollution from salt in winter (a and b) 4. Frost heaving for impervious pavement with holes (b) 5. Difficult to maintain 6. Grass or weeds could grow in porous pavement (a and b)
B. Grassed channels and vegetated strips	1. Runoff delay 2. Some runoff reduction (infil tration recharge 3. Esthetically pleasing a. Flowers b. Trees	1. Sacrifice some land areas for vegetated strips 2. Grassed areas be mowed or cut periodically (maintenance costs)
C. Ponding and detention measures on impervious pavement a. Rippled pavement b. Basins c. Constructed inlets	1. Runoff delay (a, b, and c) 2. Runoff reduction (a and b)	1. Somewhat restricted movement of vehicle (a) 2. Interferes with normal use (b and c) 3. Damage to ripped pavement during snow removal (a) 4. Depression collect dirt and debris (a, b and c)

Table 3-9 - Advantages and disadvantages of measures for reduction and delaying runoff - Continued

MEASURE	ADVANTAGES	DISADVANTAGES
D. Reservoir of detention basin	1. Runoff delay 2. Recreation benefits a. Ice skating b. Baseball, football, etc., if land is provided 3. Esthetically pleasing 4. Could control large drainage areas with low release	1. Considerable amount of land is necessary 2. Maintenance costs a. Mowing grass b. Herbicides c. Cleaning periodically (silt removal) 3. Mosquito breeding area 4. Siltation in basin
E. Ground-water recharge a. Perforated pipe or hose b. French basin c. Porous pipe d. Dry well	1. Runoff reduction (infiltration) 2. Ground-water recharge with relatively clean water 3. May supply water to garden of dry areas 4. Little evaporation loss	1. Clogging of pores of perforated pipe 2. Initial expense of installation (materials)
F. High delay grass (High roughness)	1. Runoff delay 2. Increased infiltration	1. More difficult to mow
G. Routing flow over lawn	1. Runoff delay 2. Increase infiltration	1. Possible erosion or scour 2. Standing water of lawn in depressions

3-403 MULTIPLE REGRESSION ANALYSIS

Values for peak discharge for various recurrence intervals at long-term gaging stations can be found in "Floods in Arkansas, Magnitude and Frequency" (Reference 1), by James L. Patterson, U.S.G.S. However, information is generally required at points where data is not available. The most effective way now known for estimating floodflow characteristics of ungaged sites is to define mathematical equations between streamflow characteristics and basin parameters by multiple-regression analysis of data collected at gaged sites. Once the equation is defined, streamflow characteristics for ungaged sites can be computed by determining appropriate values of basin parameters and substituting these values in the equation.

The equation has the form:

$$Y = aA^{b1} S^{b2} L^{b3} \quad \text{(Region A, Figure 3-34)}$$

$$Y = aA^{b1} S^{b2} E^{b3} P^{b4} \quad \text{(Region B, Figure 3-34)}$$

where

A = drainage area, in square miles, as determined by planimeter from the best available maps.

L = Main channel length, in miles, from the site to the basin divide.

S = Main channel slope in feet per mile, computed from elevations at points 10 percent and 85 percent of the distance along the main channel from the site to the basin divide. When stream slopes exceed 30 feet per mile, a value of 30 feet should be used for S in equations containing this parameter.

P = Mean annual precipitation from Figure 3-33, page 3-78. The parameter used was mean annual precipitation minus 30.

E = Mean basin elevation in feet above mean sea level, measured on maps by laying a transparent grid over the map, determining the elevation at each grid intersection, and averaging those elevations. The grid spacing was selected to give at least 25 intersections within the basin boundary. For drainage area less than about 5 square miles, larger scale maps were used where available.

In deriving regression equations the State was divided into two regions (Figure 3-34) page 3-79. Region A includes most of the Mississippi Alluvial Plain in Arkansas, with the exception of Crowleys Ridge. Region B includes the rest of the State. A summary of regression equations are shown in Table 3-10 and 3-11, page 3-76, and 3-77. Graphical solutions can be found in Reference 1. It should be noted that Tables 3-10 and 3-11 do not cover some cases for small drainage areas. Those equations and graphical solutions can be found in Figures 3-35 to Figure 3-40, pages 3-80 to 3-85. Regression equations should not be used at sites where floodflow is affected by significant overbank or non-natural storage in farm ponds or reservoirs, by diversion, by urban or suburban development of a significant part of the basin or by any man made projects that are placed in the drainage basin that affect flow.

It should be understood that sites in urban areas, or areas affected by urban runoff, should be analyzed very carefully because runoff occurs much quicker in urban areas causing greater discharges. If the regression formulas were used in this situation an unrealistically low discharge would be obtained resulting in an undersized structure.

The following two examples are taken from "Floods in Arkansas, Magnitude and Frequency" by James L. Patterson.

EXAMPLE 1 -- Assume that the peak discharge for a 50-year flood for Strawberry River at bridge on U.S. Highway 167, near Evening Shade (Station 730) is to be determined. This discharge is computed as follows:

1. Floodflows of this stream are virtually unaffected by manmade changes; hence, regression equations are applicable.

2. The site is in Region B (Figure 3-34), therefore, one of the equations for streams in this region is applicable (Table 3-11, equation 13(a), 13(b), 13(c) or 13(d). Assume that it is decided to used equation 13(a).

$$Q_{50} = 21.9\ A^{0.62} S^{0.33} E^{0.31} P^{0.45}$$

which contains all statistically significant variables. It will be necessary to determine values for drainage area (A), main-channel slope (S), mean basin elevation (e), and mean annual precipitation minus 30 (P).

3. The drainage area upstream from the site, measured from the best available maps, is 225 square miles. Drainage areas for most large streams in the Arkansas, White, Red River, Ouachita and St. Francis basins can be obtained from USGS "Drainage Area" publications.

4. The main-channel slope is computed as 6.02 feet per mile by (a) determining elevations at points 10 percent and 85 percent of the distance along the main channel from the site of investigation to the basin divide, (b) computing the arithmetic difference, in feet, between these elevations and dividing by distance, in miles, between the points. The best available topographic maps should be used for this purpose.

5. The mean basin elevation is determined as 740 feet above mean sea level by using USGS topographic maps.

6. The mean annual precipitation is detemined as 44 inches from Figure 3-33. The value of P, to be used in the regression equation, is 44 minus 30 = 14.

7. The peak discharge of the 50-year flood is computed as:

$$Q_{50} = 21.9\ (225)^{0.62}(6.02)^{0.33}(740)^{0.31}(14)^{0.45} = 30{,}200 \text{ cfs}$$

If it is decided to use equation 13(c), in which the only parameters considered are area and channel slope, the 50-year flood is computed as:

$$Q_{50} = 164\ A^{0.75}S^{0.63} = 164(225)^{0.75}(6.02)^{0.63} = 29{,}500 \text{ cfs}$$

Equation 13(c) can also be solved by using diagram in Figure A13 in Appendix A of Reference 1. Based on 30 years of record, the 50-year peak discharges at this site, analyzed by log Pearson Type III Method and listed in Table Al, Reference 1, is 30,100 cfs.

EXAMPLE 2 -- Assume the peak discharges for floods having recurrence intervals of 10, 25 and 50 years for Gun Springs Creek at U. S. Highway 67, near Higginson (Station 768.2) are to be determined.

1. The site is in Region B (Figure 3-34); therefore, equation ll(b), or 11(c) should be used to compute the value of the 10-year peak flow. An examination of standard errors shown in Table 3-11 indicates that equation 11(b) should be used.

$$Q_{10} = 112A^{0.78}S^{0.52}$$

2. The drainage area upstream from the site is determined to be 4.94 square miles.

3. The main channel slope is computed as 35.2 feet per mile by the method described in Example 1. As this slope exceeds 30 feet per mile, use 30 in the equation.

4. The peak discharge for the 10-year flood is then computed as:

$$Q_{10} = 112(4.94)^{0.78}(30)^{0.52} = 2{,}280 \text{ cfs}$$

This value can be determined by using diagram in Figure AII in Appendix A, Reference 1.

5. Equations 12 and 13, used to determine peak flows having recurrence intervals of 25 and 50 years, are based on peak flow data for only the long-term large-area gaging stations and should not be used for areas less than about 25 square miles. To compute values of peak discharge graphically for recurrence intervals of 25 and 50 years at the site, refer to Figure 3-38, and Figure 3-39. If the Q_{100} is needed it can be obtained from Figure 3-40 or Figure 3-41.

As an arbitrary rule, Patterson says to use gaging-station data if the period or record exceeds 15 years. Use regression equations if there is no peak flow data or if the period is 15 years or less. A work sheet to help solve regression equations is located at the end of the section (page 3-87).

If information is desired at a site that is close to a long term gaging station but not at the station, then the following method taken from "Floods in Arkansas, Magnitude and Frequency" (Reference 1) by James L. Patterson should be used.

If information is desired at a point between two long-tern gaging stations on a stream, peak-flow values may be computed by interpolation, on the basis of drainage area, between values taken from Table A1, Reference 1. Interpolation can be done by plotting discharge versus drainage area on logarithmic plotting paper for the two gaged sites, connecting these two points with a straight line and then entering this relation with the value of the drainage area at the site where information is desired. Interpolation should be not be used if the drainage area of the upstream station is less than half the downstream station.

When flood frequency data is needed at a site upstream or downstream from a long-term gaging station for which flood frequency relations have been defined, it is recommended that the appropriate regression equation listed in Table 3-10 or 3-11 be used with an adjusted value of regression constant "a" computed on the basis of the discharge obtained from Table A1, Reference 1. An example of this procedure follows:

Assume the value of the 50-year flood on Mulberry River at State Highway 23 near Cass is to be determined. The 50-year flood, Table A1, Reference 1, for Mulberry River near (Station 2520), based on 30 years of record, is 50,500 cfs. Mulberry River is in Region B (Figure 3-34). The regional equation for the 50 year flood (Table 3-11 for areas greater than 25 square miles, Equation 13(a)) is:

$$Q_{50} = aA^{0.62}S^{0.33}E^{0.31}P^{0.45}$$

Values of A, S, E and P are listed in Table A2, Reference 1, as 372 square miles, 18.1 feet per mile, 1,430 feet, and 22 (52 minus 30) inches, respectively. Substituting 50,500 cfs for Q_{50}, the value of "a" can be computed thus:

$$50{,}500 = a(372)^{0.62}181^{0.33}1430^{0.31}22^{0.45},$$

resulting in

$$a = 12.9$$

Values of A, S, E and P for the site near Cass are determined as 311 square miles, 23.0 feet per mile, 1,590 feet, and 23 (53 minus 30) inches, respectively, then

$$Q_{50} \text{ at Cass} = 12.9\ (311)^{0.62}(23.0)^{0.33}(1{,}590)^{0.31}(23)^{0.45} = 51{,}500 \text{ cfs}$$

Depending upon the degree of accuracy desired, the same procedure can be followed using equation 13(b), 13(c) or 13(d). It is recommended that this method not be used for drainage areas more than twice, or less than one-half, the size of the drainage area of the gaged site.

TABLE 3-10

Summary of regression equations for region A

(Model is $Y = aA^{b1}S^{b2}L^{b}$)

Equation number	Peak-flow characteristic, Y	Regression constant, a	Exponent of Basin Characteristic			Standard Error of Estimate, Percent	
			Drainage area, A	Main channel slope, S	Main channel length, L	Areas 100 sq. mi. or more	Areas 100 sq. mi. or less
Drainage areas, 100 square miles to 3,000 square miles							
1(a)	*Q_2	119	0.99	------	-0.58	23	-----
(b)	Q_2	76.7	.64	------	------	34	-----
2(a)	*Q_5	168	1.05	------	- .67	23	-----
(b)	Q_5	102	.64	------	------	36	
3(a)	*Q_{10}	215	1.08	------	- .74	23	-----
(b)	Q_{10}	124	.64	------	------	39	-----
4(a)	*Q_{25}	264	1.10	------	- .78	24	-----
(b)	Q_{25}	148	.63	------	------	41	-----
5(a)	*Q_{50}	295	1.11	------	- .78	26	-----
(b)	Q_{50}	164	.63	------	------	42	-----
Drainage areas, 0.2 square mile to 100 square miles							
6(a)	*Q_2	105	0.59	0.24	------	-----	32
(b)	Q_2	129	.55	------	------	-----	37
7(a)	*Q_5	152	.59	.30	------	-----	35
(b)	Q_5	197	.54	------	------	-----	38
8(a)	*Q_{10}	185	.58	.31	------	-----	34
(b)	Q_{10}	241	.53	------	------	-----	38

* See diagram for graphical solution in "Reference (1)"

TABLE 3-11

Summary of regression equations for region B

(Model is $y=aA^{b1}S^{b2}E^{b3}P^{b4}$; where S is greater than 30 feet per mile, use 30)

Equation number	Peak-flow characteristic, y	Regression constant, a	Exponent of basin characteristic				Standard error of estimate, percent	
			Drainage area, A	Main channel slope, S	Mean basin elevation, E	Mean annual precipitation minus 30, P	Areas 25 sq. mile or more	Areas less than 25 sq mi
Drainage area, 0.1 square mile to 3,000 square miles								
9(a)	Q_2	4.99	0.72	0.32	0.20	0.59	25	46
(b)	$*Q_2$	58.1	.77	.46	-----	--------	30	45
(c)	Q_2	276	.68	----	-----	--------	41	50
10(a)	Q_5	11.8	.72	.35	.21	.43	22	40
(b)	$*Q_5$	91.8	.78	.50	-----	--------	26	36
(c)	Q_5	498	.68	-----	-----	--------	40	40
11(a)	Q_{10}	17.2	.73	.37	.21	.36	22	40
(b)	$*Q_{10}$	112	.78	.52	-----	--------	26	36
(c)	Q_{10}	653	.68	----	-----	--------	40	40
**12(a)	Q_{25}	10.8	.62	.29	.36	.55	23	----
(b)	Q_{25}	65.6	.69	.45	.22	--------	24	----
(c)	$*Q_{25}$	117	.77	.63	-----	--------	26	----
(d)	Q_{25}	2680	.48	----	-----	--------	40	----
**13(a)	Q_{50}	21.9	.62	.33	.31	.45	25	----
(b)	Q_{50}	96.4	.68	.46	.20	--------	26	----
(c)	$*Q_{50}$	164	.75	.63	-----	--------	27	----
(d)	Q_{50}	3620	.46	----	-----	--------	41	----

* See diagrams for graphical solution in "Reference 1"

** Not applicable for drainage areas less than 25 square miles.
For drainage areas less than 25 square miles see Figure 38 to 40.

Fig. 3-33

MEAN ANNUAL PRECIPITATION

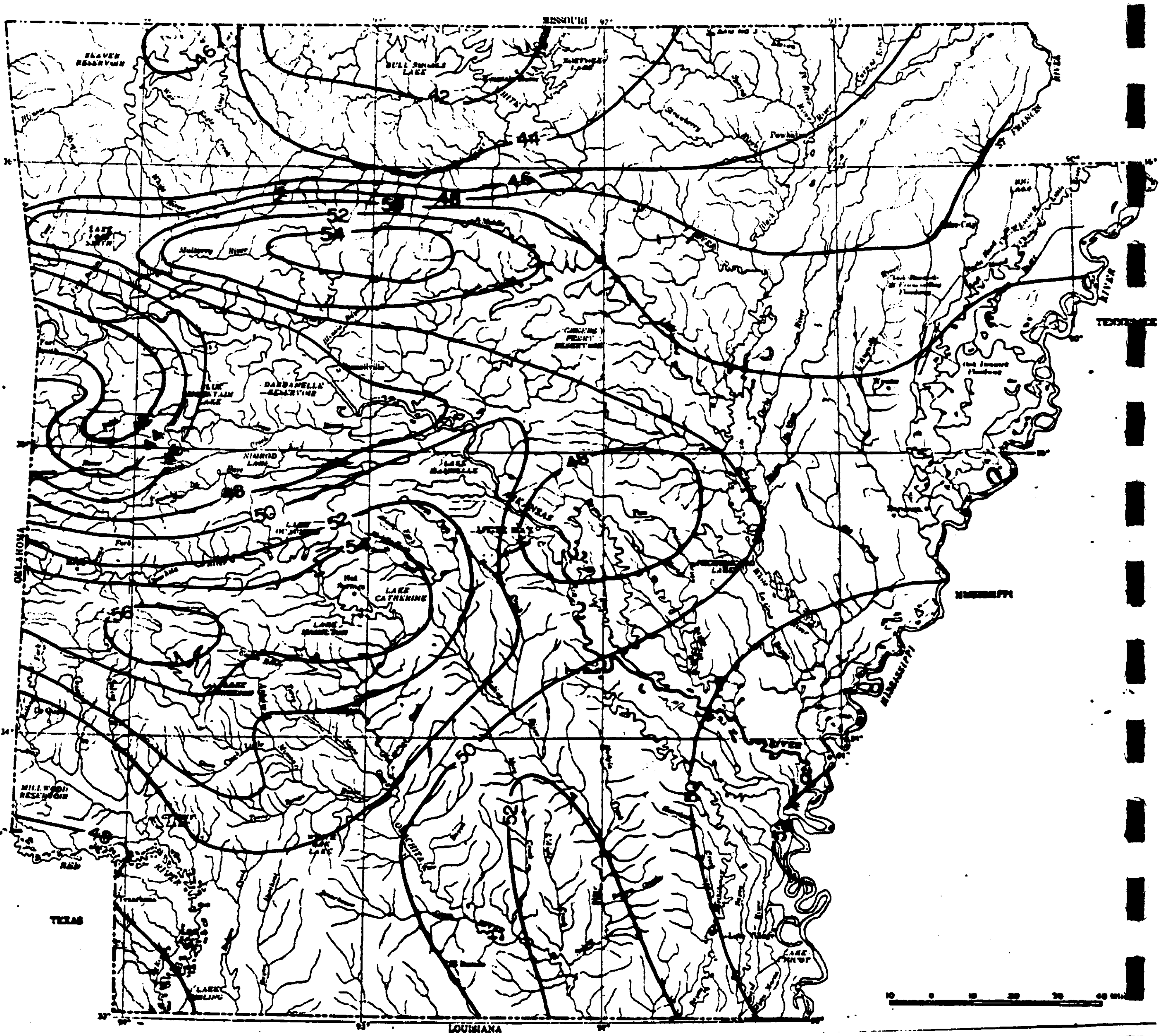
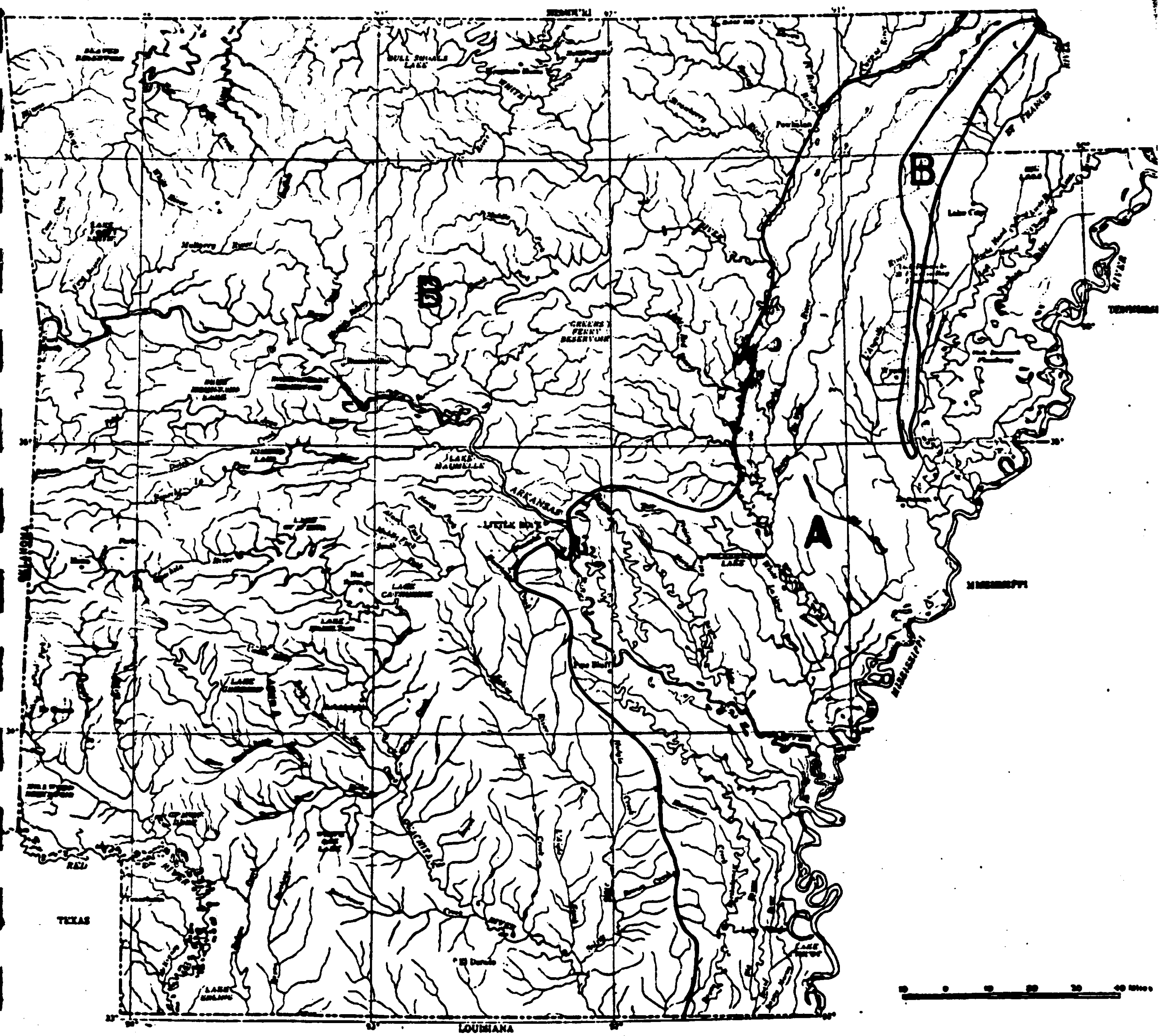


Fig. 3-34

REGIONAL BOUNDARY MAPS



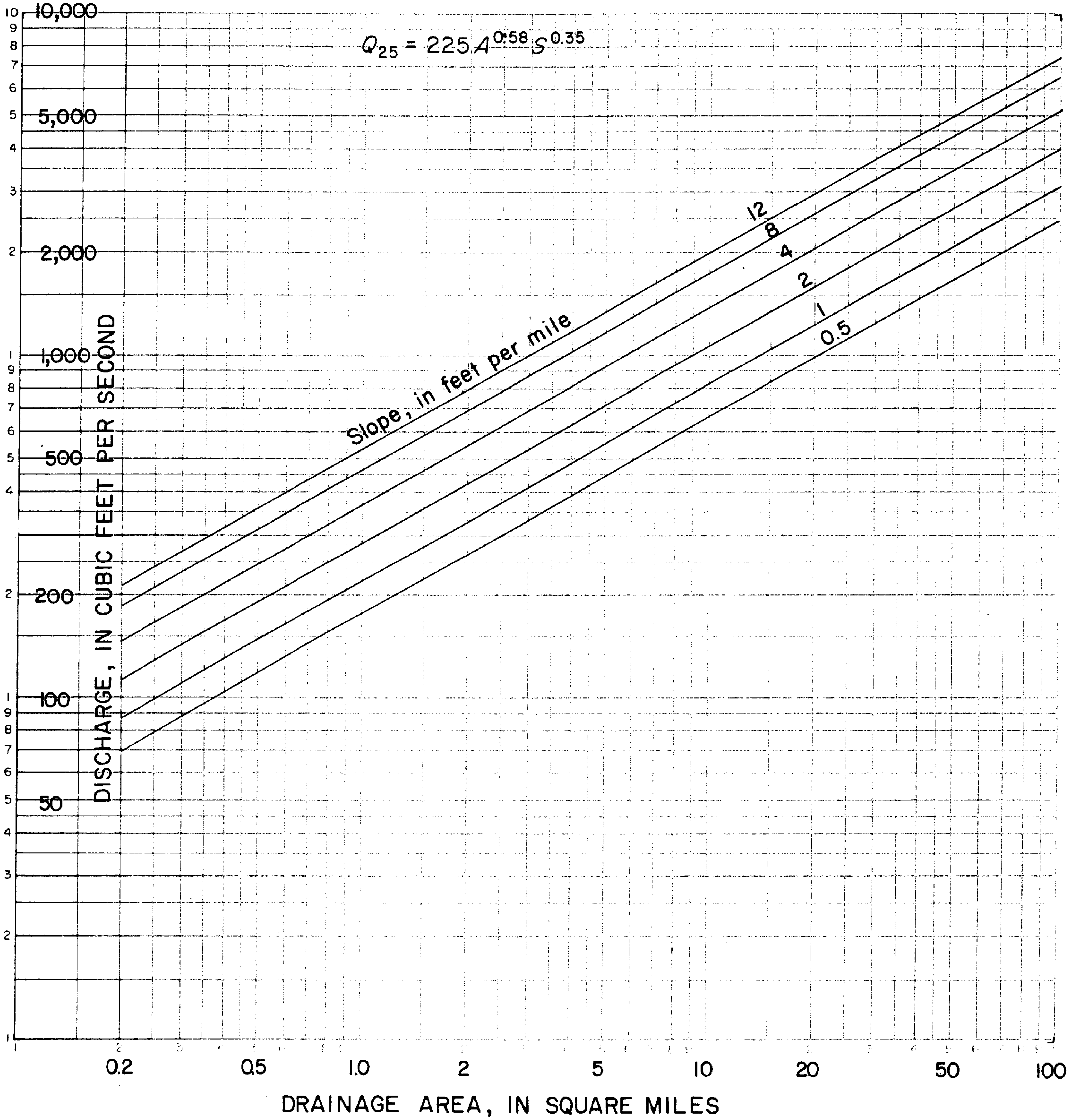


Figure 3-35--Graphical solution of a 25-year-flood equation for region A (drainage areas 0.1-100 square miles)

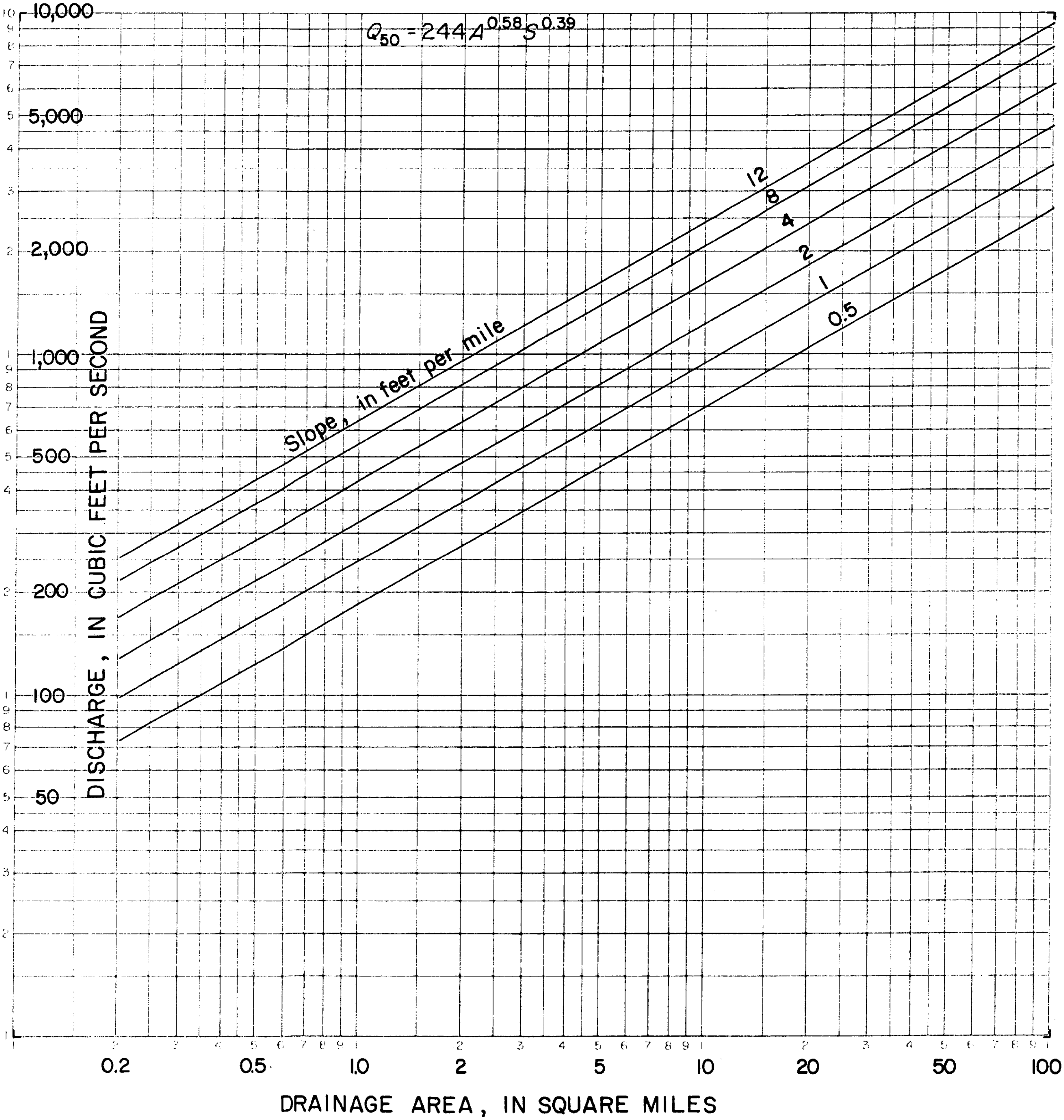


Figure 3-36--Graphical solution of a 50-year-flood equation for region A (drainage areas 0.1-100 square miles)

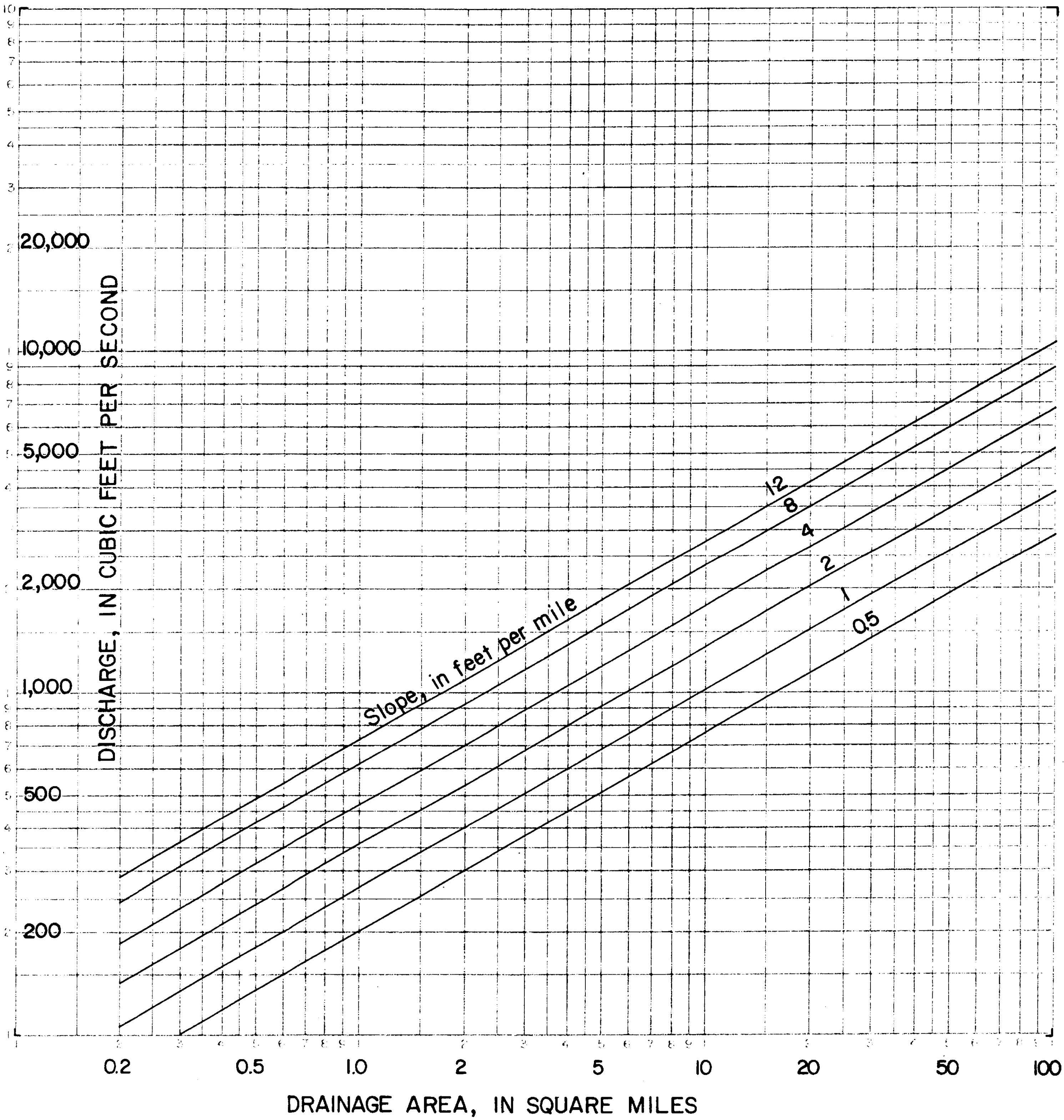


Figure 3-37--Graphical solution of a 100-year-flood equation for region A (drainage areas 0.1-100 square miles)

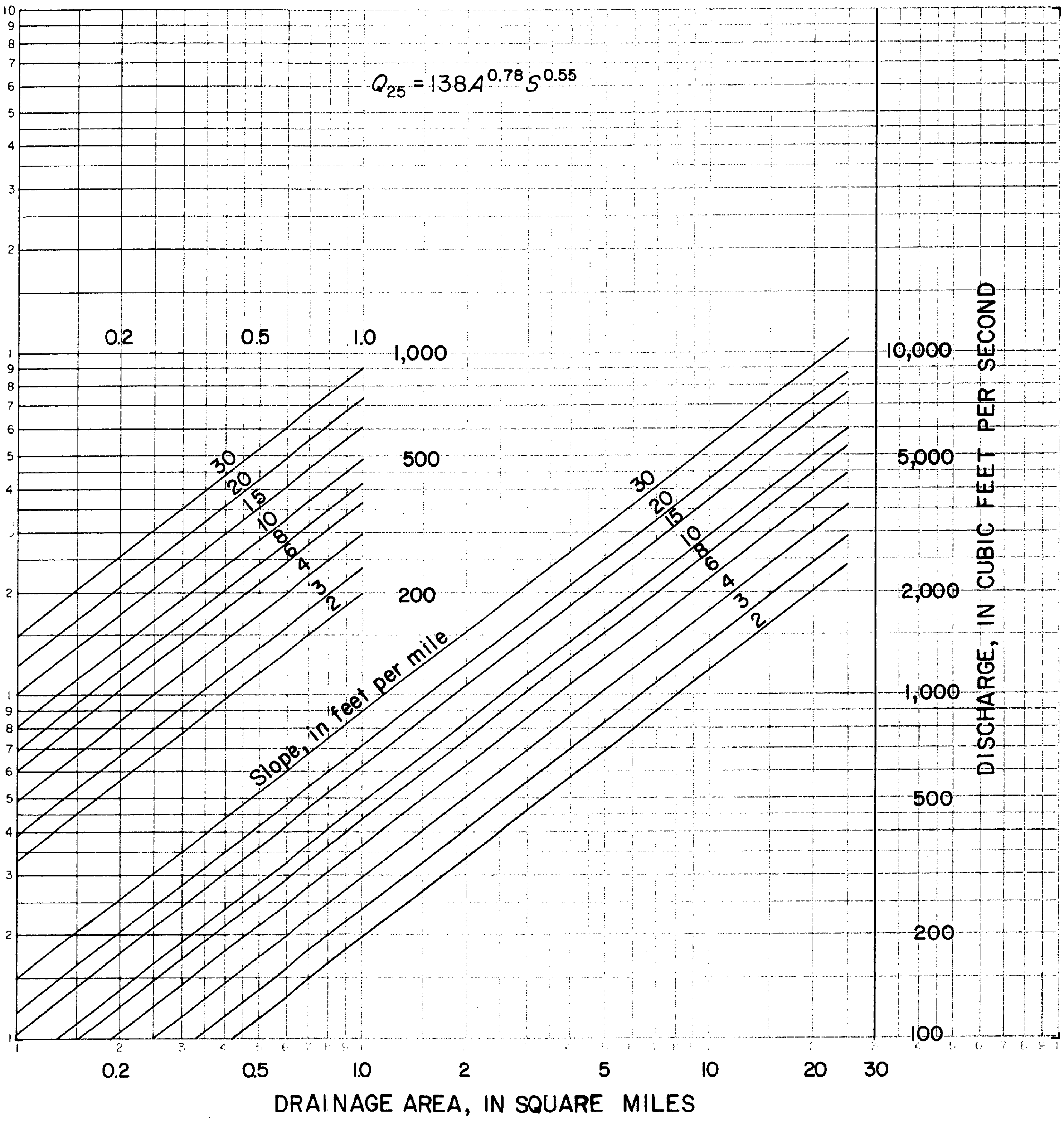


Figure 3-38--Graphical solution of a 25-year-flood equation for region B (drainage areas 0.1-25 square miles)

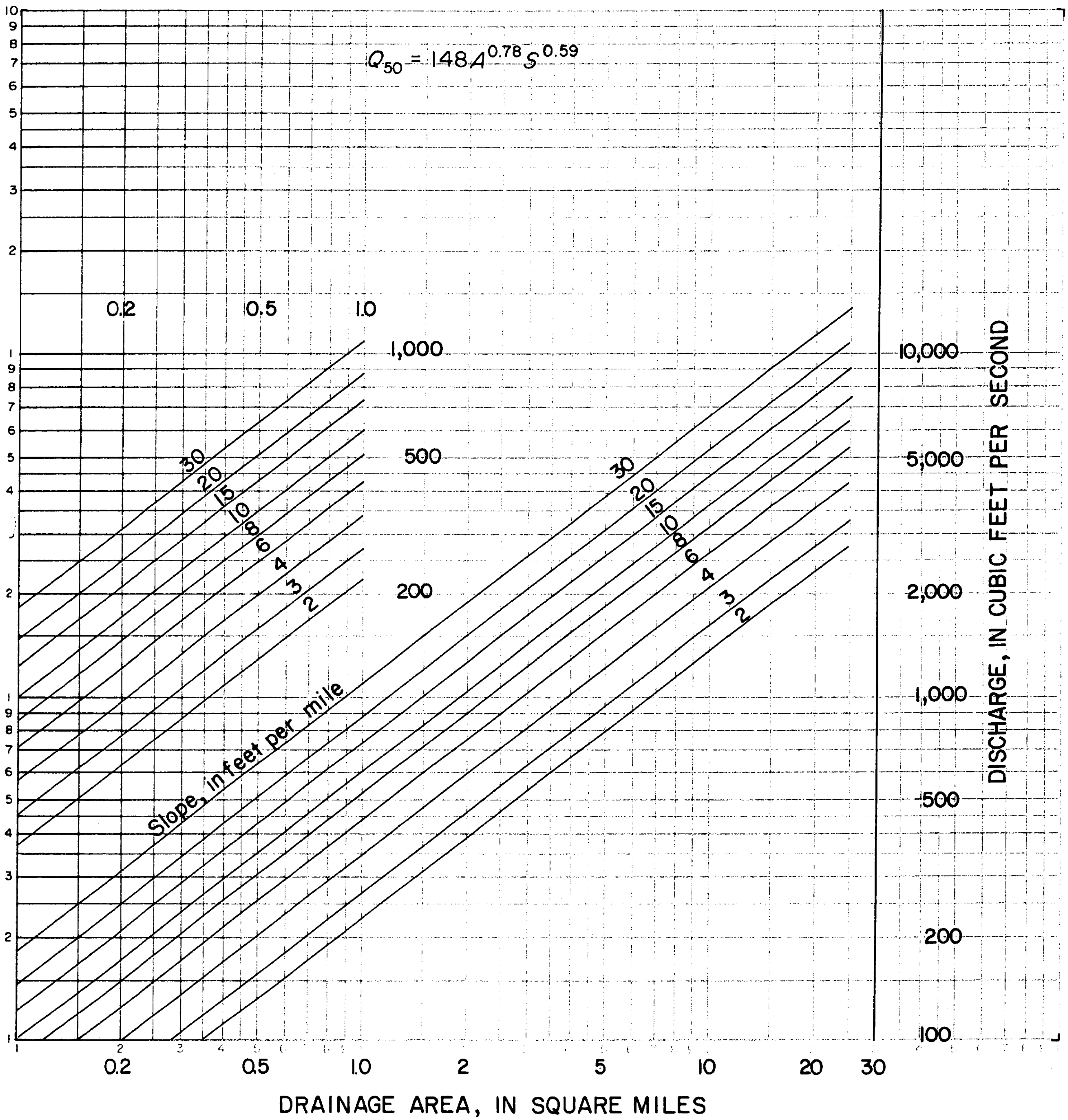


Figure 3-39--Graphical solution of a 50-year-flood equation for region B (drainage areas 0.1-25 square miles)

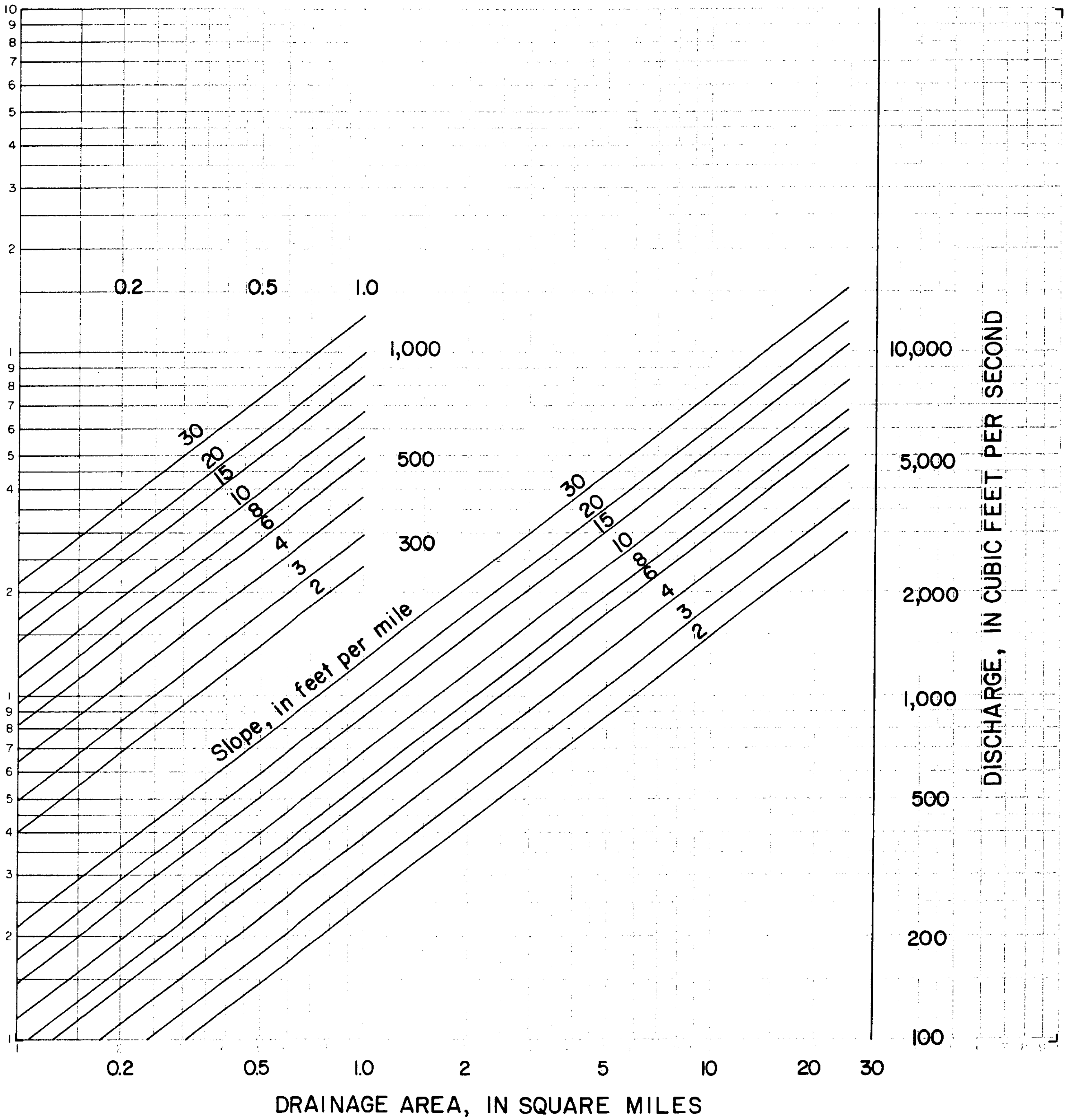
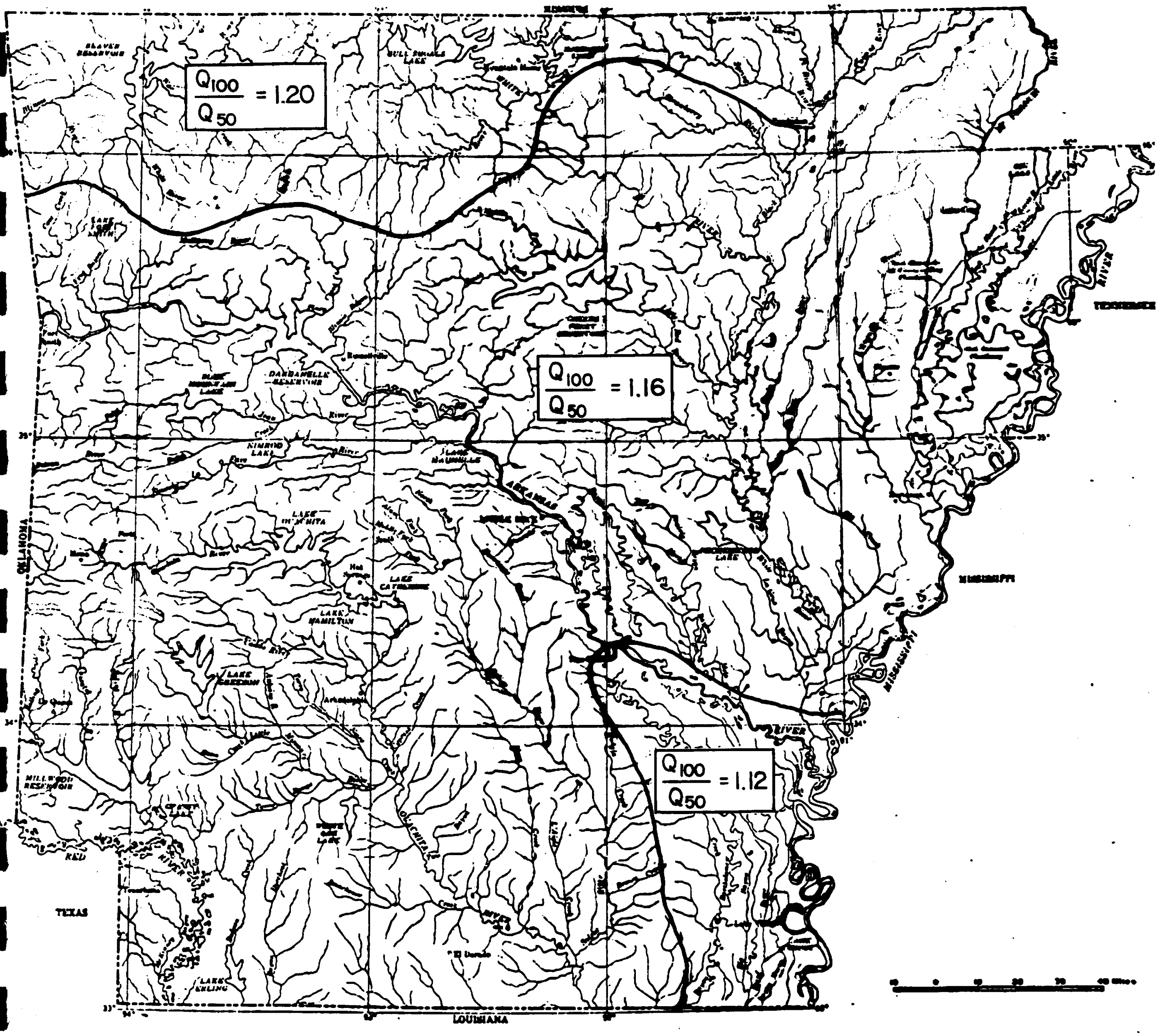


Figure 3-40--Graphical solution of a 100-year-flood equation for region B (drainage areas 0.1-25 square miles)

Fig. 3-41

Q_{100} DISCHARGE RATIOS



HYDROLOGY WORK SHEET

FOR USGS REGRESSION EQUATIONS

LOCATION:

Refer to "Floods in Arkansas, Magnitude and Frequency:, 1971, James L. Patterson, U.S.G.S.

REGION ______________________

LATITUDE ____________________

LONGITUDE ___________________

DRAINAGE AREA ____________ SQ. MI. or __________ ACRES

MEAN BASIN ELEVATION ________________

MEAN ANNUAL PRECIPITATION ______________

MAIN CHANNEL LENGTH __________ MI. ______________ FT.

ELEVATION AT BASIN DIVIDE ____________________

ELEVATION AT 85% ______________________________ FT.

ELEVATION AT 10% ______________________________ FT.

ELEVATION AT BRIDGE ___________________________ FT.

H ELEVATION AT 85% - ELEVATION AT 10% __________ FT.

SLOPE = $\dfrac{\rule{4cm}{0.4pt}}{(0.75)\ (\text{Main Ch. Length (mi.)})}$ = $\dfrac{\rule{4cm}{0.4pt}}{(0.75)\ (\quad)}$ = ________ FT./MI.

$Q = aA^{b}S^{b} = p^{b3}$

Q_{25} =

Q_{50} =

Q_{100} =

SLOPE AT BRIDGE

ELEVATION AT ____________ FT. UPSTREAM ____________ FT.

ELEVATION AT ____________ FT. DOWNSTREAM ____________ FT.

= LENGTH = H

SLOPE = $\dfrac{H}{\text{Length}}$ = ________________ ft./ft.

3-500 SOURCES OF HYDROLOGIC DATA

Our confidence in hydrology as a scientific discipline is based upon the availability of sufficient field data to verify theories concerning the natural phenomena. The variation of precipitation, streamflow, temperature and other variables is too great for theories alone to be adequate. The very foundation of programs of flood control or water managment is based on the adequacy of supporting data.

The practicing hydrological engineer is beset with two opposing problems with respect to data: first, there is never enough data available in the right location and form to fulfil this need; second, there is so much other data in existence that it is difficult to manage storage and retrieval. Besides the raw primary data, such as, stream gage records and rainfall records, there is much secondary calculated data which has previously been developed and which, if properly stored, could be a valuable tool in the future.

It is of interest to look at the types of data available and to examine the characteristics and reliability of information of interest to the hydrologist. For the purposes of highway hydrology the three primiary data types of interest are runoff, rainfall and drainage basin characteristics. Runoff data is available from different sources, however, the U.S. Geological Survey has about 70 percent of all the surface water records available. These include some 30-40 thousand different sites with over 400,000 station-years of data. The Corps of Engineers and Bureau of Reclamation also have a substantial amount of surface water data. The USGS, the Corps, and the Bureau together account for perhaps 90 percent of all surface water data available in the United States.

For large streams, federal water agencies should always be contacted including the Corps of Engineers, Bureau of Reclamation, Soil Conservation Service, Forest Service, Geological Survey, Federal Insurance Administration and Bureau of Land Management.

The amount of data available is startling, so much so that the federal government has recently formed a National Water Data Exchange (NAWDEX) to serve as a Central Clearing House and Management Center for the many different sources of data that are available. Their telephone number and address are given below:

National Water Data Exchange
U. S. Geological Survey
421 National Center
Reston, Virginia 22092
Telephone: (703) 860-6031 or FTS 928-6031

3-600 DOCUMENTATION

It is very important that all design computations be done in an orderly and clear procedure and all pertinent data and design computations be saved. The data assembled during design are often needed after the actual design in finished, also, after construction is complete-sometimes several years later. Thorough documentation of this data is invaluable for use in adjacent projects, utility problems, claims, allegations and litigations. A design checklist that can be used for documentation is given in Section 3-601.

3-601 DOCUMENTATION CHECK LIST

I. Hydrology

   A. References (State name and/or description of study or the absence of a study).

      1. Other studies by AHTD
      2. Studies by other agencies
      3. Gaging data available
      4. Other

   B. Fluvial Geomorphology

      1. Drainage area
      2. Basin length
      3. Basin slope

   C. Discharge calculation

      Use as many methods as are available-show computations in detail. State reasons for final selection of discharge used.

II. Influence and Control of Site

   A. Other streams, reservoirs, etc. that exert influence.
   B. Other structures that exert influence.
   C. Description of channel and floodplain

      1. Development of applicable design criteria.
      2. Level of flooding, degree of property damage and/or pertinent control elevations.

   D. Risk Evaluation: Design risk factor and applicable criteria including flood prone hazard zoning (if applicable).

III. Historical Data

A. High water marks

1. Determine discharge and frequency for as many high water marks as possible. Describe any unusual circumstances that may be contributed to causing a particular high water mark (i.e., a large release from a dam or backwater from a river downstream).

3-700 HEC-1 COMPUTER PROGRAM

The HEC-1 Flood Hydrograph Package computer program was orginally developed in 1967 by Leo R. Beard and other members of the Hydrologic Engineering Center staff. The first version of the HEC-1 package program was published in October 1968. It was expanded and revised and published again in 1969 and 1970. The first package version represented a combination of several smaller programs which had previously been operated independently.

3-701 PROGRAM CAPABILITIES

Most ordinary flood hydrograph computations associated with precipitation and runoff on a complex, multisubbasin, multichannel river basin can be accomplised with this package. Precipitation patterns must be related to a single hypothetical or recorded storm as there is no provision for precipitation loss rate recovery during periods of no precipitation.

HEC-1 is capable of performing five major types of flood hydrograph analyses. These five types of jobs are outlined on the next page and discussed in the paragraphs immediately following.

Optimization of routing parameters;

Optimization of unit hydrograph and loss rate parameters;

Generalized precipitation, runoff, routing and combining operations to simulate the hydrologic response of the watershed and its stream network;

Stream system computations for specified precipitation depth-area storm relationships for the entire watershed or region;

Specialized precipitation streamflow network simulation relative to multiple floods for multiple plans of basin development and the economics analysis of flood damages.

The program may be used to optimize specified parameters of precipitation-runoff or routing processes to achieve a best-fit with respect to an observed hydrograph and known precipitation or a known inflow hydrograph. Each derivation is a single job, the results of which should be examined before further use.

In the process of modeling a basin, the program provides several techniques with which to input and distribute precipitation, treat the precipitation as rainfall or snowfall, compute rainfall and snowmelt losses and excess, determine subbasin outflow hydrographs from unit graph techniques and route hydrographs by hydrologic methods. Many different techniques for each process may be combined in the same job if that is appropriate for the basin being modeled. Graphical display or intermediate or summary hydrographs and precipitation can be called where desired.

The precipitation depth-area stream system computation procedure includes a maximum of five base floods which are computed simultaneously, each representing average rainfall intensity corresponding to a specified area size. An interpolated hydrograph is automatically established for each concentration point based on the size of the area tributary to the point. This routine is useful in urban storm drainage computations as well as for river basin computations.

The routine for evaluating reservoir and channel development plans for one or more locations includes the computation of average annual damages at each damage center for each plan of development as well as for existing conditions. This involves simultaneously computing a number of system floods for each plan, covering the entire range of floods that significantly contribute to damages. The floods may be either multiples of runoff from a single representative storm or runoff from multiples of a typical storm rainfall pattern. Flow-damage relations for each type of damage and flood-peak frequency relations for existing conditions must be specified for each damage center. Unit hydrograph coefficients, loss coefficients, degree of imperviousness and routing coefficients for each plan must also be specified.

TABLE 3-12

LIST OF SYMBOLS

A	Drainage area in acres or square miles
a	U.S.G.S. regional equation coefficient
$b_{1,2,3}$	U.S.G.S. regional equation coefficient
C	A coefficient representing the ratio of runoff to rainfall in the rational method
CN	Soil Conservation Service curve number
E	Mean basin elevation
i	Rainfall rate in in./hr.
I_a	Initial abstractions prior to runoff
K	56 for English units
L	Main channel length, miles
L_o	Overland flow length in feet
n	Manning roughness coefficient
p	Accumulated rainfall
P	Mean annual precipitation
Q	Peak discharge, cfs
Q	Accumulated direct runoffs, cfs
Q_b	Basic discharge, cfs
Q_{adj}	Products of basic discharge and peak factors
S	Main channel slope, ft./mile
S	Potential maximum retention
S_o	Overland flow slope in feet per foot
T_c	Time of concentration in seconds

REFERENCES

1. FLOODS IN ARKANSAS, MAGNITURE AND FREQUENCY CHARACTERISTICS THOUGH 1968, James L. Patterson, U. S. Geological Surveys, Little Rock, Arkansas, 1971.

2. HYDROLOGY, National Engineering Handbook, Section 4, Hydrology, U. S. Department of Agriculture, Soil Conservation Servive, Washington, D.C., August 1972.

3. URBAN HYDROLOGY FOR SMALL WATERSHEDS, Technical Release No. 55, U. S. Department of Agriculture, Soil Conservation Service, Washington, D.C., January, 1975.

4. A METHOD FOR ESTIMATING VOLUME AND RATE OF RUNOFF IN SMALL WATERSHEDS, K. M. Kent, Hydrology Branch, Soil Conservation Service, Washington, D. C., April, 1973.

5. HYDROLOGY FOR TRANSPORTATION ENGINEERS, Edited by Thomas G. Sanders, prepared by Colorado State University for the FHWA (FHWA-IP-80-1), Washington, D.C., January 1980.

6. DESIGN OF URBAN HIGHWAY DRAINAGE, U.S. Department of Transportation, Federal Highway Administration, (FHWA-TS-79-225), Washington, D.C., August 1979.

7. TECHNICAL PAPER NO. 40, Prepared by David M. Hershfield for Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C., May 1961.

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CULVERTS

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CULVERTS

A	Tributary area in acres
AHW	Allowable headwater at culvert entrance in feet
B	Width of culvert barrel or diameter of pipe culvert in feet
B_f	Width of face section of improved inlets in feet
C	Coefficient in the Rational Formula
cfs	Cubic feet per second
CMP	Corrugated metal pipe
D	Height of box culvert or diameter of pipe culvert in feet
d	Dimension of top bevel in inches
d_n	Normal depth of flow in feet
d_c	Critical depth of flow in feet
fps	Feet per second
F	Froude number
FALL	Approximate depression of control section below the stream bed in feet
g	Acceleration of gravity; 32.2 feet per second per second

LIST OF SYMBOLS

Chapter 4

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H	Head of energy required to pass a given quantity of water through a culvert (outlet control) in feet
h_e	Entrance head loss in feet
H_f	Depth of pool, or head, above the face section invert in feet
h_f	Friction head loss in feet
h_o	Empirical approximation of equivalent hydraulic grade line in feet
H_t	Depth of pool, or head, above the throat section invert in feet
HGL	Hydraulic grade line in feet
HW	Headwater elevation; HW is equivalent to H_f
HW_f	Headwater elevation required for flow to pass face section in face control in feet
HW_o	Headwater elevation required for culvert to pass flow in outlet control in feet
HW_t	Headwater elevation required for flow to pass throat section in throat control in feet
h_o	Elevation of equivalent hydraulic grade line referenced to the outlet invert in feet
h_v	Velocity head $V^2/2g$ in feet (V is based on full flow in culvert)
I	Average rainfall rate in inches

LIST OF SYMBOLS

Chapter 4

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K_e	Entrance energy loss coefficient
L	Approximate total length of culvert, including inlet in feet
L_1, L_2, L_3	Dimensions relating to the improved inlet as shown in computation charts of the different types of inlets in feet
N	Number of barrels
n	Manning's roughness coefficient
P	Perimeter in feet
Q	Volume rate of flow in cubic feet per second
q	Discharge per foot of width for rectangular channels in cubic feet per second
Q_d	Design Volume rate of flow in cubic feet per second
R	Hydraulic radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$ in feet
S	Slope of culvert barrel in feet/foot
S_c	Critical slope in feet/foot
S_f	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)) in feet per foot
S_o	Slope of natural channel in feet per foot
T_c	Time of concentration in minutes

LIST OF SYMBOLS

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Taper	Sidewall flare angle (also expressed as the cotangent of the flare angle) in feet per foot
TW	Tailwater depth at outlet of culvert referenced to outlet invert elevation in feet
V	Mean velocity of flow in feet per second
V_c	Critical velocity in feet per second
V_o	Velocity of outlet flow in feet per second
W	Width of channel in feet

4-100 GENERAL

The selection of any structure should be based on hydraulic principles, on the most economical size and shape, and with a resulting headwater depth which will not cause damage to adjacent property. The resultant outlet velocity should be taken into consideration for any possible damaging effects. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the highway. It is this allowable headwater depth that is primarily the basis for sizing a culvert.

The cost of maintaining highways in good condition is directly related to the adequacy of the means provided for drainage. When adequate provisions are not made, storm water may cause severe erosion of embankment slopes and may undermine culvert outlets. Good drainage design depends on determining the proper frequency and amount of run-off and providing adequate facilities to remove the runoff at such a rate as to avoid undue interference with traffic and also keep maintenance costs at a minimum. In addition, these facilities should be provided at a minimum cost of intitial investment.

4-200 DEFINITIONS

(a) CONJUGATE DEPTH is that depth at which the same discharge would be flowing in a subcritical state after the occurrence of the hydraulic jump.

(b) CRITICAL DEPTH can best be illustrated as the depth at which water flows over a weir, this depth being attained automatically because it is the depth at which the energy content of flow is a minimum. For a given discharge and channel shape there is only one critical depth. The formula for calculating critical depth in rectangular channels:

$$d_c = \left[\frac{q^2}{32.2}\right]^{1/3}$$

where

q = Discharge per foot of width for rectangular channels in cubic feet per second (Q/W).

For rectangular culverts the critical depth may either be computed from the above formula or read from Chart 4-5, page 4-62.

For circular sections the critical depth may be obtained from Chart 4-14, page 4-71; Pipe-Arch, Chart 4-21, page 4-79.

(c) CRITICAL SLOPE is that slope at which a given discharge will pass through a structure at critical depth and critical velocity. Increasing the slope above the critical slope does not increase the discharge because culvert capacity is determined by the inlet geometry. It merely makes the water flow at a depth less than critical depth and at a greater velocity. Decreasing the slope to less than critical slope will have a retarding effect upon the discharge, causing the depth of flow to be higher than critical depth and the velocity to be less than critical.

(d) FREE OUTLETS are those outlets whose TW is equal or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge. (See Type I operation, Figure 4-1, page 4-15).

(e) FRICTION SLOPE is the only one profile slope that will produce uniform flow for a particular channel cross-section and depth of flow. The equation for friction slope is the modified Manning Formula:

$$\text{Friction Slope} = (Q_n/1.486AR^{2/3})^2$$

(f) INVERT refers to the flowline of pipe or box (inside bottom).

(g) PARTIALLY SUBMERGED OUTLETS are those outlets whose TW is higher than critical depth and lower than "D", the height of culvert. (See Type II operation, Figure 4-2, page 4-16).

(h) SIDE OR DROP TAPERED INLET indicates a special entrance condition as illustrated in Sections 4-803.1 and 4-803.2, pages 4-30 and 4-33, respectively.

(i) SOFFIT refers to the inside top of pipe of box.

(j) SUBMERGED OUTLETS are those outlets having a TW elevation higher than the soffit of the culvert. (See Type IV A operation, Figure 4-4, page 4-19).

(k) TANDEM CULVERTS refers to culverts aligned across a roadway in such a manner that it may be possible for the headwater of the downstream culvert to influence the tailwater of the culvert immediately upstream.

(l) UNIFORM FLOW is possible only in a channel of constant cross section having the same discharge, velocity and depth of flow throughout the reach. This type of flow will exist in a Type III culvert operation, (Figure 4-3, page 4-17), provided the culvert is sufficiently long to reach a uniform depth of flow.

4-300 DETERMINATION OF RUNOFF

The first step to be considered in the hydraulic design of a culvert is the determination of runoff. There is no single method for determining peak discharge that is applicable to all watersheds. The method chosen should be a function of drainage area size, availability of data, and the degree of accuracy desired.

It is recommended that the following methods as described in Chapter 3 (Hydrology), normally be used as comparison against the others within their respective parameters for increased "power" in hydrology decisions.

a. RATIONAL METHOD

This method may be used for drainage areas less than 200 acres (Reference 4-9).

b. SOIL CONSERVATION METHOD

This method may be used for drainage areas less than 2000 acres.

c. MULTIPLE REGRESSION ANALYSIS

This method should be used in rural areas for drainage areas greater than 600 acres, including large bridge-type watersheds.

d. SYNTHETIC HYDROGRAPH ANALYSIS

This method may be used on areas greater than one square mile.

River routing and reservoir routing may be used to adjust storm hydrographs for the effects of storage, channels, long pipes, and the operation of other control structures.

The HEC-1, Flood Hydrograph Computer Program can be used to compute hydrographs by Snyder's and Clark's methods, from historical data, and by the SCS Instantaneous Unit Hydrograph Method. It's capable of performing reservoir and river routing by five different methods, and computing channel diversions where split flow occurs.

The SCS Technical Release No. 20 is a computer adaptation of NEH-4 which can compute surface runoff resulting from any synthetic or natural rainstorm and route the flow through stream channels and reservoirs.

4-400 DESIGN FREQUENCY POLICIES

The following flood frequencies are recommended for design:

a. INTERSTATE PROJECTS: 50 year

b. PRIMARY PROJECTS: 50 year

c. SECONDARY PROJECTS: 25 year

d. NON-FEDERAL AID PROJECTS: 10 year*

*Drainage area less than two square miles; ADT less than 750. If either is exceeded, use 25-year flood frequency.

4-500 CULVERT OPERATION

The hydraulic and physical operation of any culvert may be broken down into several basic types. Even though there are numerous combinations of culvert operations, the most common types of culvert operations are shown in Figures 4-1 to 4-5, pages 4-15 to 4-20.

TYPE I

OUTLET CONTROL

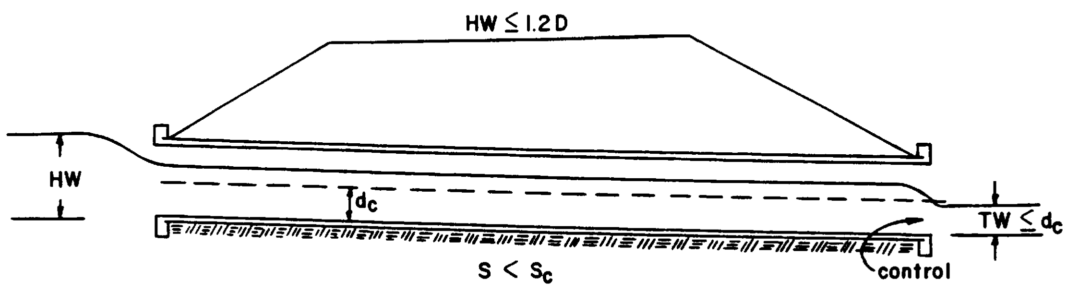


FIGURE 4-1
CONDITIONS

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

NOTE: 1.2D is an empirical value determined by research (Reference 4-3 and 4-4).

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

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - SL$$

where

d_c = critical depth

V_c = critical velocity (based on d_c)

g = 32.2 feet per second per second

h_e = entrance head = $K_e \dfrac{V_c^2}{2g}$

K_e = entrance coefficient found in Table 4-10, page 4-60

h_f = friction head = friction slope times "L"

L = length of culvert

friction slope= that slope at which 1.1 d_c would produce uniform flow

SL = veritical drop in culvert from upstream flow-line to downstream flowline

The outlet velocity is the discharge divided by the area of flow at critical depth which is critical velocity (V_c).

TYPE II

OUTLET CONTROL

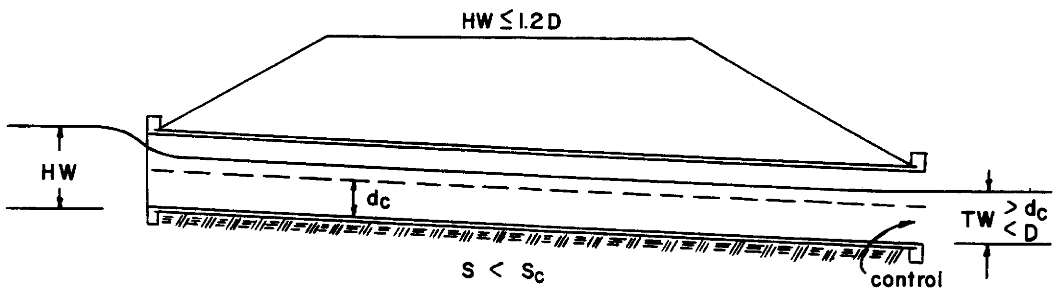


FIGURE 4-2

CONDITIONS

The entrance is unsubmerged (HW ≤1.2D), the slope at design discharge is subcritical ($S < S_c$), the tailwater is above critical depth ($TW > d_c$) and tailwater is less than D ($TW < D$).

The above condition is a common occurrence where the channel is deep, narrow and well defined. The control is tailwater at the culvert outlet.

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

where:

TW = tailwater depth at outlet

V_{TW} = velocity based on TW depth

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

h_f = friction head = friction slope times "L"

friction slope = slope at which TW depth would be uniform depth. All other terms are as previously defined.

Outlet velocity (V_{TW}) is the discharge divided by the area of flow in the culvert at tailwater depth.

TYPE III

INLET CONTROL (a)
INLET OR OUTLET CONTROL (b)

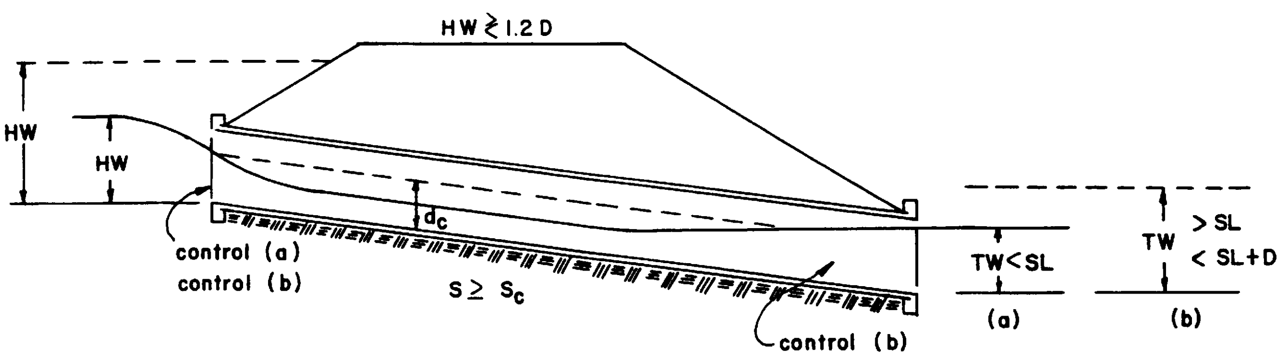


FIGURE 4-3

CONDITIONS

The entrance may be submerged or unsubmerged ($HW \lesseqgtr 1.2D$) and the slope at design discharge is equal to or greater than critical slope ($S \geq S_c$). Tailwater should be less than "SL" when "TW" elevation is lower than upstream flowline. Tailwater will be less than "SL" when "TW" elevation is above upstream flowline and less than "SL+D" (TW elevation is below upstream soffit).

This condition is a common occurrence for culverts in rolling or mountainous country. The control is critical depth at the entrance for HW values up to about 1.2D. The control is the entrance geometry for HW values over about 1.2D.

In some cases, control for this type of operation may be at the entrance or the outlet or control may transfer itself back and forth between the two. For this reason, it is recommended that HW be determined for both entrance control and outlet control and the higher of the two determinations be used.

Entrance control HW is determined from empirical curves in the form of nomographs (See Charts 4-7, 4-15, 4-16 and 4-23 , pages 4-64, 4-72, 4-73 and 4-81, respectively).

OUTLET VELOCITY

(a) If TW is greater than D, outlet velocity is based on full flow at the outlet. If TW is less than D, outlet velocity is based on uniform depth for the culvert. Uniform depth is simply that depth of water for a given discharge, culvert slope, and geometry at steady flow.

(b) If TW depth is less than D, outlet velocity should be based on TW depth. If TW depth is greater than D, outlet velocity should be based on full flow at the outlet.

TYPE IV

OUTLET CONTROL

(a)

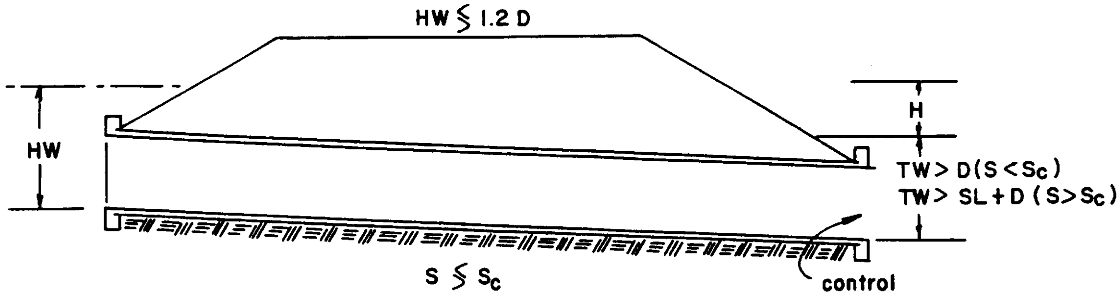


FIGURE 4-4

CONDITIONS
(Submerged Outlet)

The entrance is submerged (HW $\gtrless$1.2D). The tailwater completely submerges the outlet.

$$HW = H + TW - SL$$

where:

H = total head loss of discharge through culvert and

$H = h_v + h_e + h_f$

where:

h_v = velocity head $\frac{v^2}{2g}$ (where v is based on full flow in culvert).

h_e = entrance head $K_e\ h_v$

h_f = friction head = S_f L (where S_f is based on full flow in culvert). All other terms have been previously defined.

H may be determined directly from nomographs on Charts 4-4, 4-12 and 4-13, pages 4-61, 4-69 and 4-70, respectively.

Outlet velocity is based on full flow at the outlet.

TYPE IV
OUTLET CONTROL

(b)

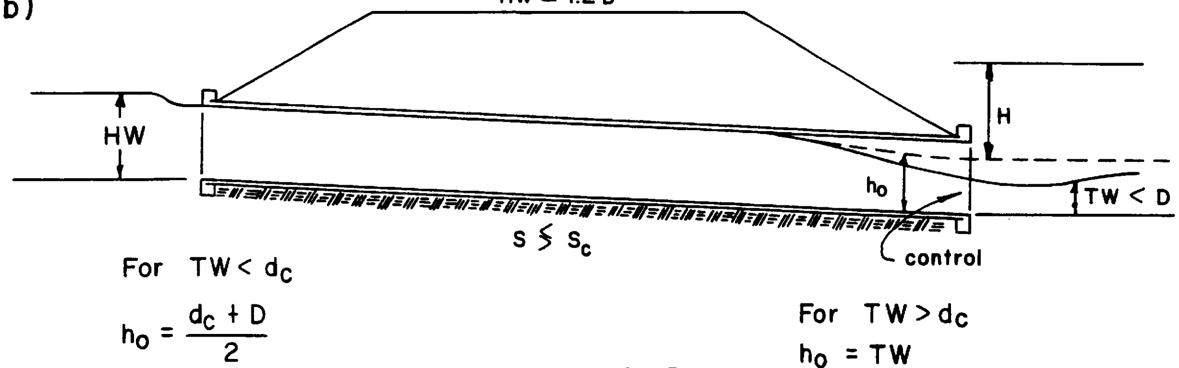


FIGURE 4-5

CONDITIONS
(Partially Submerged Outlet)

The entrance is partially submerged (HW $\leq$1.2D). The tailwater depth is less than D (TW < D).

$$HW = H + h_o - SL$$

where:

h_o = empirical approximation of equivalent hydraulic grade line.

h_o = $(d_c+D)/2$ if TW depth is less than critical depth at design discharge. If TW is greater than critical depth, then h_o = TW.

Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

4-600 CULVERT FLOW CONTROLS

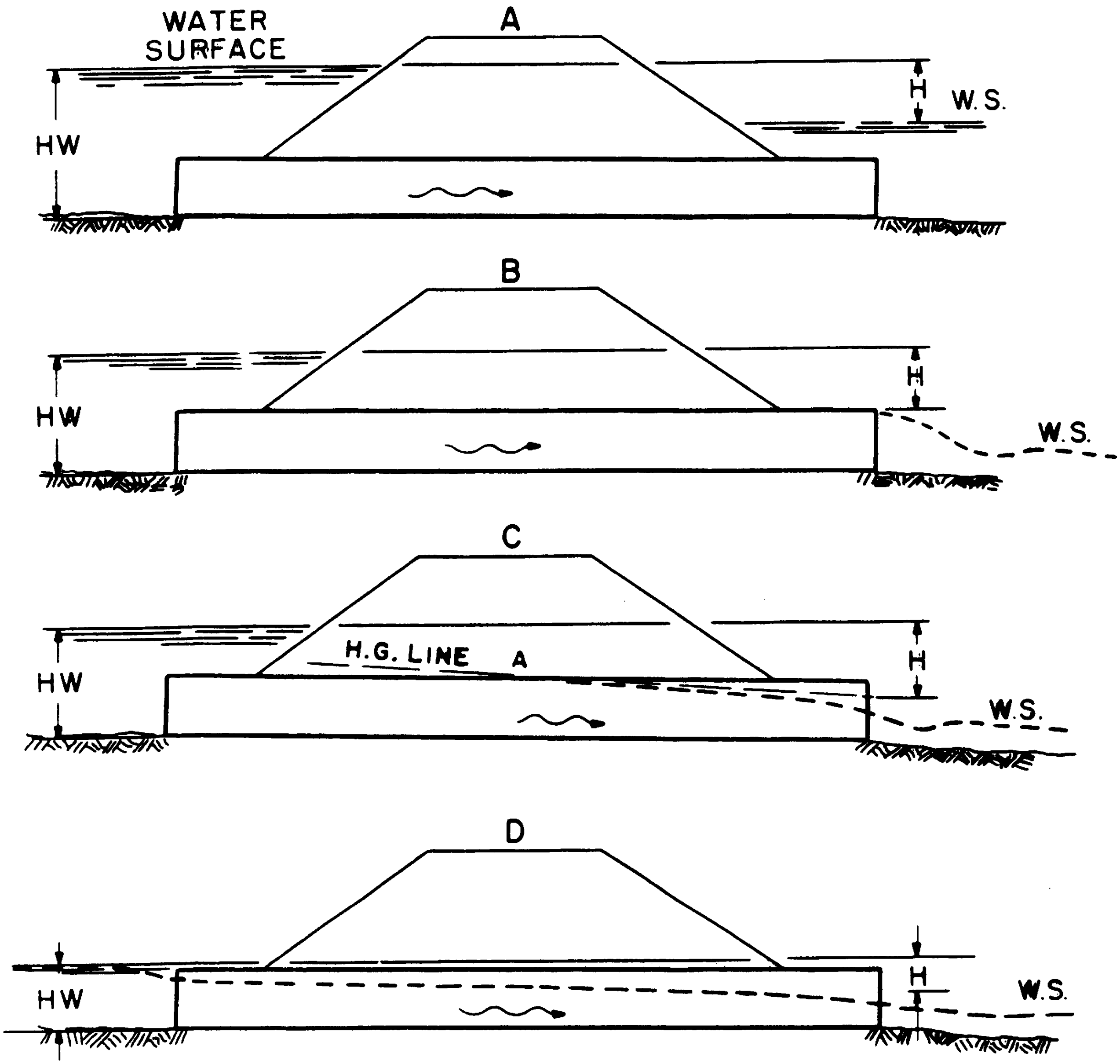
Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert slope is less than the critical slope. Inlet control usually governs if the culvert slope is greater than the critical slope. Other factors that determine how the culvert performs can be listed as physical makeup of the structure and the leading and trailing water surface profiles.

4-601 OUTLET CONTROL

For outlet control, factors such as type of opening, cross section area, barrel slope, barrel length, barrel roughness, head losses due to tailwater are predominate in controlling the headwater of the culvert as shown in Figure 4-6, page 4-22. These separately or conjointly, create physical resistances that retard the flow of water. As the resistance accumulates, the flow begins to slow and increase in depth. As some point, when the resistances mount, the water may cease to flow freely and back up in the structure and flood the upstream drainage basin.

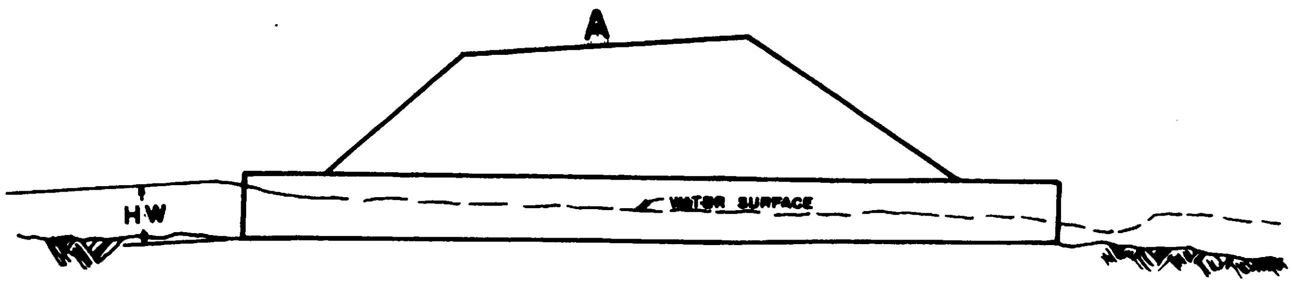
4-602 INLET CONTROL

For inlet control, the entrance characteristics of the culvert are such that the entrance head losses are predominate in determining the headwater of the culvert as shown in Figure 4-7, page 4-23. The barrel will carry water through the culvert faster than the water can enter the culvert.

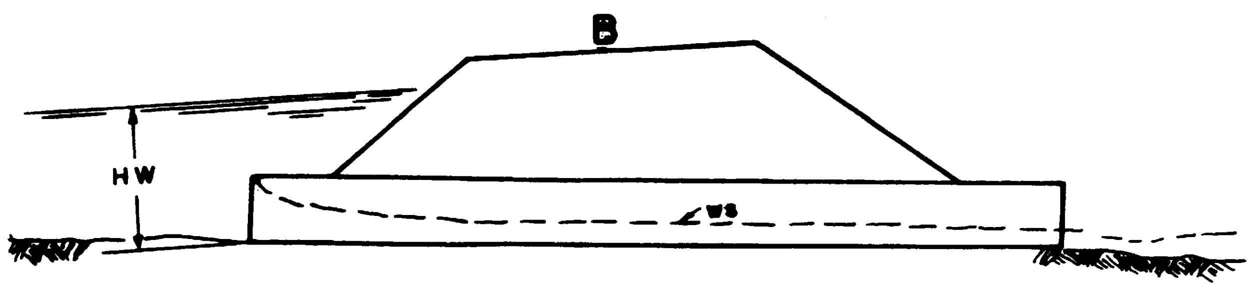


OUTLET CONTROL

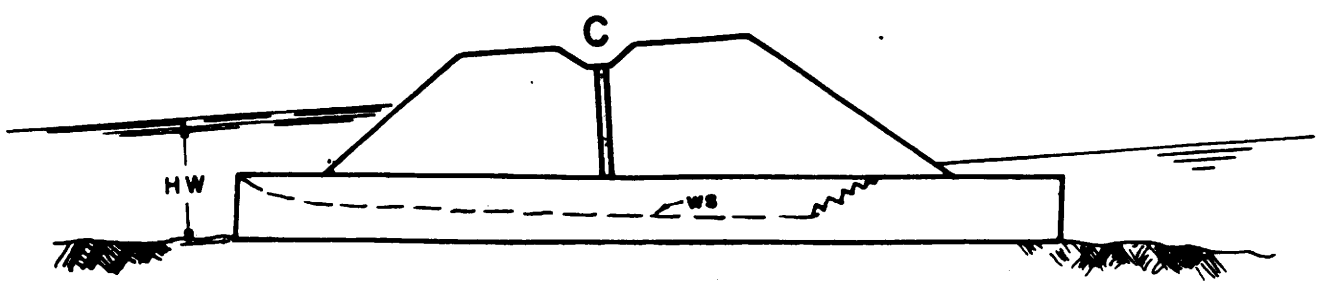
FIGURE 4-6



INLET UNSUBMERGED



INLET SUBMERGED



OUTLET SUBMERGED

INLET CONTROL

FIGURE 4-7

If the slope of the culvert is greater than critical slope and the tailwater depth is less than SL or tailwater elevation is lower than the upstream flowline of the culvert, headwater is based on inlet control for all ranges of discharges.

4-700 OUTLET VELOCITY

The design of any culvert should take into consideration the soil type, the outfall velocity and the depth of flow.

Outlet velocity in any properly designed culvert is normally greater than the velocity in the natural channel. The engineer must bear in mind, while attempting to control flow at a structure outlet, that the main objective is to return the flow to the normal flow in the natural stream, in an economical and efficient manner. A recommended threshold of erosive velocity is 8fps. Ranges of velocities, relative to soil types, lined and unlined can be found in Chapter 6 of this manual.

It should be noted at this point that if the culvert has been properly sized according to allowable headwater elevation, it is almost always more economical to protect against excessive outlet velocity with riprap and/or velocity control devices than to try to adjust the culvert size to reduce the excessive outlet velocity.

4-800 CULVERT DESIGN PROCEDURE

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of the varibles discussed in the following paragraphs.

4-801 CULVERT DESIGN DATA

The proper design of a culvert requires the definite knowledge of some items and the assumption of other items. The items which should be know either by observation or calculation include:

1. design discharge - Q_d
2. design tailwater - TW
3. culvert slope - S
4. allowable headwater - AHW

The items which must be assumed or estimated include:

5. allowable outlet velocity - V_o
6. culvert length - L
7. entrance conditions
8. culvert material and shape (box, pipe, metal, concrete, etc.)
9. maximum allowable depth of barrel should usually not be greater than AHW

Only after all the basic culvert design data is assimilated should the culvert size be attempted.

4-802 EXAMPLE PROBLEM NO. 1

The following is a step by step culvert design procedure. It should be noted that the procedure incorporates the items as shown on tabulation sheets, Tables 4-7 and 4-8, pages 4-47 and 4-48, respectively.

GIVEN:	Design Discharge (Q_{50})	= 1,000 cfs
	Slope of stream bed (S_o)	= 0.071 ft./ft.
	Allowable headwater elevation	= 200.0
	Elevation outlet invert	= 182.9
	Culvert length (L)	= 100 ft.

Downstream channel approximates an 8 ft. wide trapezoidal channel with 2:1 side slopes, Manning "n" of 0.03 and a slope (S_o) of 0.071 feet per foot.

REQUIRED: Design a culvert that will adequately convey the design discharge. The culvert should have the smallest possible barrel to pass design Q without exceeding AHW elevation. Try a reinforced concrete box culvert with "n" = 0.012.

SOLUTION:

4-802.1 STEP 1 - SELECTING CULVERT SIZE

The computations involved in selecting the smallest feasible barrel which can be used without exceeding the design headwater elevation is summarized in the tabulation sheet, titled "Culvert Computations", Table 4-1, page 4-38.

INITIAL DATA:

Enter initial data and complete required information for first approximation. The square feet of opening for the initial trial size may be estimated by the ratio of design discharge divided by 10.

TAILWATER:

The tailwater depth is influenced by conditions downstream of the culvert outlet. If the culvert outlet is located near the inlet of a downstream culvert, then the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert. If the culvert outlet is operating in a free outfall condition then the tailwater is taken as 0.0.

If the culvert discharges into an open channel, then the tailwater is equal to the normal depth of flow in that channel. Chart 4-3, page 4-59 provides a graphical solution for normal depth or may be calculated by Manning's Formula:

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

In any case, the tailwater depth is defined as the depth of water measured from the flow line of the culvert (invert) at the outlet, to the water surface elevation at the outlet.

Enter tailwater depth in Column 8 and applicable stream data in upper left hand portion of Culvert Computation Form.

4-802.2 STEP 2 - PERFORM OUTLET CONTROL CALCULATIONS

These calculations are performed before inlet control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control exceeding the allowable headwater elevation.

Column 1: Enter the span times height dimensions (or diameter of pipe) of culvert.

Column 2: Enter the type of structure and design of entrance.

Column 3: Enter the design discharge or quotient of design discharge divided by the applicable denominator.

Column 4: Enter the Entrance Loss Coefficient from Table 4-10, page 4-60.

Column 5: Enter the head from the applicable outlet control nomograph, in the example problem use Chart 4-4, page 4-61.

Column 6: Enter the critical depth from appropriate nomograph, in the example problem use Chart 4-5, page 4-62. Critical depth cannot exceed height of culvert opening.

Column 7: For tailwater elevations less than the top of the culvert at the outlet, headwater is found by solving for h_o using the following equation:

$$h_o = \frac{d_c+D}{2}$$

where: h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line in feet.

d_c = critical depth in feet

D = height of culvert opening in feet

Column 8: Enter the tailwater elevation from initial data shown at top of form. Refer to tailwater comments under STEP 1 for additional guidelines.

Column 9: Enter the product of culvert length times the slope.

Column 10: Headwater elevation required for culvert to pass flow in outlet control (HW_o) is computed by the following equation:

$$HW_o = H + h_o - LS$$

NOTE: Use TW elevation in lieu of h_o where $TW > h_o$.

Additional trials may be required. Space for additional trials is provided on Culvert Computations Form.

4-802.3 STEP 3 - PERFORM INLET CONTROL CALCULATIONS FOR CONVENTIONAL AND BEVELED EDGE CULVERT INLETS

After minimum barrel size has been determined under STEP 2, the next procedure is similar to that used in FHWA's Hydraulic Engineering Circular Number 5, "Hydraulic Charts for the Selection of Highway Culverts".

The computations involved in computing inlet headwater elevation is summarized in the tabulation sheet used in STEP 2, titled "Culvert Computations", Table 4-1, page 4-38.

Column 11: Enter ratio of headwater to height of structure from Chart 4-7, page 4-64.

Column 12: HW is derived by multiplying Column 11 by the height (or diameter) of culvert.

Column 13: Enter greater of two headwaters (Column 10 or 12).

Column 14: If inlet control governs, outlet velocity equals Q/A, where A is defined by the cross-sectional area of normal depth of flow in the culvert barrel at "S". Charts 4-6, 4-17, 4-24, and 4-26 to 4-39, pages 4-63, 4-74, 4-82 and 4-86 to 4-99 respectively; and Charts 5-14 to 5-26, Chapter 5, provides a graphical solution for estimating normal depth of flow and velocity. Manning's Formula may also be used:

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

If outlet control governs, outlet velocity equals Q/A, where A is the cross-sectional area of flow in the culvert barrel at the outlet.

Column 15 & 16: Figures shown in this column are believed to be self-explanatory.

4-803 IMPROVED INLETS

(a) If headwater (inlet control) is lower than headwater required for outlet control, the barrel operates in outlet control at design "Q", ADDITIONAL ANALYSIS for improved inlets NOT REQUIRED.

(b) If computed headwater is within the designer's judgement of acceptable limits and headwater does not exceed allowable headwater (AHW) elevation, ADDITIONAL ANALYSIS for improved inlets NOT REQUIRED.

(c) If the computed headwater is excessive in the designer's judgement from the standpoint of aesthetics, economy, risk and other engineering reasons, a need for inlet geometry refinement is indicated. If square edges were used in STEP 3 above, repeat with beveled edges. If beveled edges were used, CALCULATIONS FOR SIDE-TAPERED and SLOPE-TAPERED INLETS SHOULD BE PERFORMED.

4-803.1 STEP 4 - PERFORM SIDE-TAPERED INLET DESIGN CALCULATIONS

The same concept is involved here as with conventional and beveled edge inlet culvert design.

The computations involved in computing side-tapered inlets (Figure 4-8, page 4-32) are summarized in the tabulation sheet titled "Improved Inlet for R.C. Box Culverts", Table 4-4, page 4-43.

INITIAL DATA:

Enter initial data and information obtained from STEPS 2 and 3.

Column 1: Enter ratio of headwater above the face section invert (H_f) to the height of box culvert (D) using the formula:

$$H_f = \frac{HW_f - \text{Elevation Throat Invert} - 1}{D}$$

where:

H_f = headwater above the face section invert in feet

D = height of box culvert in feet

HW_f = allowable headwater elvation required for flow to pass face section in face control in feet.

Column 2: The ratio of $Q/B_f\ D^{3/2}$ is derived by the use of Chart 4-9, page 4-66.

Column 3: Enter the height of box culvert (D) to the 3/2 power.

Column 4: The minimum width of face section (B) is the quotient of design discharge (Q) divided by the product of Column 2 times Column 3. The derived width should be rounded up to the nearest foot.

Column 5: Enter the length of taper (L_1) by using the formula:

$$L_1 = \frac{B_f - NB}{2} \text{ Taper}$$

where:

L_1 = length of taper, in feet.

B_f = width of face section, in feet.

NB = number of barrels times width of culvert barrel

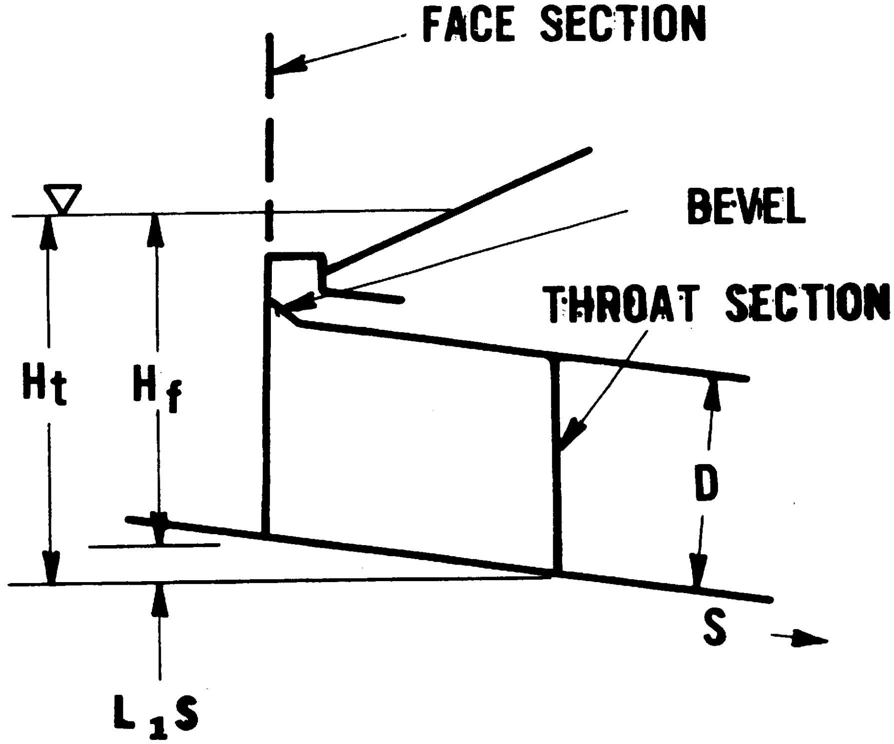
Taper = design taper slope. Side taper ratios may range from 6:1 to 4:1. The 4:1 taper is recommended as it results in a shorter length. Tapers greater than 6:1 may be used, but performance will be underestimated. For 4:1 tapers, multiply the quotient by 4; for 6:1 tapers, multiply the quotient by 6.

Column 6: Enter the design slope of culvert in feet per foot. The product of (S) times L_1 should be <1 foot. If >1 foot, redesign or proceed to STEP 5.

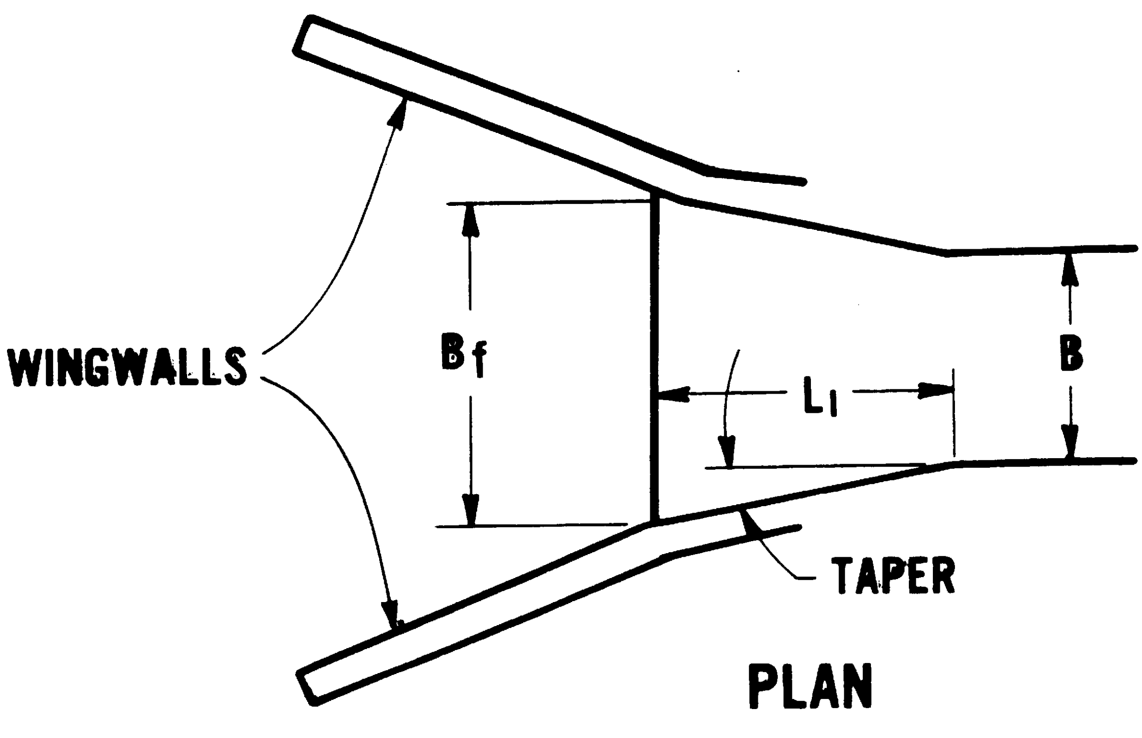
4-803.2 <u>STEP 5 - PERFORM SLOPE-TAPERED INLET DESIGN CALCULATIONS</u>

A slope-tapered inlet design (Figure 4-9, page 4-37) may be used if a FALL is required on the throat by use of a FALL in the inlet of the slope-tapered inlet.

SIDE - TAPERED INLET



ELEVATION



PLAN

FIGURE 4-8

The minimum face design is one whose performance does not exceed the allowable headwater elevation at design "Q". However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various falls.

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat.

The steps followed in the slope-tapered inlet design are:

a. Compute depression (FALL) below the stream bed elevation required to pass design dicharge.

b. Compute allowable head above the face section invert (H_f) by taking the difference between the allowable headwater elevation and stream bed elevation.

c. Determine dimensions for trial options.

d. Compare construction costs for various options, including the cost of FALL on the throat.

e. Select design with incremental cost warranted by increased capacity and improved performance.

The detailed computations involved in computing slope-tapered inlets are summarized in the tabulation sheet used in STEP 4, titled "Improved Inlet for R.C. Box Culverts", Table 4-2, page 4-39.

Column 7: Enter face invert elevation. Normally this is the approximate stream bed elevation at face.

Column 8: Enter the ratio of $Q/NBD^{3/2}$ where:

Q = design discharge
N = number of barrels
B = width of culvert barrel
D = height of culvert

Column 9: The depth of pool, or head (H_t), above the throat invert is derived graphically by the use of Chart 4-10, page 4-67.

Column 10: Elevation of throat invert is derived by subtracting Column 8 from allowable headwater elevation. If the vertical distance below the stream bed elevation (FALL) exceeds an allowable amount (as determined by the designer) the normal barrel dimensions should be increased. Improved inlet dimensions should be recalculated beginning with Column 1 for side-tapered inlet.

Column 11: Enter ratio of headwater above the face section invert (H_f) to the height of box culvert (D) using the formula:

$$H_f/D = (HW_f - \text{Elevation face invert})/D$$

Column 12: The ratio of $Q/B_f\ D^{3/2}$ is derived by the use of Chart 4-11, page 4-68.

Column 13: The minimum width of face section (B_f) is the quotient of design discharge (Q) divided by the product of Column 3 times Column 12. The derived width should be rounded up to the nearest foot.

Column 14: "L_3" is the dimension relating to the slope bend point of the culvert barrel soffit to the point of normal barrel section. Refer to Figure 4-9, page 4-37 for a pictorial definition. Minimum "L_3" = 0.5 NB

where: N = number of barrels
B = width of culvert barrel

Column 15: "L_2" is the horizontal dimension of the improved inlet from the face to the soffit slope bend point of the culvert barrel. Refer to Figure 4-9, page 4-37 for a pictorial definition. The length of slope "L_2" should be derived by using the formula:

$$L_2 = (\text{face invert elevation} - \text{throat invert elev.})S_f$$

where:

S_f = slope of FALL (use 2 for 2:1 slope and 3 for 3:1 slope).

Column 16: Column 15 should be checked for dimension adjustments that may be required of "L_3" or design Taper by using the formula:

$$L_2 = \frac{B_f - NB}{2} (\text{Taper} - L_3)$$

where:

B_f = width of face section in feet.

NB = number of barrels times width of culvert barrel in feet.

Taper = sidewall flare

$L_3 = 0.5$ NB

Column 17: If Column 16 is greater than Column 15, "L_3" should be adjusted using the formula:

$$L_3 = \frac{B_f - NB}{2} (\text{Taper} - L_2)$$

All terms are as previously defined.

Column 18: If Column 15 is greater than Column 16, TAPER length should be adjusted using the formula:

$$\text{Taper} = \frac{L_2 + L_3}{(B_f - NB)/2}$$

Column 19: Enter sum of Columns 14 (or 17) and 15.

Enter selected geometry in the upper right hand corner of Table 4-2, page 4-39.

CONCLUSION - EXAMPLE PROBLEM NO. 1

Since the requirements called for the smallest possible reinforced concrete box culvert, the barrel should be a single 8 ft. x 6 ft.

Selection of the improved inlet should be based on cost and site considerations. A side or slope-tapered design meets the design discharge and AHW requirements and appears to be cost effective

Since the depression (FALL) is only 1 foot and assuming the natural streambed can be lowered at the inlet without adverse effects, it is recommended that the beveled edge-side tapered inlet be used at this location.

SLOPE - TAPERED INLET

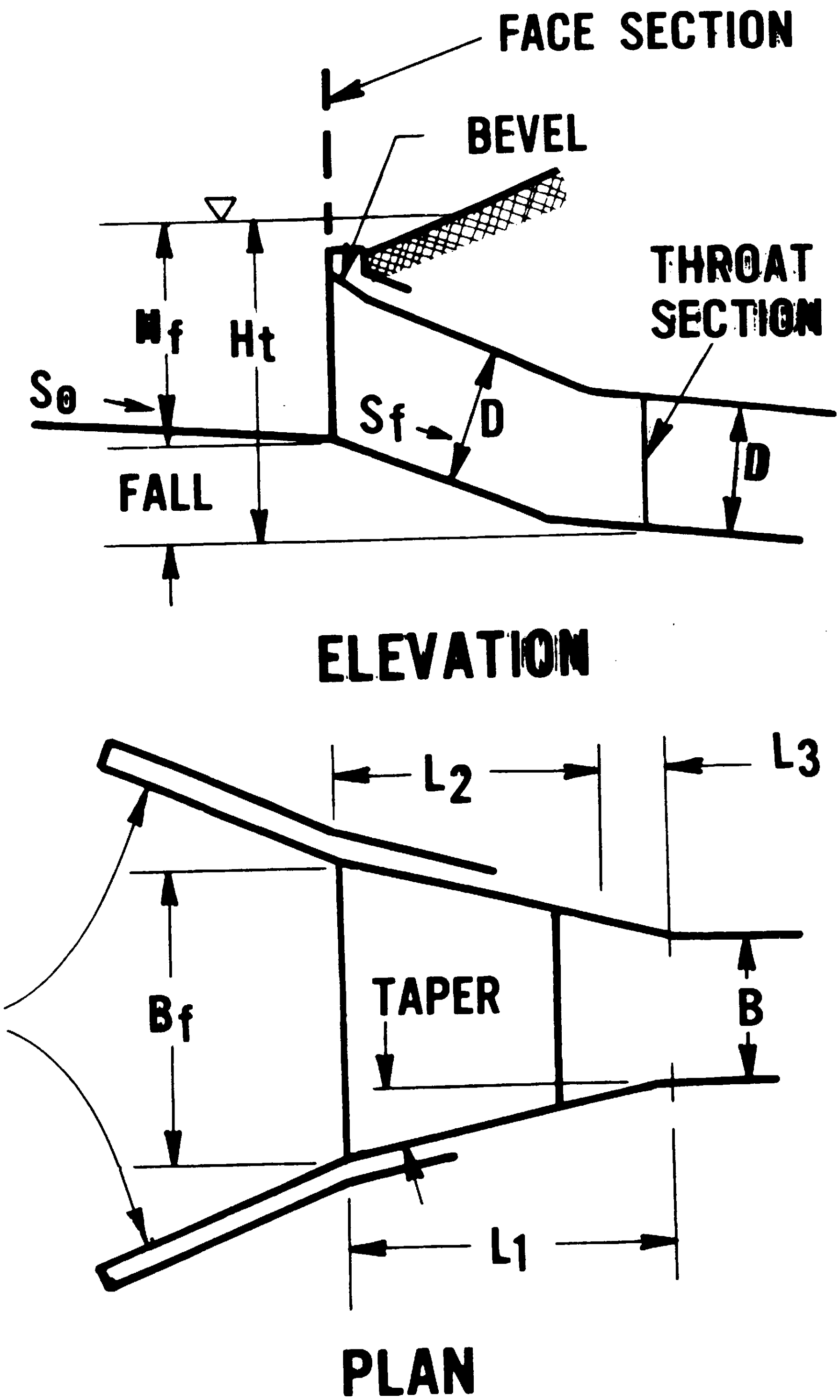


FIGURE 4-9

TABLE 4-1

FORM HYD 4-1

PROJECT: *Example No. 1*

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: *HYD*

DATE: *11-11-81*

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

Q_1 *50* = *1000* cfs

Q_2 ____ = ____ cfs

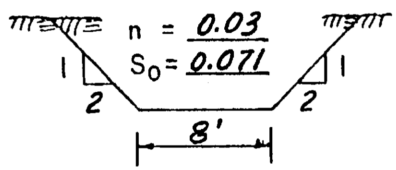
STREAM DATA

Tw_1 = *3.2*

Tw_2 = ____

n = *0.03*

S_0 = *0.071*



OUTLET CHANNEL
(APPROX. DIMENSIONS)

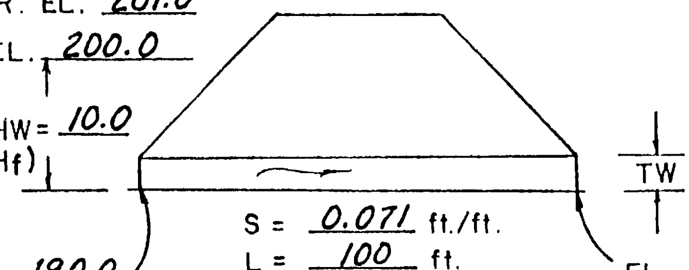
SKETCH

STATION: *30+00*

FIN. GR. EL. *201.0*

HW_f EL. *200.0*

AHW = *10.0*
(H_f)



S = *0.071* ft./ft.

L = *100* ft.

H_f EL. *190.0*

EL. *182.9*

TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q/NB $\frac{Q}{NBD^{3/2}}$	(a) Ke	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. $\frac{HW}{D}$ (f)	INLET CONT. HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
			Q	HEADWATER COMPUTATION — OUTLET CONTROL												
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	8x6	SQ. EDGE	125	0.4	11.0	>6	6	3.2	7.1	9.9						CLOSE TO AHW - TRY 7'x6'
2	7x6	"	143	0.4	15.0	>6	6	3.2	7.1	13.9						EXCEEDS AHW - CHECK TRIAL 1 FOR BEVELED EDGE
3	8x6	BEVELED	125/8.5	0.2	9.5	>6	6	3.2	7.1	8.4	3.1	18.6	18.6	21		LOWERED HW_o 1.5'. H_f EXCEEDS AHW - TRY SIDE-TAPERED

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

TABLE 4-2

FORM HYD 4-2 | IMPROVED INLET FOR R.C. BOX CULVERTS | DESIGNER: *HYD*

PROJECT: *Example No. 1* | | DATE: *11-11-81*

IMPROVED INLET FOR R.C. BOX CULVERTS

INITIAL DATA

Q *50* = *1000* cfs S_o *0.071*

AHW EL. *200* ft. L = *100* ft.

El. Stream bed at face *190.0* ft.

Allowable Headwater *10.0* ft.

TAPER = *4* :1 (4:1 to 6:1)

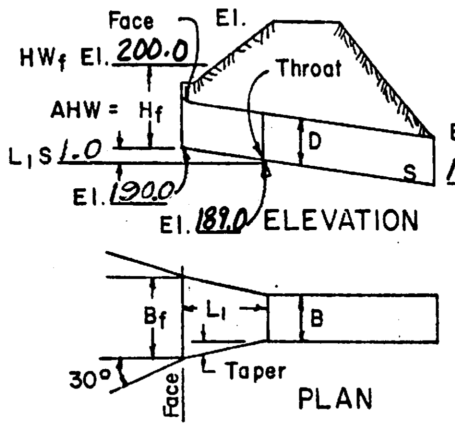
S_f = *3* :1 (2:1 to 3:1)

Barrel Shape and Material *R.C.B.*

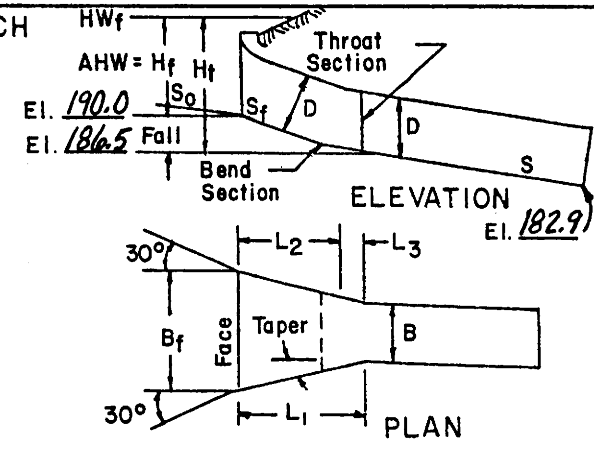
Inlet Edge Description *BEVELED*

N = *1* B = *8* ft, D = *6* ft.

SKETCH



SIDE - TAPERED



SLOPE-TAPERED (Vertical Face)

SELECTED DESIGN

B_f = *15.0* ft. L_3 *—* ft.

L_2 = *—* ft. L_1 *14.0* ft.

Bevels: Angle *30* °

d = *6* in.

TAPER *4* :1

S_f ____ :1

S *0.071* ft./ft.

IMPROVED INLET

SIDE TAPERED (n)						SLOPE TAPERED ($D/4 \leq$ Fall $\leq 1.5D$)													COMMENTS
$\frac{H_f}{D}$ (a)	$\frac{Q}{B_f D^{3/2}}$ (b)	$D^{3/2}$	Min B_f (c)	L_1 (d)	S ft./ft.	Elev. Face Invert	$\frac{Q}{NBD^{3/2}}$	H_t (e)	Elev. Throat Invert	H_f/D (f)	$\frac{Q}{B_f D^{3/2}}$ (g)	Min B_f (c)	Min L_3 (h)	L_2 (i)	Check L_2 (j)	Adj L_3 (k)	Adj Taper (l)	L_1 (m)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1.7	*4.8*	*14.7*	*15.0*	*14.0*	*0.071*	*190.0*	*8.5*	*13.5*	*186.5*	*1.7*	*5.1*	*14.0*	*4.0*	*10.5*	*8.0*	*—*	*4.83:1*	*14.5*	*OK*

(a) $H_f/D = (HW_f - \text{Throat Invert Elev.} - 1)/D$

(b) Use Chart 4-9, page 4-66.

(c) Min $B_f = \dfrac{Q}{(D^{3/2})\dfrac{Q}{B_f D^{3/2}}}$

(d) $L_1 = \dfrac{B_f - NB}{2}$ (TAPER)

(e) Use Chart 4-10, page 4-67.

(f) $H_f/D = (HW_f - \text{Face Invert Elev.})/D$

(g) Use Chart 4-11, page 4-68.

(h) Min $L_3 = 0.5\ NB$

(i) $L_2 = (\text{Elev. Face Invert} - \text{Elev. Throat Invert})\ S_f$

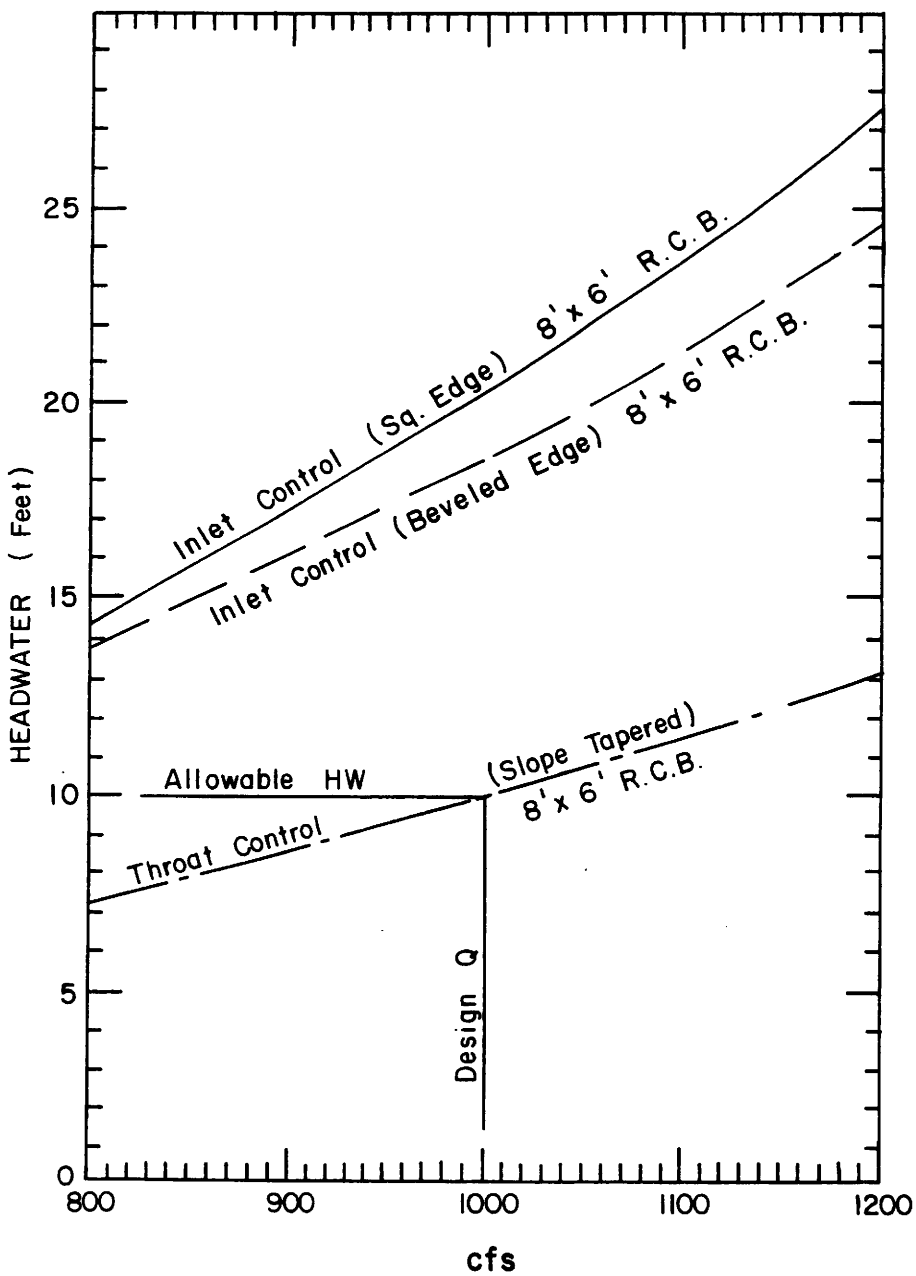
(j) Check $L_2 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER $- L_3$

(k) If Column 16 > 15, Adjust $L_3 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER $- L_2$

(l) If Column 15 > 16, Adjust TAPER $= (L_2 + L_3)/\left[\dfrac{B_f - NB}{2}\right]$

(m) $L_1 = L_2 + L_3$

(n) Elev. Face Invert − Elev. Throat Invert $\leq$ 1 ft.



EXAMPLE PROBLEM NO. 1
DESIGN
PERFORMANCE CURVES
8' x 6' R. C. BOX CULVERT

FIGURE 4-10

4-804 EXAMPLE PROBLEM NO. 2

GIVEN: Design Discharge (Q_{25}) = 1,500 cfs
Slope of streambed (S_o) = 0.02 ft./ft.
Allowable headwater elevation = 300.0
Elevation outlet invert = 290.0
Culvert length (L) = 100 ft.

Downstream channel approximates a 12 ft. wide trapezoidal channel with 2:1 slopes, Manning "n" of 0.03 and a slope (S_o) of 0.02 feet per foot.

REQUIRED:

Design a culvert that will adequately convey the design discharge. The culvert should have the smallest possible barrel to pass design Q without exceeding AHW elevation. Try a reinforced concrete box culvert with "n" = 0.012.

SOLUTION:

Use procedures as outlined in STEPS 1 through 5 of Example Problem No. 1

COMPUTATIONS:

See Table 4-3 and 4-4, pages 4-42 and 4-43, repectively for detailed computations.

CONCLUSION - EXAMPLE PROBLEM NO. 2

Again, the requirements called for the smallest possible reinforced concrete box culvert. The barrel should be a double 10 ft. x 7 ft. R.C. box culvert.

As in problem No. 1, selection of the improved inlet should be based on cost and site considerations. A side-tapered design meets the design discharge and AHW requirements.

The existing stream bed slope restricted FALL to $D/_4$. Therefore slope-tapered inlet computations are not applicable.

It should be noted that the face width (B_f) is the total clear face width needed. The width of the division wall must be added to this value in order to size the face correctly.
No design procedure is available for side-tapered inlet culverts with more than two barrels.

TABLE 4 - 3

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

FORM HYD 4-1

PROJECT: *Example No. 2*

DESIGNER: *HYD*

DATE: *11-11-81*

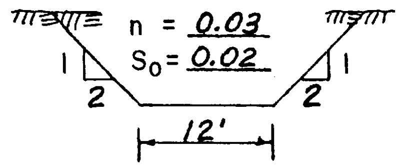
HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

Q_1 *25* = *1500* cfs
Q_2 ____ = ____ cfs

STREAM DATA

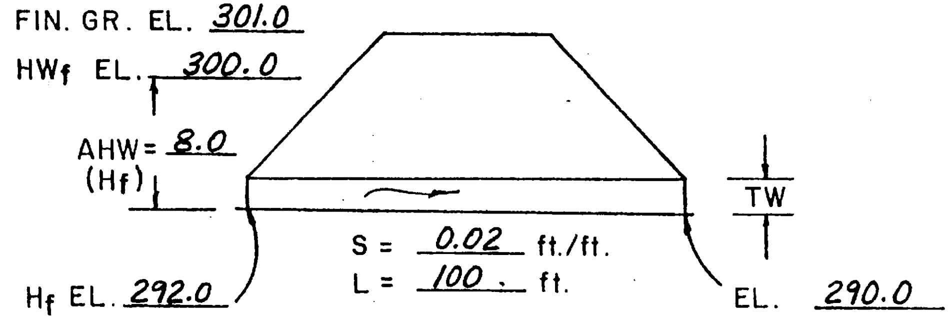
Tw_1 = *5.0*
Tw_2 = *4.8*



OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

STATION: *200+00*



TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q / Q/NB: $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION – OUTLET CONTROL: (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT.: $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	DBL. 6x6	SQ. EDGE	125	0.4	10.5	>6	6	4.8	2.0	14.5						HW_o > AHW - TRY DBL. 7x7
2	DBL. 7x7	"	107	0.4	5.7	>7	7	4.8	2.0	10.7						" " - TRY DBL. 8x7
3	DBL. 8x7	"	94	0.4	4.6	6.5	6.8	4.8	2.0	9.4						" " - TRY DBL. 9x7
4	DBL. 9x7	"	83	0.4	3.6	6.0	6.5	4.8	2.0	8.1						CLOSE TO AHW - CHECK BEVELED EDGE
5	DBL. 9x7	BEVELED	83 / 4.5	0.2	3.0	6.0	6.5	4.8	2.0	7.5						LOWERED HW_o 0.6 - TRY DBL. 10x7
6	DBL. 10x7	"	75 / 4.0	0.2	2.3	5.5	6.25	4.8	2.0	6.55	1.3	9.1	9.1	11		H_f EXCEEDS AHW - TRY SIDE TAPERED

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

TABLE 4 - 4

FORM HYD 4 - 2

PROJECT: *Example No. 2*

IMPROVED INLET FOR R.C. BOX CULVERTS

DESIGNER: *HYD*

DATE: *11-11-81*

INITIAL DATA

Q *25* = *1500* cfs S_o *0.02*

AHW EL. *300* ft. L = *100* ft.

El. Stream bed at face *292.0* ft.

Allowable Headwater *8.0* ft.

TAPER = *4* :1 (4:1 to 6:1)

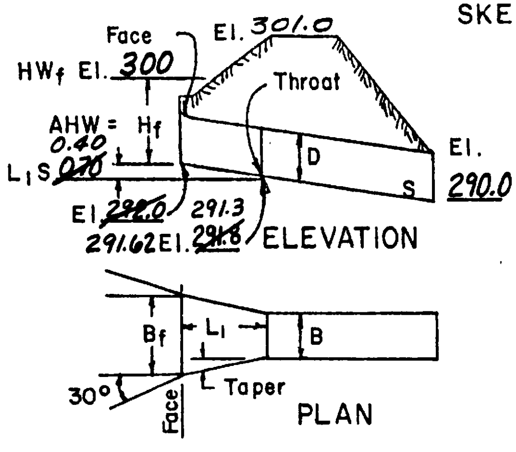
S_f = *—* :1 (2:1 to 3:1)

Barrel Shape and Material *R.C.B.*

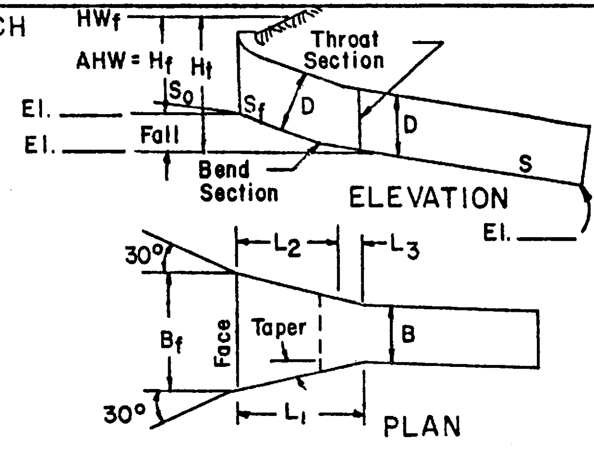
Inlet Edge Description *BEVELED*

N= *2* B= *10* ft, D= *7* ft.

SKETCH



SIDE - TAPERED



SLOPE - TAPERED (Vertical Face)

SELECTED DESIGN

B_f = *30.0* ft. L_3 *—* ft.

L_2 = *—* ft. L_1 *20.0* ft.

Bevels: Angle *30* °

d = *7* in.

TAPER *4* :1

S_f ____ :1

S ____ ft./ft.

IMPROVED INLET

SIDE TAPERED (n)						SLOPE TAPERED ($D/4 \leq$ Fall $\leq 1.5\,D$)													COMMENTS
$\frac{H_f}{D}$ (a)	$\frac{Q}{B_f D^{3/2}}$ (b)	$D^{3/2}$	Min B_f (c)	L_1 (d)	S ft./ft.	Elev. Face Invert	$\frac{Q}{NBD^{3/2}}$	H_t (e)	Elev. Throat Invert	H_f/D (f)	$\frac{Q}{B_f D^{3/2}}$ (g)	Min B_f (c)	Min L_3 (h)	L_2 (i)	Check L_2 (j)	Adj L_3 (k)	Adj Taper (l)	L_1 (m)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1.0	*2.4*	*18.5*	*34.0*	*28.0*	*0.025*		*EL. Hf INVERT ABOVE STREAM BED - RECOMPUTE*												
1.1	*2.7*	*18.5*	*30.0*	*20.0*	*0.016*	*292.0*	*4.0*	*8.4*	*291.6*	*—*	*FALL < D/4 - USE SIDE TAPERED*								
				NOTE: MIN. Bf SHOWN IN COLUMN 4 DOES NOT INCLUDE DIVISION WALL															

(a) H_f/D = (HW$_f$ − Throat Invert Elev. − 1)/D

(b) Use Chart 4-9, page 4-66.

(c) Min $B_f = \dfrac{Q}{(D^{3/2})\dfrac{Q}{B_f D^{3/2}}}$

(d) $L_1 = \dfrac{B_f - NB}{2}$ (TAPER)

(e) Use Chart 4-10, page 4-67.

(f) H_f/D = (HW$_f$ − Face Invert Elev.)/D

(g) Use Chart 4-11, page 4-68.

(h) Min L_3 = 0.5 NB

(i) L_2 = (Elev. Face Invert − Elev. Throat Invert) S_f

(j) Check $L_2 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER − L_3

(k) If Column 16 > 15, Adjust $L_3 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER − L_2

(l) If Column 15 > 16, Adjust TAPER = $(L_2 + L_3)/\left[\dfrac{B_f - NB}{2}\right]$

(m) $L_1 = L_2 + L_3$

(n) Elev. Face Invert − Elev. Throat Invert ≤ 1 ft.

4-805 EXAMPLE PROBLEM NO. 3

GIVEN:	Design Discharge (Q_{50})	= 1,000 cfs
	Slope of streambed (S_o)	= 0.175 ft./ft.
	Allowable headwater elevation	= 200.0
	Elevation outlet invert	= 172.5
	Culvert length (L)	= 100 ft.

Downstream channel approximates an 8 ft. wide trapezoidal channel with 2:1 slopes, Manning "n" of 0.03 and a slope (S_o) of 0.175 feet per foot.

REQUIRED: Design a culvert that will adequately convey the design discharge. The culvert should have the smallest possible barrel to pass design Q without exceeding AHW elevation. Try a reinforced concrete box culvert with "n" = 0.012.

SOLUTION: Use procedures as outlined in STEPS 1 through 5 of Example Problem No. 1.

COMPUTATIONS:

See Table 4-5 and 4-6, pages 4-45 and 4-46, respectively, for detailed computations.

CONCLUSION - EXAMPLE PROBLEM NO. 3

The selected design for the smallest barrel section is a 6' x 6' R.C. box culvert.

All improved inlet types were reviewed for discharge and AHW requirements. An on site inspection and structure type economics should be considered by the designer as parameters for the hydraulic decision.

The existing streambed slope is steep (0.175 ft./ft.). The side-tapered inlet must have FALL upstream of the inlet face. If this is not permissible, then the designer should omit the computations for the side-tapered inlet and check the slope-tapered inlet, which incorporates a bend within the barrel. Headwater control will be at the throat instead of the face.

TABLE 4-5

FORM HYD 4-1

PROJECT: *Example No. 3*

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: *HYD*

DATE: *11-11-81*

HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

Q_1 *50* = *1000* cfs

Q_2 ____ = ____ cfs

cfs

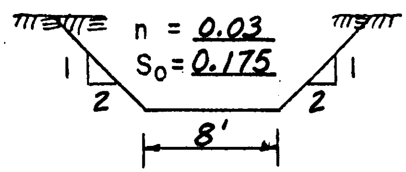
STREAM DATA

Tw_1 = *4.6*

Tw_2 = ____

n = *0.03*

S_0 = *0.175*



OUTLET CHANNEL
(APPROX. DIMENSIONS)

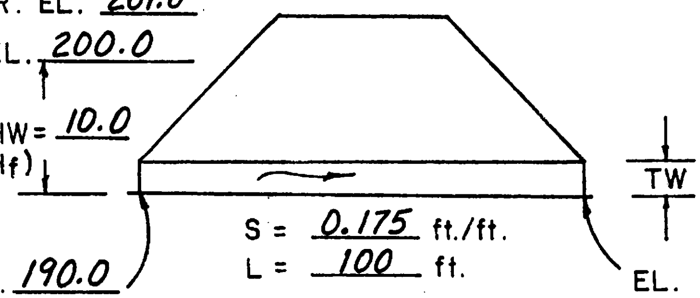
SKETCH

STATION: *400+00*

FIN. GR. EL. *201.0*

HW_f EL. *200.0*

AHW = *10.0*
(H_f)



S = *0.175* ft./ft.

L = *100* ft.

H_f EL. *190.0*

EL. *172.5*

TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q — Q/NB — $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION — OUTLET CONTROL — (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. — $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1 (Q_1)	*7x6*	*SQ. EDGE*	*143*	*0.4*	*14.6*	*> 6*	*6*	*4.6*	*17.5*	*3.1*						*6.9' LOWER THAN AHW TRY 6x6*
2 (Q_1)	*6x6*	*"*	*167*	*0.4*	*20.0*	*> 6*	*6*	*4.6*	*17.5*	*8.5*						*CLOSE TO AHW CHECK 5x6*
3 (Q_1)	*5x6*	*"*	*200*	*0.4*	*29.0*	*> 6*	*6*	*4.6*	*17.5*	*17.5*						*EXCEEDS AHW - CHECK 6x6 BEVELED EDGE*
4 (Q_1)	*6x6*	*BEVELED*	*167 / 11.3*	*0.2*	*18.0*	*> 6*	*6*	*4.6*	*17.5*	*6.5*	*4.1 ±*	*24.6*	*24.6*	*28*		*LOWERED HW_o 2.0'; H_f > AHW - TRY SIDE TAPERED*

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

TABLE 4-6

FORM HYD 4-2

PROJECT: *Example No. 3*

IMPROVED INLET FOR R.C. BOX CULVERTS

DESIGNER: *HYD*

DATE: *11-11-81*

INITIAL DATA

Q_{50} = *1000* cfs S_o *0.175*

AHW EL. *200* ft. L = *100* ft.

El. Stream bed at face *190.0* ft.

Allowable Headwater *10.0* ft.

TAPER = *4* :1 (4:1 to 6:1)

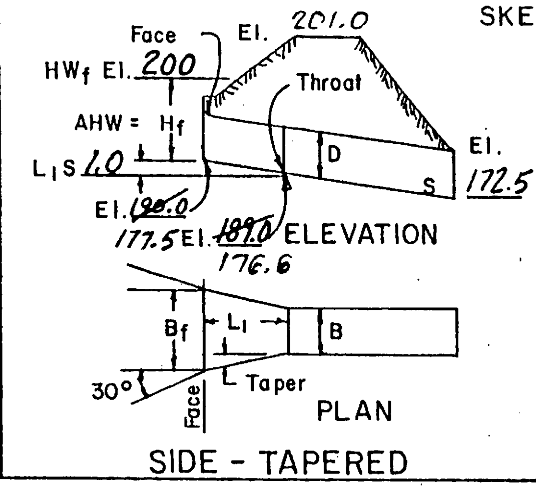
S_f = *2* :1 (2:1 to 3:1)

Barrel Shape and Material *R.C.B.*

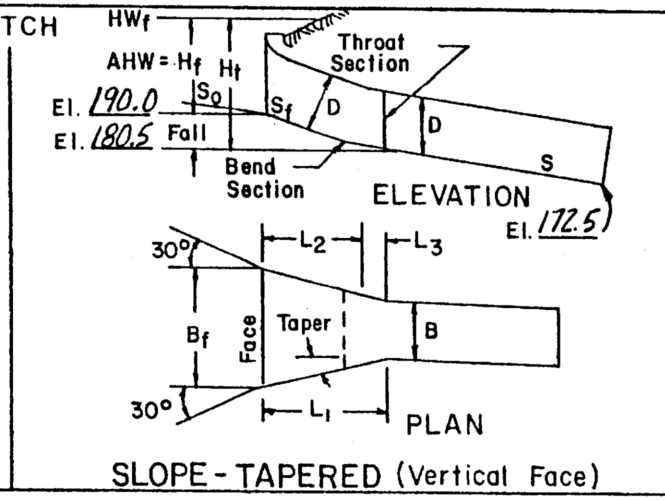
Inlet Edge Description *BEVELED*

N= *1* B= *6* ft, D= *6* ft.

SKETCH



SIDE - TAPERED



SLOPE - TAPERED (Vertical Face)

SELECTED DESIGN

B_f = *14* ft. L_3 *3* ft.

L_2 = *19* ft. L_1 *22* ft.

Bevels: Angle *30* °

d = *6* in.

TAPER *5.5* :1

S_f *2* :1

S *0.103* ft./ft.

IMPROVED INLET

SIDE TAPERED (n)						SLOPE TAPERED ($D/4 \le$ Fall $\le 1.5D$)													COMMENTS
$\frac{H_f}{D}$ (a)	$\frac{Q}{B_f D^{3/2}}$ (b)	$D^{3/2}$	Min B_f (c)	L_1 (d)	S ft./ft.	Elev. Face Invert	$\frac{Q}{NBD^{3/2}}$	H_f (e)	Elev. Throat Invert	H_f/D (f)	$\frac{Q}{B_f D^{3/2}}$ (g)	Min B_f (c)	Min L_3 (h)	L_2 (i)	Check L_2 (j)	Adj L_3 (k)	Adj Taper (l)	L_1 (m)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1.67	*4.6*	*14.7*	*15.0*	*18.0*	*0.05*	*12.5' FALL WOULD HAVE TO BE INCORPORATED UPSTREAM FOR*													
						SIDE-TAPERED INLET TO FUNCTION WITHOUT BEND SECTION													
						BECAUSE OF S (0.05%) REQUIRED TO KEEP FACE & THROAT													
						ELEV. DIFFERENCE ≤ 1 FT. TRY SLOPE TAPERED INLET.													
						190.0	*11.3*	*19.5*	*180.5*	*1.67*	*5.0*	*14.0*	*3.0*	*19.0*	*13.0*	*—*	*5.5:1*	*22.0*	*USE*

(a) H_f/D = (HW_f − Throat Invert Elev. − 1)/D

(b) Use Chart 4-9, page 4-66.

(c) Min $B_f = \frac{Q}{(D^{3/2})\frac{Q}{B_f D^{3/2}}}$

(d) $L_1 = \frac{B_f - NB}{2}$ (TAPER)

(e) Use Chart 4-10, page 4-67.

(f) H_f/D = (HW_f − Face Invert Elev.)/D

(g) Use Chart 4-11, page 4-68.

(h) Min L_3 = 0.5 NB

(i) L_2 = (Elev. Face Invert − Elev. Throat Invert) S_f

(j) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_3

(k) If Column 16 > 15, Adjust $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(l) If Column 15 > 16, Adjust TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(m) $L_1 = L_2 + L_3$

(n) Elev. Face Invert − Elev. Throat Invert $\le$ 1 ft.

TABLE 4-7

FORM HYD 4-1

CULVERT COMPUTATIONS
(SQUARE AND BEVELED EDGES)

DESIGNER: ______

PROJECT: ______

DATE: ______

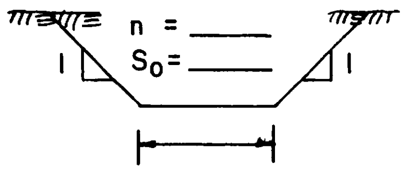
HYDROLOGIC AND CHANNEL INFORMATION

HYDROLOGY

Q_1 ____ = ____ cfs
Q_2 ____ = ____ cfs

STREAM DATA

Tw_1 = ____
Tw_2 = ____



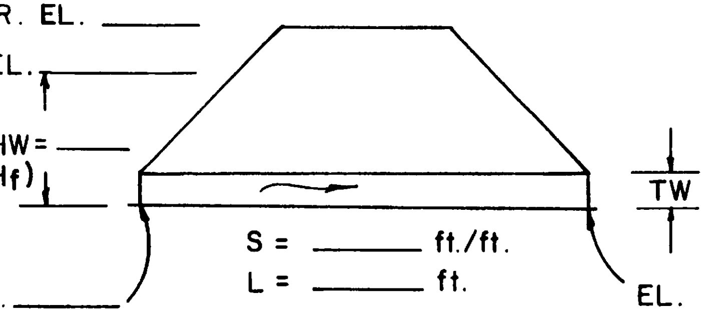
n = ____
S_0 = ____

OUTLET CHANNEL
(APPROX. DIMENSIONS)

SKETCH

STATION: ______

FIN. GR. EL. ______
HW_f EL. ______
AHW = ____
(H_f)



S = ____ ft./ft.
L = ____ ft.

TW

H_f EL. ______

EL. ______

TRIAL NO.	SIZE	STRUCTURE TYPE & ENTRANCE DESIGN	Q/NB $\frac{Q}{NBD^{3/2}}$	HEADWATER COMPUTATION: OUTLET CONTROL (a) K_e	H	(b) d_c	(c) h_o	(d) TW	LS	(e) HW_o	INLET CONT. $\frac{HW}{D}$ (f)	HW	CONTROL-LING HW	OUTLET VELOCITY ft./sec.	COST	COMMENTS
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

(a) Entrance loss coefficient, Refer to Table 4-10, page 4-60.
(b) "d_c" cannot exceed D.
(c) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.
(d) TW = d_n in natural channel, or other downstream control.
(e) $HW_o = H + H_o - LS$
(f) Use Chart 4-7, page 4-64, for Conventional face.
Use Chart 4-8, page 4-65, for Beveled Edge.

TABLE 4-8

FORM HYD 4-2

PROJECT: ______

IMPROVED INLET FOR R.C. BOX CULVERTS

DESIGNER: ______

DATE: ______

INITIAL DATA

Q____ = ________ cfs S_o ________

AHW EL. ________ ft. L = ________ ft.

El. Stream bed at face ________ ft.

Allowable Headwater ________ ft.

TAPER = ____ : 1 (4:1 to 6:1)

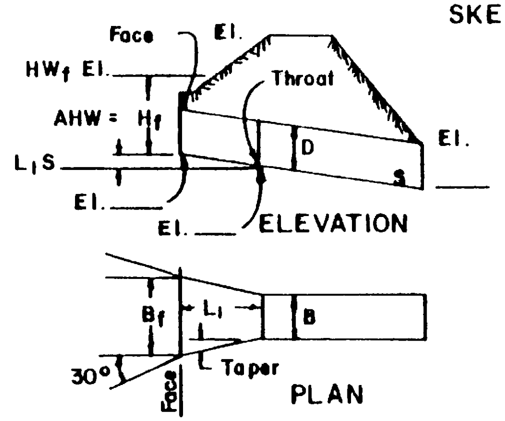
S_f = ____ : 1 (2:1 to 3:1)

Barrel Shape and Material ________

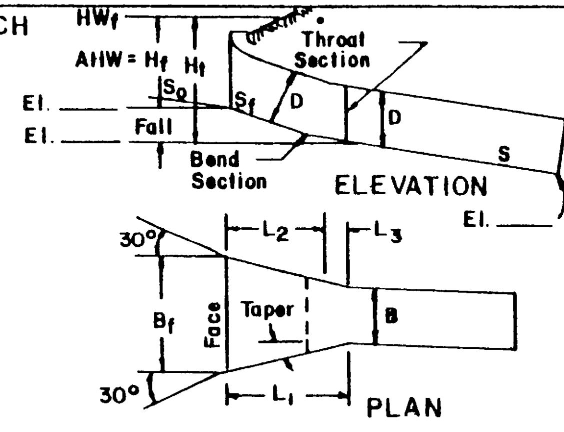
Inlet Edge Description ________

N= ______ B= ______ ft, D= ______ ft.

SKETCH



SIDE - TAPERED



SLOPE - TAPERED (Vertical Face)

SELECTED DESIGN

B_f = ______ ft. L_3 ______ ft.

L_2 = ______ ft. L_1 ______ ft.

Bevels: Angle ______ °

d = ______ in.

TAPER ______ : 1

S_f ______ : 1

S ______ ft./ft.

IMPROVED INLET

SIDE TAPERED (n)						SLOPE TAPERED ($D/4 \le$ Fall ≤ 1.5 D)													COMMENTS
$\frac{H_f}{D}$ (a)	$\frac{Q}{B_f D^{3/2}}$ (b)	$D^{3/2}$	Min B_f (c)	L_1 (d)	S ft./ft.	Elev. Face Invert	$\frac{Q}{NBD^{3/2}}$	H_f (e)	Elev. Throat Invert	H_f/D (f)	$\frac{Q}{B_f D^{3/2}}$ (g)	Min B_f (c)	Min L_3 (h)	L_2 (i)	Check L_2 (j)	Adj L_3 (k)	Adj Taper (l)	L_1 (m)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

(a) $H_f/_D$ = (HW_f - Throat Invert Elev. - 1)/D

(b) Use Chart 4-9, page 4-66.

(c) Min $B_f = \dfrac{Q}{(D^{3/2})\dfrac{Q}{B_f D^{3/2}}}$

(d) $L_1 = \dfrac{B_f - NB}{2}$ (TAPER)

(e) Use Chart 4-10, page 4-67.

(f) $H_f/_D$ = (HW_f - Face Invert Elev.)/D

(g) Use Chart 4-11, page 4-68.

(h) Min L_3 = 0.5 NB

(i) L_2 = (Elev. Face Invert - Elev. Throat Invert) S_f

(j) Check $L_2 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER $- L_3$

(k) If Column 16 > 15, Adjust $L_3 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER $- L_2$

(l) If Column 15 > 16, Adjust TAPER = $(L_2 + L_3) / \left[\dfrac{B_f - NB}{2}\right]$

(m) $L_1 = L_2 + L_3$

(n) Elev. Face Invert - Elev. Throat Invert $\le$ 1 ft.

4-806 DESIGN LIMITATIONS AND RECOMMENDATIONS

1. For multibarrel installations bevels must be sized on the basis of total clear opening rather than on individual barrel size.

2. For multibarrel installations exceeding 3:1 width to depth ratio it is recommended that the side bevel be sized in proportion to the total clear width (B), or three times the height, whichever is smaller.

3. Skewed inlets should be avoided whenever possible, and should not be used with side or slope-tapered inlets.

4. 4:1 taper is recommended for side-tapered inlets.

5. Upstream fall is normally not recommended. If fall is warranted, the plane of the barrel invert from upstream FALL (depression) of side-tapered inlets should be extended upstream from the face of wingwalls a minimum distance of D/2. The designer should always consider a slope-tapered inlet structure where fall is warranted in lieu of upstream depression.

6. For double barrel structures, the face width, as determined from Charts 9 and 11, pages 4-66 and 4-68 respectively, is the total clear face width needed. The width of the division wall must be added to this value in order to size the face correctly.

7. No design procedures are available for side-tapered and slope-tapered inlet culverts with more than two barrels.

8. The FALL slope must range from 2:1 to 3:1.

9. FALL should range from D/4 to 1.5D. FALLS less than D/4, must be designed as side-tapered inlets.

10. DO NOT interpolate between Charts 4-9 and 4-11, pages 4-66 and 4-68 respectively.

11. DO NOT use an "L_3" value less than 0.5B.

12. Throat control should govern in the design of all improved inlets.

4-900 COMPUTER PROGRAM

A computer program for "Hydraulic Analysis of Culvert", HY-6, developed by the Federal Highway Administration is available "in-house" and is "on-line" in Computer Services. A users manual is available from FHWA or Hydraulics Section.

This program is for the hydraulic analysis of concrete and circular culverts for any given hydrological data and side conditions. In addition to designs for conventional culverts, this program includes:

- Analysis and designs for improved inlets based on concepts presented in FHWA's Hydraulic Engineering Circular No. 13, "Hydraulic Design of Improved Inlets for Culverts", 1972. The improved inlet designs include bevel-edged, side-tapered and slope-tapered inlets.

Technical programming assistance may be obtained from Computer Services or Hydraulics Section. See Figure 4-11, page 4-51 for Standard Input Data Sheet.

INPUT DATA

CARD NO. 1 — PROBLEM IDENTIFICATION

Columns	Field
1 – 78	Problem identification (columns 1, 2, 3 … 50 ←→ 78)

CARD NO. 2 — SITE DATA

Columns	Field	Decimal point after column
1 – 5	Culvert Code	
6 – 12	Culvert Slope	8
13 – 19	Approx. culvert length (DIST)	18
20 – 26	Design discharge (Q_1)	25
27 – 33	Design Tailwater Depth (TW)	32
34 – 40	Allowable Headwater Elevation (AHW)	39
41 – 47	Inlet Toe-Embank. Elevation (ELIN)	46
48 – 54	Outlet Toe-Embank. Elevation (ELOUT)	53
55 – 61	Upstream Embankment Slope (SEL)	60
62 – 68	Downstream Embankment Slope (SER)	67
69 – 75	Fall Slope Slope-Taper Inlet (SFACE)	74

CARD NO. 3 — FOR BOX CULVERTS

Columns	Field
1 – 2	KBAS1
3 – 4	KBAS2
5 – 6	KDEP1
7 – 8	KDEP2 *

CARD NO. 3 — FOR PIPE CULVERTS

Columns	Field	Decimal point after column
1 – 5	DIA1	3
6 – 10	DIA2	8

*BOX CULVERT AND PIPE CULVERT SIZES-SEE INPUT SECTION

FIGURE 4-11

4-1000 OUTLET VELOCITY CONTROL

4-1001 GENERAL

If the outlet velocity of a culvert is deemed excessive by the designer, there are possible solutions available by which velocity may be either reduced or controlled. The most widely used control has been the use of riprap which covers the channel area immediately downstream from the culvert outlet. However, there are other design possibilities available. Certain special culvert types have the chief function of maintaining acceptable outlet velocities which would be excessive if a straight profile culvert were used. One such special culvert type is the broken-back culvert.

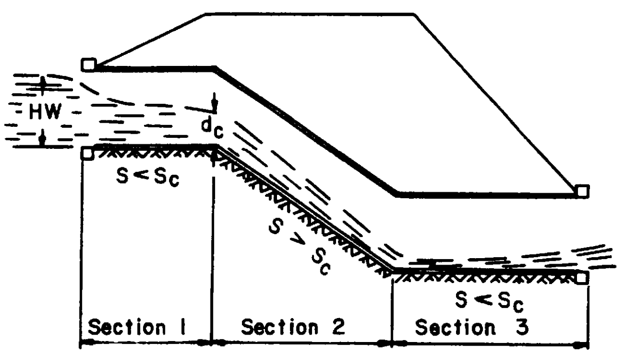


FIGURE 4-12

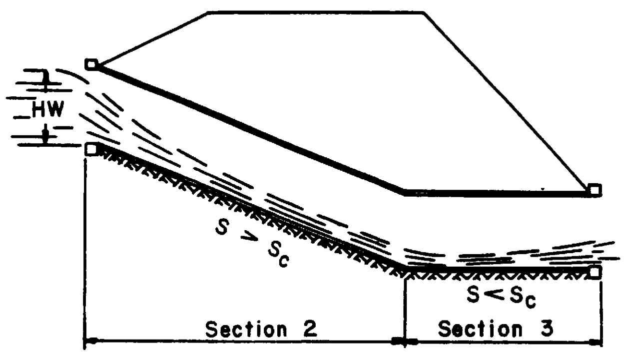


FIGURE 4-13

It is recommended that Section 1 either not be used or be very short. This allows for more adjustment in the profiles of Section 2 and Section3.

4-1002 VELOCITY CONTROL DEVICES

4-1002.1 BROKEN-BACK CULVERT DESIGN

The designer may occasionally encounter culvert locations at which the difference in elevation from one side of the highway to the other is so great as to cause outlet velocities which are unacceptably high. In this event, the engineer may choose to design a broken-back culvert as depicted in Figures 4-12 or 4-13, page 4-52. The advantage of a broken-back culvert towards reducing outlet velocities lies in the theory of the hydraulic jump.

The broken-back culvert employs the hydraulic jump as an energy dissipator (or velocity reducer). But the formation of the jump requires that:

a. there be sufficient natural tailwater greater than critical depth of culvert (not usually the case since the broken-back culvert is usually located on a steep profile such as a hillside and stream depth is rather shallow).

b. there be sufficient friction in Section 3 (see Figures 4-12 and 4-13, page 4-52. This is possible only if the culvert material is very rough and/or the Section 3 length is relatively long. Normally, in a broken-back culvert, neither of the above circumstances exist. It is therefore necessary to employ special energy dissipator devices to assist in forming the hydraulic jump and reduce velocities.

The hydraulic jump is a feature in hydraulic flow by which flow in a super-critical condition is abruptly changed to flow in a subcritical condition. In other words, a given discharge traveling at some depth less than critical depth and some velocity greater than critical velocity, upon passing through a hydraulic jump, then flows at some depth greater than critical depth and some velocity less than critical velocity.

The design of a broken-back culvert is not particularly difficult but certain provisions must be made or circumstance found so that the primary intent of reducing velocity at the outlet is realized. The design and the provisions or circumstances are discussed as follows:

1. INITIAL INFORMATION NEEDED:

   a. Upstream channel or culvert slope, width, Q and roughness factor.

   b. Downstream slope, width, Q (constant), and roughness factor.

   c. Tailwater

2. DATA TO BE CALCULATED FOR ANALYSIS

   a. Normal depth (d_{n1}) and velocity (V_1) of upstream section.

   b. Normal depth (d_{n2}) and velocity (V_2) downstream section.

   c. Calculate the Froude Number (F_1) for the upstream section:

$$F_1 = \frac{V_1}{(gd_{n1})^{0.5}}$$

   d. Determine the ratio of the downstream normal depth to the pstream normal depth, i.e. d_{n2}/d_{n1}

3. HYDRAULIC JUMP DETERMINATION (Reference 4-4)

   a. If $F_1 > 2.0$ and $d_{n2}/d_{n1} > 2.4$, then a jump is indicated.

   b. If $F_1 < 1.7$ and $d_{n2}/d_{n1} < 2.0$, then a jump is not indicated.

   c. If $F_1 < 1.0$, then a jump is not possible.

   d. A hydraulic jump in horizontal rectangular channels will form if the Froude Number F_1 of the flow, the flow depth d_{n1}, and a downstream depth d_{n2} satisfy the equation $d_{n2}/d_{n1} = 0.5\ (1+8F_1{}^2)^{0.5}-1$. This equation may be represented by the curve on Chart 4-1, page 4-57.

4. CALCULATION OF LENGTH OF HYDRAULIC JUMP is graphically shown on Chart 4-2, page 4-57 or may be estimated from the following equations:

   a. If $2.0 < F_1 < 5.5$, then the approximate length of jump is equal to the Froude Number times the downstream normal depth (Reference 4-6).

   b. If $F_1 > 5.5$, then the approximate length of jump is equal to the downstream normal depth times 5.75 (Reference 4-6).

   Equations: Length (L) = $F_1\ d_{n2}$ (a)

   Length (L) = $5.75\ d_{n2}$ (b)

EXAMPLE PROBLEM:

Given: Type of Structure - 4' x 4' RCB

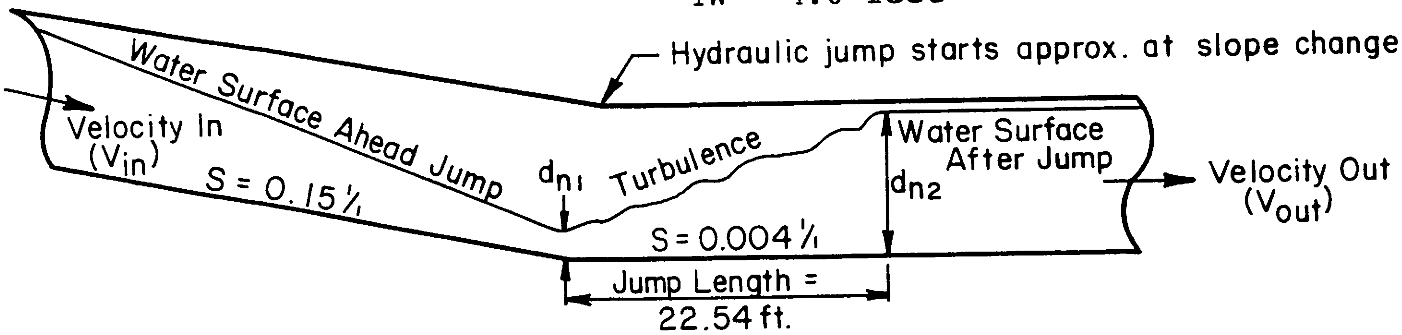
Upstream conditions - Slope = 15%

n = 0.012

Width = 4.0 feet

Q = 150 cfs

Downstream conditions - Slope = 0.4%
n = 0.012
Width = 4.0 feet
Q = 150 cfs
TW = 4.0 feet



TYPICAL CROSS-SECTION & PROFILE

Figure 4-14

CALCULATE NORMAL DEPTH FOR 150 CFS FOR UPSTREAM AND DOWNSTREAM FLOW

Determine normal depth and velocity for upstream section:

d_{n1} = From Chart 4-28, page 4-88, using 150 cfs and slope of 15%

d_{n1} = 1.0 feet*

V_1 = 150 cfs/1.0 feet (4.0 feet)

V_1 = 37.5 fps

*This depth is a conservative estimate of the water depth at the break in profile between Sections 2 and 3. Actual depth is probably slightly more than normal depth for ordinary conditions (Reference 4-3).

Determine normal depth and velocity for downstream section:

d_{n2} = From Chart 4-28, page 4-88, using 150 cfs and slope of 0.4%

d_{n2} = 3.9 feet (must be $\leq$ TW)

V_2 = 150 cfs/3.9 feet (4.0 feet)

V_2 = 9.6 fps

CALCULATE FROUDE NUMBER FOR UPSTREAM SECTION

F_1 = 37.5 fps/ $[(32.2 \text{ ft/sec/sec/}) \ (1.0 \text{ ft.})]^{1/2}$

F_1 = 6.6

CALCULATE THE RATIO OF DOWNSTREAM NORMAL DEPTH TO UPSTREAM NORMAL DEPTH

d_{n2}/d_{n1} = 3.9 ft/1.0 ft.

d_{n2}/d_{n1} = 3.9

DETERMINE IF JUMP IS PRESENT

Is F_1 greater than 2.0?	Yes, 6.6 >2.0
Is F_1 greater than 5.5?	Yes, 6.6 >5.5
Is d_{n2}/d_{n1} ratio greater than 2.4?	Yes, 3.9 > 2.4

All requirements are met, therefore, jump is present.

DETERMINE THE LENGTH OF THE HYDRAULIC JUMP

Since the Froude Number for the upstream section is greater than 5.5, the number 2 equation should be used to calculate the jump length.

Equation Number 2: Length (L) = 5.75 d_{n2}

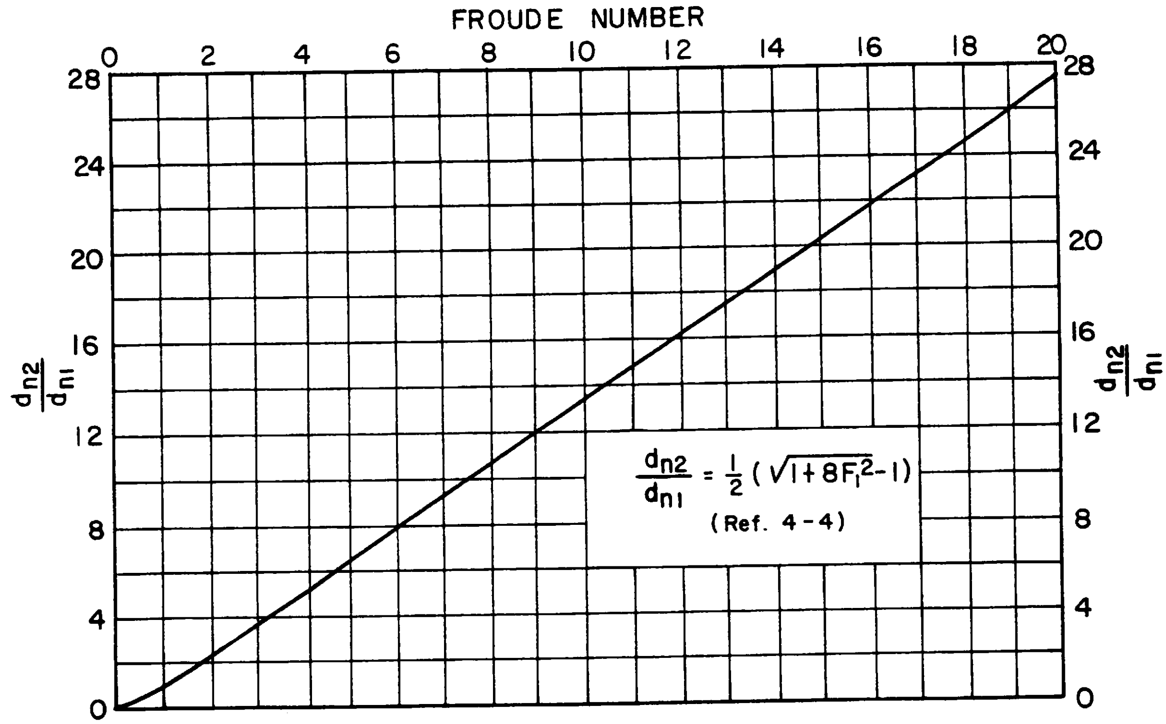
L = 5.75 (3.9 feet)

L = 22.4 feet

Refer to Figure 4-14, page 4-55 for a pictorial sketch of EXAMPLE PROBLEM.

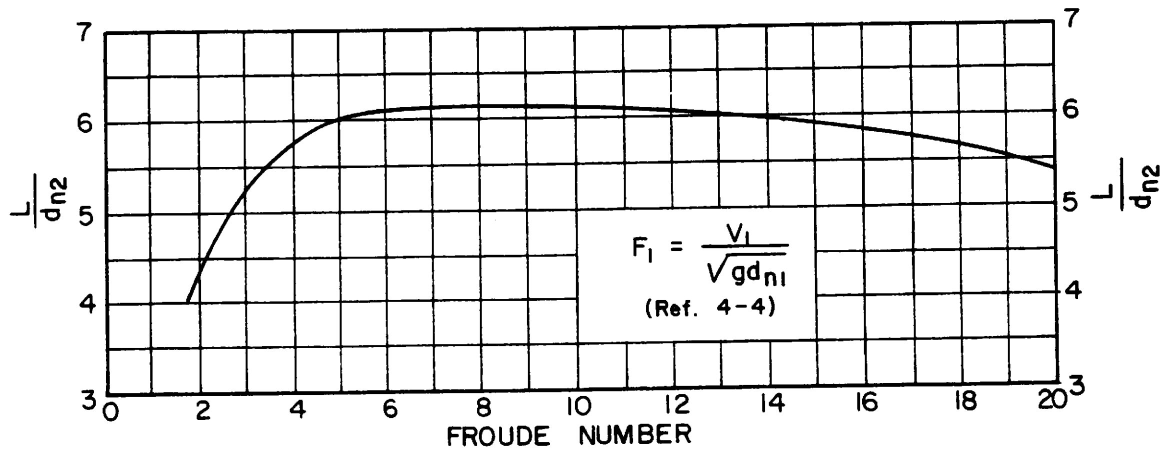
4-1002.2 RIPRAP

Riprap, when used as an outlet velocity control measure, should be applied to the channel area immediately downstream of the culvert outlet for a distance of no less than 20 feet or to the ROW, which ever is less. This arbitrary limit may be tempered by engineering judgement based on the severity of the velocity and the potential for erosion or scour. Most of the various types of riprap have proven effective as erosion an scour preventatives and soil protectors. Also, see Chapter 6, Section 6-900, "Erosion Control Channel Linings".



HYDRAULIC JUMP DETERMINATION
(in a horizontal rectangular channel)

CHART 4-1



LENGTH OF HYDRAULIC JUMP

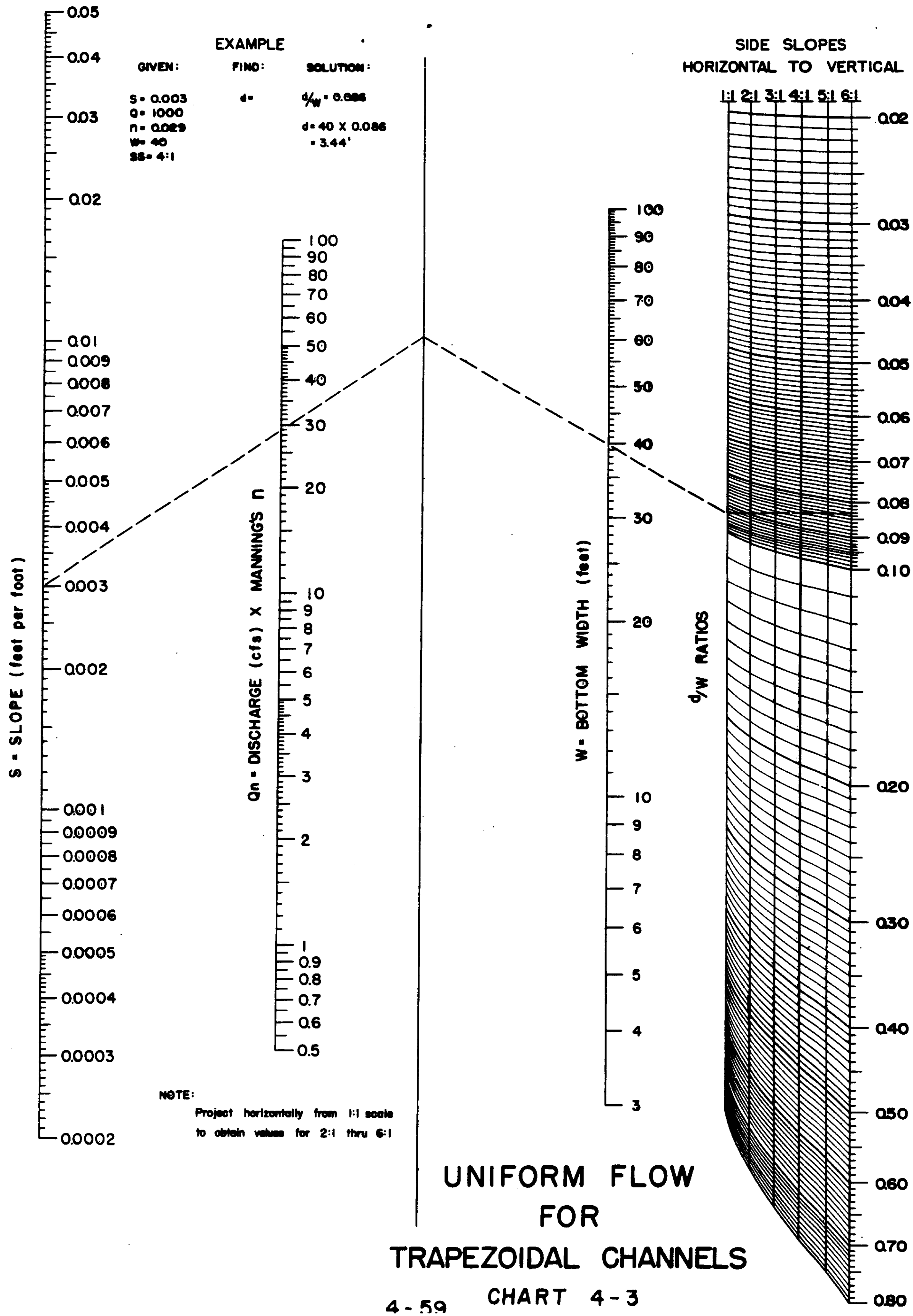
CHART 4-2

TABLE 4-9

MANNING'S n FOR NATURAL STREAM CHANNELS
(Surface width of flood stage less than 100 ft.)

1.	Fairly regular section:	
	a. Some grass and weeds, little or no brush	0.030--0.035
	b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-- 0.05
	c. Some weeds, light brush on banks	0.035-- 0.05
	d. Some weeds, heavy brush on banks	0.05 -- 0.07
	e. Some weeds, dense willows on banks	0.06 -- 0.08
	f. For trees within channel, with branches submerged at high stage; increase all above values by	0.01 -- 0.02
2.	Irregular sections, with pools, slight channel meander; increase values given above	0.01 -- 0.02
3.	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
	a. Bottom of gravel, cobbles, and few boulders . .	0.04 -- 0.05
	b. Bottom of cobbles, with large boulders	0.05 -- 0.07

(After Reference 4-8)



UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

CHART 4-3

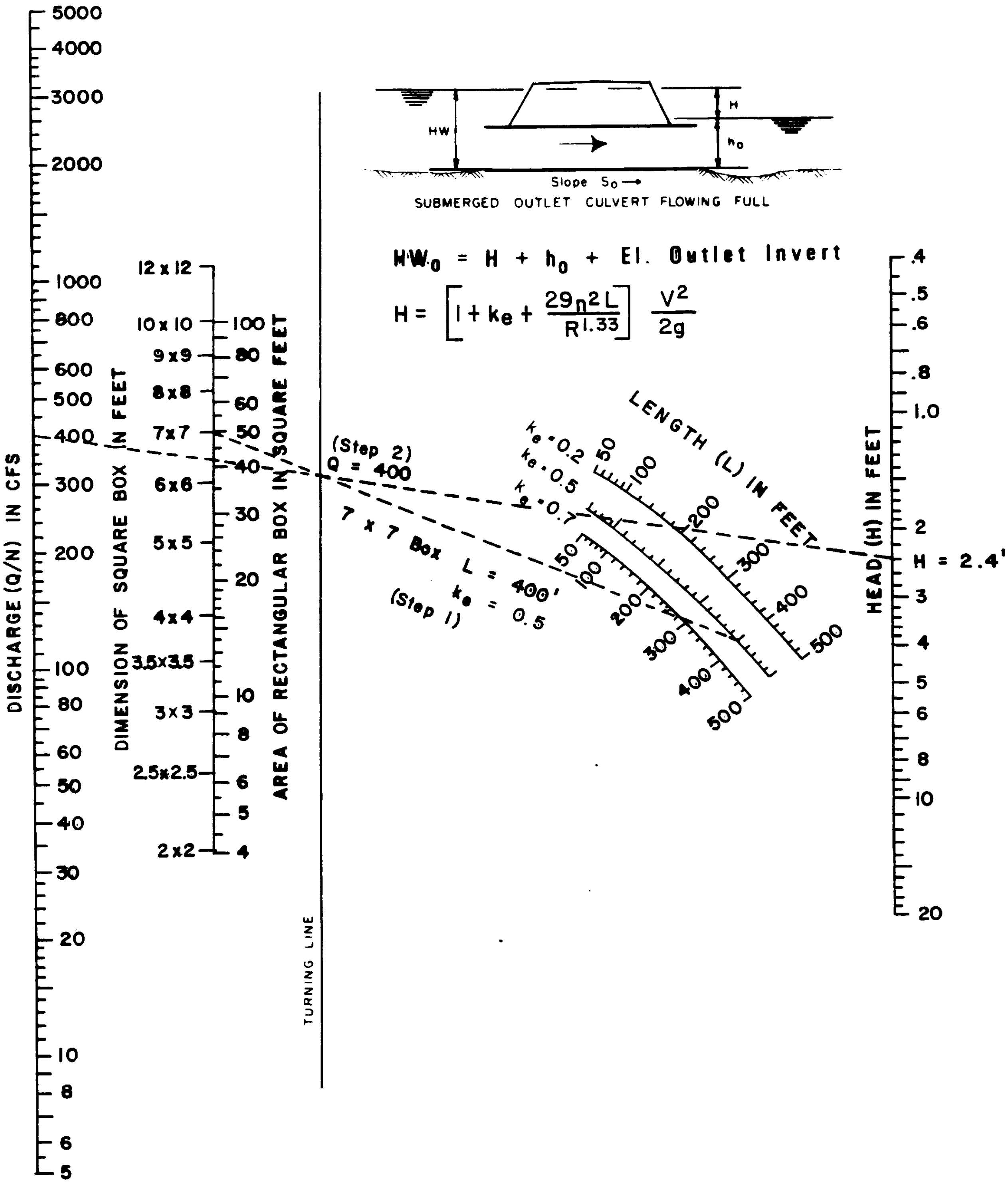
TABLE 4-10

ENTRANCE LOSS COEFFICIENTS
Outlet Control, Full or Partly Full

$$\text{Entrance head loss } H_e = K_e \frac{V^2}{2g}$$

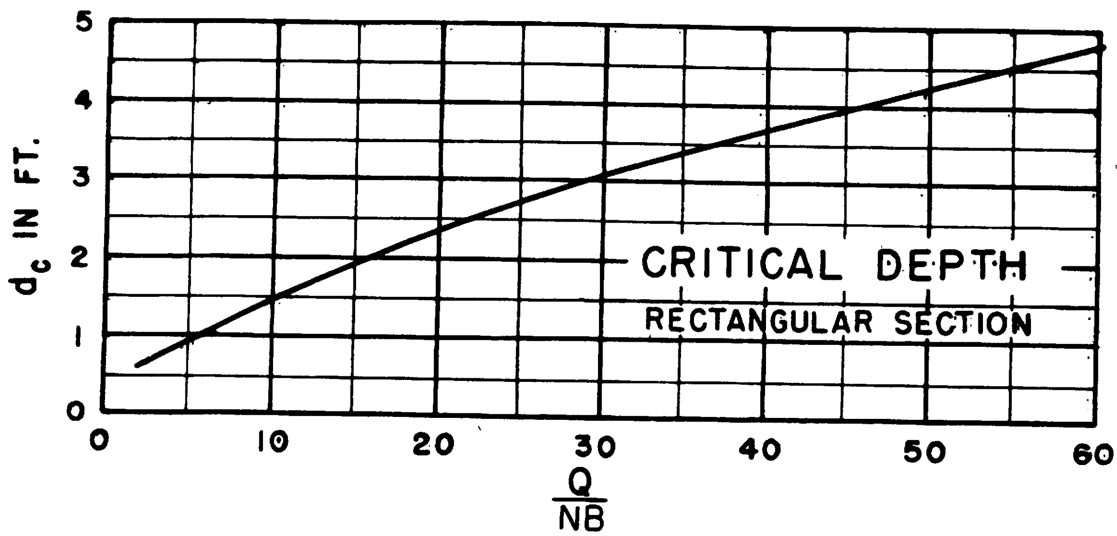
Types of Structure and Design of Entrance	Coefficient K_e
PIPE, CONCRETE	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Round (Radius - 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° to 45° bevels	0.2
Side or slope-tapered inlet	0.2
PIPE, OR PIPE-ARCH, CORRUGATED METAL	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, pave or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope-tapered inlet	0.2
BOX, REINFORCED CONCRETE	
Headwall parallel embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edges at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side or slope-tapered inlet	0.2

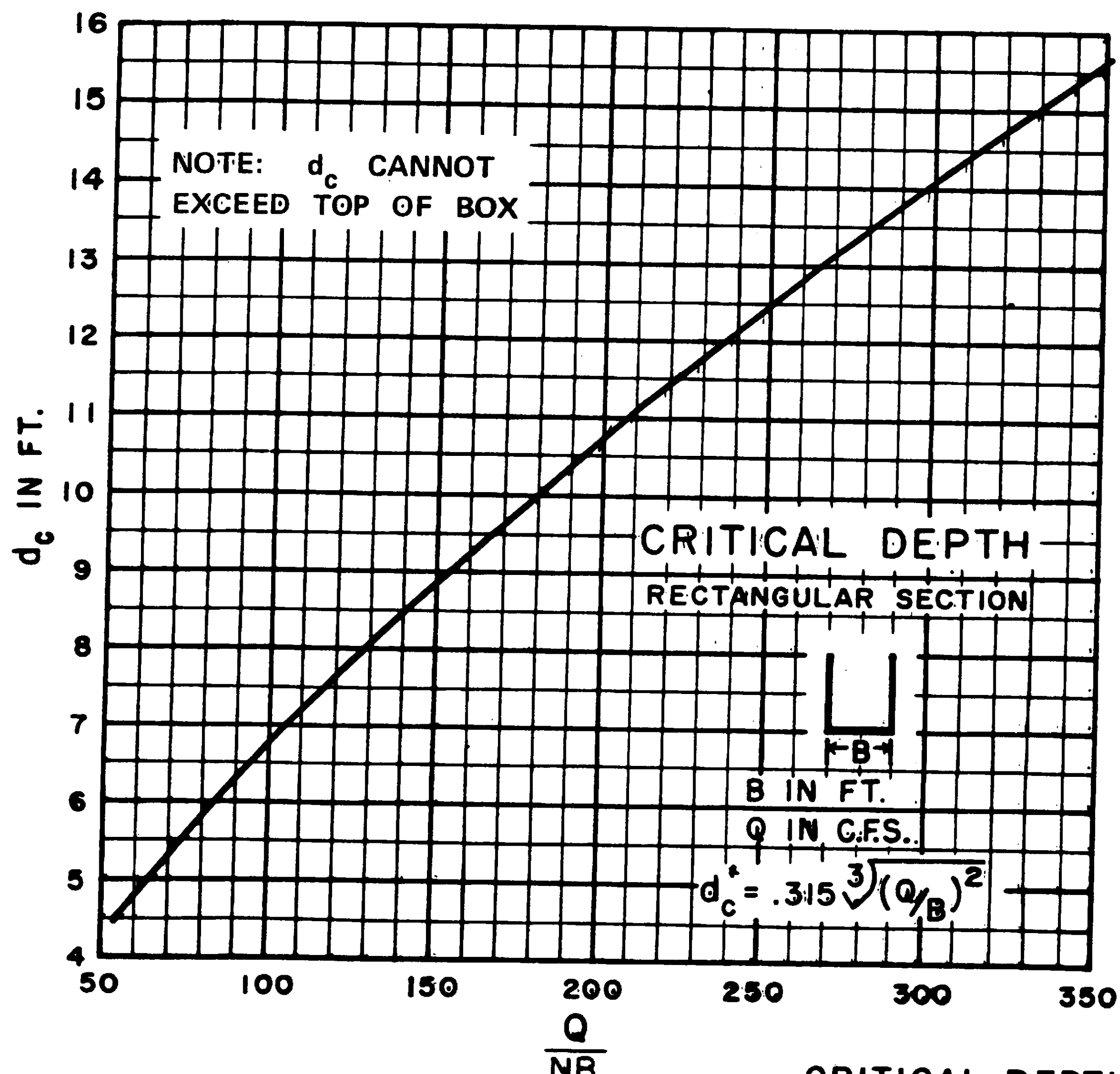
(After Reference 4-8)



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

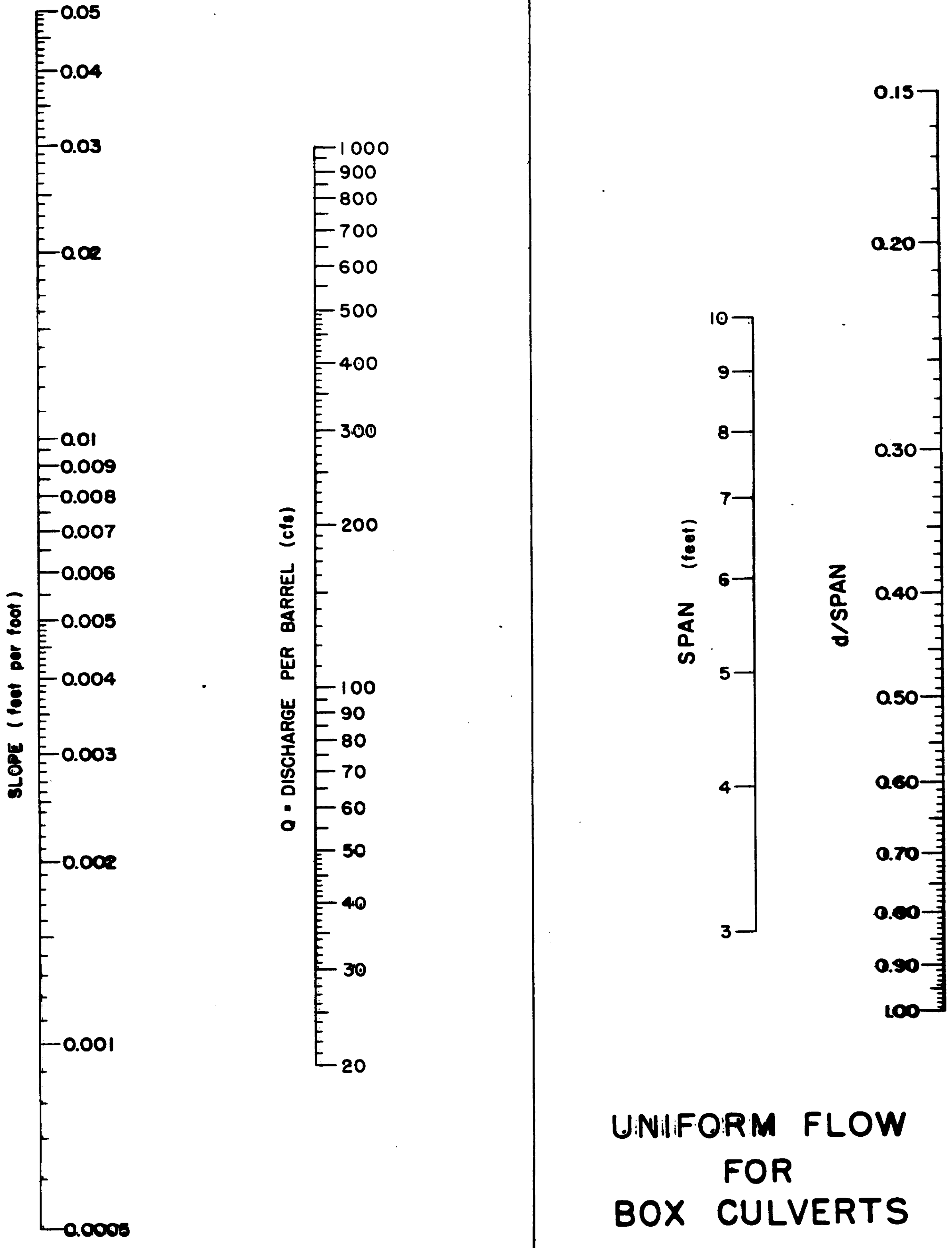
CHART 4-4





CRITICAL DEPTH
RECTANGULAR SECTION

CHART 4-5

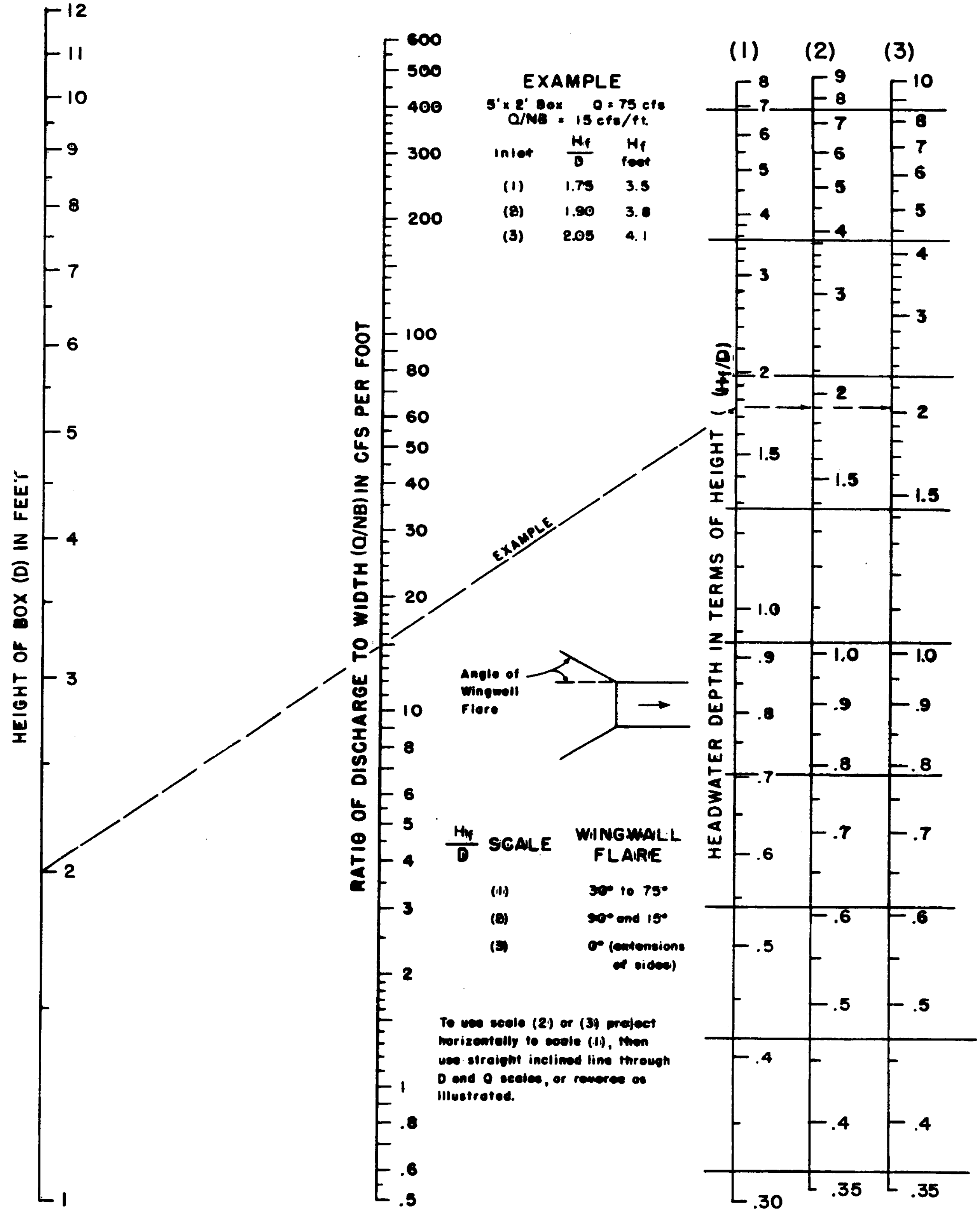


3 Sides Wetted

UNIFORM FLOW FOR BOX CULVERTS

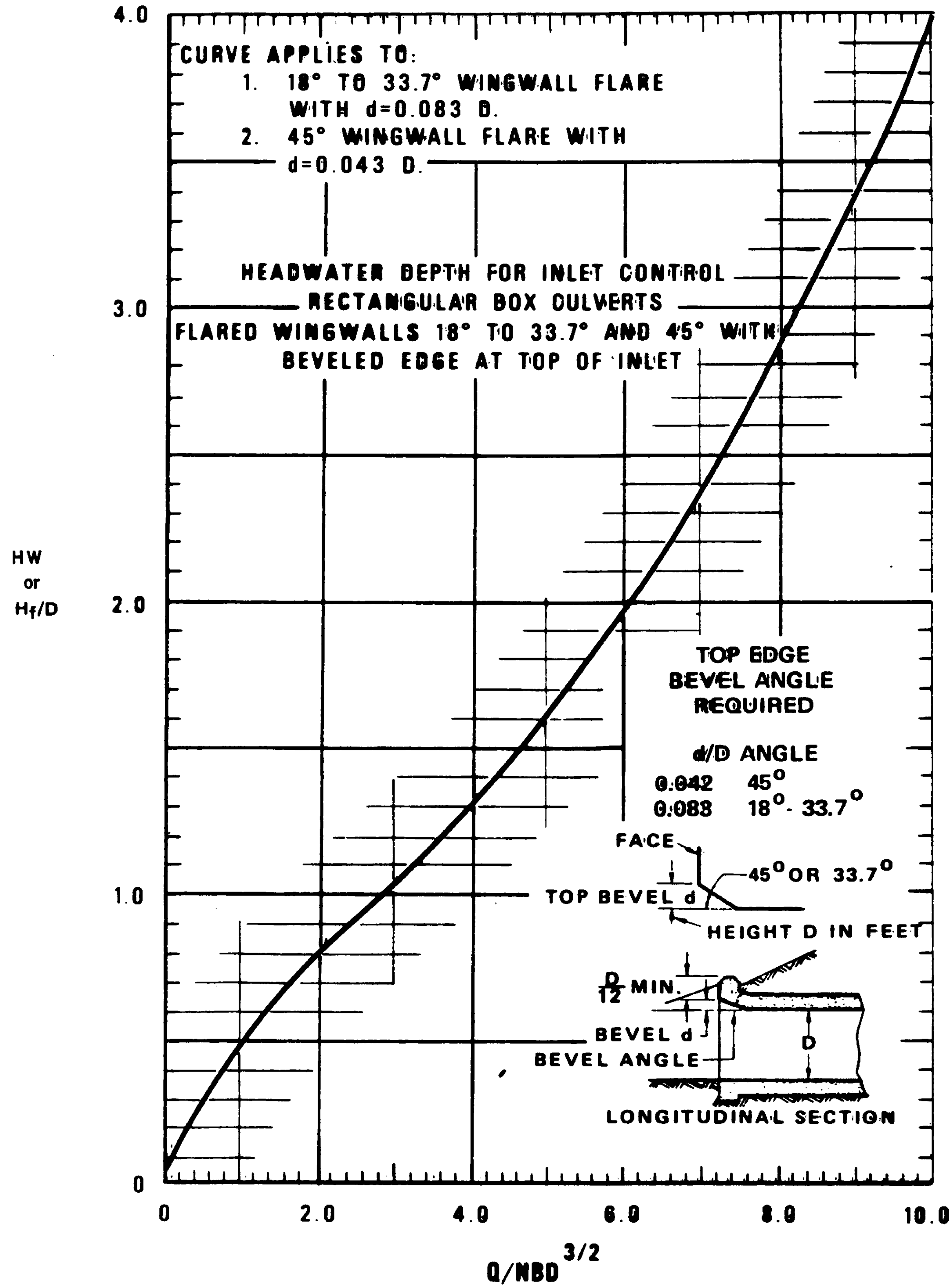
$n = 0.012$

CHART 4-6



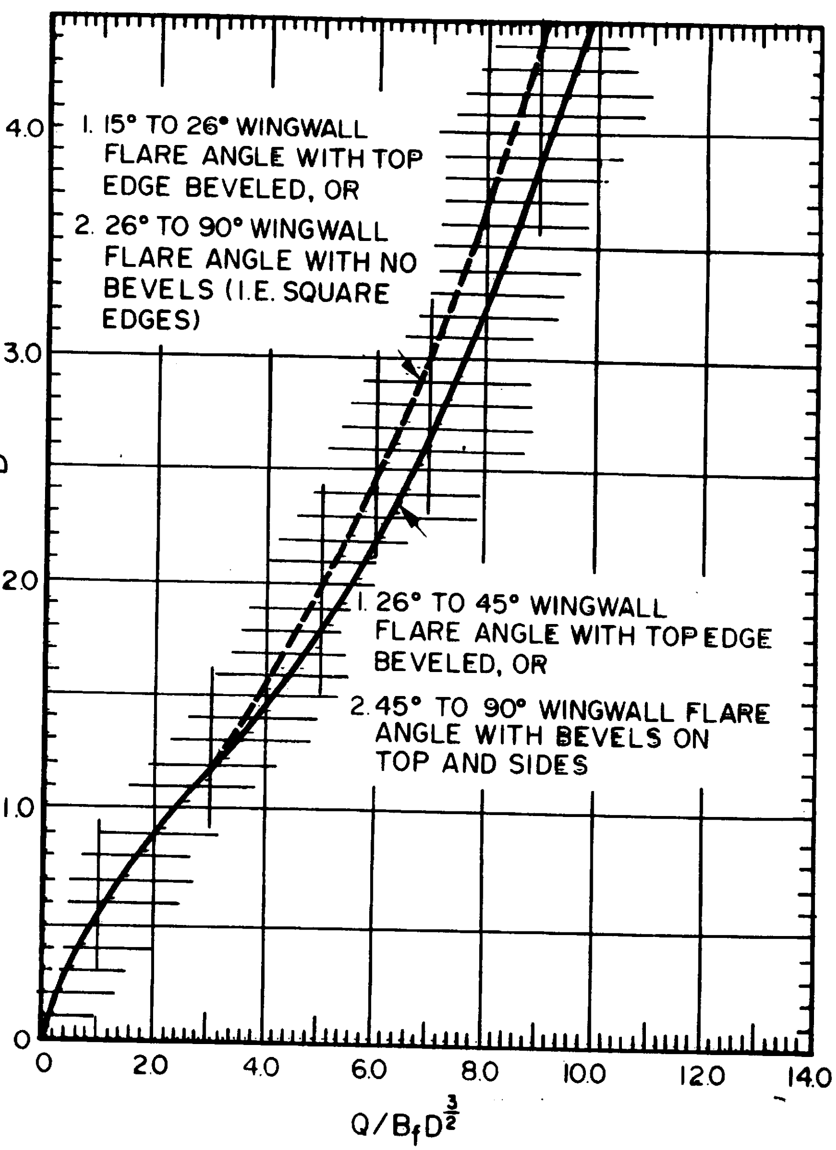
HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

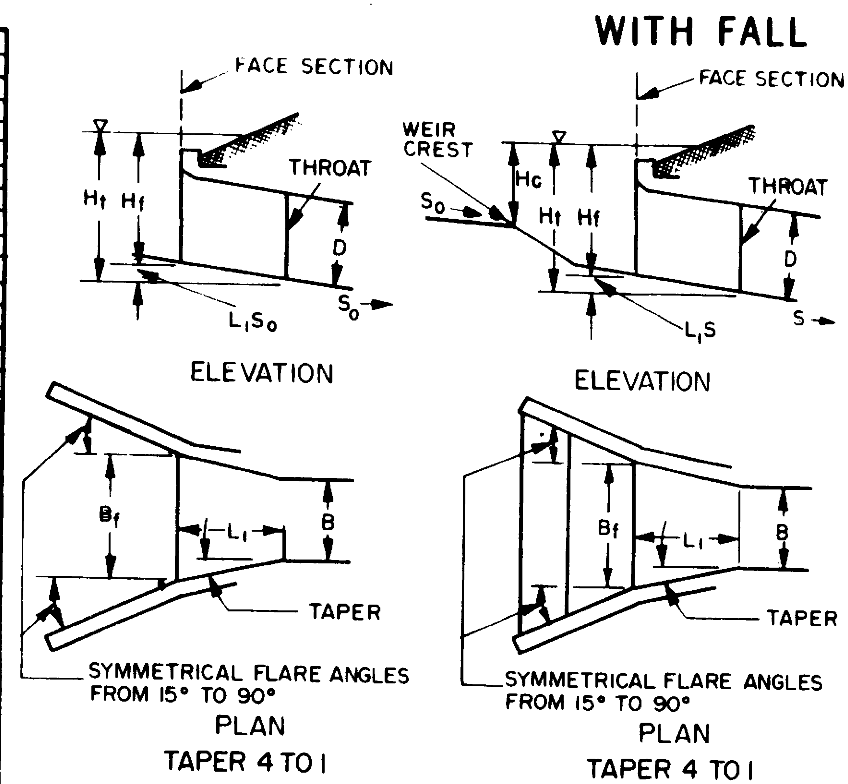
CHART 4-7



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° AND 45°
WITH BEVELED EDGE AT TOP OF INLET

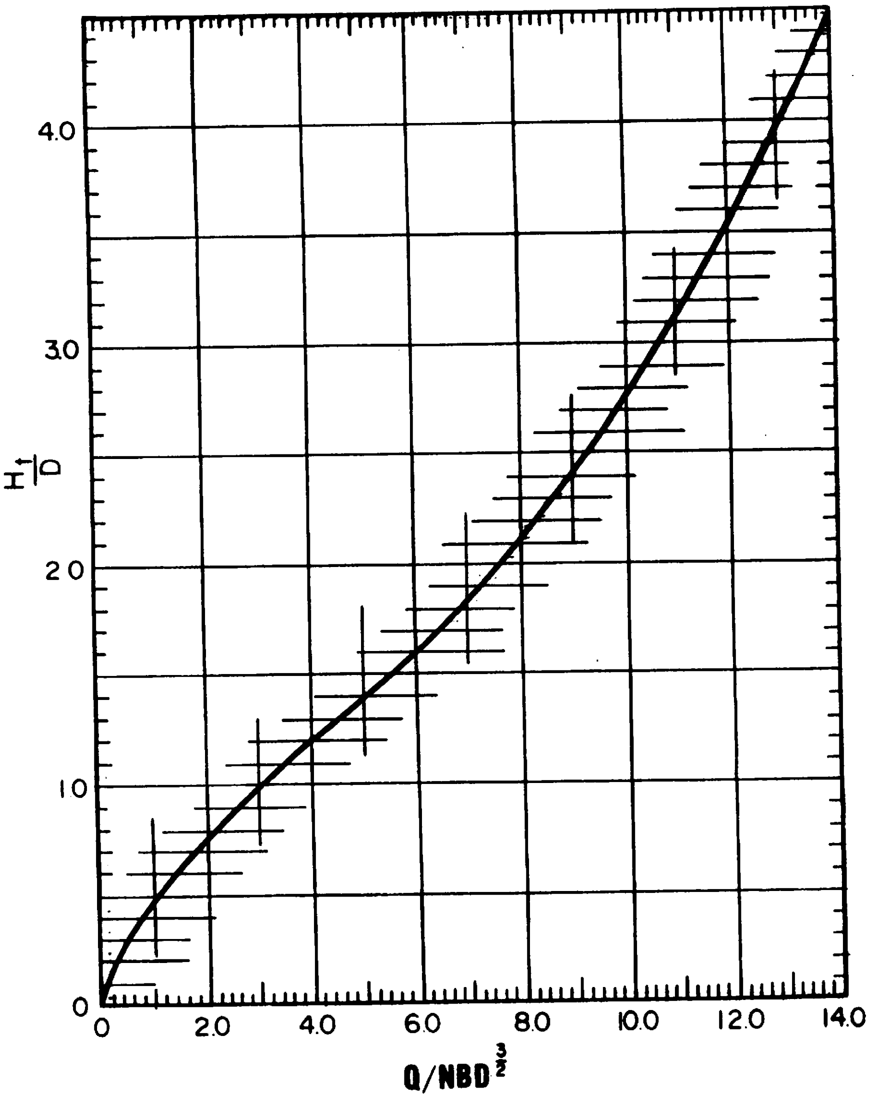
CHART 4-8

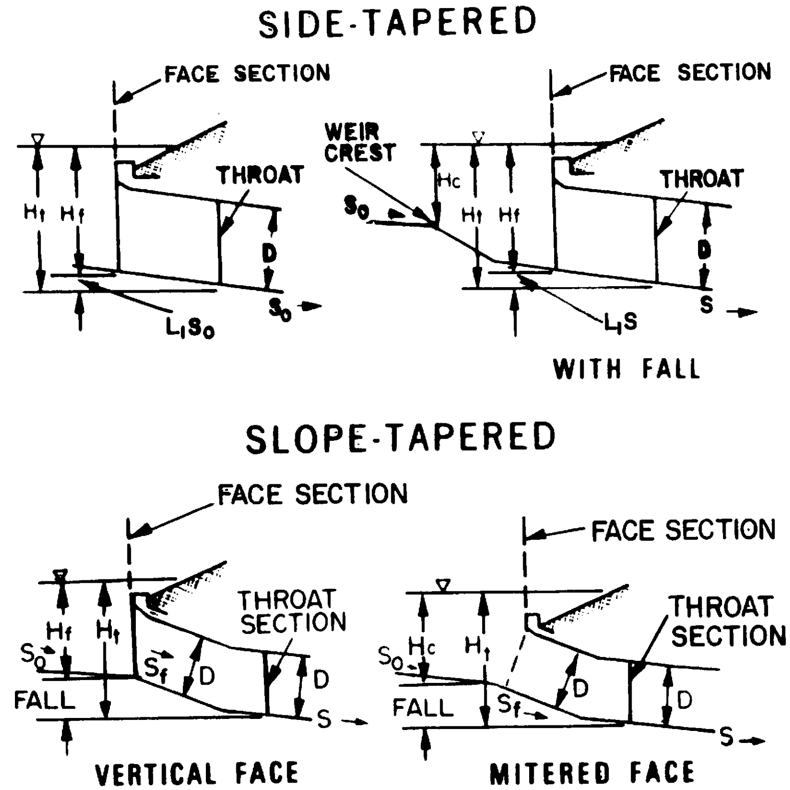




FACE CONTROL CURVES FOR BOX CULVERTS SIDE-TAPERED INLETS

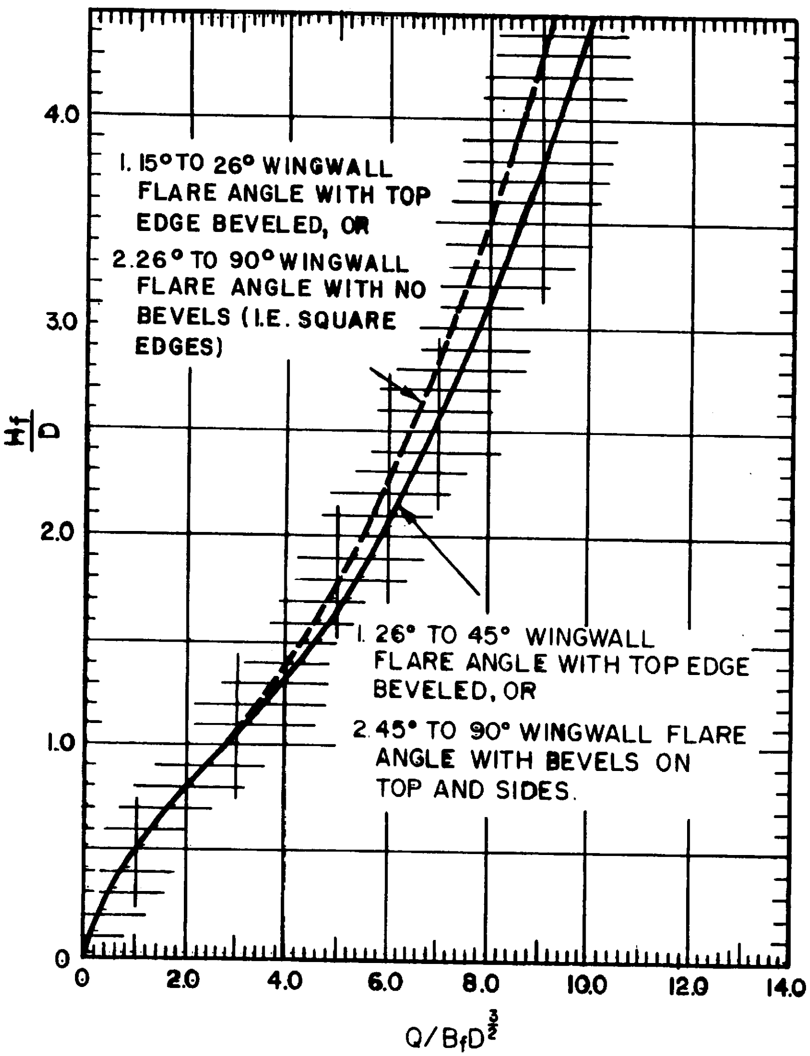
CHART 4-9

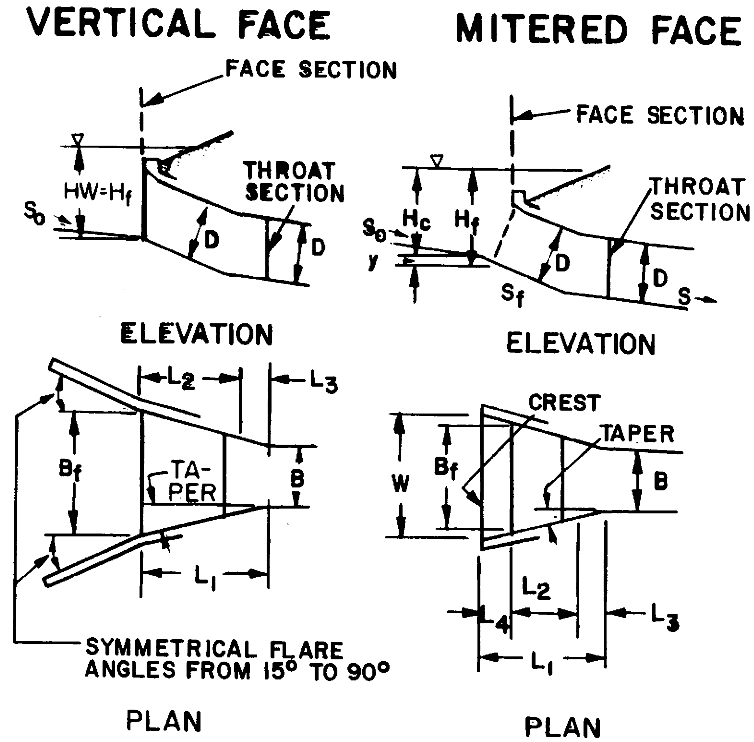




THROAT CONTROL CURVE
FOR
BOX CULVERTS
TAPERED INLETS

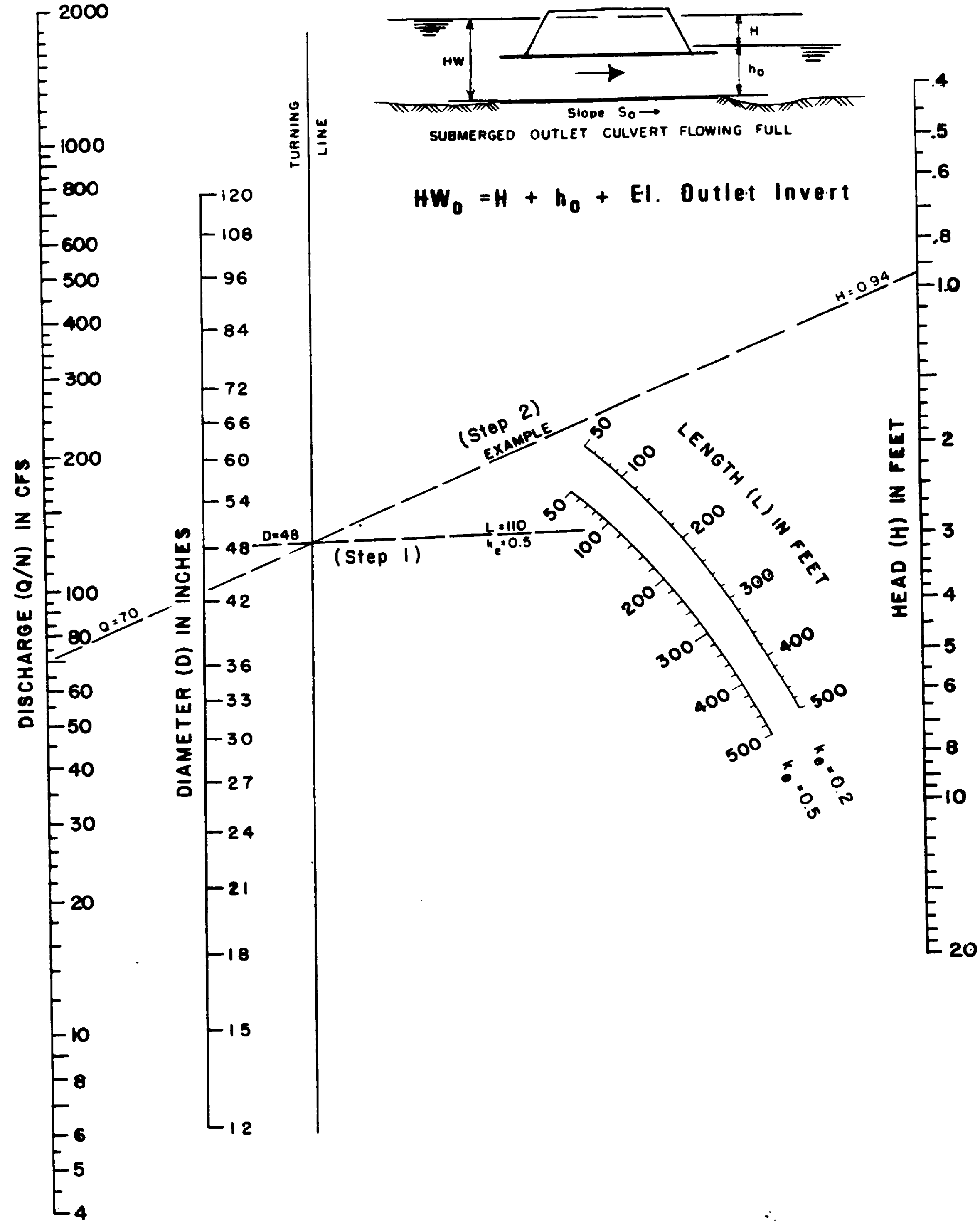
CHART 4 - 10





FACE CONTROL CURVES FOR BOX CULVERTS SLOPE TAPERED INLETS

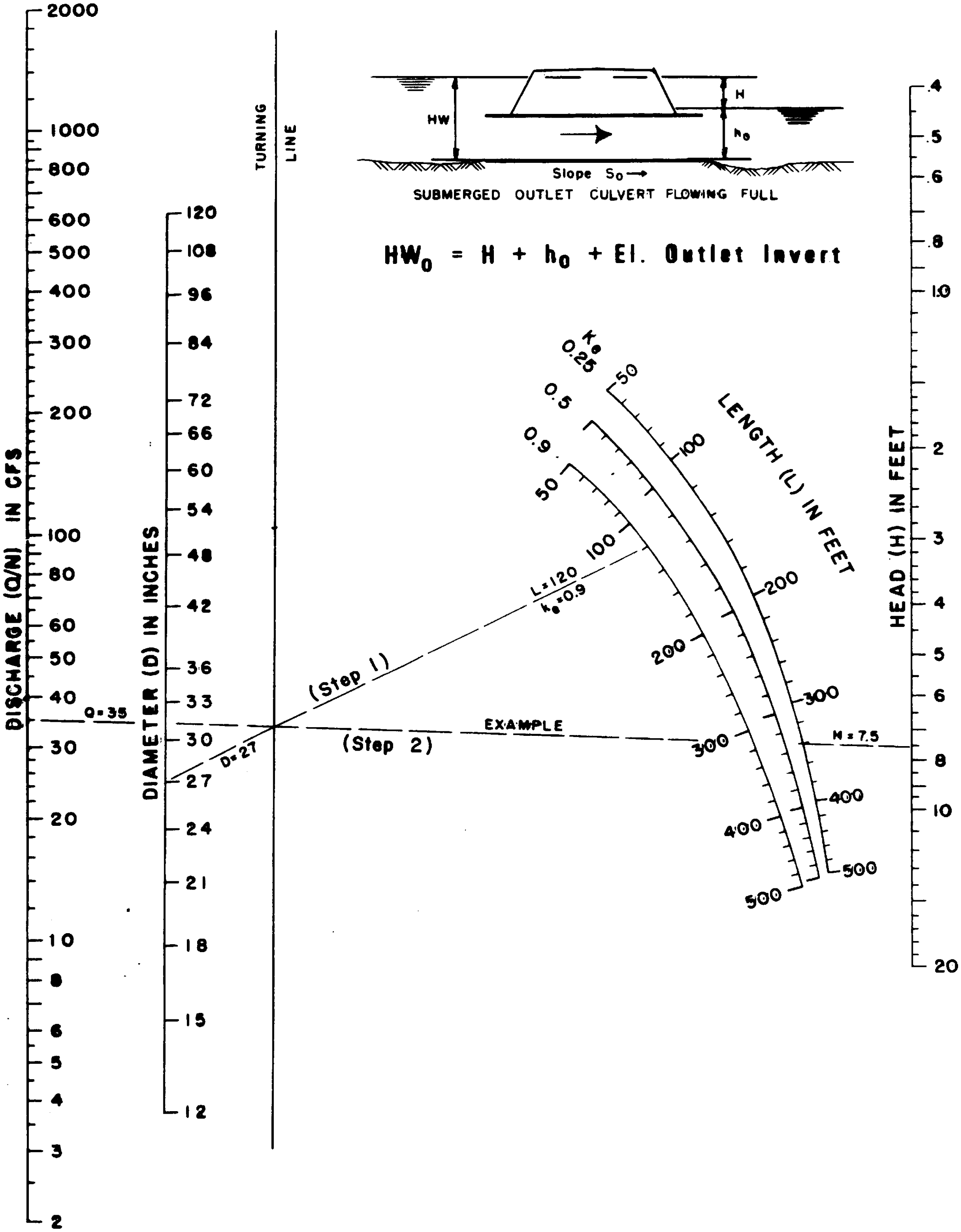
CHART 4-11



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL

n = 0.012

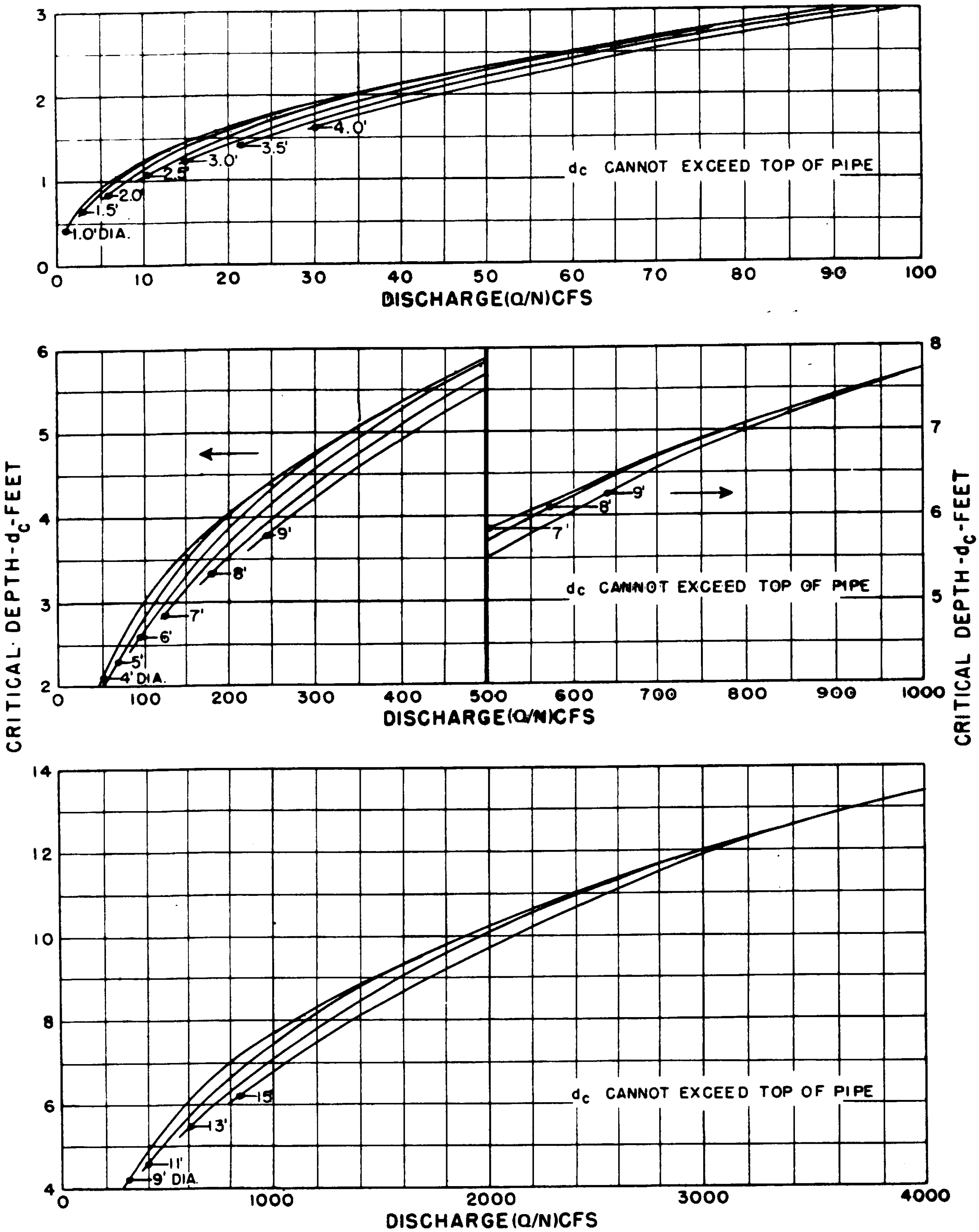
CHART 4-12



HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL

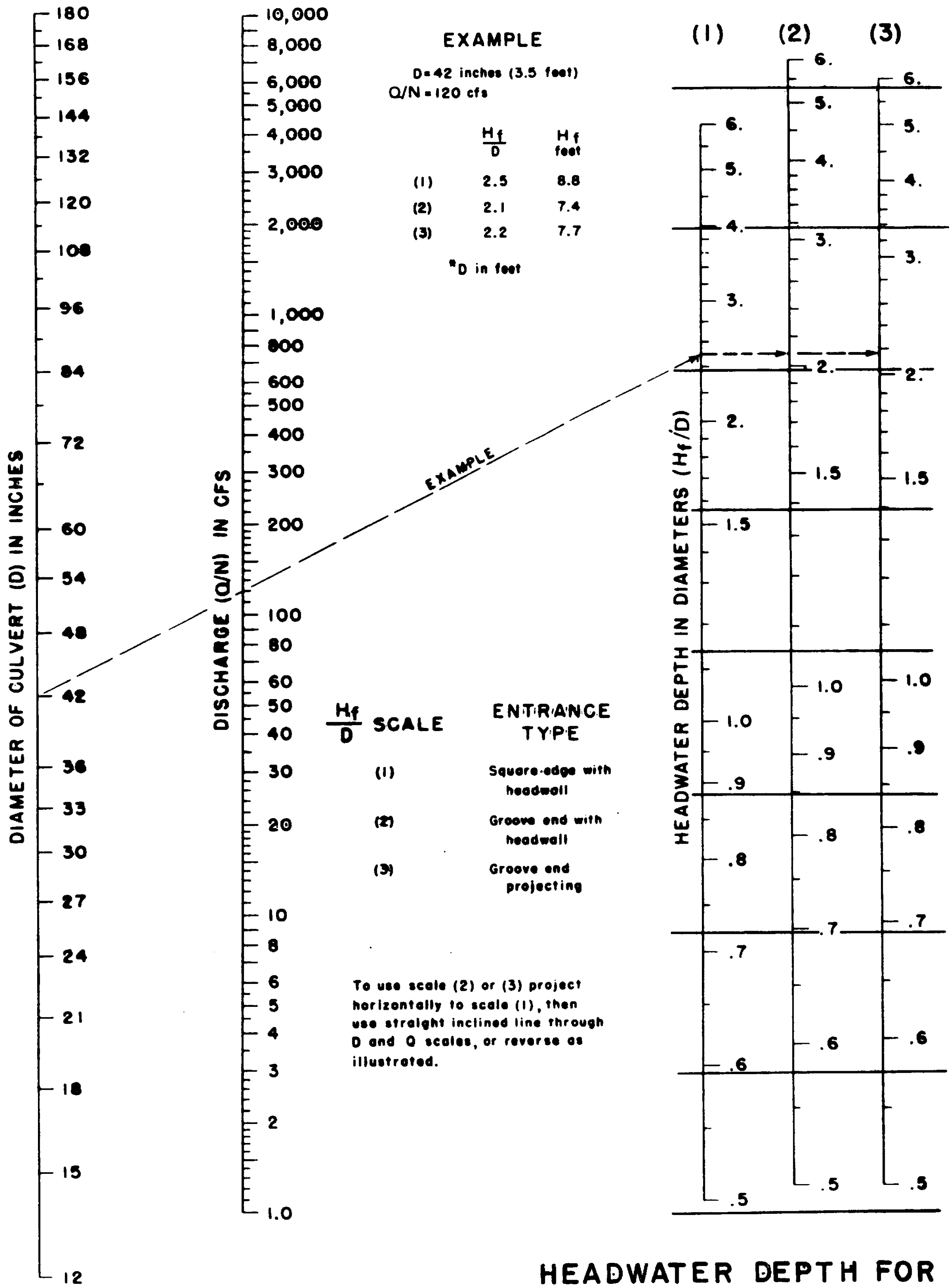
n = 0.024

CHART 4-13



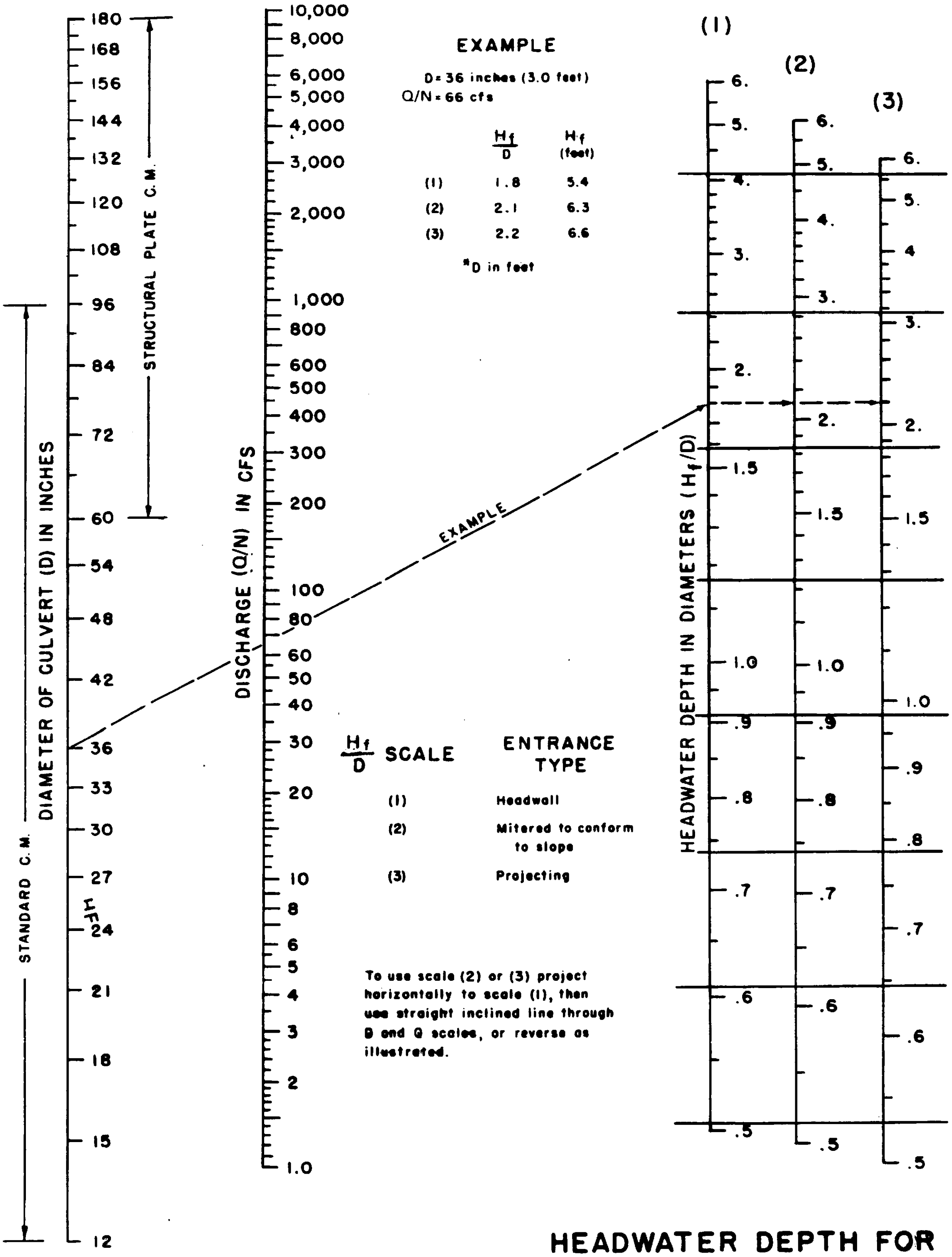
CRITICAL DEPTH
CIRCULAR PIPE

CHART 4-14



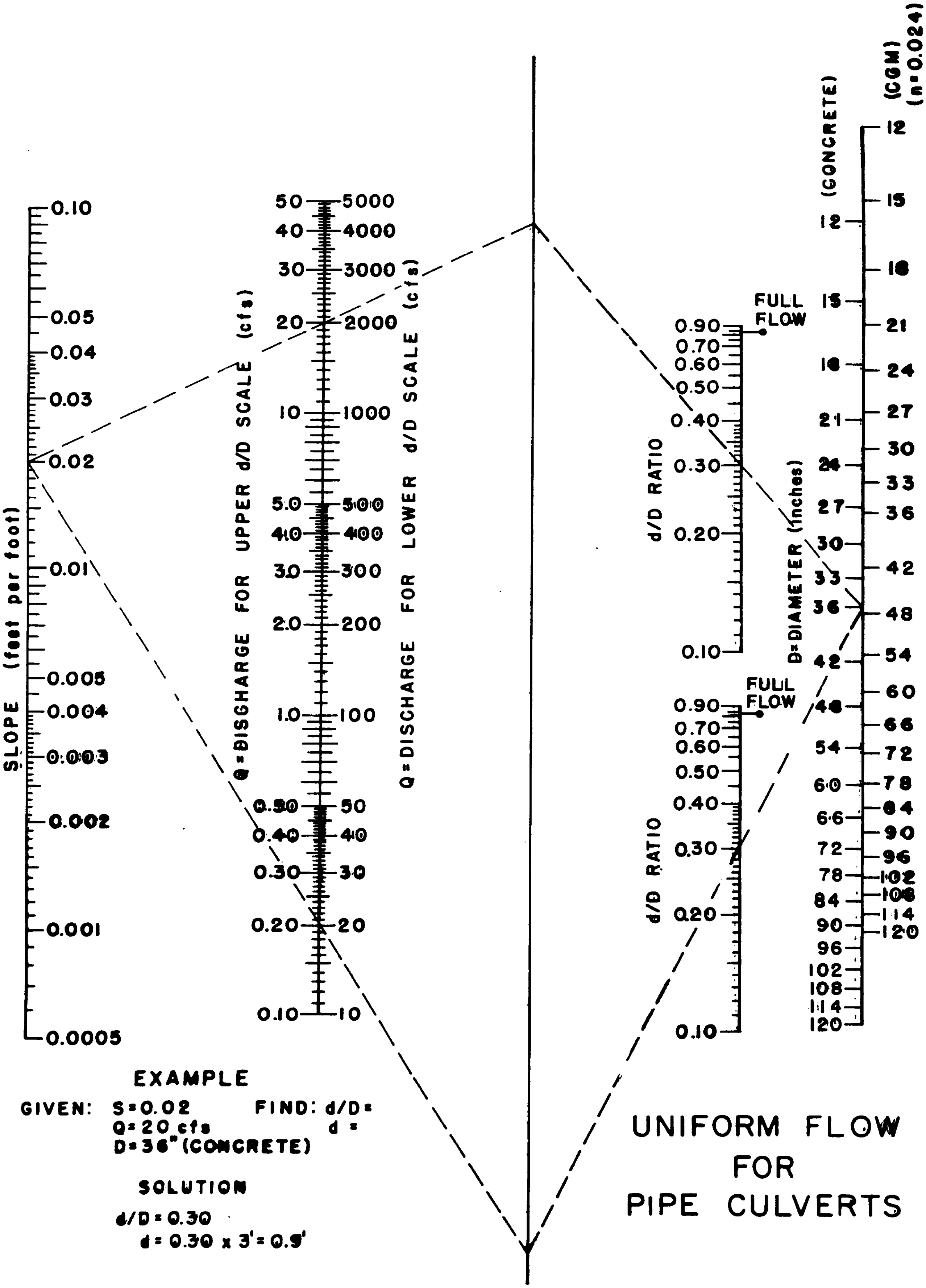
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART 4-15



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

CHART 4-16



EXAMPLE

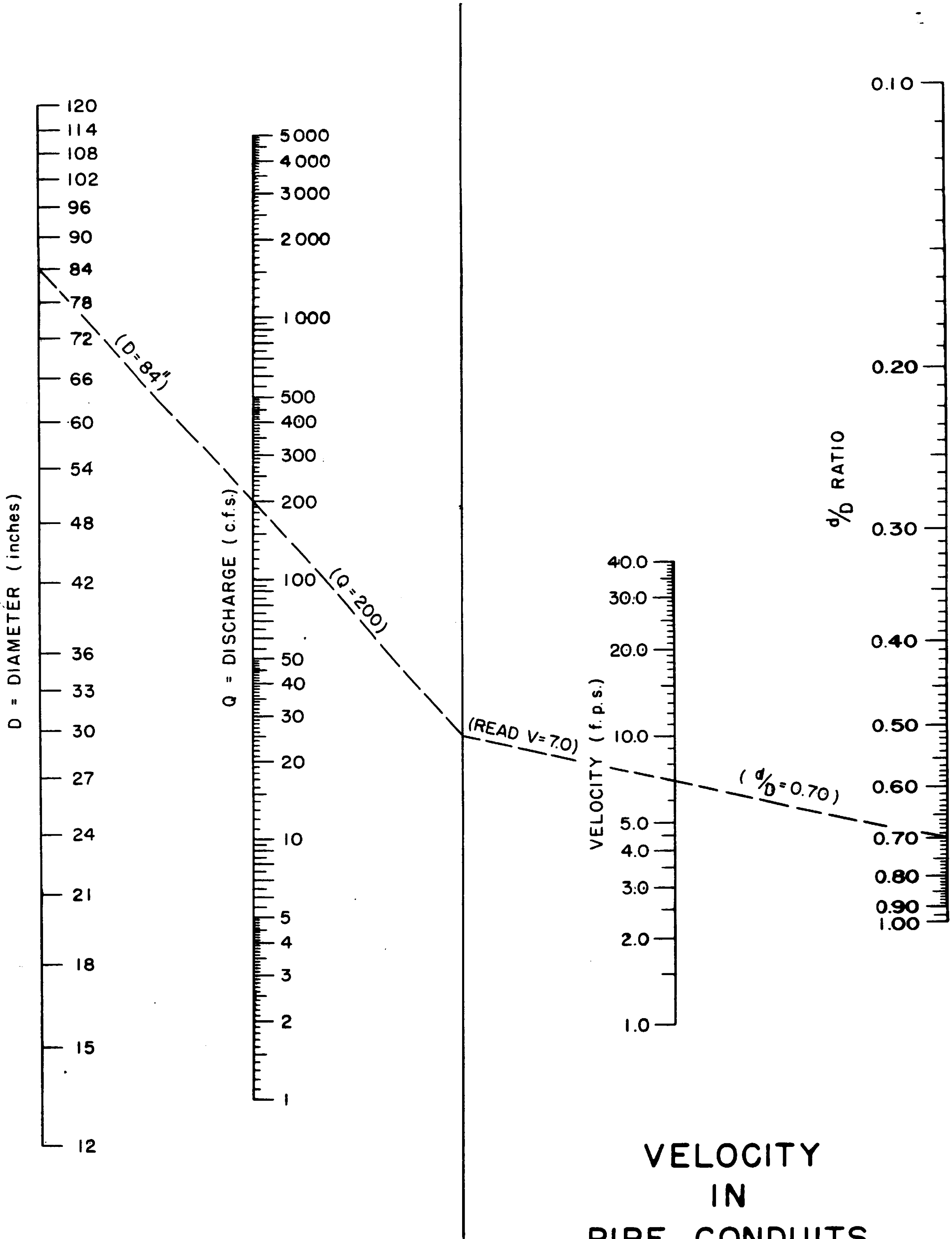
GIVEN: S=0.02 FIND: d/D=
Q=20 cfs d =
D=36" (CONCRETE)

SOLUTION

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

UNIFORM FLOW FOR PIPE CULVERTS

CHART 4-17



VELOCITY IN PIPE CONDUITS

BASED ON
Q = VA

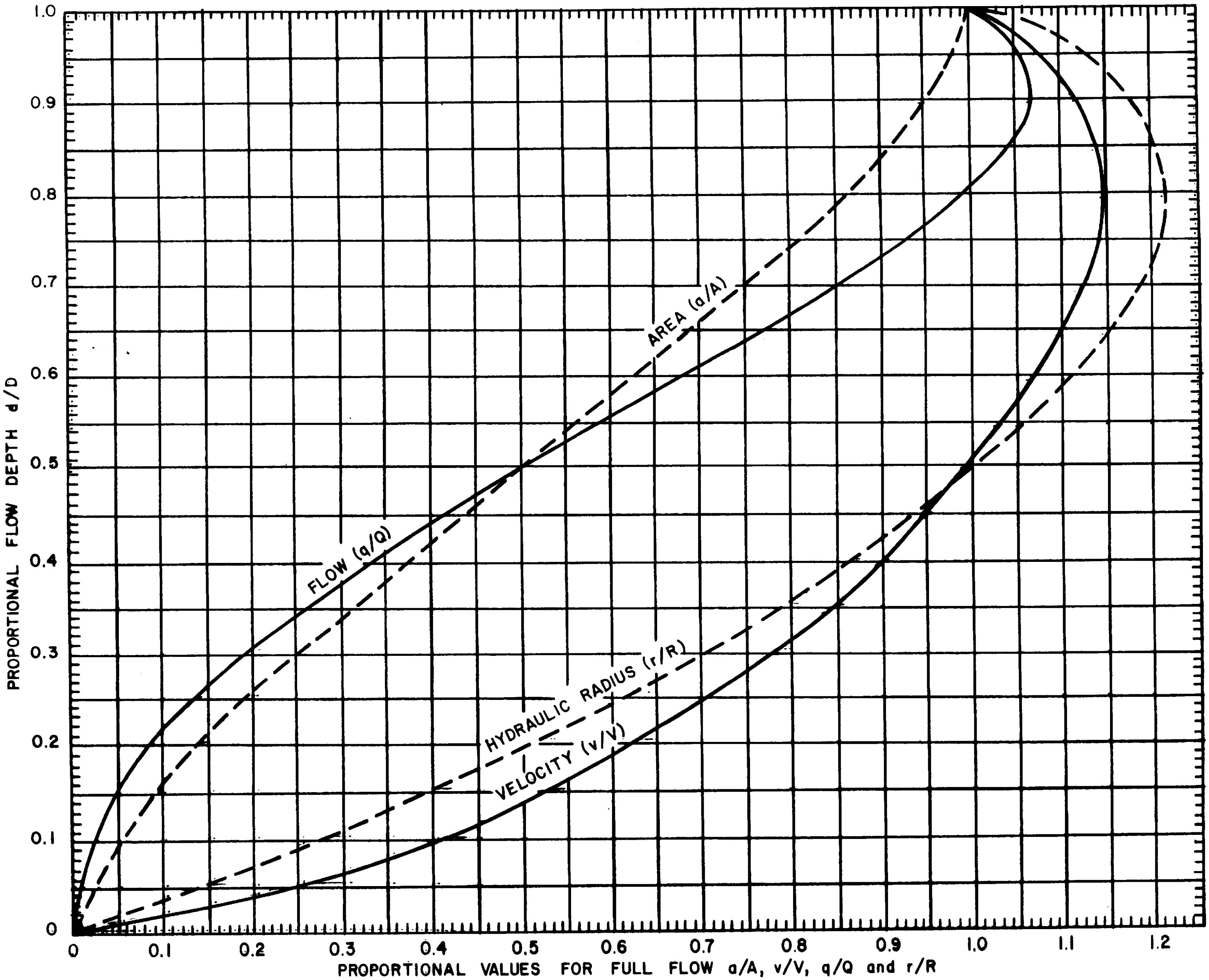
CHART 4-18

TABLE FOR SOLVING MANNING'S FORMULA FOR CONDUITS FLOWING FULL

Diameter, Inches	Area in Sq. Ft.	Hydraulic Radius	$R^{2/3}$	$AR^{2/3}$	Values of $\frac{1.486 \times R^{2/3} \times A}{n}$								
					n = .010	n = .011	n = .012	n = .013	n = .021	n = .024	n = .025	n = .030	n = .032
6	.196	.125	.250	.049	7.281	6.619	6.068	5.601	3.467	3.034	2.913	2.427	2.275
8	.349	.167	.303	.106	15.75	14.32	13.13	12.12	7.501	6.563	6.301	5.25	4.92
10	.545	.208	.351	.191	28.38	25.80	23.65	21.83	13.52	11.83	11.35	9.46	8.84
12	.785	.250	.397	.312	46.36	42.15	38.64	35.66	22.08	19.32	18.55	15.45	14.49
15	1.227	.3125	.461	.566	84.11	76.46	70.09	64.70	40.05	35.04	33.64	28.04	26.28
18	1.767	.375	.520	.919	136.6	124.2	113.8	105.1	65.03	56.90	54.63	45.53	42.69
21	2.405	.437	.576	1.385	205.8	187.1	171.5	158.3	98.01	85.75	82.32	68.60	64.31
24	3.142	.50	.630	1.979	294.1	267.4	245.1	226.2	140.0	122.5	117.6	98.03	91.91
27	3.976	.5625	.681	2.708	402.4	365.8	335.3	309.6	191.6	167.7	161.0	134.1	125.7
30	4.909	.625	.731	3.588	533.2	484.7	444.3	410.1	253.9	222.2	213.3	177.7	166.6
36	7.069	.75	.825	5.832	866.6	787.9	722.2	666.6	412.7	361.1	346.7	288.9	270.8
42	9.621	.875	.915	8.803	1308	1189	1090	1006	622.9	545.0	523.3	436.0	408.8
48	12.566	1.00	1.00	12.566	1867	1698	1556	1436	889.2	778.0	746.9	622.3	583.4
54	15.904	1.125	1.082	17.208	2557	2325	2131	1967	1218	1065	1023	852.3	799.1
60	19.635	1.25	1.16	22.777	3385	3077	2821	2604	1612	1410	1354	1128	1058
66	23.758	1.375	1.236	29.365	4364	3967	3636	3357	2078	1818	1745	1455	1364
72	28.274	1.50	1.310	37.039	5504	5004	4587	4234	2621	2293	2202	1835	1720
78	33.183	1.625	1.382	45.859	6815	6195	5679	5242	3245	2839	2726	2272	2130
84	38.485	1.75	1.452	55.880	8304	7549	6920	6388	3954	3460	3322	2768	2595
90	44.179	1.875	1.521	67.196	9985	9078	8321	7681	4755	4161	3994	3328	3120
96	50.266	2.00	1.587	79.772	11854	10776	9878	9119	5645	4939	4742	3951	3704
102	56.745	2.125	1.653	93.799	13939	12671	11615	10722	6637	5808	5575	4646	4356
108	63.617	2.25	1.717	109.230	16232	14756	13526	12486	7729	6763	6493	5411	5073
114	70.882	2.375	1.780	126.170	18749	17044	15624	14422	8928	7812	7500	6250	5859
120	78.54	2.5	1.842	144.671	21498	19544	17915	16537	10237	8957	8599	7166	6718

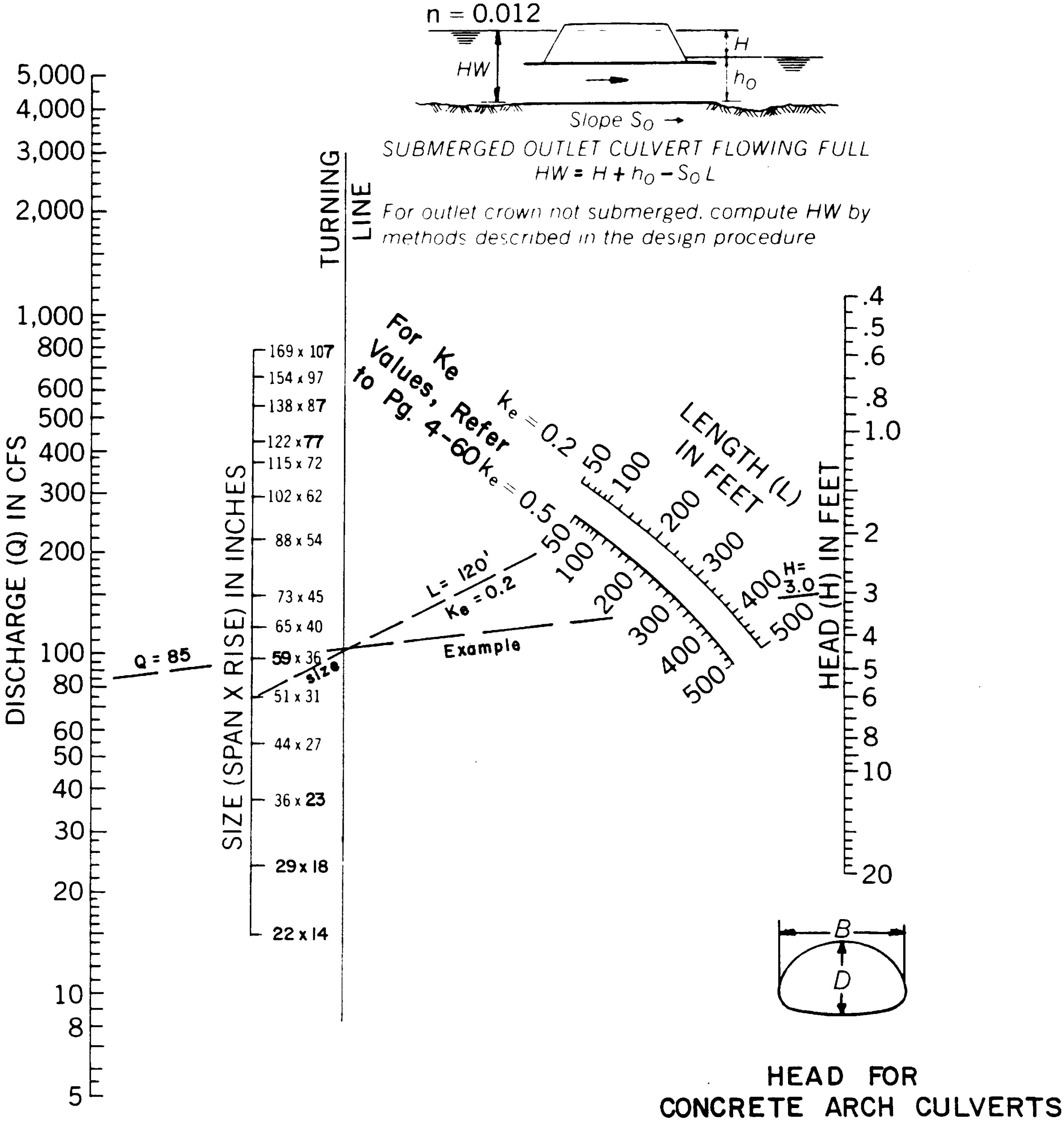
MANNING'S FORMULA: $Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$

TABLE 4-11



RELATIVE VELOCITY, AREA, AND DISCHARGE IN A CIRCULAR PIPE FOR ANY DEPTH OF FLOW

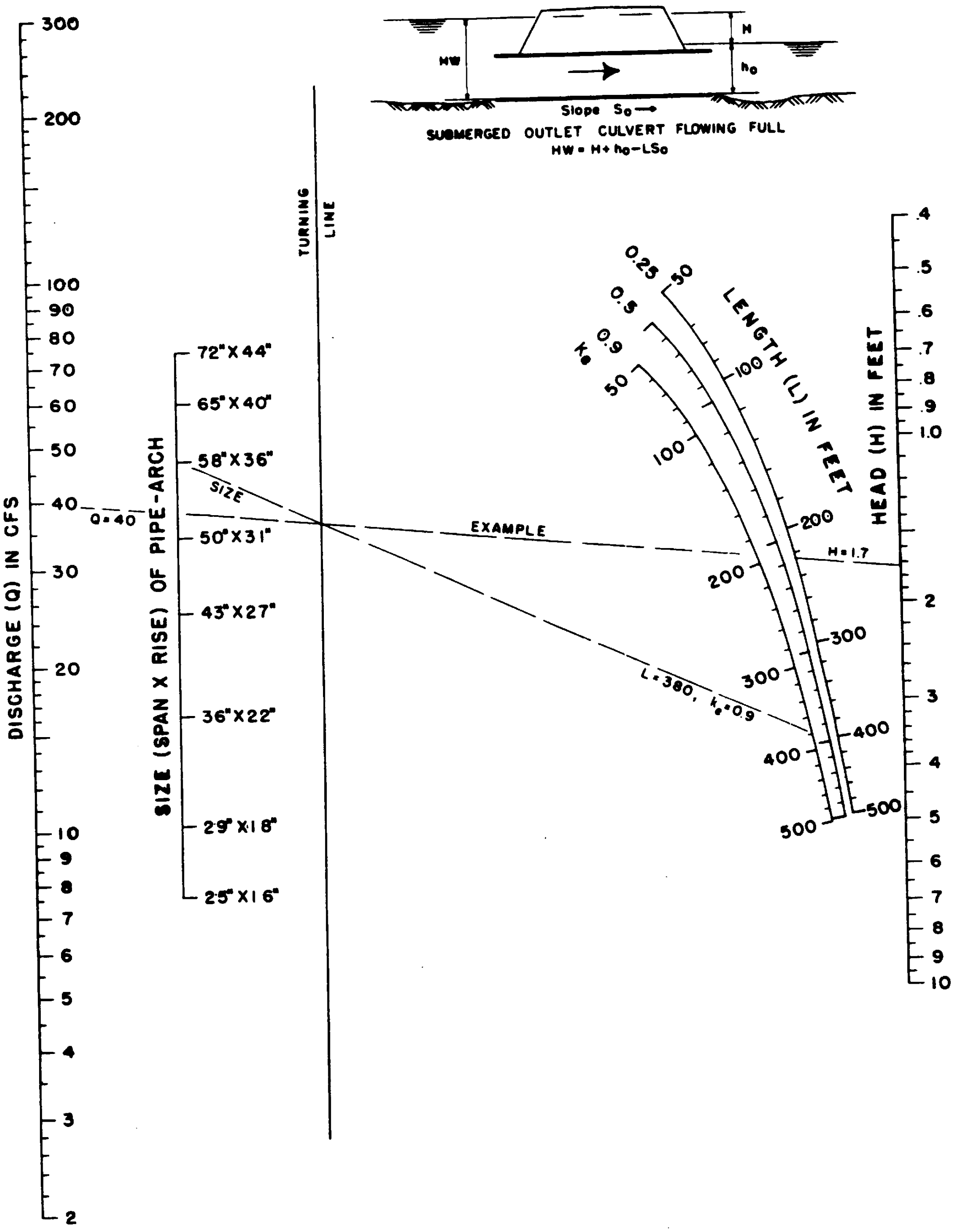
CHART 4-19



From Reference 4-12

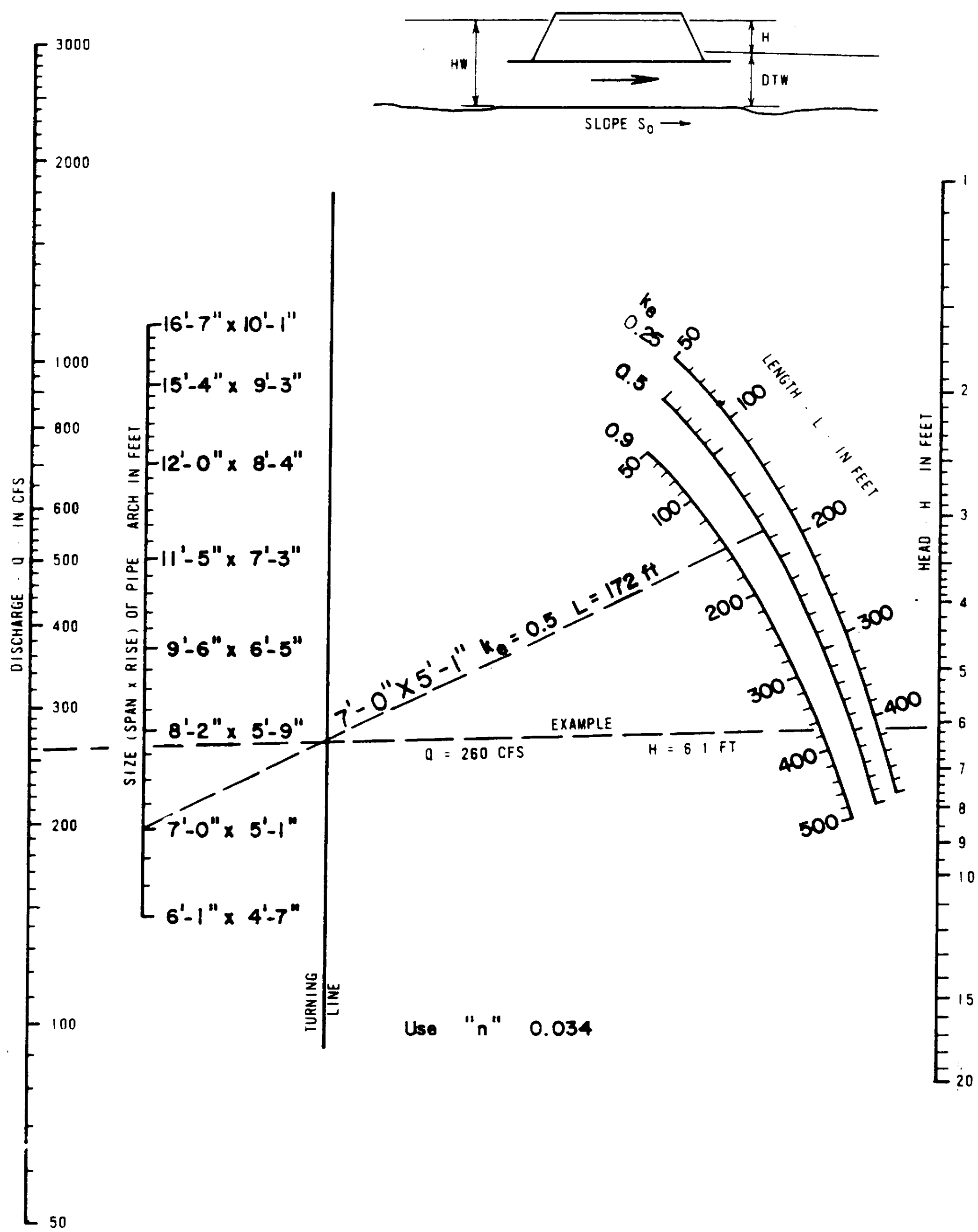
CHART 4-19.1

HEAD FOR CONCRETE ARCH CULVERTS FLOWING FULL n = 0.012



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

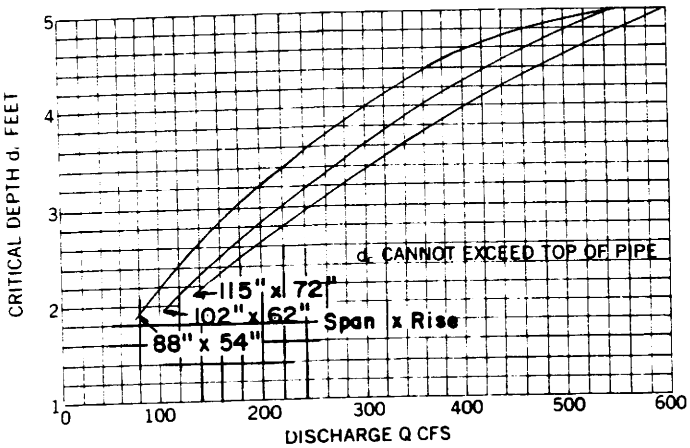
CHART 4-20

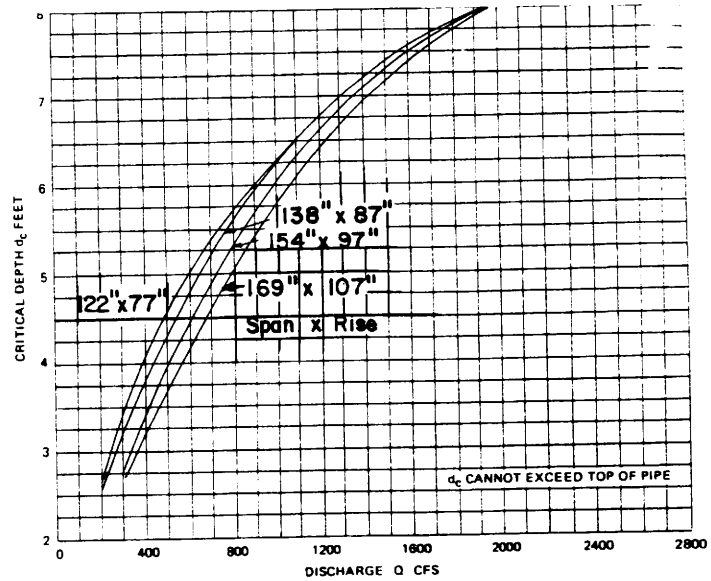


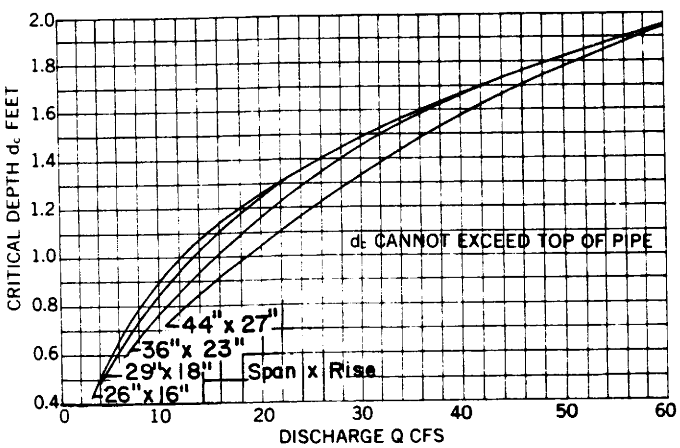
See Standard Drawing "Multi-Plate Culvert, SPA-1" for square feet of waterway opening.

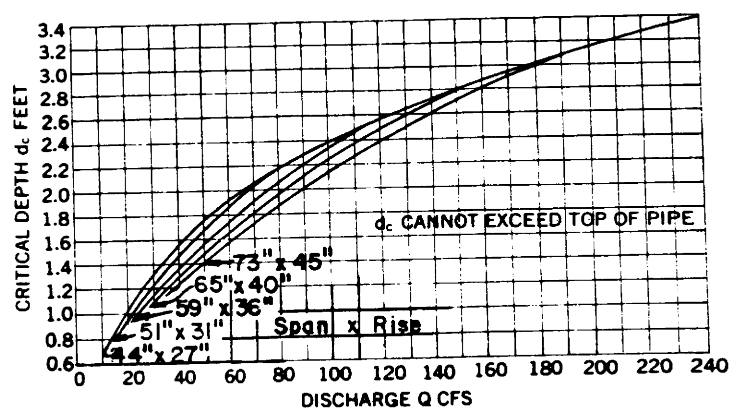
CHART 4-20.1

HEAD FOR
STRUCTURAL PLATE C.M.P. ARCH





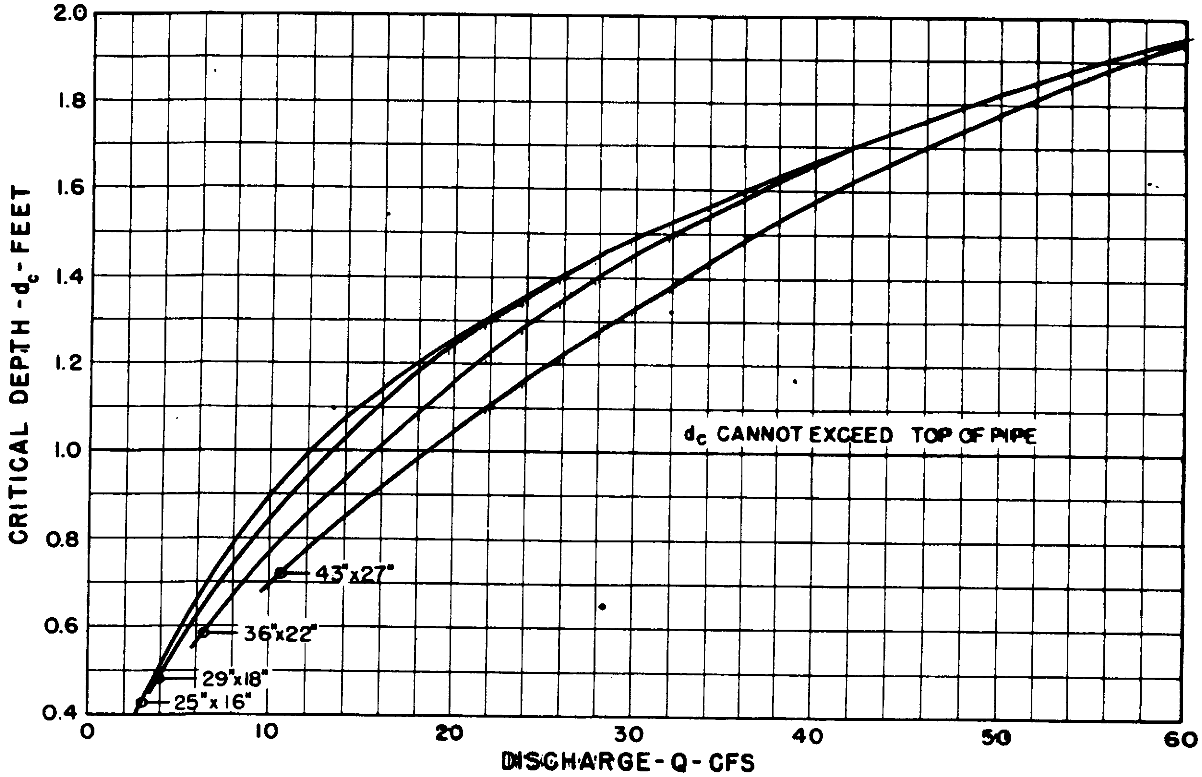


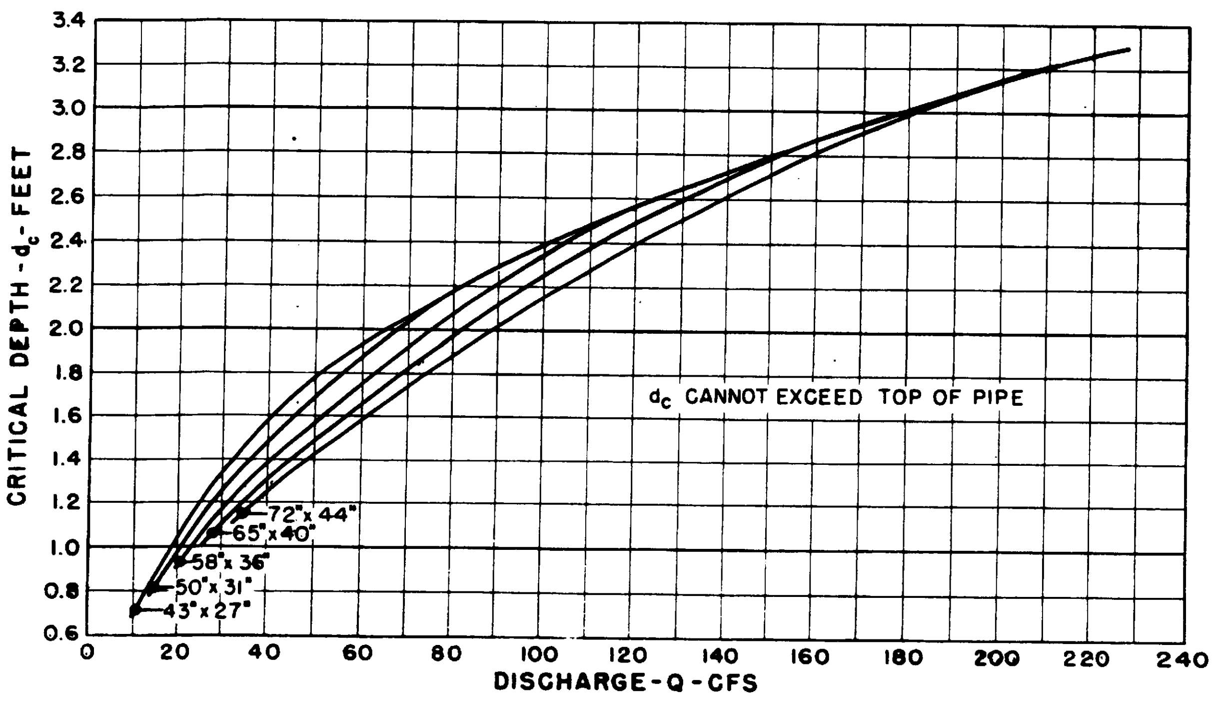


From Reference 4-12

CRITICAL DEPTH
CONCRETE ARCH PIPE

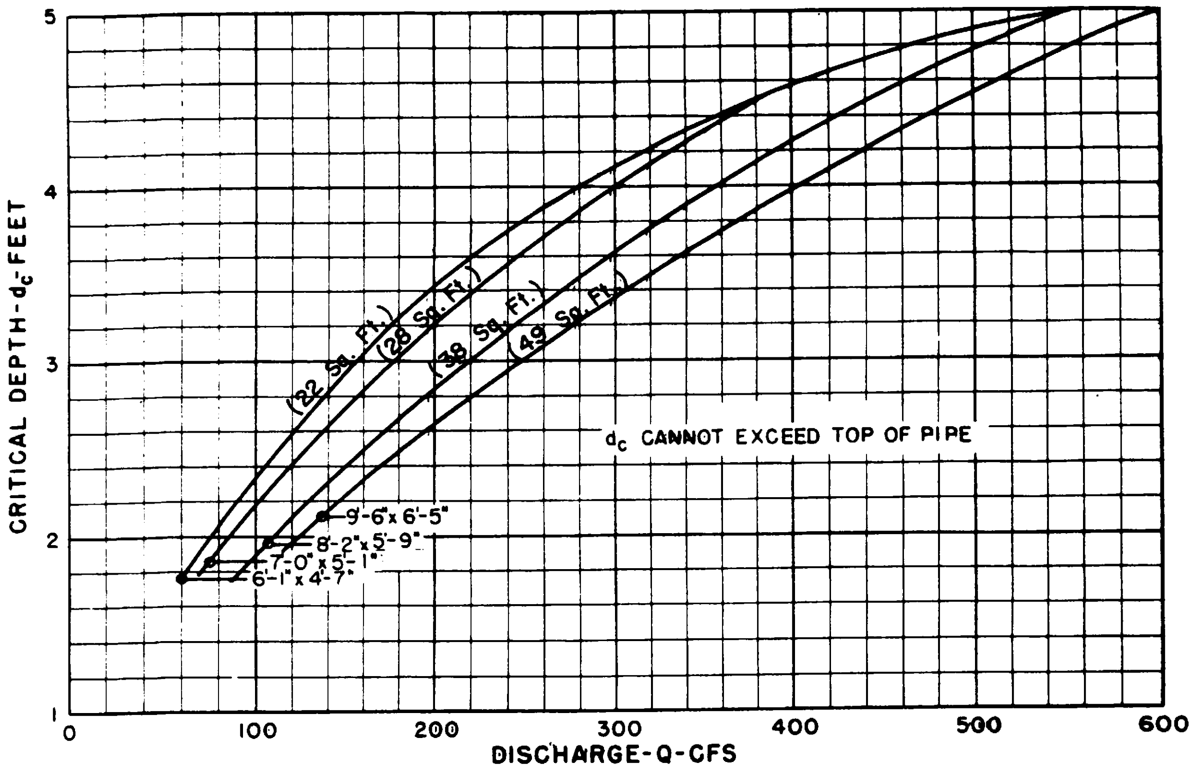
CHART 4-20.2

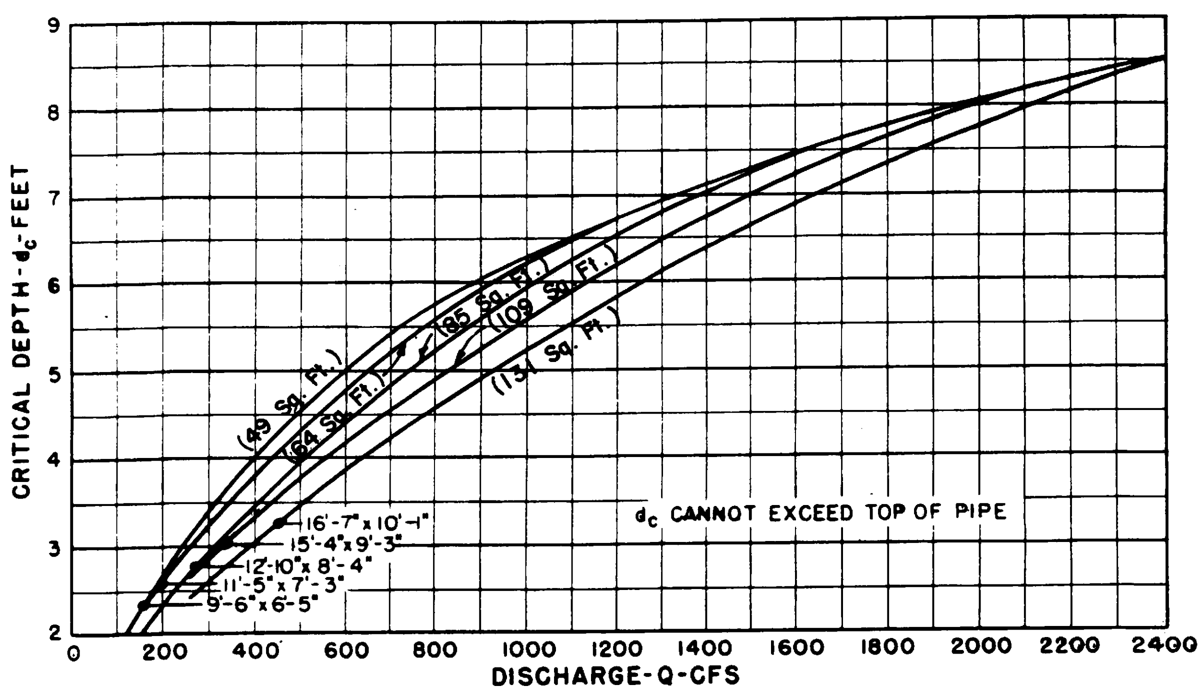




CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

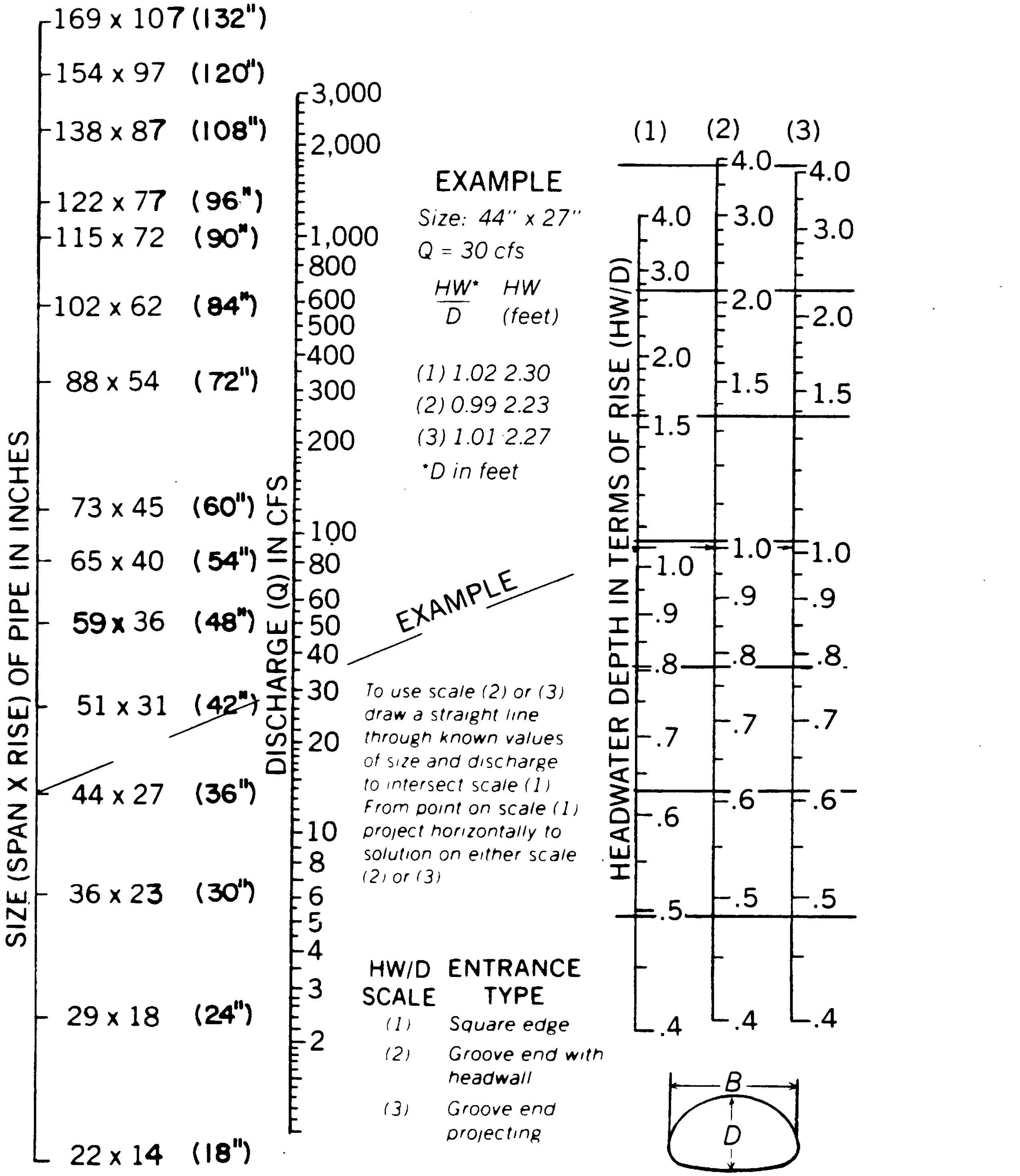
CHART 4-21





CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS

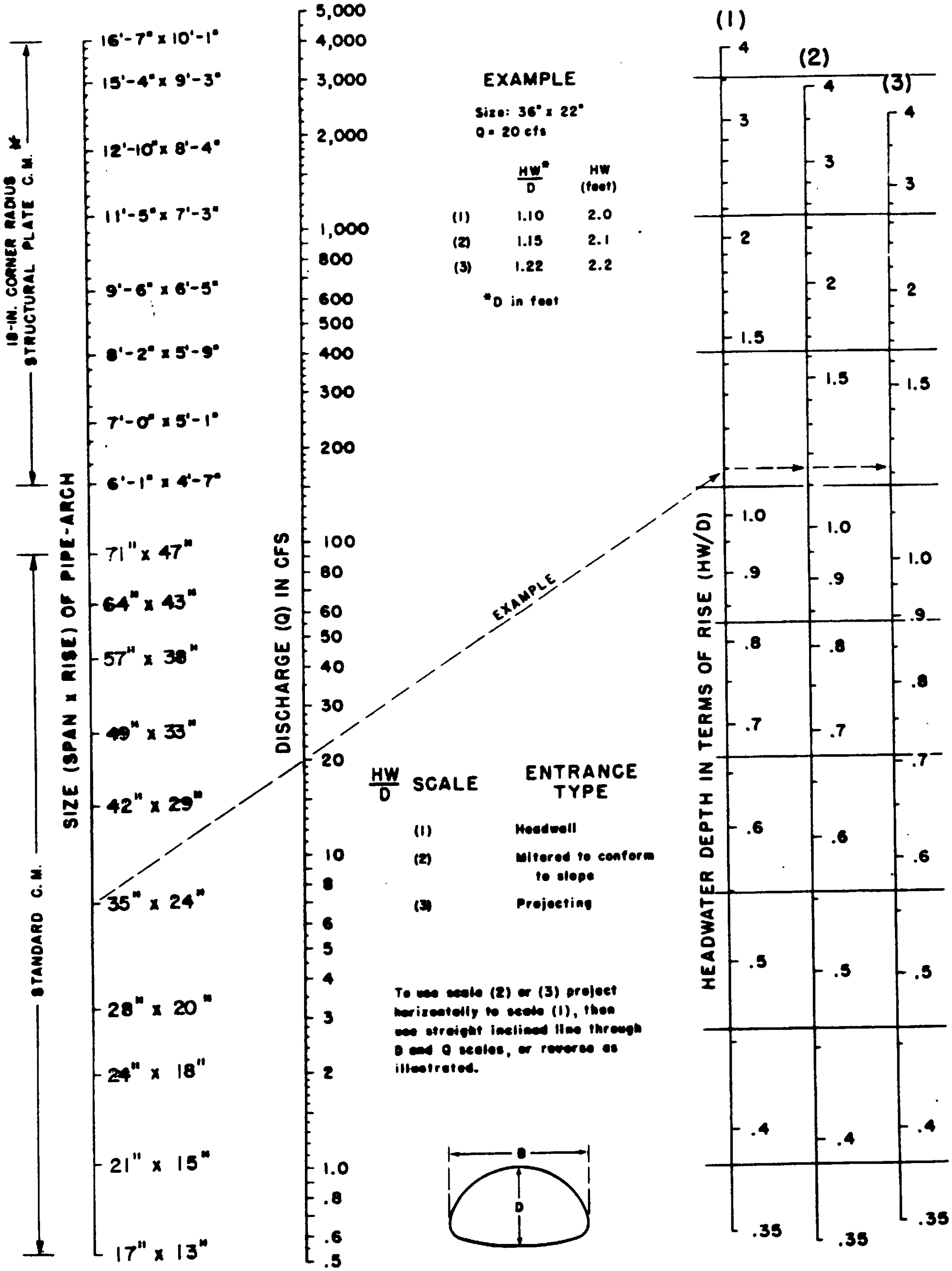
CHART 4-22



From Reference 4-12

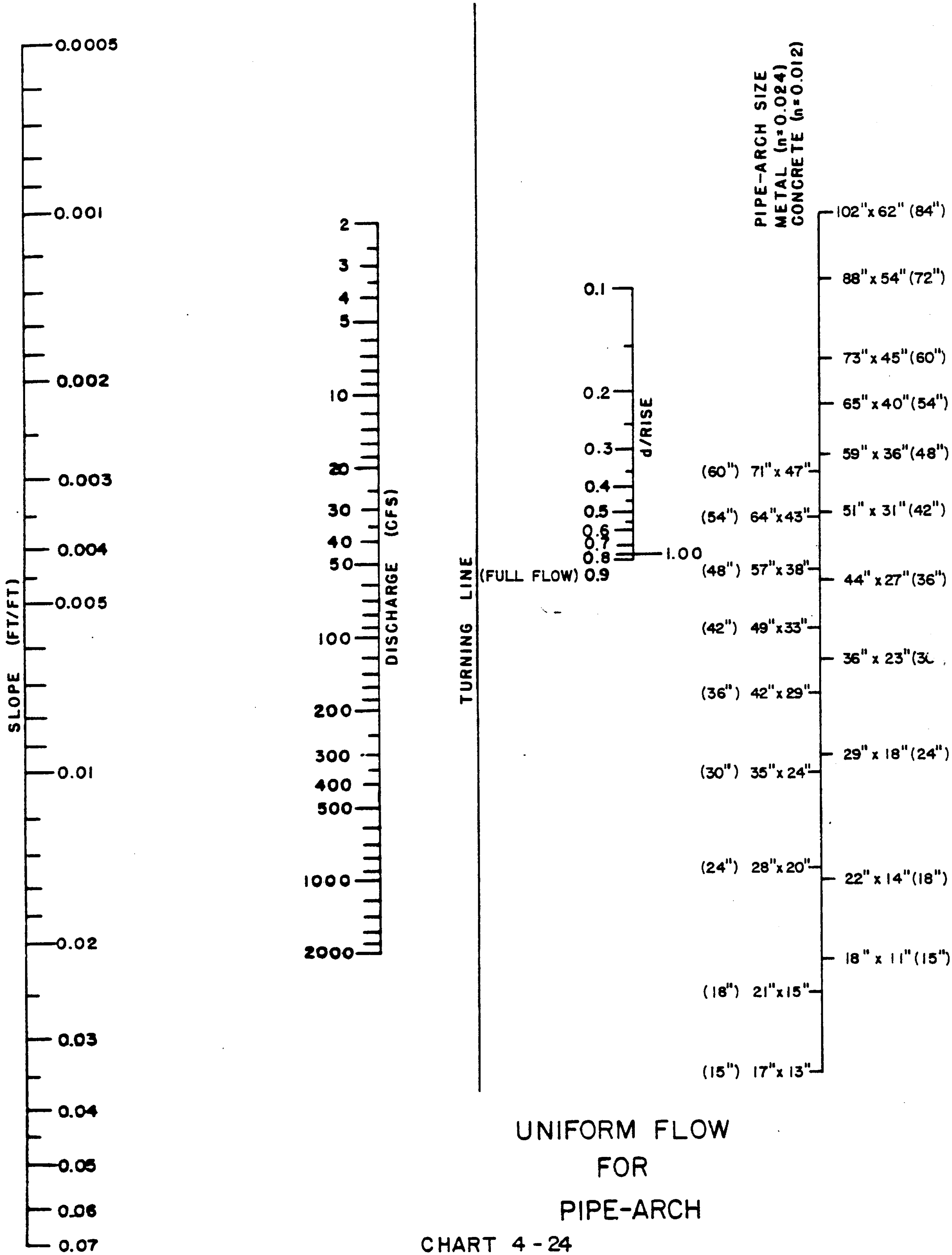
CHART 4-22.1

HEADWATER DEPTH FOR CONCRETE ARCH CULVERTS WITH INLET CONTROL



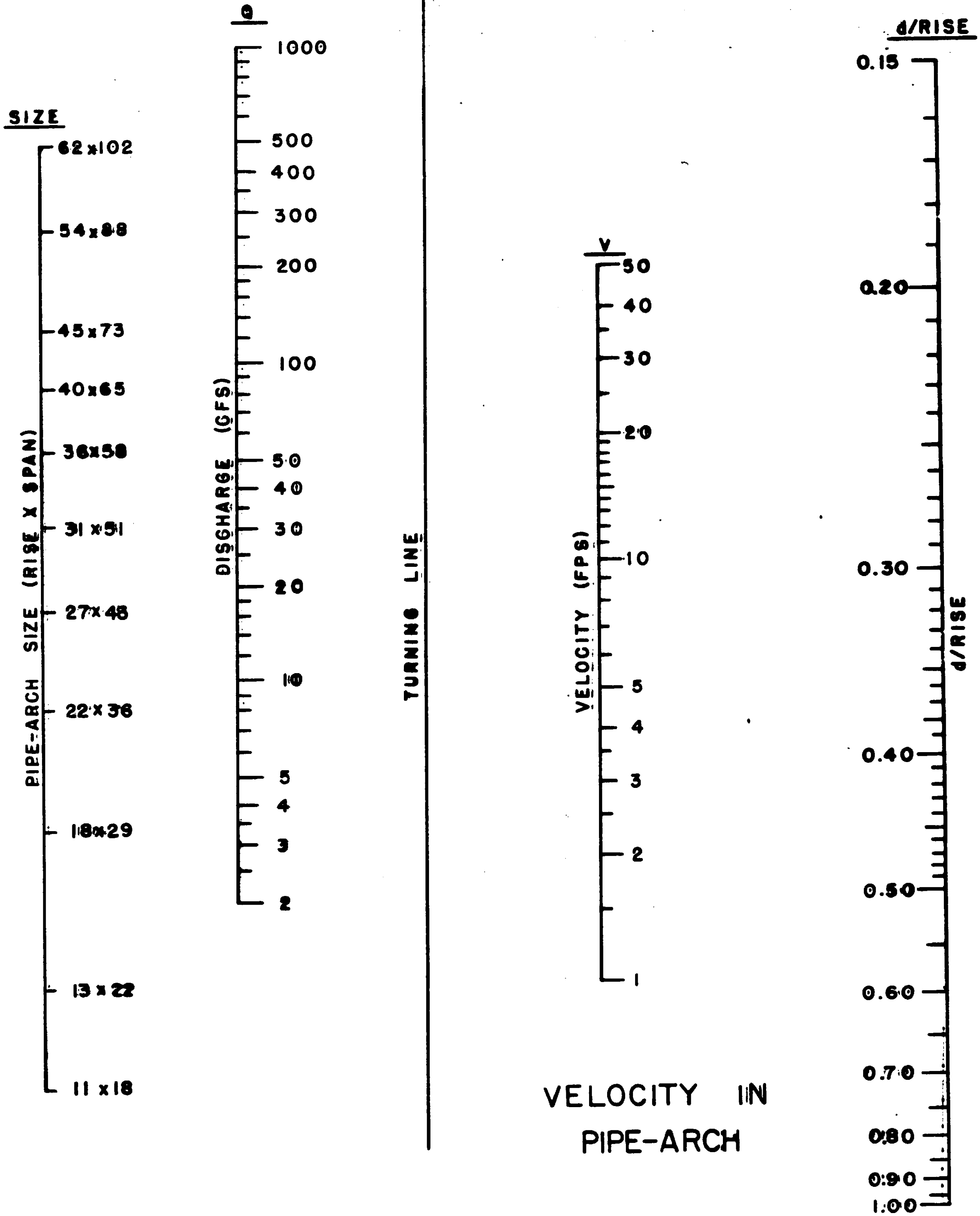
HEADWATER DEPTH FOR
C. M. PIPE-ARCH CULVERTS
WITH INLET CONTROL

CHART 4-23



UNIFORM FLOW
FOR
PIPE-ARCH

CHART 4-24



VELOCITY IN PIPE-ARCH

CHART 4-25

WATERWAY AREAS FOR STANDARD SIZES OF CORRUGATED STEEL CONDUITS

Round Pipe

Diameter (Inches)	Area Square Feet
12	.785
15	1.227
18	1.767
21	2.405
24	3.142
30	4.909
36	7.069
42	9.621
48	12.566
54	15.904
60	19.635
66	23.758
72	28.27
78	33.18
84	38.49
90	44.18
96	50.27
108	63.62
114	70.88
120	78.54
126	86.59
132	95.03
138	103.87
144	113.10
150	122.7
156	132.7
162	143.1
168	153.9
174	165.1
180	176.7
186	188.7
192	201.1
198	213.8
204	227.0
210	240.5
216	254.5
222	268.8
228	283.5
234	298.6
240	314.2
246	330.1
252	346.4
258	363.1
264	380.1
270	397.6
276	415.5
282	433.7
288	452.4
294	471.4
300	490.9

Pipe-Arch (½ in Corrugation)

Size	Area
17 x 13	1.1
21 x 15	1.6
24 x 18	2.2
28 x 20	2.9
35 x 24	4.5
42 x 29	6.5
49 x 33	8.9
57 x 38	11.6
64 x 43	14.7
71 x 47	18.1
77 x 52	21.9
83 x 57	26.0

Pipe-Arch (1 in Corrugation)

Size	Area
60 x 46	15.6
66 x 51	19.3
73 x 55	23.2
81 x 59	27.4
87 x 63	32.1
95 x 67	37.0
103 x 71	42.4
112 x 75	48.0
117 x 79	54.2
128 x 83	60.5
137 x 87	67.4
142 x 91	74.5

Structural Plate Arch

Size Feet Inches	Area Square Feet
6.0 x 3-2	15
7.0 x 3-8	20
8.0 x 4-2	26
9.0 x 4-8½	33
10.0 x 5-3	41
11.0 x 5-9	50
12.0 x 6-3	59
13.0 x 6-9	70
14.0 x 7-3	80
15.0 x 7-9	92
16.0 x 8-3	105
17.0 x 8-10	119
18.0 x 8-11	126
19.0 x 9-5½	140
20.0 x 10-0	157
21.0 x 10-6	172
22.0 x 11-0	190
23.0 x 11-6	172
24.0 x 12-0	226
25.0 x 12-6	247

Structural Plate Pipe-Arch

Size Feet Inches	Area Square Feet
18-inch Corner Radius R_c	
6-1 x 4-7	22
6-4 x 4-9	24
6-9 x 4-11	26
7-0 x 5-1	28
7-3 x 5-3	31
7-8 x 5-5	33
7-11 x 5-7	35
8-2 x 5-9	38
8-7 x 5-11	40
8-10 x 6-1	43
9-4 x 6-3	46
9-6 x 6-5	49
9-9 x 6-7	52
10-3 x 6-9	55
10-8 x 6-11	58
10-11 x 7-1	61
11-5 x 7-3	64
11-7 x 7-5	67
11-10 x 7-7	71
12-4 x 7-9	74
12-6 x 7-11	78
12-8 x 8-1	81
12-10 x 8-4	85
13-5 x 8-5	89
13-11 x 8-7	93
14-1 x 8-9	97
14-3 x 8-11	101
14-10 x 9-1	105
15-4 x 9-3	109
15-6 x 9-5	113
15-8 x 9-7	118
15-10 x 9-10	122
16-5 x 9-11	126
16-7 x 10-1	131
31 inch Corner Radius R_c	
13-3 x 9-4	97
13-6 x 9-6	102
14-0 x 9-8	105
14-2 x 9-10	109
14-5 x 10-0	114
14-11 x 10-2	118
15-4 x 10-4	123
15-7 x 10-6	127
15-10 x 10-8	132
16-3 x 10-10	137
16-6 x 11-0	142
17-0 x 11-2	146
17-2 x 11-4	151
17-5 x 11-6	157
17-11 x 11-8	161
18-1 x 11-10	167
18-7 x 12-0	172
18-0 x 12-2	177
19-3 x 12-4	182
19-6 x 12-6	188
19-8 x 12-8	194
19-11 x 12-10	200
20-5 x 13-0	205
20-7 x 13-2	211

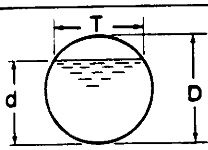
From Ref. 4-10

TABLE 4-12

Hydraulic Properties of Circular Conduits Flowing Partly Full

d = Depth of Flow
d_c = Critical depth
d_m = Mean depth

D = Diameter of pipe
A = Area of flow
R = Hydraulic radius
T = Top width of flow

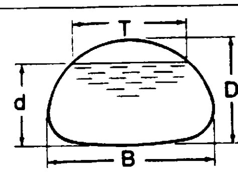


$\frac{d}{D}$ or $\frac{d_c}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{T}{D}$	$\frac{d_m}{D}$
1.00	0.7854	0.2500	—	—
0.95	0.7707	0.2865	0.4359	1.7681
0.90	0.7445	0.2980	0.6000	1.2408
0.85	0.7115	0.3033	0.7142	0.9962
0.80	0.6736	0.3042	0.8000	0.8420
0.75	0.6319	0.3017	0.8660	0.7297
0.70	0.5872	0.2962	0.9165	0.6407
0.65	0.5404	0.2882	0.9539	0.5665
0.60	0.4920	0.2776	0.9798	0.5021
0.55	0.4426	0.2649	0.9950	0.4448
0.50	0.3927	0.2500	1.0000	0.3927
0.45	0.3428	0.2331	0.9950	0.3445
.040	0.2934	0.2142	0.9798	0.2994
0.35	0.2450	0.1935	0.9539	0.2568
0.30	0.1982	0.1709	0.9165	0.2163
0.25	0.1535	0.1466	0.8660	0.1773
0.20	0.1118	0.1206	0.8000	0.1397
0.15	0.0739	0.0929	0.7142	0.1035

Hydraulic Properties of Pipe Arch Conduits Flowing Partly Full

d = Depth of flow
d_c = Critical depth
d_m = Mean depth

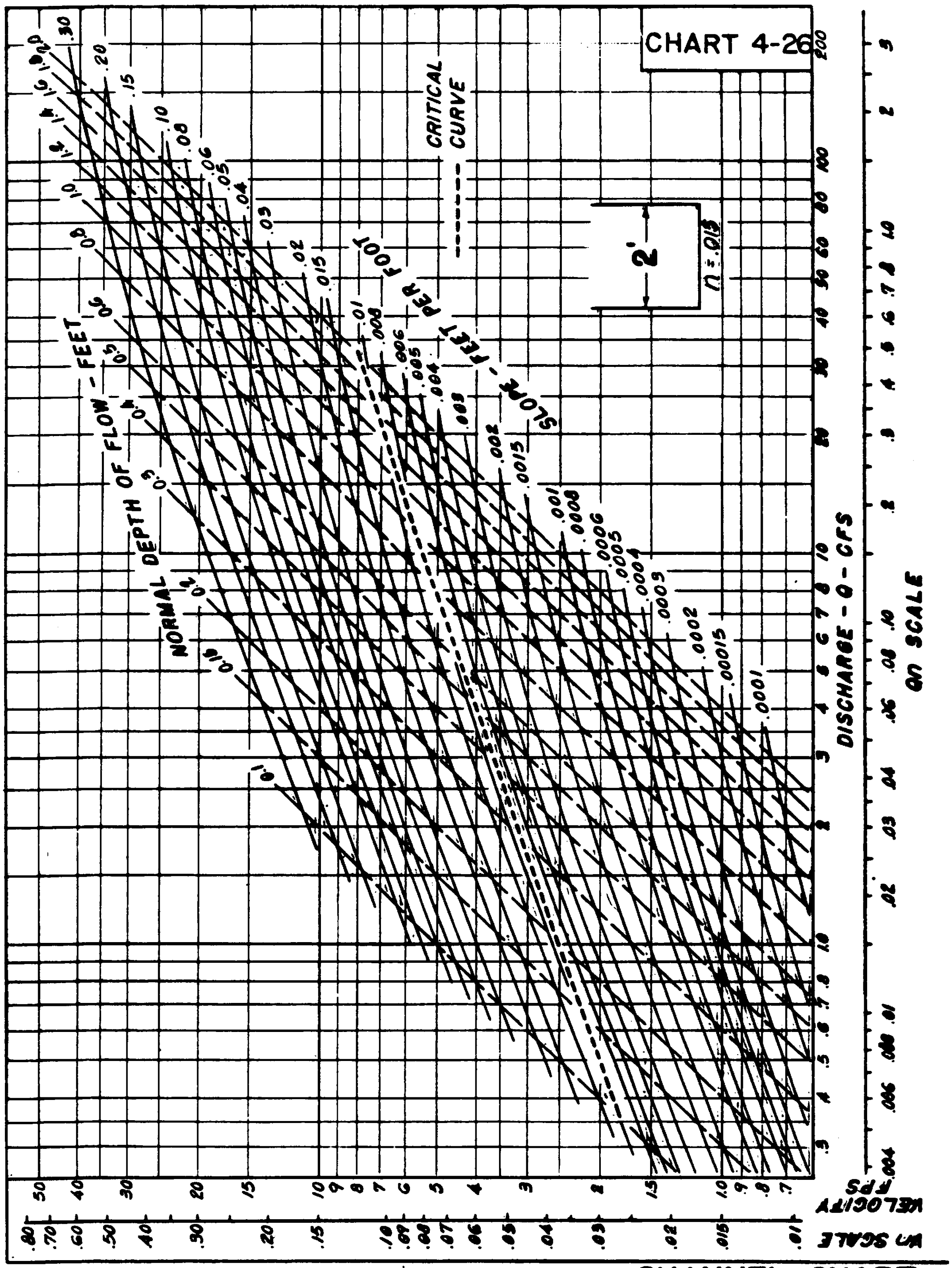
D = Diameter of pipe
A = Area of flow
R = Hydraulic radius
T = Top width of flow



$\frac{d}{D}$ or $\frac{d_c}{D}$	$\frac{A}{BD}$	$\frac{R}{D}$	$\frac{T}{D}$	$\frac{d_m}{D}$
1.00	0.7879	0.2991	—	—
0.95	0.7762	0.3408	0.3489	2.225
0.90	0.7552	0.3549	0.4855	1.555
0.85	0.7283	0.3622	0.5848	1.245
0.80	0.6970	0.3649	0.6637	1.0503
0.75	0.6621	0.3639	0.7288	0.9085
0.70	0.6243	0.3595	0.7837	0.7966
0.65	0.5839	0.3520	0.8303	0.7033
0.60	0.5414	0.3415	0.8700	0.6223
0.55	0.4970	0.3282	0.9037	0.5500
0.50	0.4511	0.3120	0.9320	0.4840
0.45	0.4039	0.2928	0.9555	0.4227
0.40	0.3556	0.2705	0.9755	0.3646
0.35	0.3065	0.2451	0.9889	0.3100
0.30	0.2568	0.2162	0.9967	0.2577
0.25	0.2069	0.1839	0.9967	0.2076
0.20	0.1574	0.1484	0.9815	0.1603
0.15	0.10908	0.11022	0.9477	0.11505

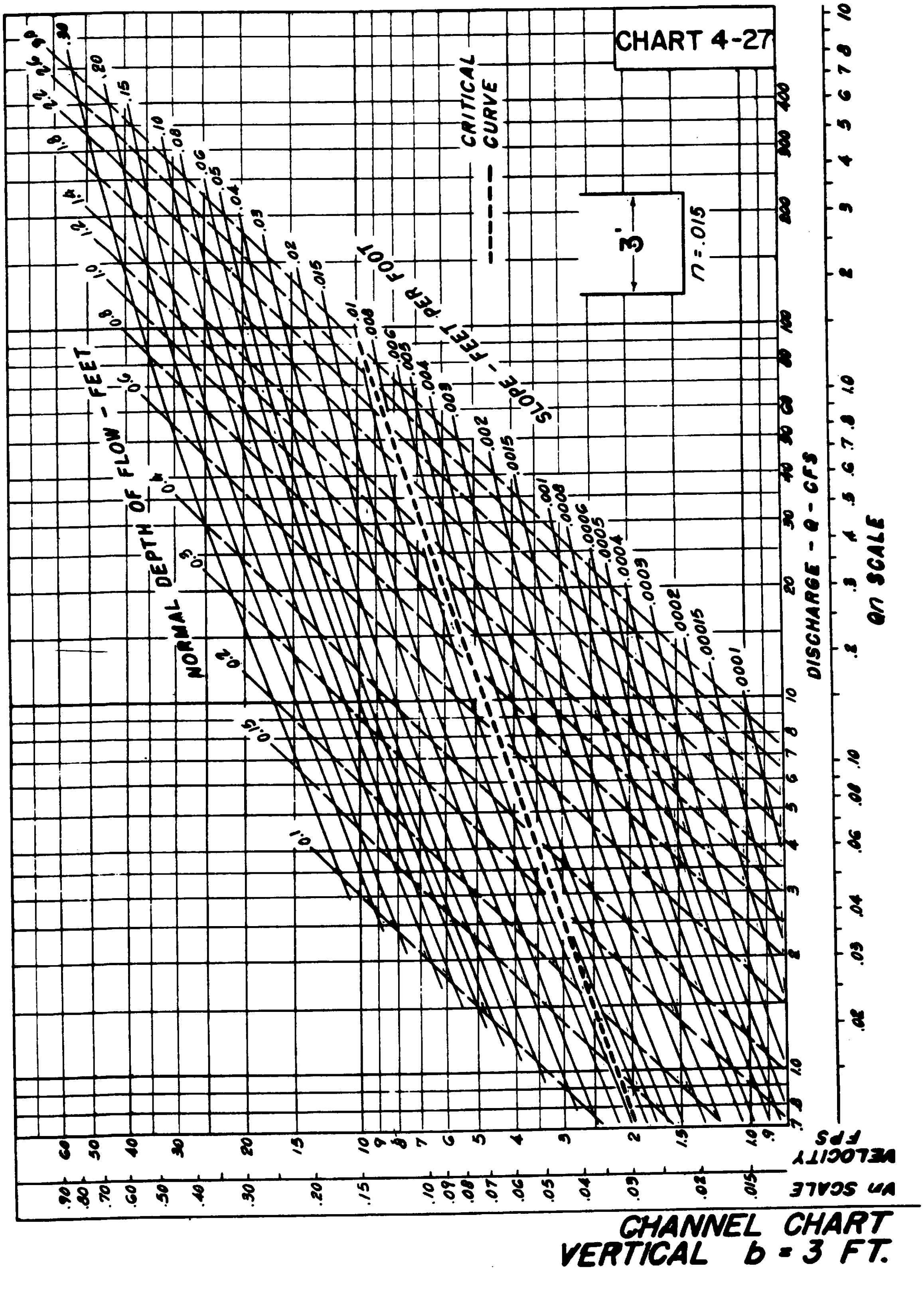
From Ref. 4-10

TABLE 4-13



CHANNEL CHART
VERTICAL b = 2 FT.

CHART 4-26



CHANNEL CHART
VERTICAL b = 3 FT.

CHART 4-27

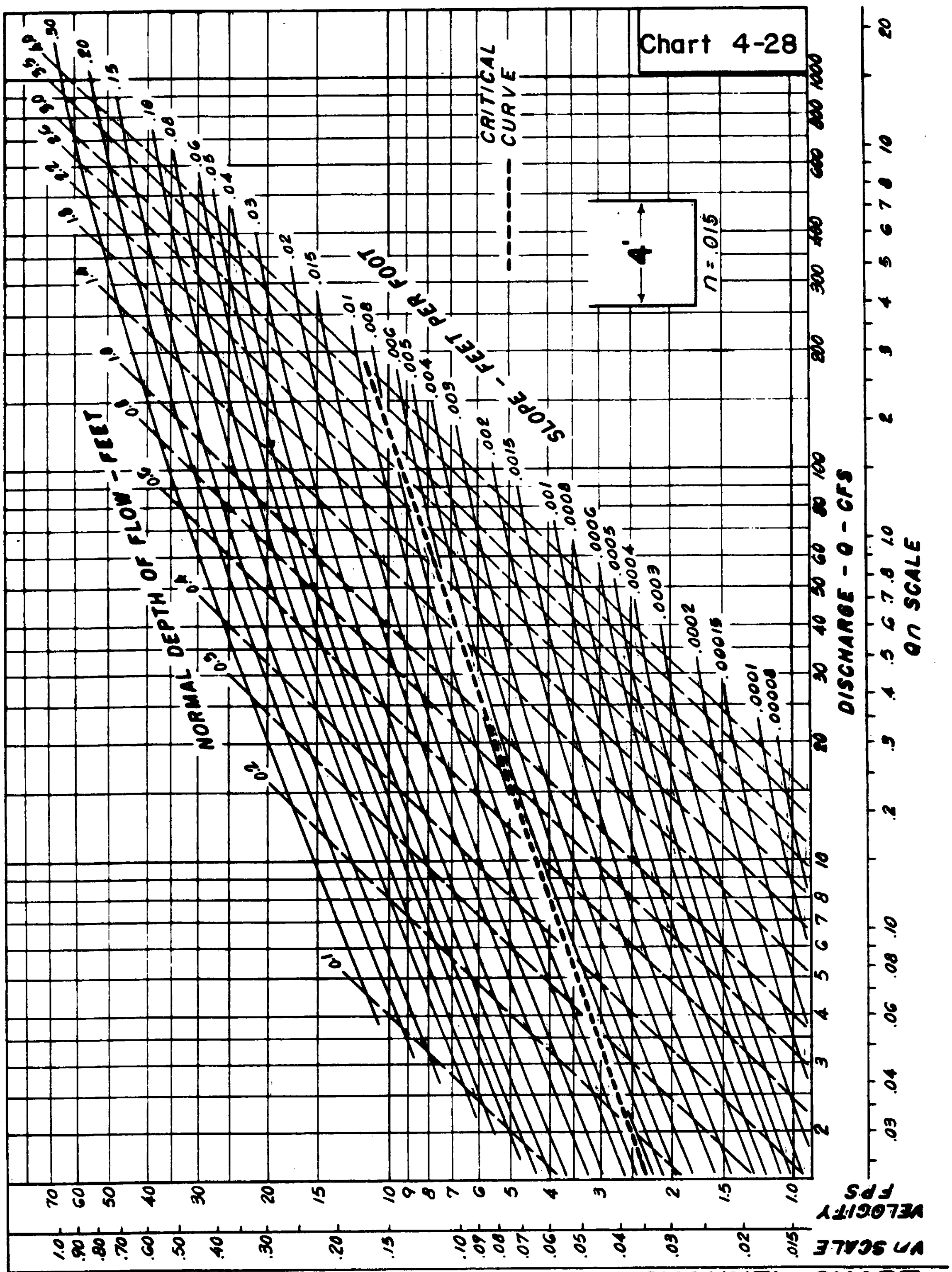


CHART 4-28

CHANNEL CHART
VERTICAL b = 4 FT.

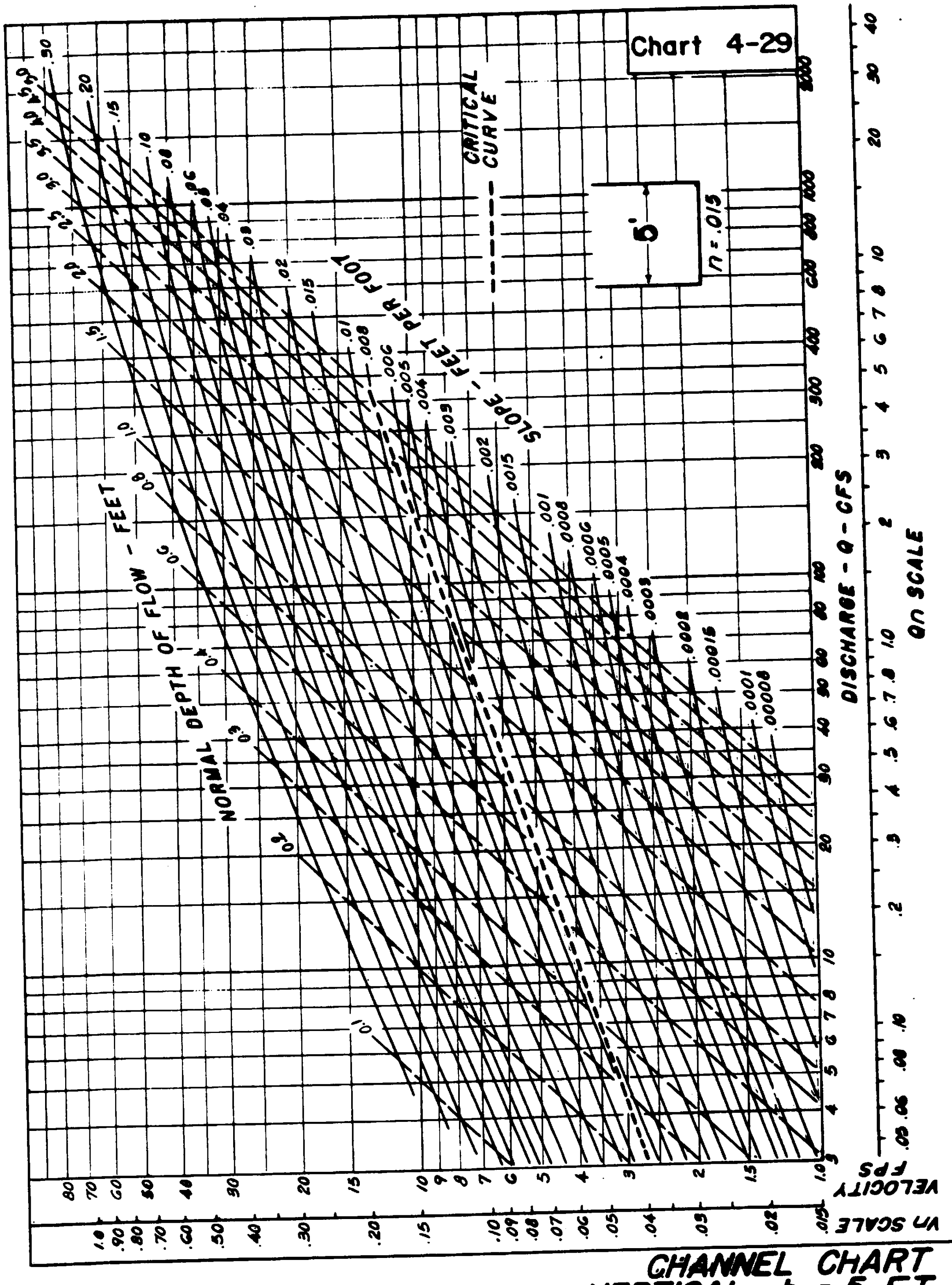
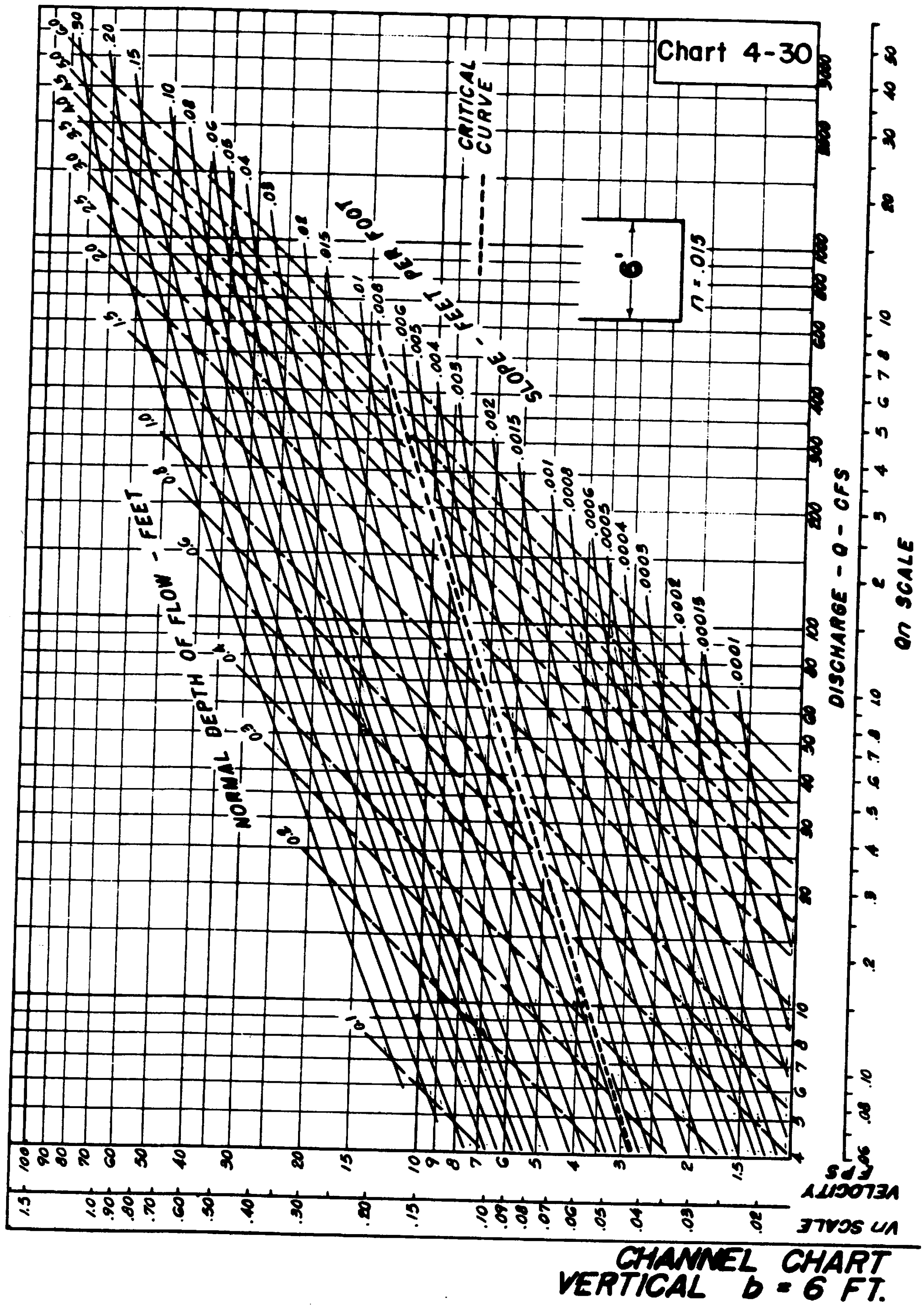


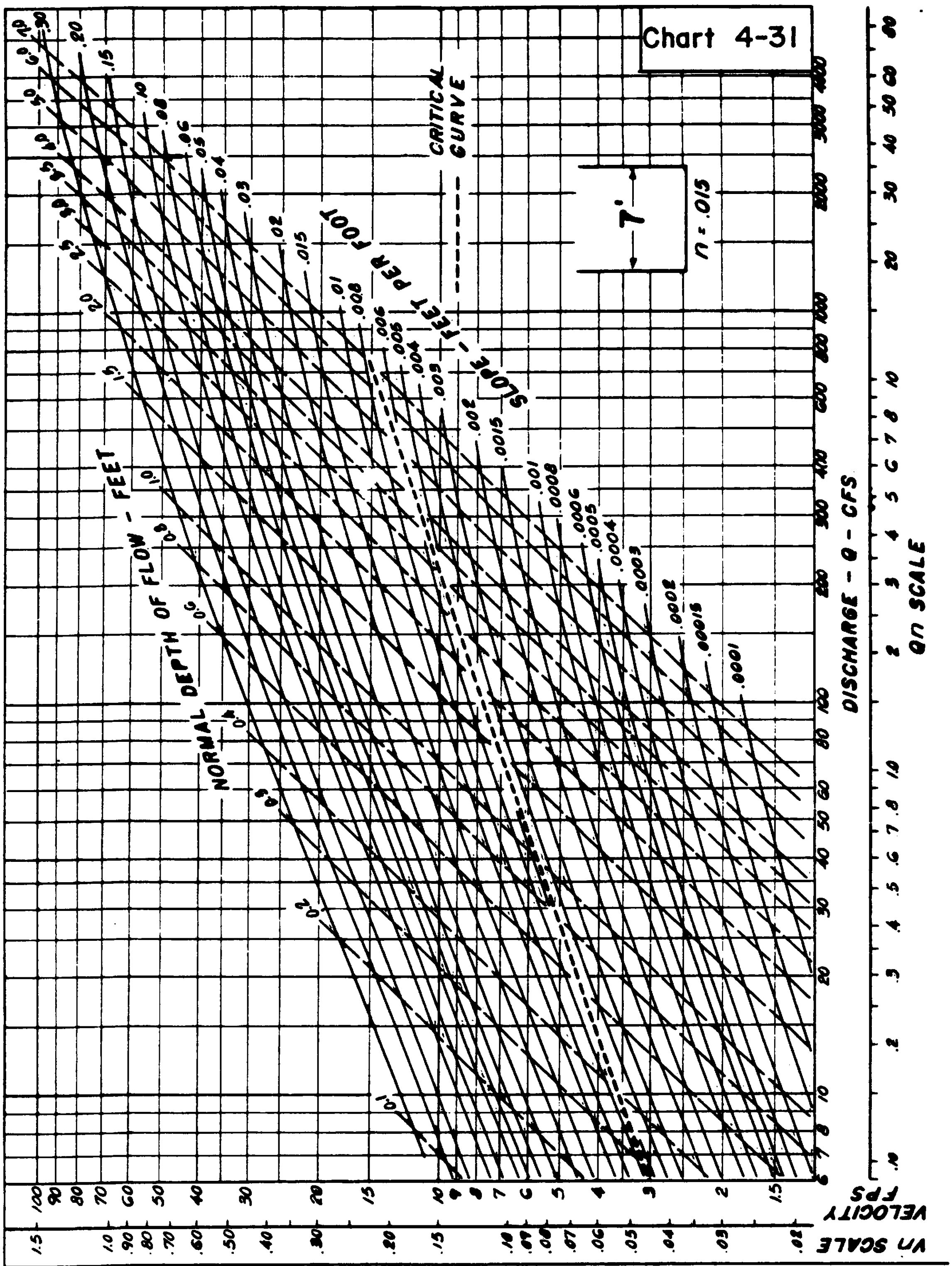
CHART 4-29

CHANNEL CHART
VERTICAL b = 5 FT.



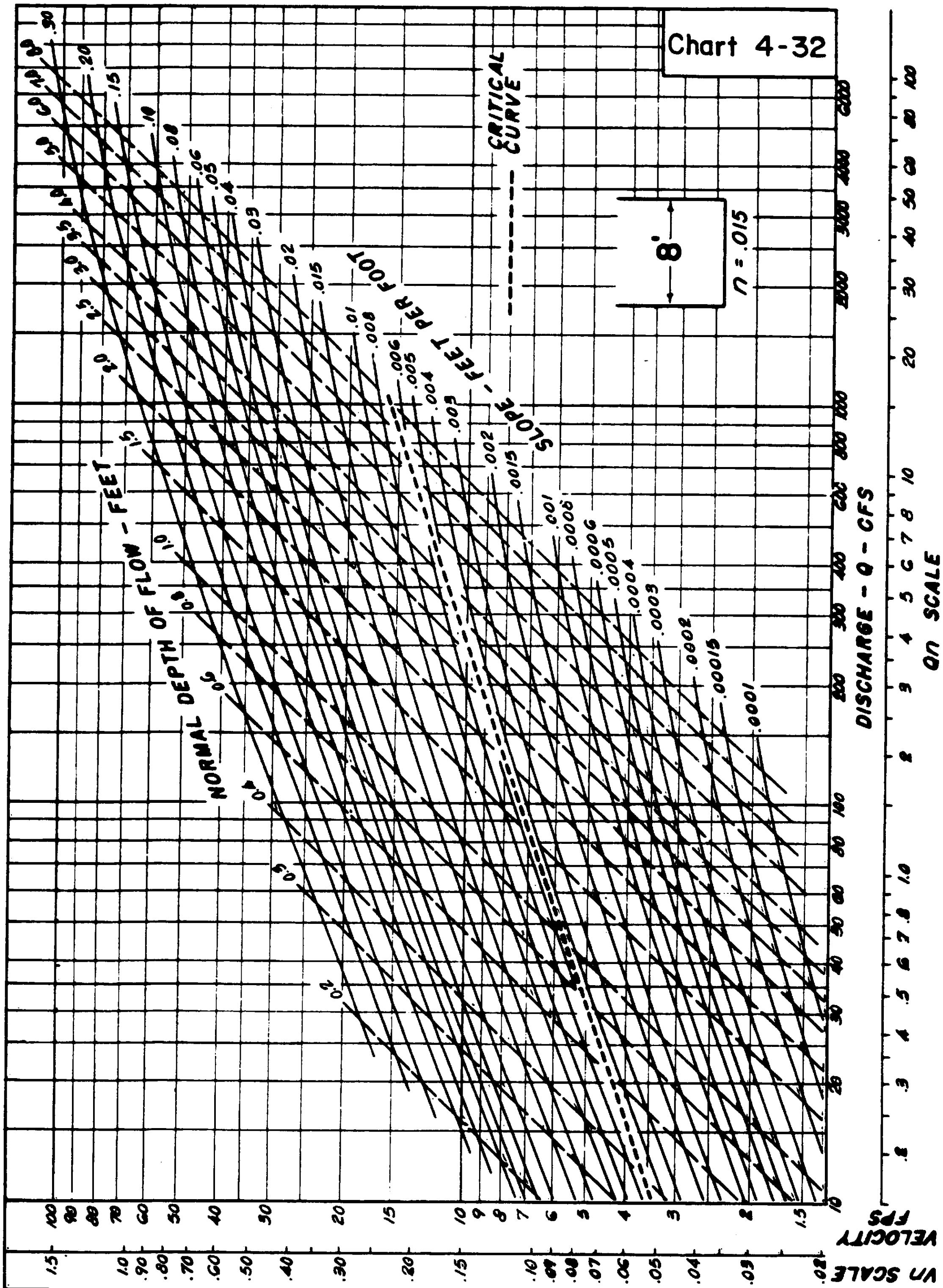
CHANNEL CHART
VERTICAL b = 6 FT.

CHART 4-30



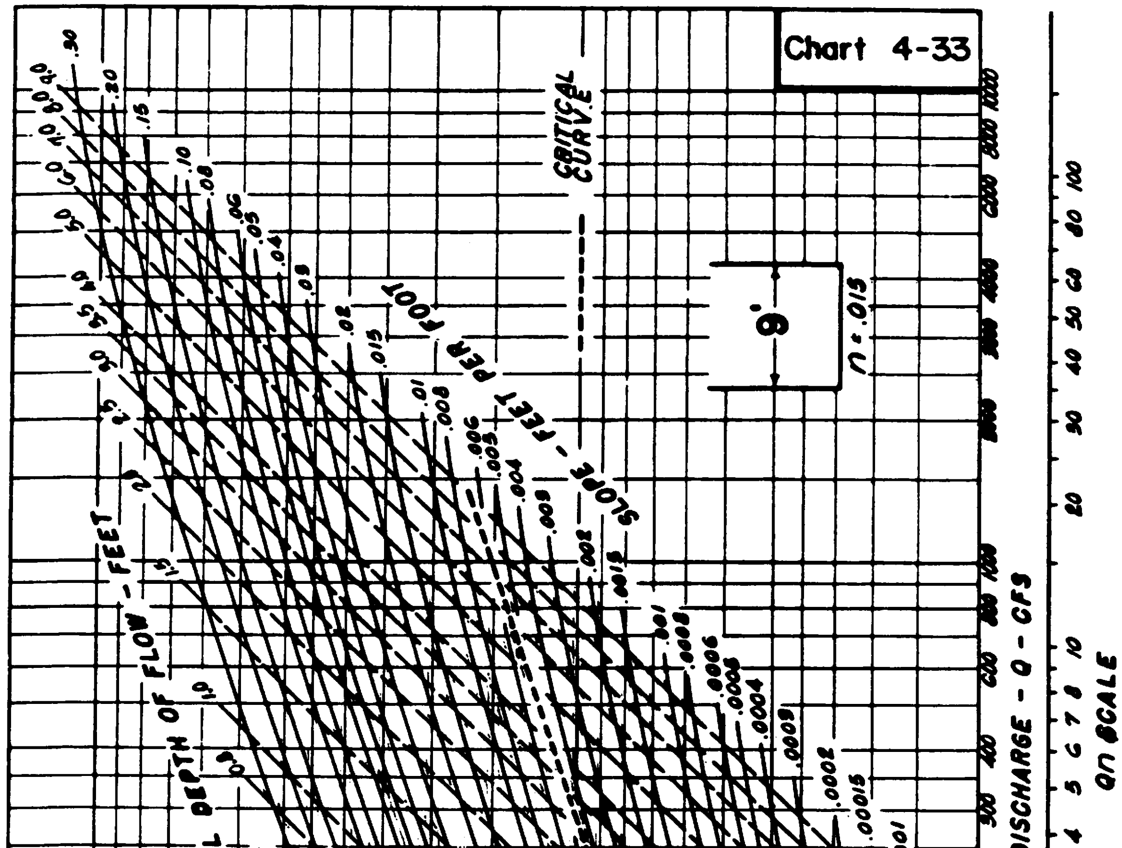
CHANNEL CHART
VERTICAL b = 7 FT.

CHART 4-31



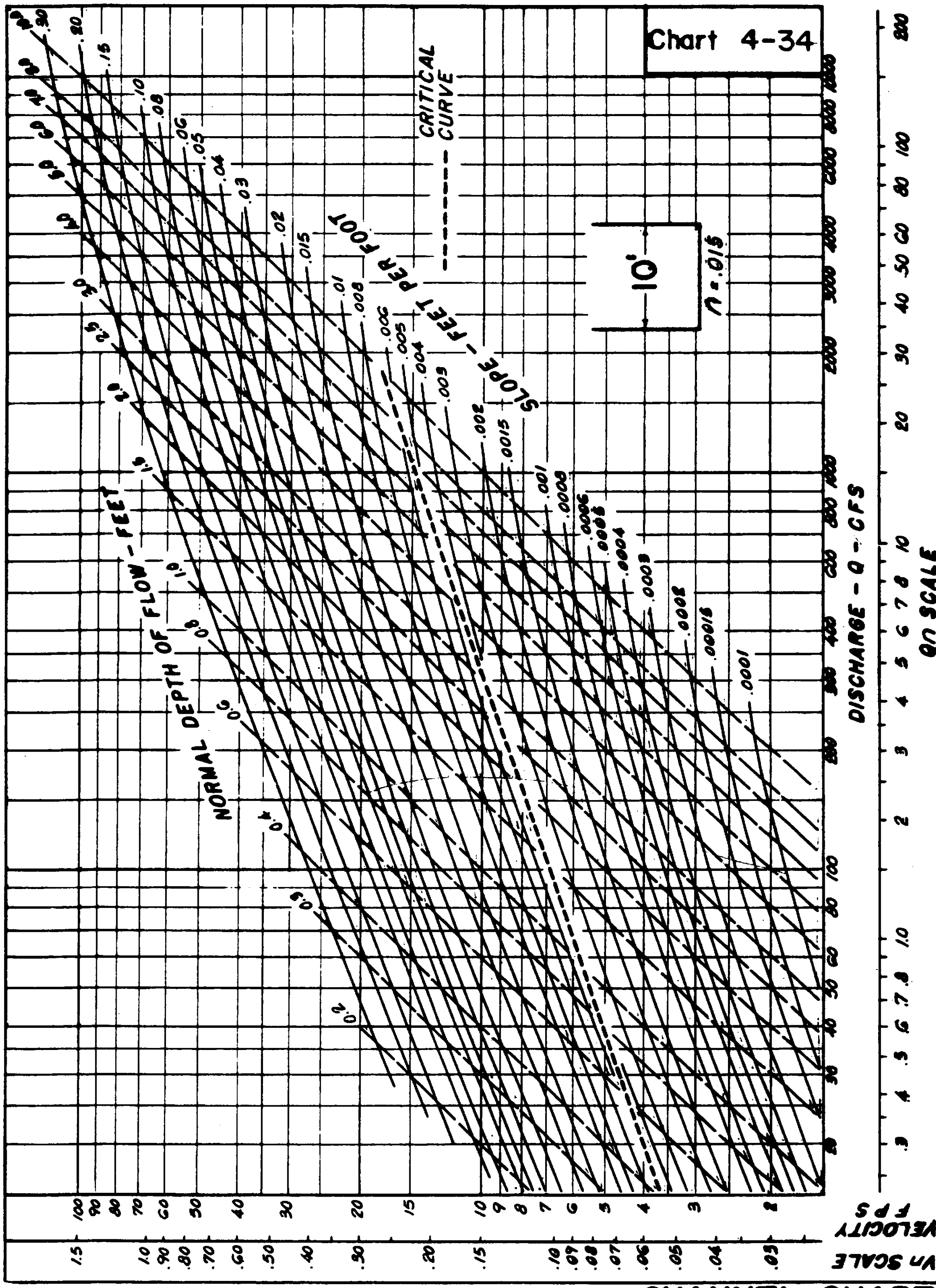
CHANNEL CHART
VERTICAL b = 8 FT.

CHART 4-32



CHANNEL CHART
VERTICAL b = 9 FT.

CHART 4-33



CHANNEL CHART
VERTICAL b = 10 FT.

CHART 4-34

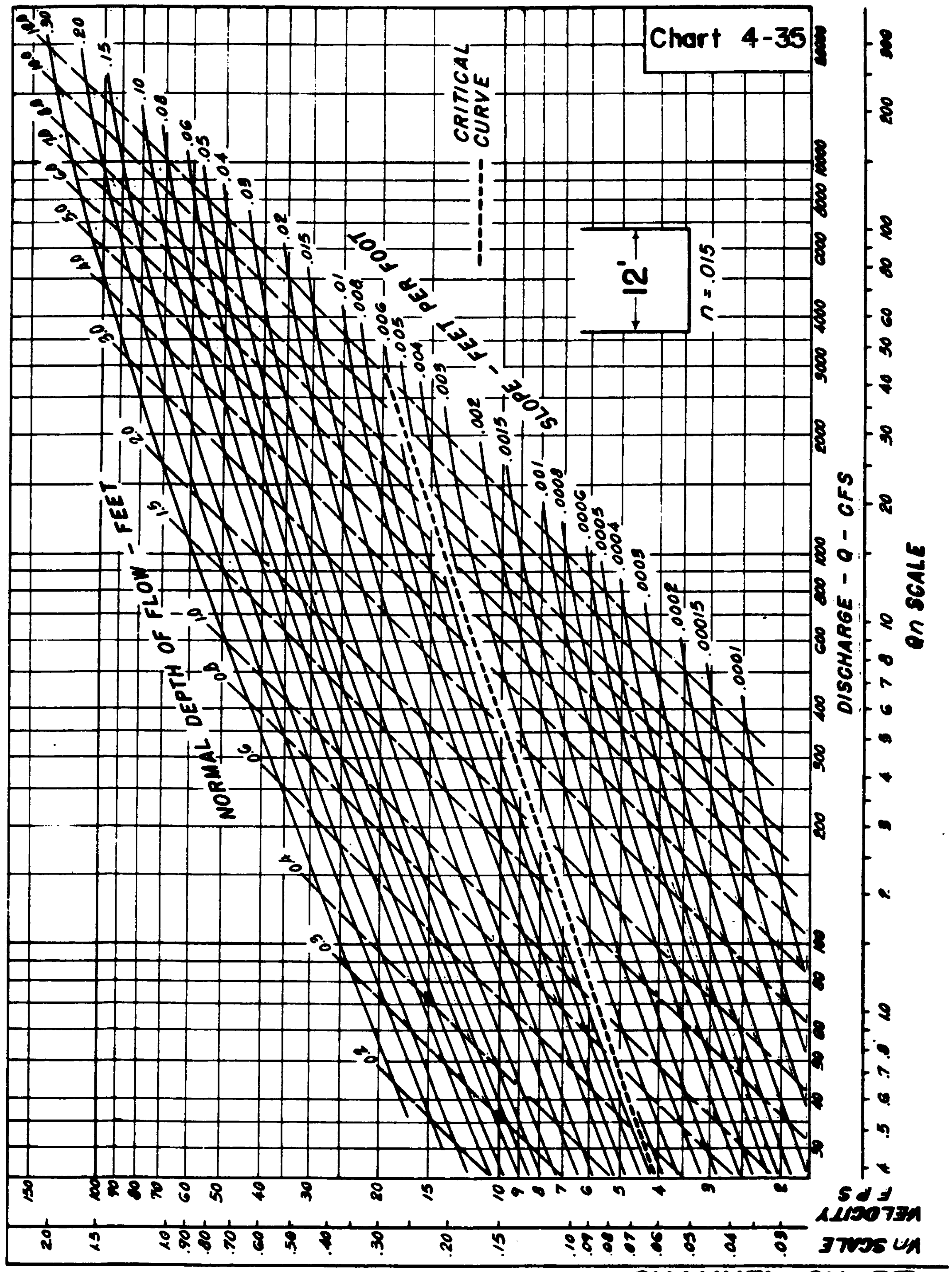
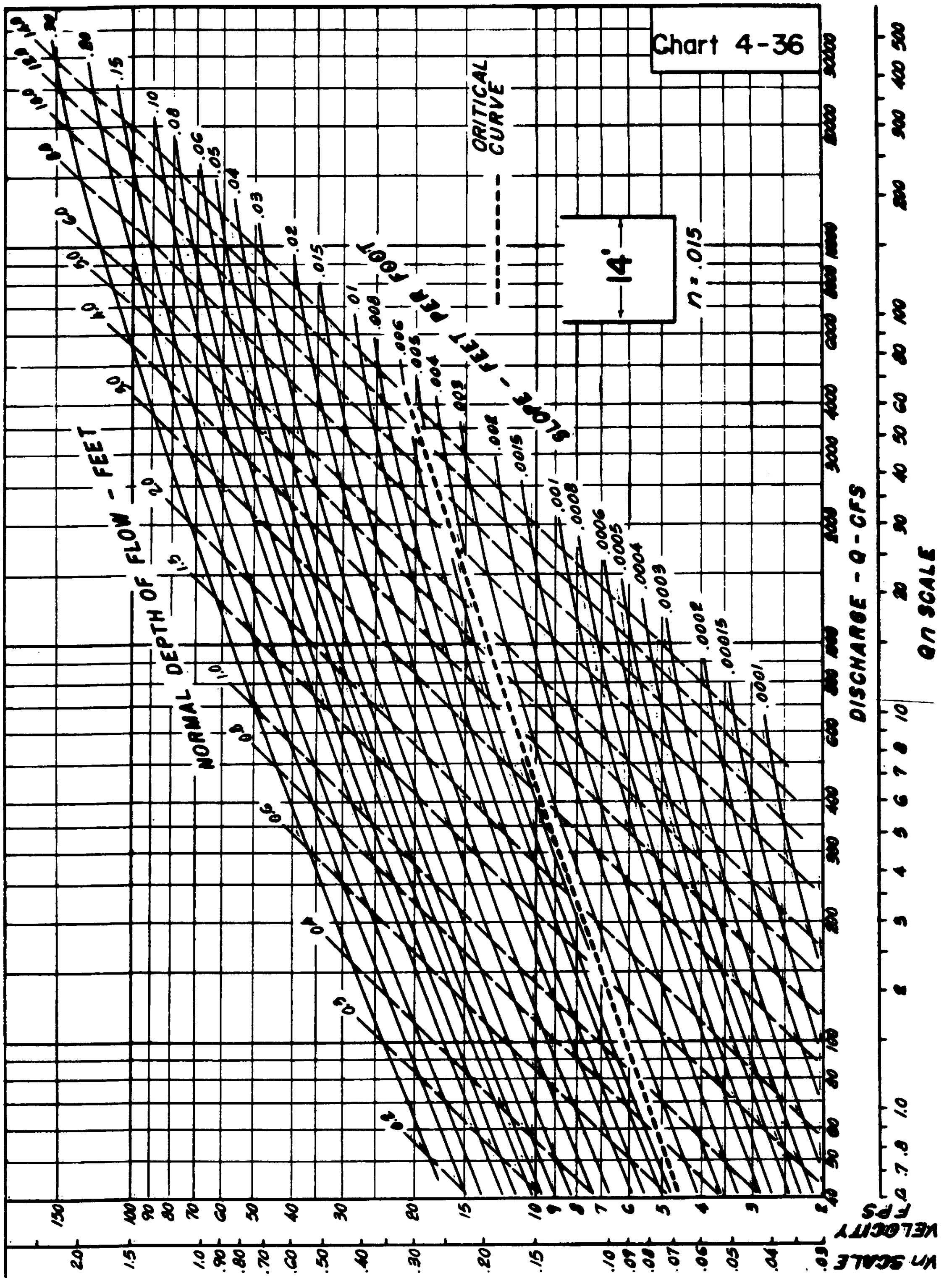


CHART 4-35

CHANNEL CHART
VERTICAL b = 12 FT.



CHANNEL CHART
VERTICAL b = 14 FT.

CHART 4-36

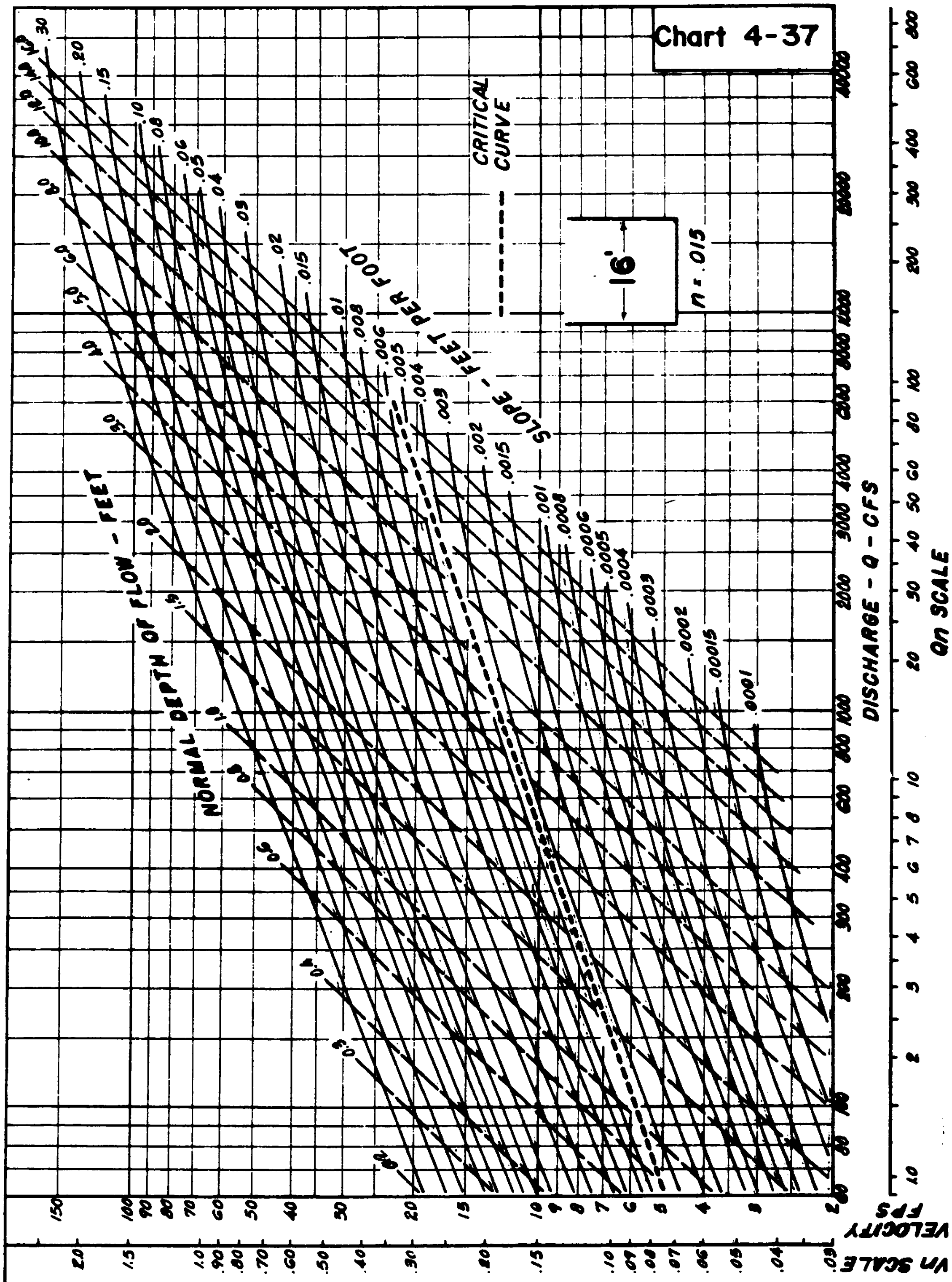
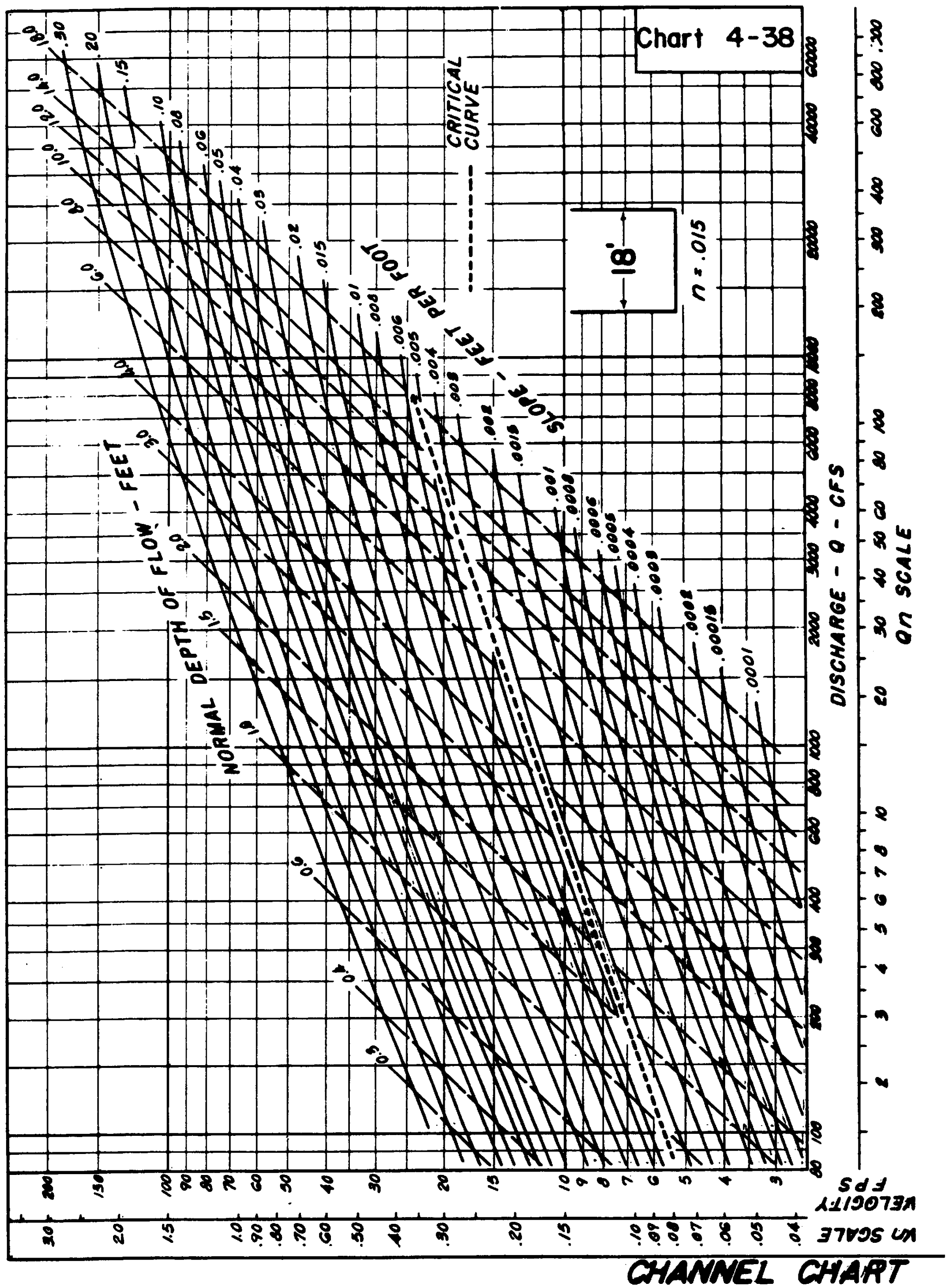


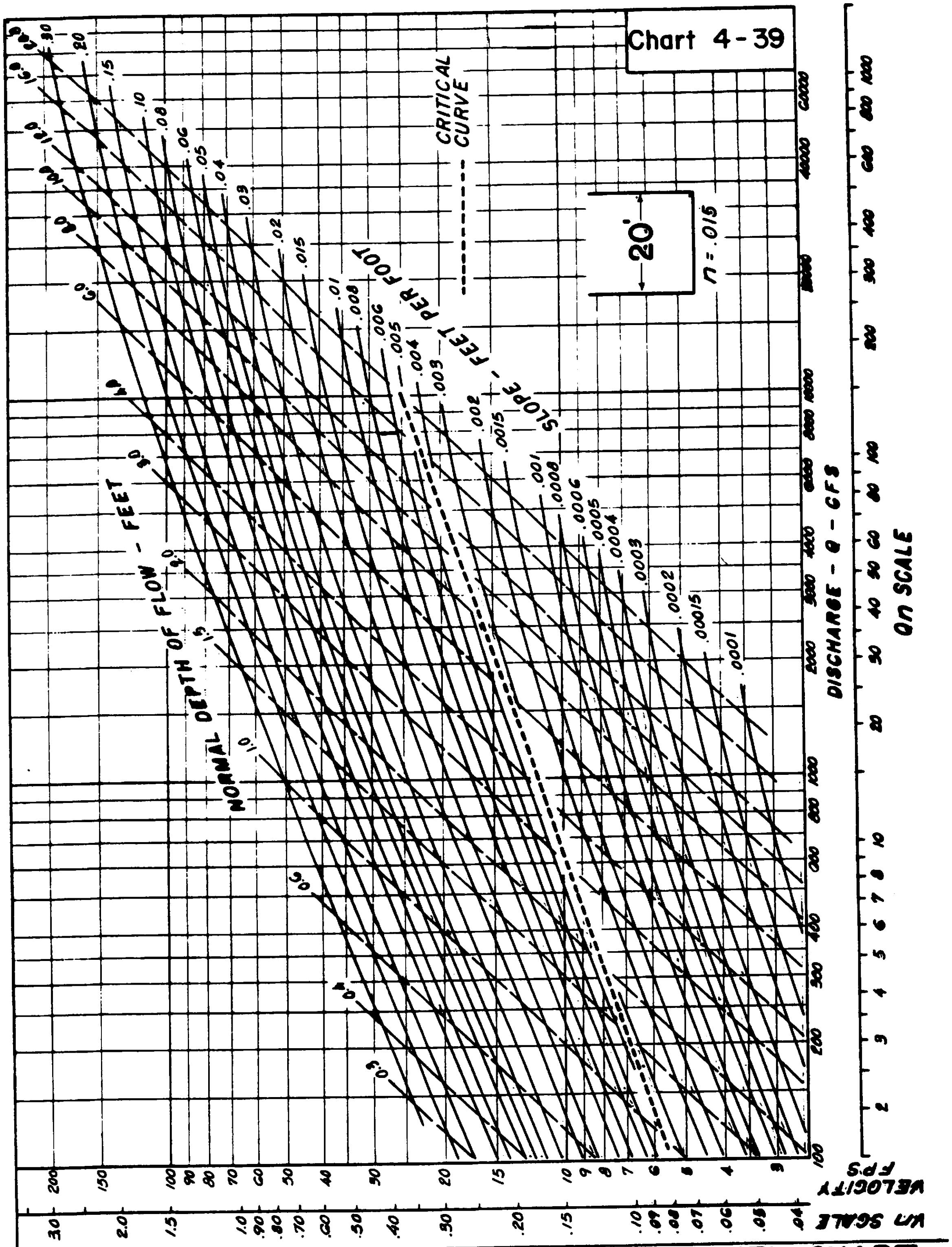
CHART 4-37

CHANNEL CHART
VERTICAL b = 16 FT.



CHANNEL CHART
VERTICAL b = 18 FT.

CHART 4-38



CHANNEL CHART
VERTICAL b = 20 FT.

CHART 4-39

REFERENCES

Chapter 4

CULVERTS

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STORM SEWERS

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STORM SEWERS

LIST OF SYMBOLS

CHAPTER 5

STORM SEWERS

A	Exponent for slotted drain pipe inlet formula
A	Tributary area in acres
A	Waterway area of the inlet opening or grate, in square feet
a	Depth of gutter depression in feet
C	Coefficient in the Rational Formula
C	Discharge coefficient
cfs	Cubic feet per second
D	Pipe diameter in feet
d	Depth of curb flow in feet
d_c	Critical depth of flow in feet
E	The hydraulic efficiency of an inlet grate or slotted drain pipe inlet, in percentage
E_o	Grate inlet effiency without splash-over
g	Gravitational constant; 32.2 feet per second per second
H	Velocity head loss
H_b	Bend head loss in feet

LIST OF SYMBOLS

CHAPTER 5

STORM SEWERS

H_e	Entrance head loss for outlet structure
HGL	Hydraulic grade line
H_T	Total head loss
h	Height of curb-opening of an inlet, in feet.
H_f	Friction loss in feet
h_{j1} or h_{j2}	Head loss at a junction in feet
h_m	Minimum height of curb opening required for weir-type operation, in feet
H_{tm}	Terminal (beginning of run) junction loss in feet
I	Average rainfall rate in inches
I_p	Intensity in pipe in inches per hour
K	Coefficient
L	Total flow capture length of slotted drain pipe inlet in feet
L	Total length or length of pipe, in feet

LIST OF SYMBOLS

CHAPTER 5

STORM SEWERS

L_A	Actual length of slotted drain pipe inlet
L_i	Length of curb-opening inlet in feet
m	Constant dependent upon the grate configuration
n	Roughness coefficient in the Manning Formula
P	Perimeter in feet
Q	Rate of flow in cubic feet per second
Q_F	Grate Approach frontal flow in cfs
Q_i	Intercepted flow at inlet, in cfs
Q_p	Flow rate in pipe in cfs
Q_t	Total flow in cfs
Q_T	Total gutter flow in cfs
R	Hydraulic radius or area of flow cross-section divided by wetted perimeter
R	Grate reduction factor
S	Slope of pipe in feet per foot
S_c	Critical slope in feet per foot
S_f	Friction slope

LIST OF SYMBOLS

CHAPTER 5

STORM SEWERS

S_o	Longitudinal slope in feet per foot
S_o^A	Longitudinal slope in feet per foot with variable exponent A
S_x	Cross-slope in feet per foot
T	Top width of water surface (spread) from curb face toward crown of pavement, in feet
T_c	Time of concentration in minutes
V	Velocity in feet per second
V_{av}	Average flow velocity in the gutter in feet per second
V_o	Velocity of outlet structure in fps
V_1	Velocity of flow in lateral in fps
V_F	Average frontal flow velocity in feet per second
W	Slot width; grate width; or gutter width in feet
W_E	Effective grate width in feet
ΔW	Extra grate width in feet which would be necessary for the inlet to have the same efficiency without side flow interception
Z	Reciprocal of pavement cross slope

5-100 GENERAL

Stormwater drainage attitudes and consequent policies have been undergoing a significant redirection. Historical practice has involved a philosophy of intercepting, collecting and disposing of stormwater runoff as rapidly as possible. The cumulative effects of such past concepts of urban storm drainage have been a principal cause of increased frequency of downstream flooding, often accompanied by diminishing groundwater supplies as a direct result of urbanization; or they have necessitated the development of large-scale downstream engineering works to prevent flood damage. There is increased attention in urban area master planning of storm drainage to the desirability of detaining or storing rainfall close to where it falls on-site, which sometimes requires tradeoffs with short-term, localized inconvenience. This will also control the effects of stormwater so that the threat not only to life and property, but also to the environment as a whole, can be minimized.

Today's urban drainage master plan should include collection, storage and disposal. The principles, objective and design considerations in current approaches to stormwater storage are outlined in Reference 5-1.

5-101 "MINOR" AND "MAJOR" DRAINAGE SYSTEMS

In the layout of an effective storm drainage system, the most important factor is to assure that a drainage path for both the minor and major systems be provided to avoid flooding and ponding in undesirable locations.

The "Minor System" might better be called the "Convenience System". This is part of the larger major storm drainage system which includes all natural and man-made drainage facilities in an entire watershed. The "Convenience System" is that scheme of curbs, gutters, inlets, pipes or other conduits, swales, channels and appurtenant facilities all designed to minimize nuisance, inconvenience and hazard to persons and property from storm runoffs which occur at relatively frequent intervals (usually all runoffs associated with a 10-year or less recurrence interval rainfall).

Current progressive engineering recognizes the need to devote more detailed attention to planning and designing supplementary aspects of the overall "Major System", which carries excess flow over and above the hydraulic capacity of various components of the "Convenience System". The initial portion of the collection process in the "Convenience System", i.e., gutters and inlets, should have as much design attention as the conveyance system after water has been removed from streets, sidewalks, parking and landscaped areas, etc. For example, when inlets, pipes or conduits become overtaxed, excess runoffs use the hydraulic capacity of roads, streets and flow overland. Past practice has not consciously recognized in design detail the functioning of supplementary facilities in the major storm system which comes into operation when less frequent higher-intensity storms occur. Lack of conscious attention to supplementary functioning of the major storm drainage system should no longer be considered acceptable.

Except for man-made changes incidental to urban development, namely, the provision of streets and grading as it affects overland flow and provides swales and artificial channels, or closed storm sewers, routing of runoff from a major storm follows minor and major valleys of the design area. Except in very flat terrain, flow paths of natural valleys can be readily determined on topographic maps. It is of critical importance that major storm system flow paths are such in location and hydraulic character that accumulated excess runoff can find its way to a suitable outlet such as a major valley or lake. Major storm flow paths should not direct flows against houses or other structures; should not fill up low areas which have no suitable outlet; should not result in scour and subsequent sedimentation; should not make it impossible for emergency vehicles to get through streets.

Whenever practicable, swales and channels should have slow flow characteristics, be wide and shallow, and natural in appearance.

Estimates of runoff rates from a 100-year rainfall should be made for various reaches of the probable flow path. The probable hydraulic behavior of critical reaches should be examined.

The possibility of flooding property, streets and highways should be examined. Can practical, economic modifications be incorporated in the major drainage system to minimize or eliminate undersirable problems?

If topography permits consideration of alternate major drainage flow routings, they should be carefully evaluated through a field check. Social impacts on neighborhoods and general environmental design constraints should be determined. Ability of the major drainage system to serve the total tributary basin when a 100-year rain occurs, should be determined.

The estimated flow capacity of streets may be calculated from the Manning equation.

When designing the major system it should be done in consideration of the minor system, with the sum of their capacities being the total system's capacity. The minor system should be first designed to handle a selected high frequency storm; i.e., 2-year, next, the major system is designed for a low frequency storm; i.e., 100-year. If the roadway cannot handle excess flow, the minor system may be enlarged accordingly or other major storm flow paths selected such that accumulated excess runoff can find its way to a suitable outlet.

5-102 STORM SEWER DEFINITION

For purposes of interpretation of policies and procedures of this chapter:

A storm sewer system is a drainage system installed to carry storm water runoff, consisting of one or more pipes connecting two or more drop inlets. Cross drain pipes "hydraulically designed" to function as a culvert or culverts are an exception to this rule.

5-200 DETERMINATION OF RUNOFF

The first step to be considered in designing a storm sewer system is determination of runoff. The Rational Method, as described in Chapter 3 (Hydrology), of this manual is the method that usually applies to a vast majority of runoffs to be handled by storm sewers. Chapter 3 also covers applicability of the Soil Conservation Service Method. Either method may be used as a comparison against the other within their respective parameters.

HEC-1 Computer Flood Hydrograph Package, may be used for unit hydrograph computations, for hydrograph combining and routing.

5-300 PONDING

Roadway ponding may limit or stop traffic, limit access to property owners, cause accidents, damage personal property or the roadway itself and therefore must be a prime consideration in the storm sewer system.

The flow of water in the gutter should be restricted to a depth, and corresponding width, which will not severely obstruct or cause a hazard to traffic. This flow is a function of the quantity of water, gutter gradient, roughness of pavement where the flow is contained, and cross section shape of the flow area.

The following limiting widths are recommended for use at design flood stage:

a. For Interstate and Fully Controlled Access Projects:

Limit ponding to one-half the width of the outer lane - 50-year flood frequency.

b. Other Federal Aid Projects: Limit ponding to the width of the outer lane - 10 year flood frequency.

c. Non-Federal Aid Projects: Limit ponding to the width of the outer lane - 2 year flood frequency.

d. For Minor Two-Lane Highways and Streets: Limit ponding to a width and depth, which will allow passage of one lane of traffic with safety.

5-400 DESIGN FREQUENCY POLICIES

The following flood frequencies and criteria are recommended for design:

a. CROSS DRAINS

Interstate Projects: 50 year
Primary Projects: 50 year
Federal Aid Urban Projects: 25 year
Secondary Projects: 25 year
Non-Federal Aid Projects: 10 year*

*Drainage area less than two square miles; ADT less than 750. If either is exceeded, use 25 year flood frequency.

b. STORM SEWERS

Interstate and/or Interstate Type Projects: 50 year
Other Federal Aid Projects: 10 year
Non-Federal Aid Projects: 2 year

NOTE: If local drainage facilities and practices have provided drains of a lesser standard to which the highway system must connect, special consideration should be given to whether it is realistic to design the highway system to a higher standard than available outlets.

c. Storm sewers should be designed with conduit slopes sufficient to develop a self-cleaning velocity of three feet per second when flowing full.

d. Concrete storm sewers should be designed using Manning's Formula "n" factor of 0.013.

e. Recommended minimum time of concentration (T_c) for determining rainfall intensity using the Rational Method is 10 minutes for overland flow and 5 minutes for paved surface flow.

f. Check local ordinances for city streets and coordinate between design policies.

5-500 INLET TYPES AND CONDITIONS

5-501 GENERAL

Either curb opening inlets, grate inlets, slotted drain pipe inlets, or a combination of curb opening and grate inlets may be used for intercepting runoff. Curb opening inlets are preferred on grades 3% or less (Ref. 5-1) because of their self-cleansing ability as grate inlets clog easily. In some instances however, the use of grates will be found necessary either with or without curb openings in combination. Where grates are used, designing and placing grates should be such that grate bars will be parallel to the direction of flow of water rather than perpendicular to the flow and should be considered only 50% effective.

5-502 CURB INLETS IN SAGS

It is desirable that three inlets be placed in a sag vertical curve: One at the low point and one on each side of this point where the grade elevation is approximately 0.2 foot higher than at the low point (Ref. 5-1).

As a result, inlets in the sag of a highway must at times be designed to remove stormwater resulting from a large storm over the contributing area minus the flow intercepted by inlets on the grade which are designed to limit spreading water to a tolerable limit. Inlets on grade will intercept a greater quantity of water during the larger storm than the quantity used to determine their spacing, but the spread of water on the pavement will exceed the spread designated as the tolerable limit for design.

When a curb opening inlet is unsubmerged, capacity is governed by weir flow and may be calculated by the following formula:

$$Q_i = 3.0\ L\ d^{3/2}$$

where

Q_i = the inlet intercept, in cfs.
L = the length of the inlet, in feet.
d = depth of water at the inlet opening, in feet.

Chart 5-1, page 5-62 provides a graphical solution for depth of gutter flow. Then Chart 5-3, page 5-64 gives the estimated hydraulic capacity for each linear foot of curb opening.

When the opening is submerged, capacity is governed by orifice flow and may be calculated as follows:

$$Q_i = 5.37\ A\ (d - h/2)^{1/2}$$

where

A = the waterway area of the inlet opening, in square feet.
h = height of inlet opening, in feet; and all other terms as previously defined.

Chart 5-4, page 5-65 gives the minimum height (h_m) of opening required for weir-type operation. If the opening height (h) equals or exceeds h_m, Charts 5-5 and 5-6, pages 5-66 and 5-67 give the depth of ponding measured at the curb, just above the depressed area.

Charts 5-5 and 5-6, pages 5-66 and 5-67, are based on experiments made at Colorado State University and apply to depressed curb opening inlets with a height of opening equal or exceeding the appropriate h_m from Chart 5-4, page 5-65.

5-503 CURB INLETS ON GRADE

The curb opening necessary to remove all or part of the flow from the system can be determined by using Chart 5-2 on page 5-63.

5-504 GRATE INLETS IN SAGS

A grate type drop inlet located at a low point in grade will behave either as a weir or as an orifice, depending upon the depth of water at the grate. Capacity is independent of geometric configuration of the grate. For depths of water less than 0.4 feet, capacity is governed by weir flow and may be calculated by the following equation:

$$Q_i = 3.0\ Pd^{3/2}$$

where

P = perimeter of the grate opening ignoring the bars, in feet.

d = depth of water at the inlet opening, in feet.

Where one side of grate is against a curb, this side must be omitted in computing the perimeter.

When the depth of water exceeds 1.4 feet, grate capacity is governed by orifice flow and may be calculated by the following equation:

$$Q = 5.37\ Ad^{1/2}$$

where

A = the clear waterway area of the grate in square feet.

d = depth of water above the top of grate in feet.

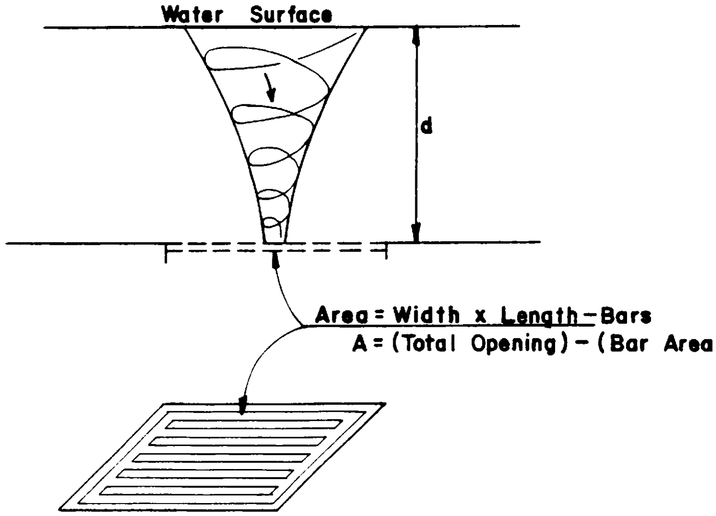


FIGURE 5-1

Between the depths of 0.4 feet and 1.4 feet, capacity of the grate is undefined and is assumed to be lesser of the values given by the above Equations. Because of uncertainty, and because of the tendency for trash to clog the opening, the effective perimeter or waterway area of the grate should be at least twice that given by the above equations. Chart 5-7, page 5-68 provides a graphical solution to these equations.

5-505 GRATE INLETS ON GRADE

Entrained water approaching an inlet at a velocity (V) will enter a sufficiently long inlet string along the direction of travel. Actual length of the grate can be estimated by the following equation:

$$L = \frac{m}{5.67} \quad Vd^{0.5}$$

where

- L = length of grate required to obtain efficient operation, in feet.
- m = a constant dependent upon grate configuration. Refer to Table 5-1 below.
- V = velocity of the approaching water in fps.
- d = maximum depth of approaching water at the curb in feet.

TABLE 5-1

VALUES OF "m" FOR VARIOUS GRATING CONFIGURATIONS

INLET DESCRIPTION	m
Combination inlets with bar width equal to or slightly less than the clearance between bars, and no large transverse bars flush with the surface.	3.3
Combination inlets with a bar width equal to or slightly less than the clearance between bars, and with several transverse bars.	6.6
Grated inlet without curb opening and with bar width equal to or slightly less than the clearance between bars, and no large transverse bars flush with the surface.	4.0
Grated inlet with a bar width equal to or slightly less than the clearance between bars, and with several transverse bars flush with the surface.	8.0

(From Reference 5-2)

The extra grate length for unpredicated hydraulic conditions or partial plugging will depend on site conditions, grate type, frequency of maintenance, etc. IT IS RECOMMENDED THAT THE DESIGN ALLOW A FACTOR OF SAFETY OF 1.5 OR MORE with respect to grate length.

The flow of water through grate openings may be estimated in the same manner as flow through rectangular orifices. The formula generally used for orifice flow is as shown below:

$$Q = CA\ (2gh)^{1/2}$$ (also known as maximum flowrate Q)

where

Q = discharge in cfs

C = coefficient of discharge, (approximately 0.7).

A = area of orifice (net opening area) in square feet.

g = acceleration due to gravity (32 feet per second per second).

h = depth of water directly over grate in feet.

This formula gives the theoretical capacity of the grate inlet but, in practice, the grate is shown to perform at about 50% less than expected. The design should be made with the knowledge that only 50% efficiency can be expected from the technical maximum design.

For hydraulic and safety characteristics of various type grates refer to Reference 5-12.

5-506 COMBINATION INLETS

Performance data on combined curb and grate inlets is very limited. Authentic data on true capacities of such combinations are insufficient to allow establishment of any very accurate factors for determining the true capacity of a combination inlet.

The capacity of a combination inlet on a continuous grade using an "efficient" grate is not appreciably greater than with the grate alone.

Combination inlets are typically used in a sag location. The curb-opening provides a relief opening if the grate should become clogged.

As a "rule of thumb" it is believed reasonable to assume that the capacity of the combination inlet will run about 50% of the sum of individual capacities of the grate and curb opening, computed in the manner described in proceeding paragraphs. In other words, compute the capacity of the curb opening inlet and the capacity of the grate inlet (without reduction for clogging) separately, add together, and the working capacity of the combination be taken as 50% of this figure.

5-507 SLOTTED DRAIN PIPE INLETS

A slotted drain inlet consists of a corrugated pipe cut along the longitudinal axis with a slot welded into the cut. When the drain is installed along a roadway, water falls through the slot and is carried to a collection point through the corrugated metal pipe. Normally, the main advantages of slotted drains are reasonable cost, ease of installation and efficiency.

Inlet capacity for slotted drains may be determined from the following equation when weir flow is controlling (minimum inlet length on grade = 10 feet):

$$L = \frac{0.706\ Q^{0.442}\ S_o^{\ A}\ Z^{\ 0.849}}{n^{0.385}}$$

where

- L = total flow capture length, in feet.
- Q = gutter flow, in cfs.
- S_o = longitudinal slope of gutter, in feet per foot.
- Z = reciprocal of cross slope, in feet per foot.
- n = Mannings roughness coefficient of gutter.
- A = exponent from Table 5-2, page 5-23 or
- $A = 0.207 + 0.007Z + 2.613\ S_o - 0.049\ S_o Z - 19.084\ S_o^{\ 2} - 0.0001Z^2.$

TABLE 5-2

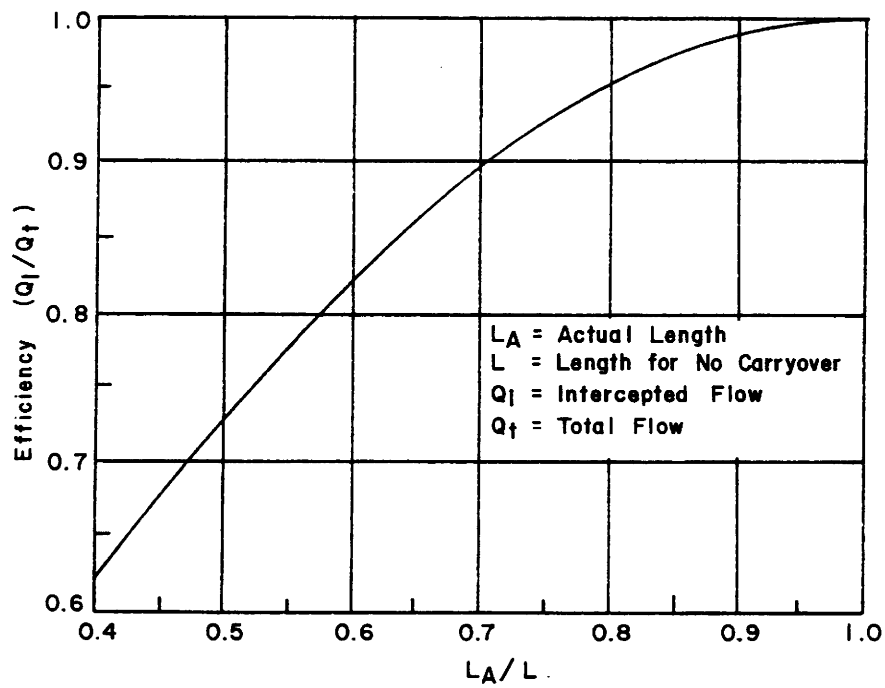
VALUES OF EXPONENT "A" FOR SLOTTED DRAIN PIPE INLETS			
LONGITUDINAL GRADE	RECIPROCAL OF CROSS SLOPE		
"S_o"	"Z"		
Feet Per Foot	33	40	50
0.005	0.334	0.330	0.308
0.010	0.337	0.332	0.306
0.015	0.340	0.333	0.305
0.020	0.341	0.332	0.302
0.025	0.342	0.331	0.299
0.030	0.341	0.329	0.294
0.035	0.340	0.326	0.289
0.040	0.338	0.323	0.283
0.045	0.336	0.318	0.276
0.050	0.331	0.312	0.267
0.055	0.326	0.305	0.258
0.060	0.320	0.297	0.248
0.065	0.313	0.289	0.237
0.070	0.305	0.279	0.224
0.075	0.297	0.269	0.212
0.080	0.287	0.257	0.198
0.085	0.276	0.245	0.183
0.090	0.263	0.231	0.166

Exponents shown were calculated by the following formula (Ref. 5-7):

$$A = 0.207 + 0.007\,Z + 2.613\,S_o - 0.049\,S_o Z - 19.084\,S_o^2 - 0.0001\,Z^2$$

A slotted drain is a more efficient flow interceptor in the upstream portion of the slot. About 70 percent of flow is captured in 46 percent of the length required to capture all the flow. The advantages of carryover are as shown:

$$E/100 = 1 - 0.918\left(1 - \frac{LA}{L}\right)^{1.769}$$



SLOTTED DRAIN CARRYOVER EFFICIENCY
(From Ref. 5-7)

FIGURE 5-2

If carryover is to be permitted, assume a length (L_A) such that L_A/L is less than 1.0, but greater than 0.4. Pipe diameter is usually not a factor but it is recommended that an 18" minimum be used. It should be carefully noted that generally, economics favor slotted drain pipe inlets designed with carryover rather than total flow interception. Make certain there is a feasible location to which the carryover may be directed.

The orifice equation can be used to determine the flow capacity of slots in a completely submerged sump condition:

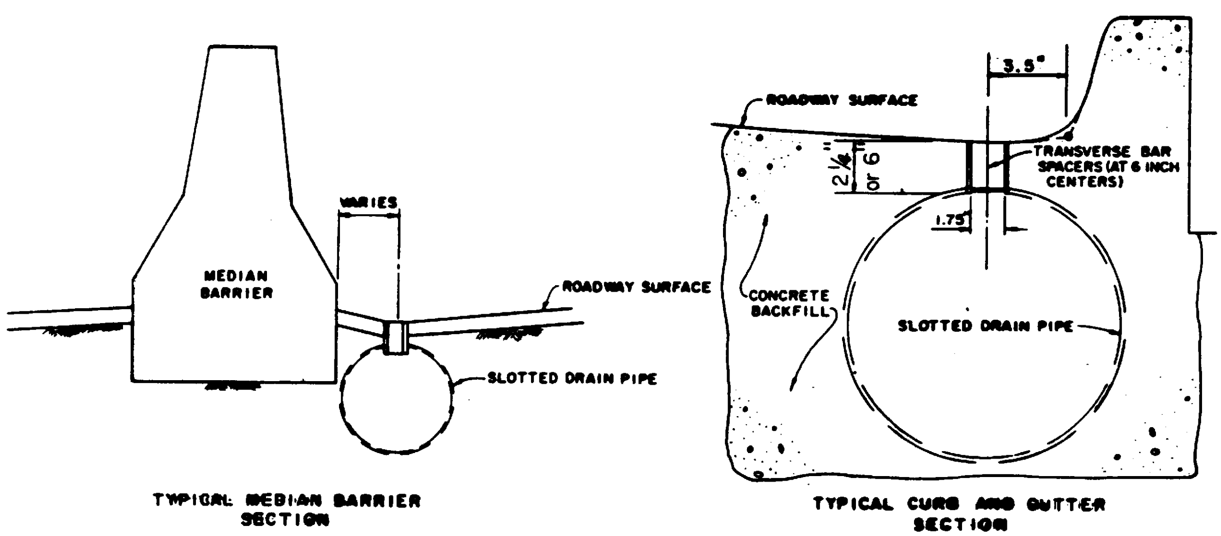
$$Q = CLW\,(2\,gd)^{0.5}$$

where

- C = coefficient of discharge (0.61 - less than or equal to 0.4 ft. depth; 0.79 - greater than 0.4 ft. depth (Ref. 5-6 and 5-7).
- L = total length, in feet.
- W = slot width, in feet.
- g = acceleration due to gravity (32 feet per second per second).
- d = flow depth over slot, in feet.

For sag inlets, length should be at least 2.0 times calculated L to insure against debris hazard. In no case for sags should L be less than 20 feet.

The slot should be paralled to the curb and located in the gutter approximately as shown:

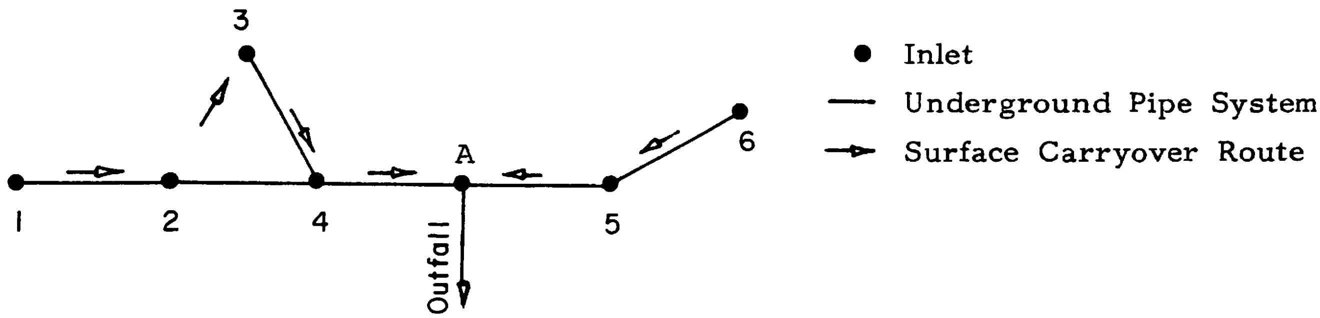


Typical slotted drain installations.

FIGURE 5-3

A hypothetical pipe layout is analyzed to demostrate the method of application of formulas and to provide an overview of the final hydraulic design procedure.

The storm sewer segment shown exhibits two basic situations encountered in slotted inlet design. One is inlets on grade and two is an inlet in a sag.



INLET COMPUTATIONS

INLET ID	Q	C.O.	Q_T	S_o	Z	d	P	A(a)	L(b)	L_A	E(c)	C.O.	C.O. to
1	3.2	-	3.20	0.0125	50	0.20	10.0	0.306	45.56	30	0.86	0.45	2
2	2.3	0.45	2.75	0.0125	50	0.19	9.5	0.306	42.62	25	0.81	0.52	3
3	1.0	0.52	1.52	0.005	50	0.18	9.0	0.307	24.65	15	0.82	0.27	4
4	1.2	0.27	1.47	0.0125	50	0.15	7.5	0.306	32.32	20	0.84	0.24	A
6	3.9	-	3.90	0.0125	50	0.21	10.5	0.306	49.73	30	0.82	0.70	5
5	1.6	0.7	2.30	0.005	50	0.23	11.5	0.307	29.61	20	0.87	0.30	A
A	6.1	0.24/0.30	6.64	-	50	0.24	12.0	-	18.99(d)	35	1.04(e)	-	low point

(a) From Values of Exponents, Table 5-2, page 5-23

(b) $L = 0.706\ Q^{0.442}\ S_o^{A}\ Z^{0.849} / n^{0.385}$

(c) E = Efficiency - See Figure 5-2, page 5-24.

(d) $L = 2.86\ Q_t$, from $Q = CA\ (2\ gh)^{0.5}$, $C = 0.61$, $A = L\ \frac{1.75}{12}$,

$h = 0.24$

(e) Safety factor for low point inlet.

use: n = 0.013, maximum ponding = 12 feet

5-508 CURVED VANE INLET GRATES

For purposes of hydraulic analysis, it is convenient to consider the flow intercepted by an inlet grate as consisting of two parts: (1) frontal flow or that portion of the interecepted flow which passes over the upstream front edge of the grate, and (2) side flow or that portion of the intercepted flow which passes over the edge of the grate parallel to and away from the curb.

The hydraulic efficiency, E, of a grate is defined as the ratio of the total flow intercepted, Q_i in cfs to the total gutter flow, Q_T also in cfs,

$$E = Q_i/Q_T$$

The percent of frontal flow intercepted depends mainly on bar configuration, grate length and velocity of flow. On mild slopes normally 100% of the frontal flow will be intercepted. On steep slopes the higher velocity flow may cause the water to splash over the grate. When splash-over occurs, only a portion of the frontal flow is intercepted.

The amount of side flow intercepted increases as the length of the grate increases and it decreases as the velocity of flow increases.

For grates on a continuous grade, the quanity of flow intercepted increases as the spread increases and for this reason, economy of design usually requires that a percentage of the approach gutter flow be allowed to flow around the inlet and be subsequently picked up by downstream inlets or at the sag. The spacing of grate inlets on continuous grades is therefore determined by the allowable width of water on the pavement and the efficiency of the inlets.

From the modified Manning equation for gutter flow it can be derived that the ratio of approach frontal flow, Q_F in cfs, to total gutter flow, Q_T in cfs, is:

$$\frac{Q_F}{Q_T} = 1 - \left[1 - \frac{W}{T}\right]^{8/3}$$

where W is the width of the grate in feet and T is the width of water or spread in feet on the pavement. This ratio is equal to the theoretical efficiency of a grate inlet assuming 100% frontal flow interception and no side flow interception.

Side flow may be considered in the above equation by substituting an effective width W_E for W. The effective width W_E in feet is equal to W plus ΔW where ΔW is the extra grate width in feet which would be necessary for the inlet to have the same efficiency without side flow interception. W is a constant for any given longitudinal slope, cross slope, grate size and bar configuration. The equation for estimating grate inlet efficiency, E_O, without splash-over is therefore:

$$E_O = 1 - \left[1 - \frac{W_E}{T}\right]^{8/3}$$

This equation may be solved graphically using Figure 5-3-2, page 5-26.7. Values of ΔW may be obtained from Figure 5-3.3a, page 5-26.8.

The efficiency of the inlet under slash conditions depends on frontal velocity and is computed by multiplying E_O by a reduction factor R. Frontal flow velocity is the average flow velocity of that portion of the intercepted flow which passes over the upstream front edge of the grate. Figure 5-3.3b, page 5-26.8 give R as a function of V_F, the average frontal flow velocity in feet or per second. This latter can be obtained by multiplying the average flow velocity in the gutter, V_{av} in feet per second by the coefficient K as given in Figure 5-3.4, page 5-26.9. The equation for computing frontal flow velocity is

$$V_F = KV_{av} = K(2Q_T Z/T^2)$$

in which Q_T is the total gutter flow in cfs: Z is the reciprocal of the cross slope; T is the spread in feet.

5-508.1 FACTOR OF SAFETY

Grate inlets should be designed to allow for unpredicted hydraulic conditions or partial plugging. The latter may considerably reduce inlet efficiency. Grate lengths longer than necessary for 100% frontal flow interception will allow for some debris accumulation. The grate length necessary to intercept 100% of the frontal flow is given in Figure 5-3.5, page 5-26.9.as a function of frontal flow velocity. In this figure L' is the effective or unclogged grate length, which is assumed in the design.

The extra grate length needed will depend on site conditions, grate type, frequency of maintenance, etc. It is recommended however, that the design allows a factor of safety of 1.5 or more with respect to grate length (Reference 5-1).

5-508.2 SELECTION OF GRATE TYPE

Grate type selection should consider such factors as hydraulic efficiency, pedestrian and bicycle safety, debris handling characteristics and fabrication costs.

Reference 5-1 compares the relative hydraulic efficiencies of the various grate types. The curved vane grate has good hydraulic characteristics with high velocity flows. The other grates tested are hydraulically effective at lower velocities.

Debris-handling capabilities as determined in Reference 5-1 show a clear difference in efficiency between the grates with the 3-1/4 inch longitudinal bar spacing and those with small spacings. In general, the increased flow velocity at the 4% slope results in a higher debris-handling efficiency. The curved vane grate ranks highest in debris-handling capabilities of all grates tested.

The curved vane grate ranks sixth according to relative bicycle and pedestrian safety. Whereas bicycle safety gratings were based on a test program, evaluation of pedestrian safety was arrived at subjectively (Reference 5-1).

5-508.3 EXAMPLE PROBLEM

GIVEN: Q_T = 3.5 cfs; S_o = 4.5%; Z = 50; n = 0.015

Try 24" x 48" curved vane grade Figure 5-3.1, page 5-26.6.

FIND: Intercepted flow, Q_i assuming:

a) no clogging and b) only 50% of the grate length is effective due to clogging.

SOLUTION:

1. Compute T and V_F

$$T = \left[\frac{Qn\ Z^{5/3}}{0.56\ S_o^{1/2}}\right]^{3/8} = \left[\frac{3.5(0.015)(50)^{5/3}}{0.56\ (0.045)^{1/2}}\right]^{3/8} = 8.5 \text{ ft.}$$

K = 1.23 (From Figure 5-3.4, page 5-26.9).

$$V_F = \frac{2KQZ}{T^2} = \frac{2(1.23)(3.5)(50)}{(8.5)^2} = 6.0 \text{ ft./sec.}$$

2. Determine Q_i without clogging:

$L = 4.0'$

$\Delta W/L' = 0.11$ (From Figure 5-3.3a, page 5-26.8)

$W_E = W + \Delta W = 2 + 0.11(4) = 2.44$

$E_o = 0.60$ (From Figure 5-3.2, page 5-26.7).

$R = 1.00$ (From Figure 5-3.3b, page 5-26.8).

$E = E_o R = 0.60(1.00) = 0.60$

$Q_i = E\ Q_T = 0.60(3.5) = 2.10$ cfs

3. Determine Q_i with 50% of the length effective due to clogging:

$L' = 4(0.5) = 2.0$ ft.

$\Delta W/L' = 0.11$ (From Figure 5-3.3a, page 5-26.8).

$W_E = W + \Delta W = 2+0.11(2.0) = 2.22$

$E_o = 0.56$ (From Figure 5-3.2, page 5-26.7).

$R = 0.97$ (From Figure 5-3.3b, page 5-26.8)

$E = E_o\ R = 0.56(0.97) = 0.54$

$Q_i = E\ Q_T = 0.54\ (3.5) = 1.89$ cfs

Since the 4-foot grate length is over 60% longer than the minimum grate length without splash-over (see Figure 5-3.5, page 5-26.9); and side flow interception is small, only a slight reduction in efficiency (1.9 verses 2.2 cfs) results from the reduced effective length.

4. Determine actual grate size required to intercept 70% of the gutter flow assuming 50% of the grate length is effective.

$\underline{W_E = \text{REQUIRED}}$

E_O = (grate inlet efficiency 70%) = 0.7

W_E = 3.08 (from Figure 5-3.2, page 5-26.7).

GRATE LENGTH REQUIRED (From Figure 5-3.3b, page 5-26.8).

V_F = 6.0 ft./sec.

$\Delta L'$ = 2.2 ft. Therefore, the required grate length is:

2.2/0.50 = 4.4 ft. = or approximately 53 inches

GRATE WIDTH, W, based on L'

$\Delta W/L'$ = 0.11 (Figure 5-3.3a, page 5-26.8).

ΔW = 0.11 (2.2) = 0.24 ft.

W = 3.08-0.24 = 2.84 (say 34")

A curved vane grate, 34" wide and 53" long is required. Use 36" x 48". Since this grate is wider than one normally used by the Department, it is suggested that another grate upstream be installed to capture a portion of this flow, thereby reducing the required grade size at this location.

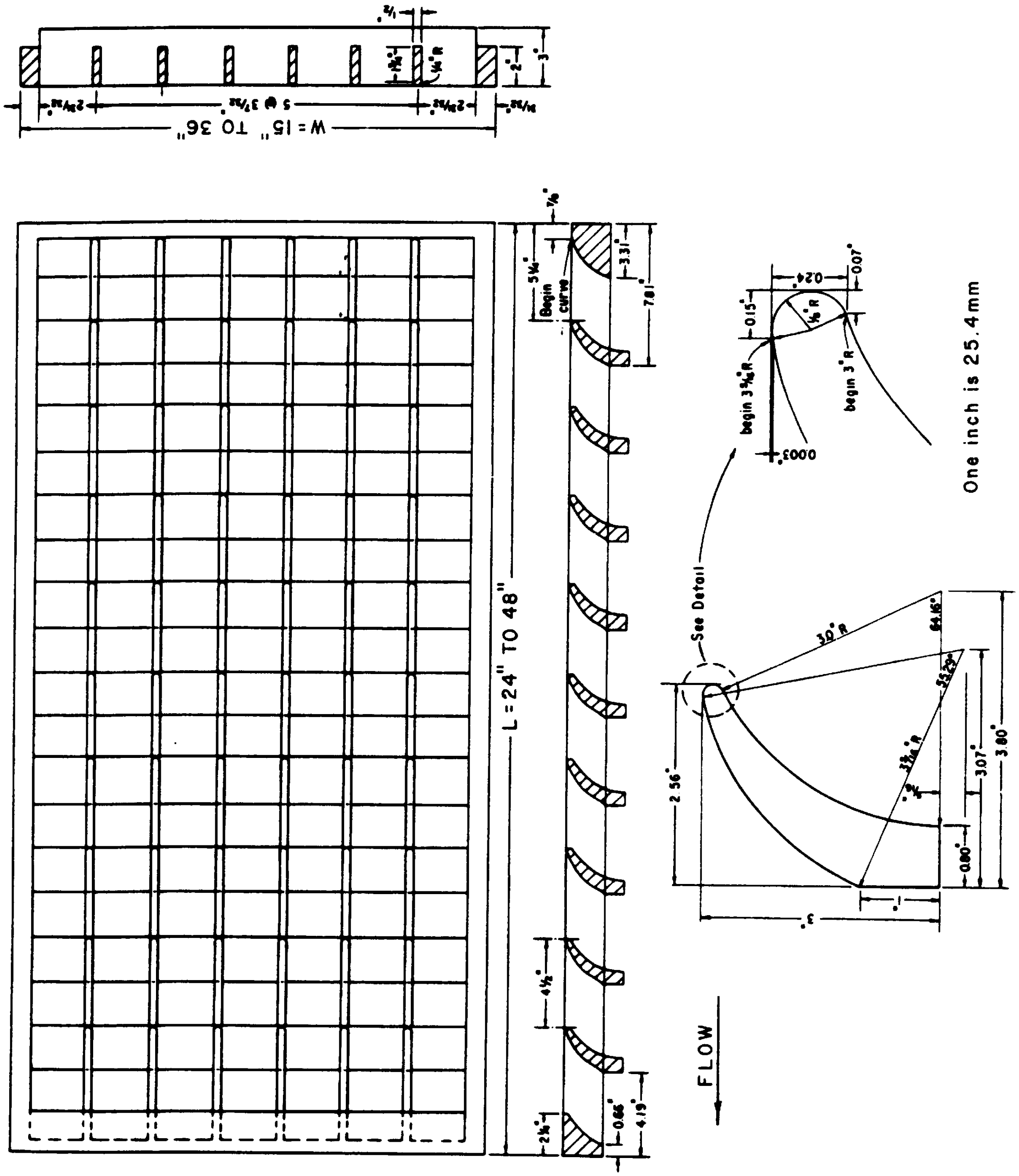


FIGURE 5-3.1

CURVED VANE GRATE

GRATE INLET DESIGN CURVES

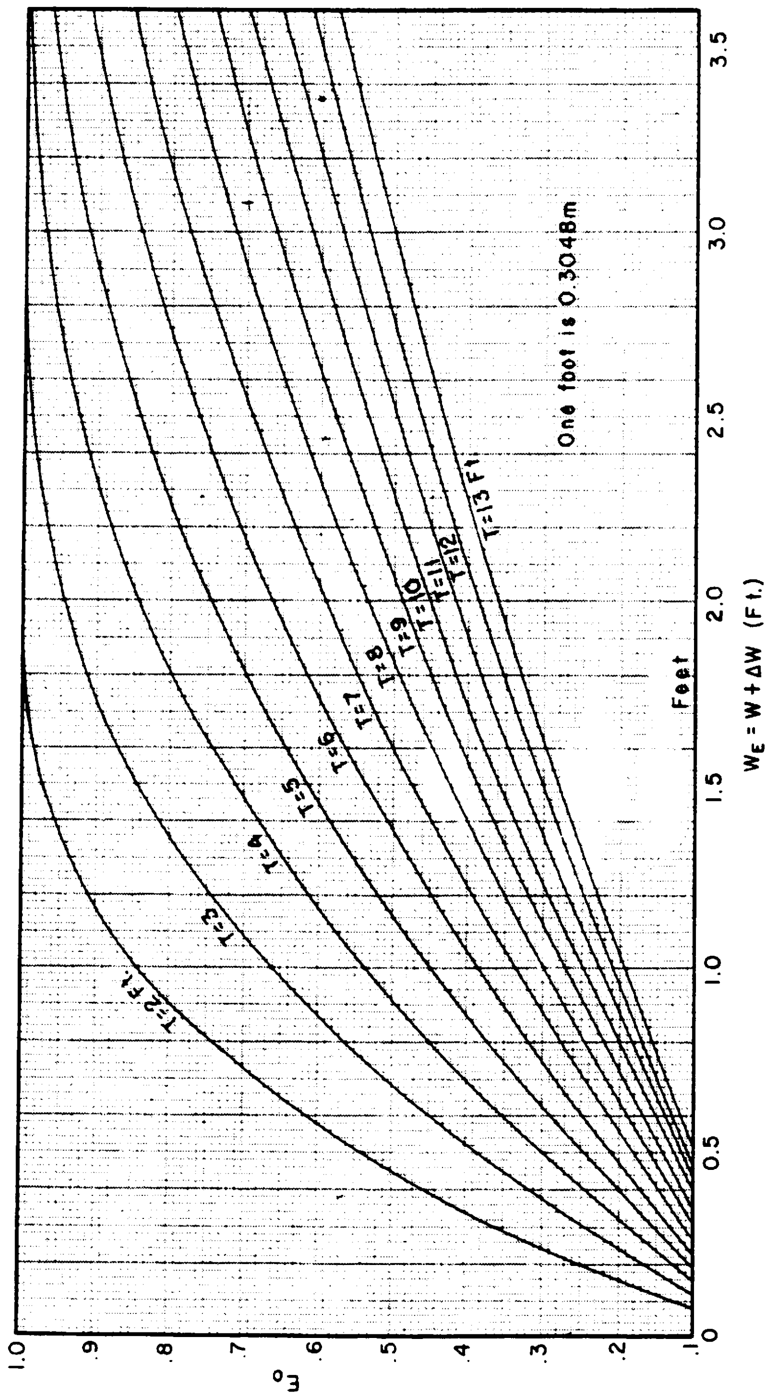
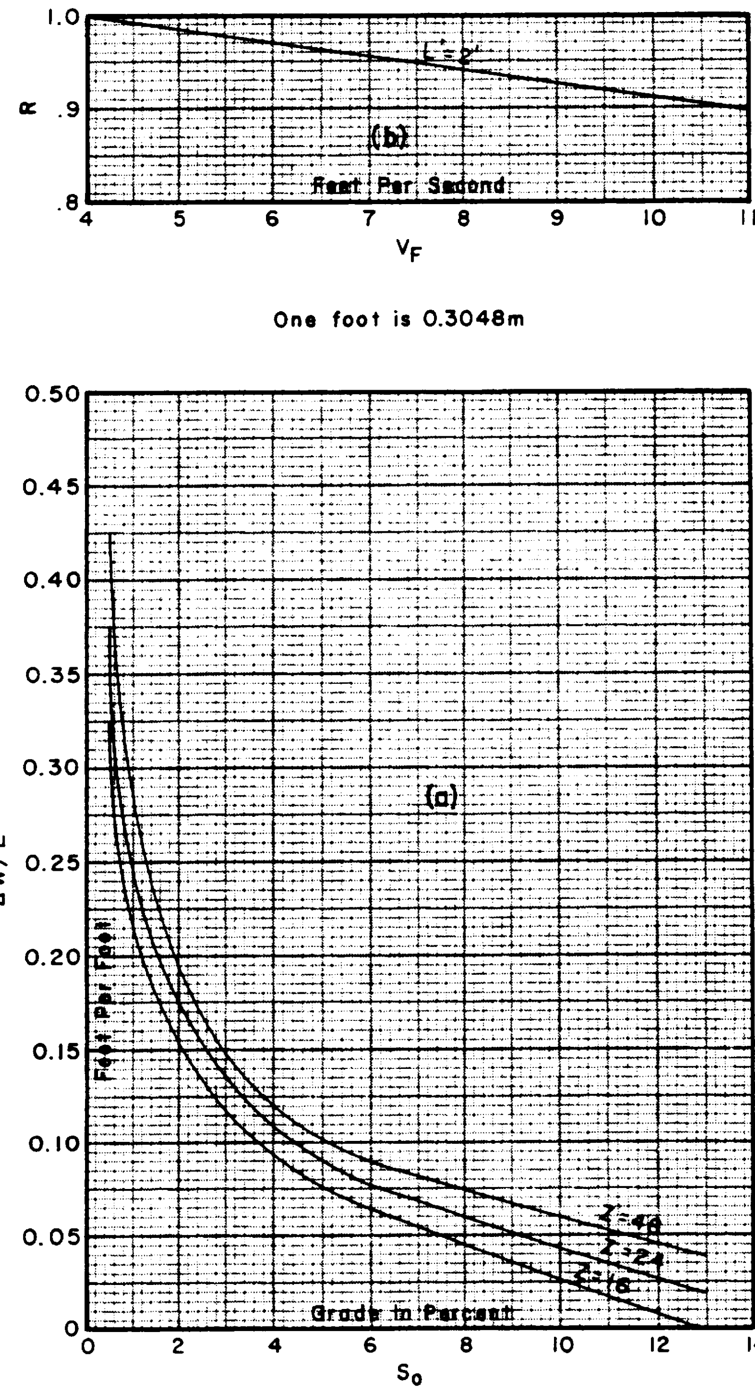


FIGURE 5-3.2

GRATE INLET DESIGN CURVES



CURVED VANE GRATE

FIGURE 5-3 3(a) and (b)

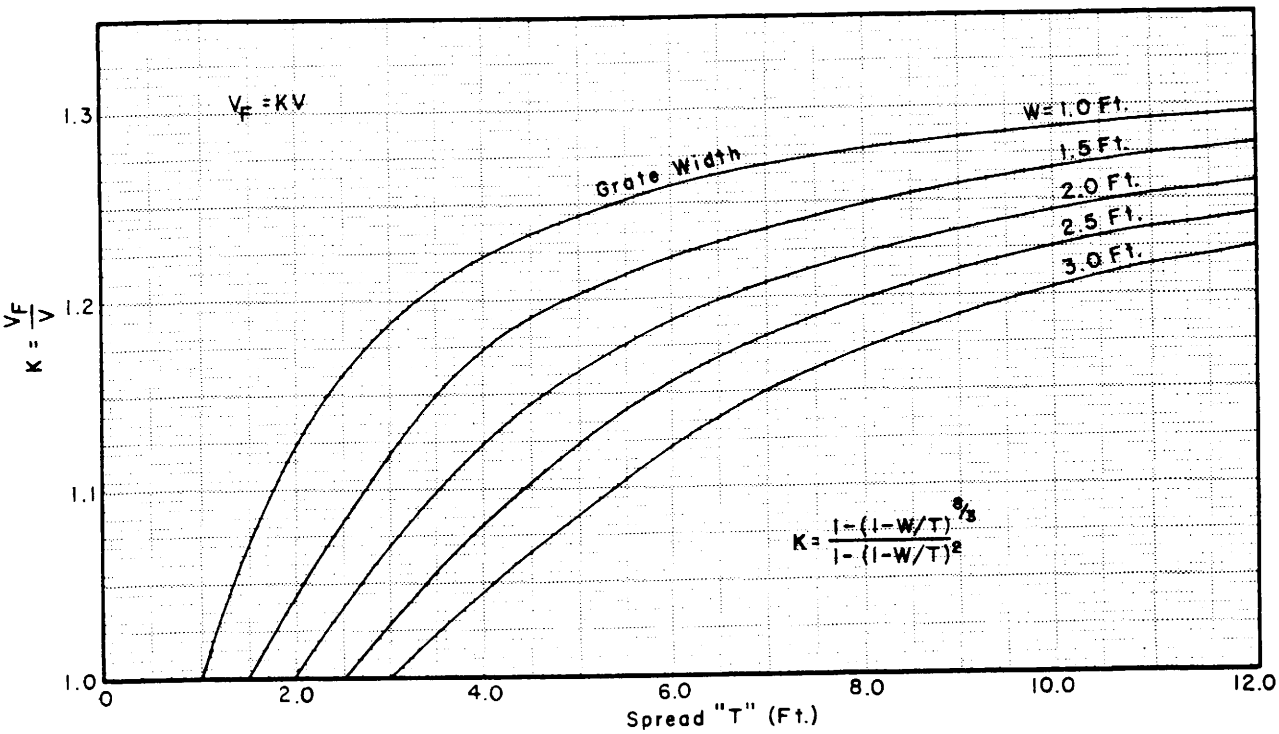


FIGURE 5-3 4 SPREAD vs K (Frontal Flow Velocity Coefficients)

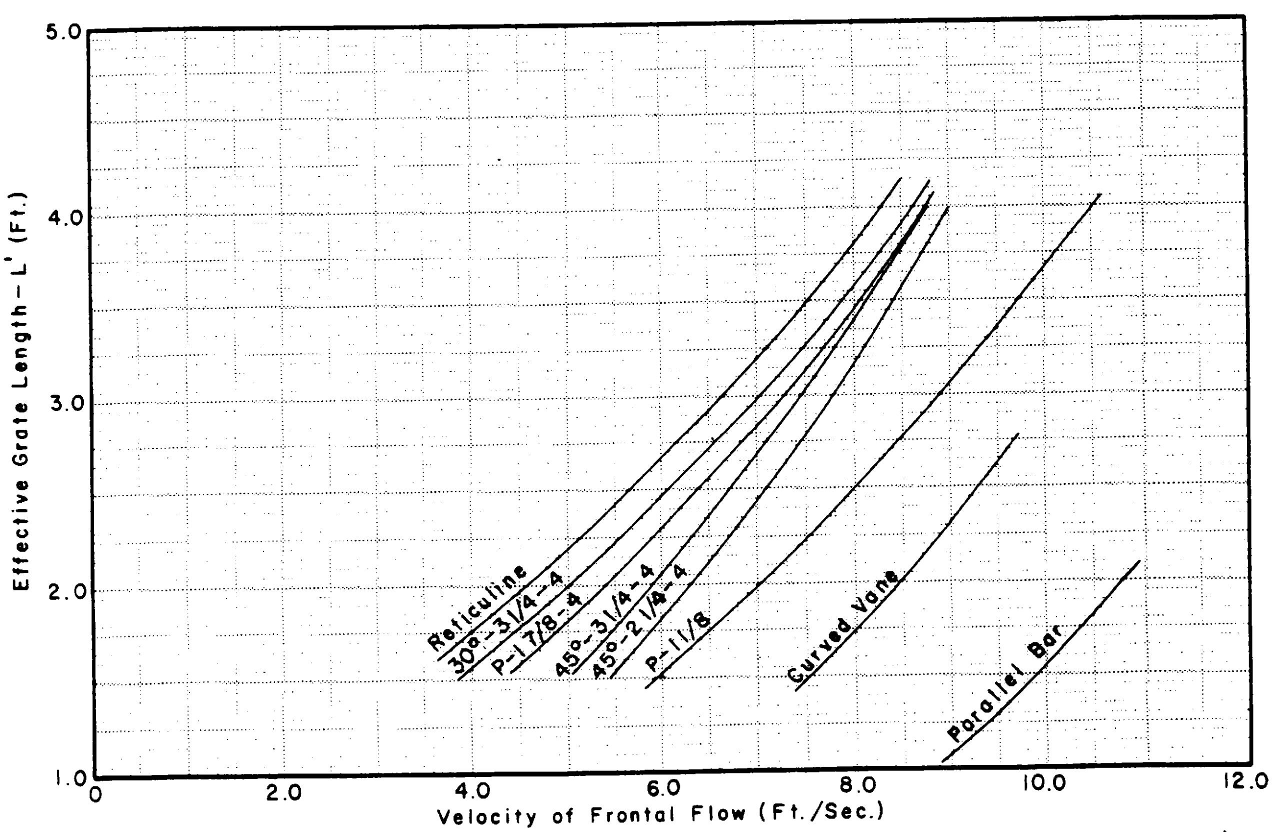


FIGURE 5-3.5 MINIMUM EFFECTIVE GRATE LENGTH (Without Splashover)

5-600 DESIGNING STORM SEWER RUNS

5-601 BASIC CONCEPTS OF STORM SEWER DESIGN

a. Do not use pipe sizes less than 18" diameter except in special cases.

b. To determine minimum pipe size relative to flow, use critical depth of flow (circular) Chart 5-8, page 5-69, with d_c/D ratio equal to 0.8; where d_c is critical depth and D is the diameter.

c. If the pipe gradient is known (or set by physical controls), size pipe on full flow, but the diameter determined by this step shall not be less than the diameter in Item b above.

d. At changes in size of pipe or box, always place soffits or top inside surfaces of the two pipes at the same level rather than placing flow lines at the same level. This technique will help prevent backwater profiles from rising and upstream velocities from decreasing. Naturally this rule cannot be followed in every instance, but it should be adhered to where practicable.

e. Do not discharge the contents of a larger pipe or box into a smaller one even though the capacity of the smaller pipe or box may be greater due to a steeper slope. Special consideration should be given to available outlets of an existing system.

f. Calculate the hydraulic gradient of each storm sewer run to determine water surface elevations at critical points, such as junction structures and drop inlets.

5-602 MINIMUM GRADE

All pipes or boxes should be designed such that velocities of flow will not be less than 3 feet per second at design flow or lower (Refer to Table 5-3 below). For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm system should have flatter slopes than slopes in lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deters settling of particles due to steadily increasing flow streams.

TABLE 5-3

SLOPES REQUIRED FOR V = 3fps AT FULL AND HALF FULL FLOW

"n" = 0.013

Pipe Diameter (Inches	Slope in Percent
18"	0.26
24"	0.17
30"	0.13
36"	0.10
42"	0.08
48"	0.07
54"	0.06
60"	0.05
66"	0.05

The minimum slopes are calculated by the modified Manning Formula:

$$S = \frac{(nV)^2}{2.208\ R^{4/3}}$$

where

S = slope in ft./ft.

N = Manning's roughness coefficient

V = velocity in feet per second

R = hydraulic radius

5-603 DESIGN PROCEDURES

Generally each drainage basin contribution to a roadway system will have its own time of concentration, average slope, total area, rainfall intensity, runoff C-factor, and dump point. Water arriving at the dump point may be picked up in curb openings, curb inlets, side ditches, or any combination of the three. It may become necessary to place several standard size inlets in sequence to pick up the flow, in parts, as the total flow travels down the gutter next to the curb. The accumulating water in the gutter must be kept in check and not be allowed to exceed maximum gutter flow width as shown on page 5-14.

Several pipes may deliver flow from multiple inlet locations to one inlet or junction box. For each inlet location, water will have traveled a given path over a given length of time. This total run time is used in calculating the intensity which, in turn, is used in the $Q = CIA$ equation to calculate the amount of flow at that inlet location.

The storm sewer segment shown in Figure 5-4 below exhibits three basic situations encountered in storm sewer design. One, a junction box is receiving flow directly from another junction box. Two, a junction box is receiving flow directly from a single standard inlet. Three, a junction box is receiving flow from an in-line combination of multiple inlets.

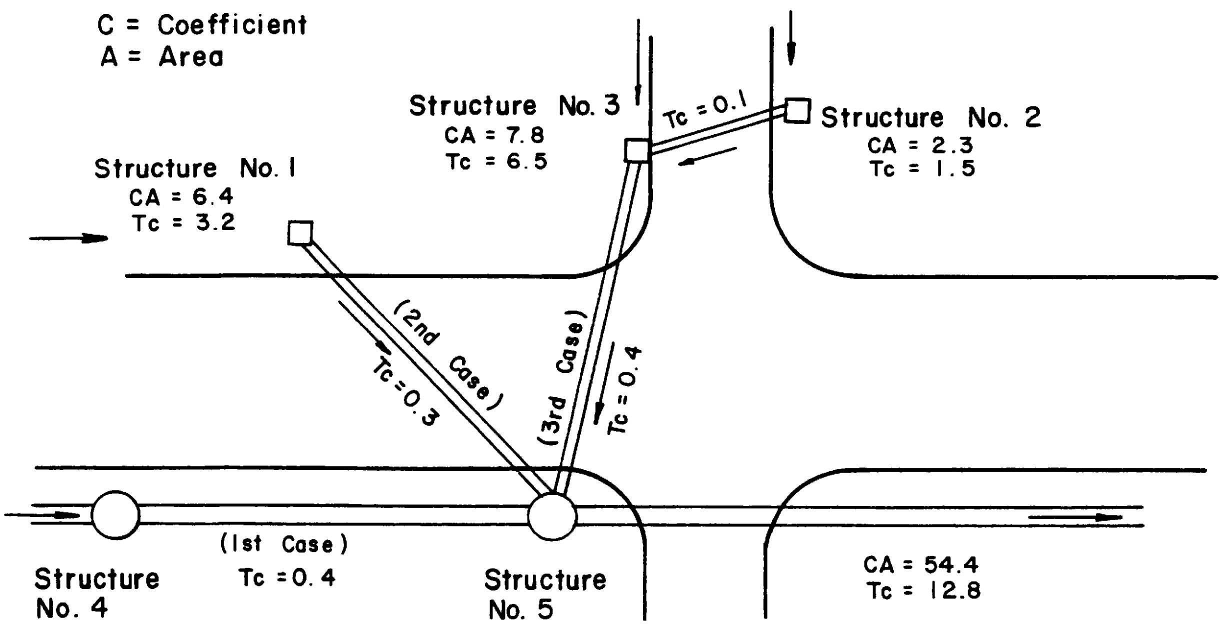


FIGURE 5-4

In the first case, flow coming from an upstream junction box is an accumulation of captured flow from upstream inlets and structures. Associated with incoming flow is overland time of concentration plus actual run time in the pipe. The procedure is to add the rational formula CA products - as each inlet contributes to the storm sewer and use the longest time of concentration up to that point in the system.

Up to Structure 4, the accumulated CA product is given as 32.6 and the time of concentration is 12.4 minutes. Using the appropriate intensity chart for the location of the system, intensity is calculated for the system ahead of Structure 4. The flowrate Q calculated by the rational formula Q = CIA will be the design Q for sizing the pipe leaving Structure 4.

The pipe leaving Structure 4 should be flowing no more than 80 percent full for the assumed construction slope. Travel time within the pipe is calculated by dividing the length of pipe string by the velocity of flow in the pipe. The total time of concentration at the junction box is the accumulated total of 12.4 minutes plus the time in the pipe of 0.4 minutes.

In the second case, the drainage basin CA product of 6.4 is carried to the inlet, Structure 1. This inlet may or may not be able to accept all of the flow reaching it. If all the flow is accepted then the CA product carried forward is the same CA product as the drainage area. If the inlet capacity is smaller than the Q reaching the inlet, then the flowrate Q accepted by the inlet divided by intensity gives an adjusted CA product to be carried in the pipe to the junction box of Structure 5.

The third type of condition is shown as multiple inlets feeding into a junction box. Shown in the example is inlet Structure 2 passing through inlet Structure 3, which in turn runs directly into the junction box, Structure 5.

The example shows overland time of concentration to the inlet to be 1.5 minutes and a CA product of 2.3. These values are used to calculate flowrate Q arriving at the inlet location. The amount of flow accepted by the inlet may be total or partial. In either case the run time is shown to be 0.1 minute to the next inlet in the string which is Structure 3.

Structure 3 must pick up any bypass flow from Structure 2 and pick up any direct flow from adjacent drainage areas. Drainage area data shows an overland time of concentration of 6.5 minutes and a CA product of 7.8 to the inlet from the drainage basin. Since the pipe runs directly through the second inlet, CA products are added and a CA of 10.1 is carried down the pipe to the junction box. Since Structure 3 has a larger time of concentration than Structure 2, the time of concentration of Structure 3 is carried down the pipe string to the junction box. Using the time of concentration for Structure 3 and the combined CA product of Structures 2 and 3, a flowrate Q using Q = CIA, is determined for sizing the pipe running to Structure 5.

At this point, the junction box is being fed by three strings, each having its own associated time of concentration and CA product. The pipe leaving Structure 5 is sized using a flowrate Q based on the Q = CIA formula. The accumulated total of CA products from all three strings is used in the Q determination and the longest time of concentration of all strings is used as the time of concentration value, from which intensity is derived. The final flowrate Q is calculated from Q = CIA or CA x I.

In tabular form the data would appear as follows:

At 5 from 1,	Tc	=	3.2	+ 0.3	=	3.5	min.,	CA Prod.	= 6.4
At 5 from 3,	Tc	=	6.5	+ 0.4	=	6.9	min.,	CA Prod.	= 10.1
At 5 from 4,	Tc	=	12.4	+ 0.4	=	12.8	min.,	CA Prod.	= 32.6
								TOTAL CA PROCUCTS	= 49.1

From preceeding table, the largest time of concentration is shown to be 12.8 minutes and the accumlated CA product is 49.1 units. For a 10-year design storm at a Tc of 12.8 minutes, intensity is found to be 6.45 inches per hour. By multiplying the CA product times intensity factor "I", flowrate "Q" can be determined for the junction box location.

5-604 HYDRAULIC GRADIENT

The hydraulic gradient is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a storm sewer run. The difference in elevation for water surfaces in successive tubes represents friction loss for that length of sewer, and the slope of the line between water surfaces is the friction slope. Therefore, if a storm sewer run were placed on a calculated friction slope corresponding to a certain quantity of water, cross-section, and roughness factor, the surface of flow (hydraulic gradient) would be parallel to the top of the conduit, and the sewer run would not be under pressure. This is desirable. If there is resson to place the sewer run on a slope less than friction slope, then the hydraulic gradient would be steeper than the slope of the sewer run. Depending on the elevation of the hydraulic gradient at the downstream end of the run in question, it is possible to have the hydraulic gradient go above the top of the conduit which would mean the sewer is under pressure until, at some point upstream, the hydraulic gradient is once again at or below the top of the conduit.

Hydraulic design of a storm sewer system is a trial and error procedure. For example, the first choice of pipe sizes may require modification in one or more reaches after a trial hydraulic gradient is computed. The estimated hydraulic losses in some structures may be excessive. The most likely will involve too large a velocity head in one or more of the pipes entering or leaving a manhole, junction or inlet. An increase in pipe size may be the most satisfactory way to correct this problem. If a pipe size is increased, time of flow in the reach will increase because of the slower velocity; and, consequently, the time of concentration and runoff will change.

Therefore, it becomes necessary to compute the hydraulic grade line of the entire storm sewer system. This is done by starting with the tailwater elevation (no lower) at the point where the storm sewer is finally discharged or 0.8 diameter of pipe plus invert out elevation of the outfall pipe, whichever is greater and working back up along the entire length of the storm sewer, computing head losses (friction and/or form losses) for each run and plotting the elevation of the total head thus computed at each manhole, junction or inlet.

If the hydraulic grade line as thus plotted does not rise above the top of the gutter line or top of grate minus 9 inches (freeboard recommended for design) the storm sewer system is considered satisfactory. Wherever it does rise above these points, blowouts through inlet throats, grates or manhole covers will occur and pipe sizes or gradients should be increased as necessary to eliminate such blowouts.

Empirical head loss equations based on testing and detailed application of hydraulic theories have been developed to aid the designer in his analysis. Following is a list of various types of head losses and equations that may be used for their determination.

5-604.1 FRICTION LOSSES

Energy losses from pipe friction may be determined by rewriting the Manning Equation.

$$S_f = \left[\frac{Qn}{1.486\ AR^{2/3}}\right]^2$$

Then the head losses due to friction may be determined by the formula:

$$H_f = S_f L$$

where

H_f = friction head loss

S_f = friction slope

L = length of outflow pipe

5-604.2 VELOCITY HEAD LOSSES - GENERAL

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach Equations.

$$H = K \frac{V^2}{2g}$$

where

H = velocity head loss

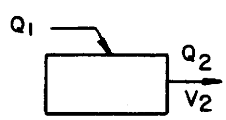
K = coefficient for the particular structure

V = velocity of flow

g = acceleration due to gravity (32.2 feet per second per second)

5-604.3 ENTRANCE LOSSES FOR BEGINNING FLOWS

$$H_{tm} = \frac{V^2}{2g}$$



$$H_e = K \frac{V^2}{2g}$$

where

H_{tm} = terminal (beginning of run) loss

H_e = entrance loss for outlet structure

K = 0.5 (assuming square-edge); and all other terms as previously defined.

5-604.4 JUNCTION LOSSES FOR INCOMING OPPOSING FLOWS

The head loss at a junction, H_{j1} for two almost equal and opposing flows meeting head on with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$H_{j1} = \frac{V_3^2}{2g} \text{ (outflow)}$$

An example and application is described as follows:

> A junction with a 90 degree turn has two 30" R.C. pipes feeding into it with a composite flow of 46 cfs (based on the CA products multiplied by the proper intensity factor at this junction). Since we know the velocity loss is total within the junction (K = 1.0) and by sizing the outlet pipe relative to pipe sizing rules, we find that the pipe required is a 36" diameter with a velocity of 8.2 fps. The loss is computed as follows:

$$H_{j1} = \frac{(8.2)^2}{2(32.2)} = 1.04 \text{ feet}$$

This means that within the junction the water surface must draw-down 1.04 feet to develop the energy necessary to get the flow moving through the outlet pipe. We may drop the outlet pipe 1.04 feet below the top of the lowest feeding pipe to develop this draw-down as shown in Figure 5-5, page 5-36.

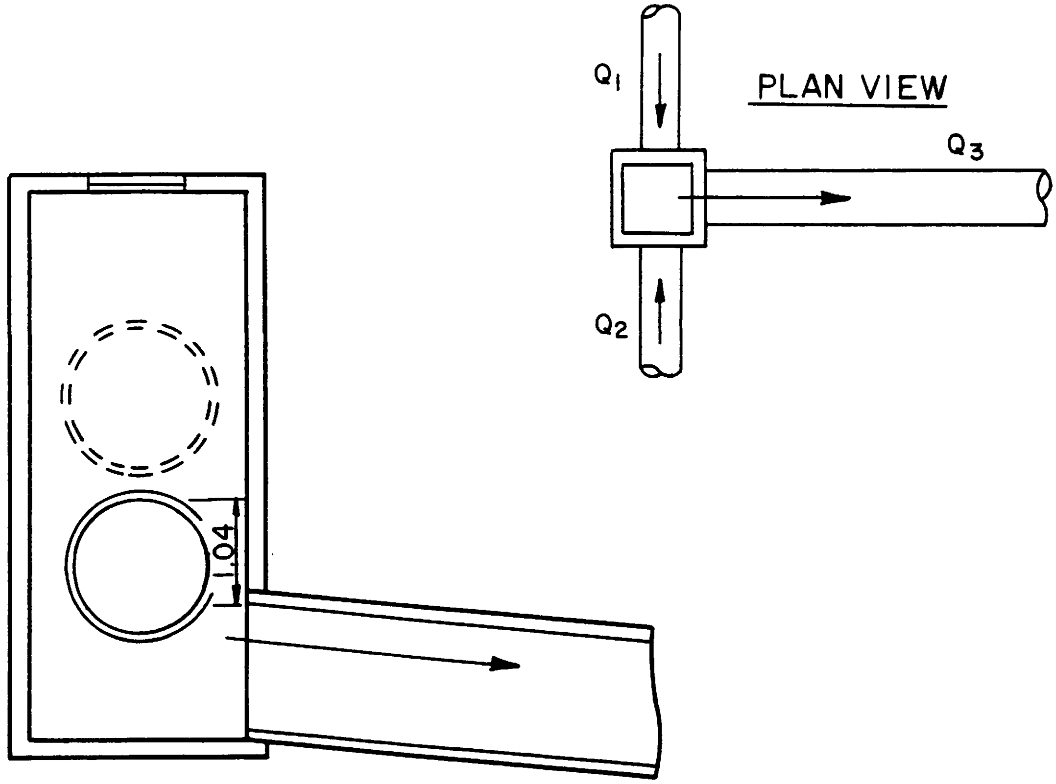


FIGURE 5-5

5-604.5 JUNCTION LOSSES FOR CHANGES IN DIRECTION OF FLOW

When storm sewer main or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90°) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$H_b = K \frac{V^2 \text{ (outlet)}}{2g}$$

where

H_b = bend head loss, in feet
K = junction loss coefficient

Listed in Table 5-4, page 5-37 are values of K for various junction angles.

TABLE 5-4

VALUES OF K FOR CHANGE IN DIRECTION OF FLOW IN LATERAL

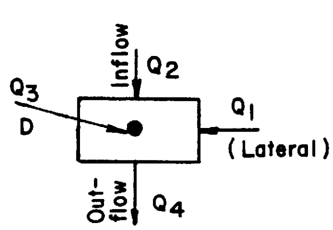
K	Degree of turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

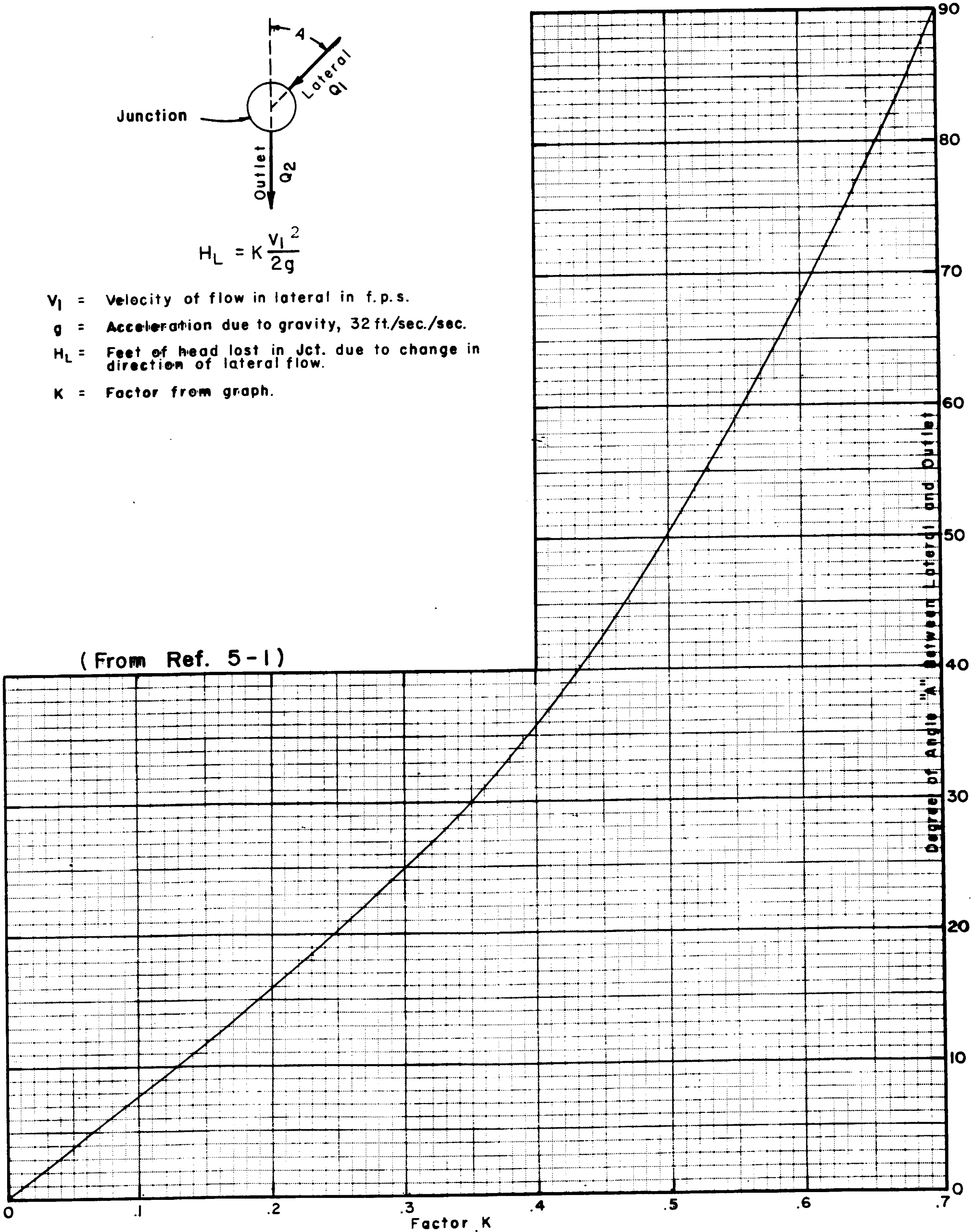
For a graphic solution to other degree of turns refer to Figure 5-6, page 5-38.

5-604.6 JUNCTION LOSSES WITH SEVERAL ENTERING FLOWS

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy, involving both position energy (elevation of water surface) and momentum energy (mass times velocity head). Thus, for a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction (Ref. 5-1).

The total junction losses at the sketched intersection is as follows:





$$H_L = K\frac{V_1^{\,2}}{2g}$$

V_1 = Velocity of flow in lateral in f.p.s.

g = Acceleration due to gravity, 32 ft./sec./sec.

H_L = Feet of head lost in Jct. due to change in direction of lateral flow.

K = Factor from graph.

(From Ref. 5-1)

Loss in Junction due to change in direction of flow in lateral.

FIGURE 5-6

$$H_{j2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 + K Q_1 V_1^2}{2 g Q_4}$$

where

H_{j2} = junction losses

Q_4, Q_1, Q_2 & Q_3 = discharges in cfs

V_4, V_1, V_2 & V_3 = horizontal velocities of foregoing flows (V_3 is assumed to be zero).

g = acceleration due to gravity (32.2 feet per second per second)

K = bend loss factor from Figure 5-6, page 5-38

where subscript nomenclature

Q_1 = 90° lateral

Q_2 = straight through inflow

Q_3 = vertical dropped-in flow from an inlet

Q_4 = main outfall = total computed discharge

are respective discharges in cfs and V_1, V_2, V_3 and V_4 are the horizontal velocities of foregoing flows, respectively in feet per second; V_3 is assumed to be zero.

Assume water surface of inflow and outflow pipes in junction to be level.

Assume $H_b = \frac{K \quad V_1^2}{2g}$ for change in direction

Assume no velocity head of an incoming line is greater than the velocity head of the outgoing line.

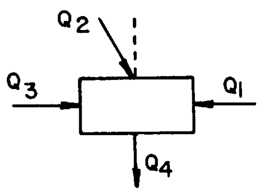
When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present.

If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

An example and application is described below assuming no vertical dropped-in flow from an inlet:

The total junction losses are computed by the foregoing formula:

$$H_{j2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 - Q_3 V_3^2 + 0.7\ Q_1 V_1^2 + 0.7\ Q_3 V_3^2}{2gQ_4}$$



Q_1	= 3.42 cfs	V_1	= 2.79 fps
Q_2	= 36.60 cfs	V_2	= 9.21 fps
Q_3	= 3.15 cfs	V_3	= 2.57 fps
Q_4	= 42.24 cfs	V_4	= 8.61 fps

$$H_{j2} = \frac{3131.3-26.6-3104.6-20.8+18.6+14.6}{2720.3} = 0.0046 \text{ (negligible)}$$

$$H_{j2} = 0.0$$

BEND LOSS FOR Q_2 (change in direction of flow):

$$H_b = K \frac{V^2}{2g}; \quad 18^o \text{ skew, } K = 0.23$$

$$H_b = \frac{0.23 \times 9.21^2}{64.4}$$

$$H_b = 0.3 \text{ ft.}$$

BEND LOSS FOR Q_3:

$$H_b = \frac{0.7\ V_3^2}{2g} = \frac{0.7 \times 2.57^2}{64.4} = 0.1 \text{ ft.}$$

BEND LOSS FOR Q_1:

$$H_b = \frac{0.7\ V_1^2}{2g} = \frac{0.7 \times 2.79^2}{64.4} = 0.1 \text{ ft.}$$

TOTAL LOSSES: $= H_{j2} + H_b + H_b + H_b$

$= 0.0 + 0.3 + 0.1 + 0.1$

$= 0.5$ ft.

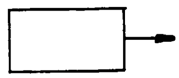
The final step in designing a storm sewer is to check the Hydraulic Grade Line (HGL). Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc.

The HGL should be computed for all storm sewer systems and may be tabulated on Form HYD 5-4, page 5-61 using the procedures listed at the end of the example problem.

Refer to Figure 5-7, page 5-42 for summary of energy losses.

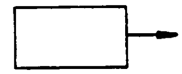
Refer to Figure 5-8, page 5-43 for improper and proper design of energy and hydraulic grade lines.

SUMMARY OF ENERGY LOSSES



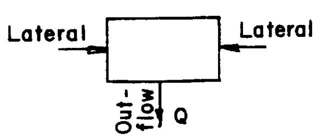
$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL JUNCTION LOSSES
(at beginning of run)
Where g = gravitational constant, 32.2 feet per second per second.



$$H_e = 0.5\,\frac{V^2}{2g}$$

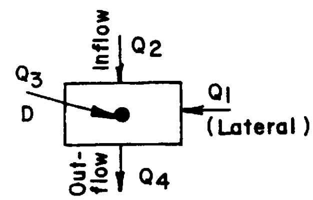
ENTRANCE LOSSES
(for structure at beginning of run)
Assuming square-edge



$$H_{j1} = \frac{V^2\,(\text{Outflow})}{2g}$$

JUNCTION LOSSES

Use only where flows are identical to above, otherwise use H_{j2} Equation.

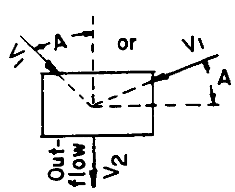


$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90° (H_b).

Where K = Bend loss factor (Figure 5-6 page 5-38)
Q_3 = Vertical dropped-in flow from an inlet
V_3 = Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$$H_f = S_f \times L$$

Where H_f = friction head
S_f = friction slope
L = length of conduit

$$S_f = \left(\frac{Qn}{1.486\,AR^{\frac{2}{3}}}\right)^2$$

Where Q = discharge of conduit
n = Mannings coefficient of roughness (use 0.013 for R.C. Pipes)
A = area of conduit
R = hydraulic radius of conduit ($D/4$ for round pipe)

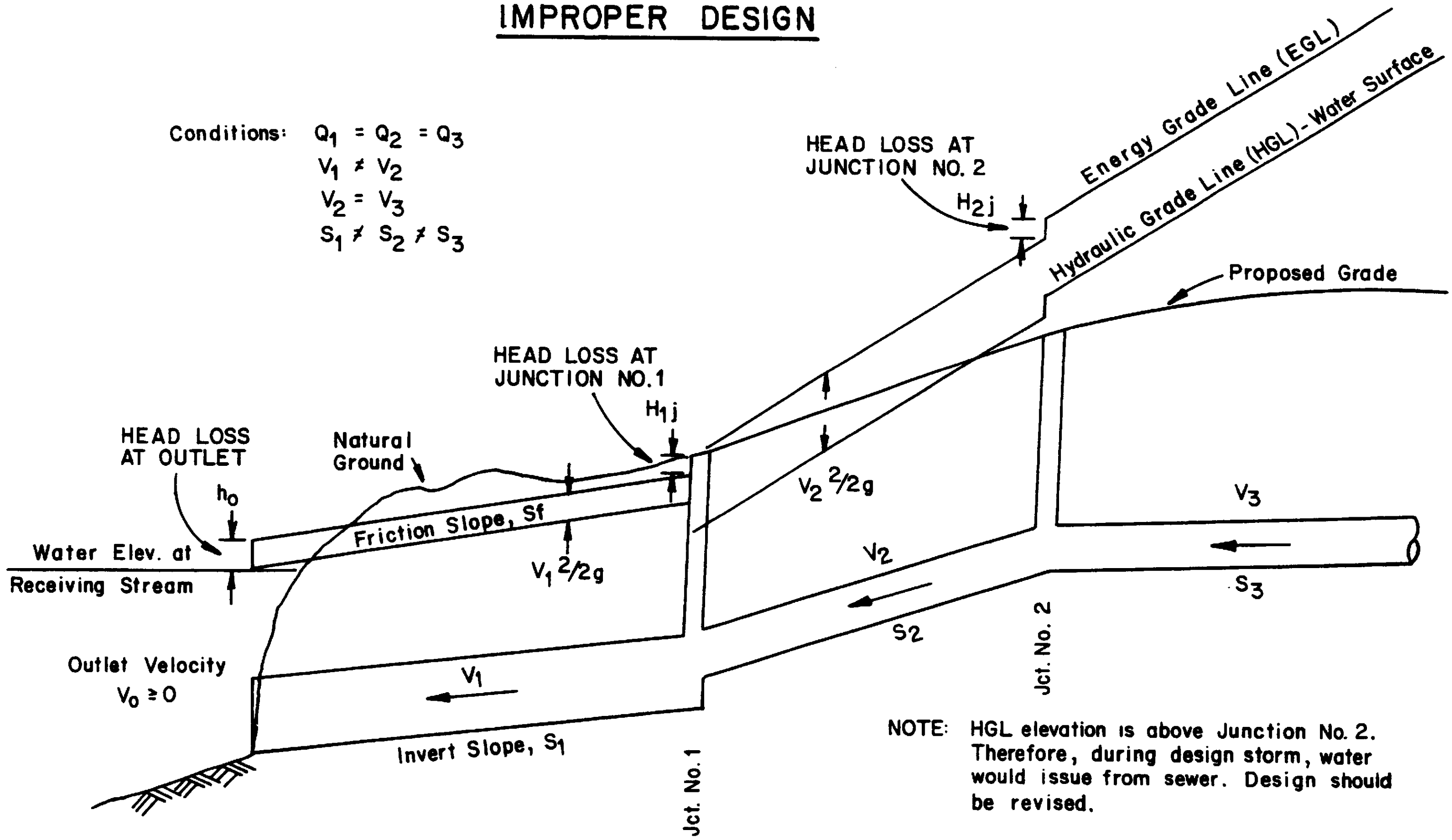
TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + H_{j1} \text{ or } H_{j2} + H_b + H_f$$

FIGURE 5-7

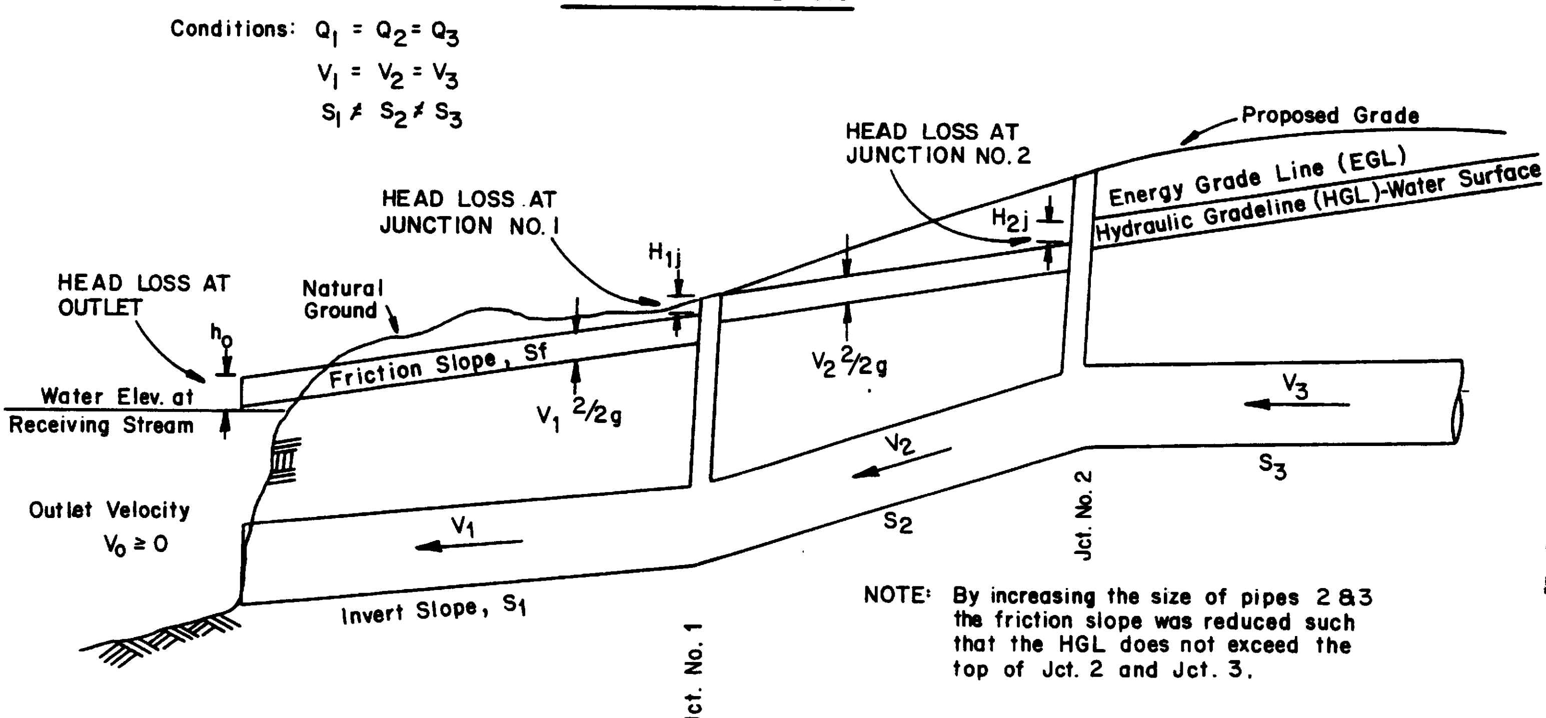
ENERGY AND HYDRAULIC GRADE LINES
FOR
STORM SEWER UNDER CONSTANT DISCHARGE

IMPROPER DESIGN



NOTE: HGL elevation is above Junction No. 2. Therefore, during design storm, water would issue from sewer. Design should be revised.

PROPER DESIGN



NOTE: By increasing the size of pipes 2 & 3 the friction slope was reduced such that the HGL does not exceed the top of Jct. 2 and Jct. 3.

FIGURE 5-8

5-700 EXAMPLE PROBLEM

Given: Table 5-5, page 5-54, shows the layout of the area to be drained, the typical section of improvement and run-off computations.

Design Frequency:

10 years
Area III rainfall intensity

Minimum time of concentration:
10 minutes (overland flow)
5 minutes (paved surface flow)

Minimum inlet length:
5 feet

Gutter depression:
Minimum 2 inches

Permissible width of ponding:
12 feet

Required: Design a "minor" storm sewer system that will adequately dispose of the design discharge.

Solution:

STEP 1 - RUNOFF COMPUTATIONS. Runoff computations are tabulated in Table 5-5, page 5-54. The tabulations are believed to be self-explanatory. The total CA as shown in the last column is computed by multiplying each incremental area by its corresponding coefficient of runoff and totaling the sum of these increment products. As an example, the total CA for Drainage Area No. 1 is computed as follows:

TYPE	ACREAGE		C		CA
Paved	0.09	x	0.95	=	0.08
Commercial	0.21	x	0.70	=	0.15
Unimproved	0.20	x	0.30	=	0.06
			TOTAL CA	=	0.29

This value shall be inserted in Column 4 of Table 5-7, page 5-56.

STEP 2 - RUNOFF AND INLET COMPUTATIONS. The total runoff Q and inlet computations are shown in Table 5-7, page 5-56. A detailed explanation of these tabulations follows:

Column 1: Enter the drainage area either as a composite (total) or as a component (sub-area).

Column 2: Enter the inlet number.

Column 3: Enter the station and location of inlet.

Column 4: Enter the total CA unit from runoff computations shown in Table 5-5, page 5-54.

Column 5: Enter the amount of by-pass CA unit flow which has passed the last preceeding inlet to the inlet under consideration. This CA product is obtained from Column 18.

Column 6: Enter the sum of Columns 4 and 5.

Column 7: Derive the time of concentration by the use of Figure 3-1, Chapter 3, or it may be computed by dividing the distance from the most remote point of the drainage area by the assumed velocity of approach flow. Since we have already established that a minimum time of concentration of 10 minutes for overland flow will be used, this value shall be the minimum value inserted.

Column 8: The intensity of rainfall can be determined from applicable Intensity Chart, Chapter 3.

Column 9: The total flow "Q_t" is determined by products of Columns 6 and 8.

Column 10: "z" is the reciprocal of the cross slope "S_x" and is determined in the following manner:

Slope 0.02 feet/foot: $Z = \frac{1}{0.02} = 50$

Column 11: The ratio $\frac{z}{n}$ is self-explanatory. In the example problem, a roughness coefficient "n" of 0.013 is used. This coefficient may vary from 0.012 to 0.015 depending upon the roughness and possible sediment of the gutter. For the example problem.

$$\frac{z}{n} = \frac{50}{0.013} = 3846$$

Column 12: Enter the grade "S_o", expressed (in feet) per foot, and is obtained from established grade lines as shown on the plan-profile sheets.

Column 13: The value of "d" is the depth of flow (in feet) in the gutter for a certain discharge, z/n ratio, and longitudinal slope. It is determined from Chart 5-1, page 5-62. The procedure to be used in determining the value of "d" is clearly explained on that chart.

Column 14: The width of ponding must be kept within the previously determined acceptable limits. In the example problem, the maximum permissible ponded width is 12 feet. The ponded width is the product of "d" times "z" as determined in Columns 13 and 10 respectively.

Column 15: Enter the type of inlet under consideration.

Column 16: Enter the throat opening length of proposed drop inlet.

Column 17: The value "Q_i" shown in the column is the amount of water in cubic feet per second which the inlet in question actually intercepts. It is the product of "Q_t" from Column 8 and the percentage of interception graphically computed from Chart 5-2, page 5-63 or as determined from Charts 5-3 to 5-7 pages 5-64 to 5-68. For the example problem the combination inlets were treated as separate inlets, intercepted flow computed for each and then assumed 50% efficiency for each inlet using Charts 5-2 and 5-7, pages 5-63 and 5-68 respectively.

Column 18: The carry-over is the amount of water which passes an inlet and is the difference between the total runoff, "Q_t" (Column 9), and the intercepted flow, "Q_i" (Column 17) divided by the "I" of Column 9. More simply stated:

$$\text{Carry over CA product} = \frac{\text{Column 9 less 17}}{\text{Column 8}}$$

This unit value is entered in Column 18 and 5.

STEP 3 - PROPORTIONING STORM SEWERS PIPES. The computations involved in proportioning various runs of sewer pipe are summarized in the tabulation sheet titled "Storm Sewer Computations", Table 5-9, page 5-58.

Column 1: Enter the inlet number - same as used in Inlet Form.

Column 2: Enter the station and location of the inlet.

Column 3: For the Runoff Form, the quotient of Column 17/8 is used to obtain the CA product to be entered in Column 3. Only structures contributing flow to the system should have values in Column 3.

Column 4: Enter the CA product of flow from any contributing upstream structure.

Column 5: Number the inflowing structure.

Column 6: Enter the sum of Columns 3 and 4.

Column 7, 8 and 9: The time of concentration is the time required for water to flow from the most remote part of the drainage area or areas involved to the upper end of the pipe run under consideration. For the first run time of concentration is the inlet time for the first inlet. For all succeeding runs, time of concentration may be either the time as computed along the sewer line or the inlet time of the inlet at the beginning of the run under consideration, depending upon which of these two periods is longer. Accordingly, the larger of the two is used in determining "I" and "Q", unless this larger value is less than 10 minutes, in which case the established minimum time of 10 minutes is used.

The time of concentration shown in Column 7 is computed by taking the time of concentration for the preceding run and adding to it the time required for water to flow through the preceding run to the beginning of the run under consideration.

At junctions of lines, the larger value of the time of concentration is used. For example, "Lines A and B" join to discharge into "Line C." Since the time of concentration for "Line A" is greater than that for "Line B", this larger value is carried through in figuring times for "Line C."

Column 10: Using the "T_c" from Colum 9, enter the intensity value as determined from Figures 3-3 to 3-7, Chapter 3.

Column 11: Enter the product of Column 6 times Column 10.

Column 12, 13 and 14: The size and gradient of pipe as shown in Columns 12 and 13 must be chosen in such manner that the pipe when flowing full, but not under head, will carry an amount of water approximately equal to or greater than the computed discharge, "Q". In other words, the "Capacity" shown in Column 14 must be approximately equal to or greater than the value "Q" shown in Column 11.

The capacity may be calculated by Manning's formula:

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

or capacity can be taken directly from the appropriate nomographs in Charts 5-9 to 5-14, 5-16 to 5-26, pages 5-70 to 5-75 and 5-77 to 5-87, respectively.

Wherever a pipe run is designed in such a manner that the capacity of a pipe as shown in Column 14 is less than the computed discharge shown in Column 11, a check of the hydraulic gradient above this run should be made to make such that the backwater head created by such a design is not large enough to cause blowouts at inlets or junctions above the run.

Column 15: The velocities shown in this column can be calculated by Manning's forumla:

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

or the velocities can be taken directly from Chart 5-14, page 5-75 or Chart 5-15, page 5-76.

Column 16: The length of each run as shown in this column is the length center to center of inlets or junctions. This length is used in determining the time of flow from one inlet or junction to another.

Column 17: The time of concentration in the pipe under consideration is actual flow time, in minutes from the present inlet to the next junction point. Run time is calculated by dividing the length of run (Column 15) by velocity of flow (Column 15) and converting the answer into minutes by dividing by 60.

Column 18 to 23: Figures shown in these columns are believed to be self-explanatory.

STEP 4 - HYDRAULIC GRADE LINE. The final step in designing a storm sewer is to check the Hydraulic Grade Line (HGL). Computing the HGL will determine the elevation under design conditions to which water will rise in various inlets, manholes, junctions and etc.

The HGL should be computed for all storm sewer systems. Computations are summarized in tabulation sheet entitled "Hydraulic Grade Line", Table 5-11, page 5-60.

Column 1: Enter 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.

Column 2: Enter tailwater elevation if the outlet will be submerged during the design storm or 0.8 diameter of pipe plus invert out elevation of the outflow pipe, whichever is greater.

Column 3: Enter diameter of outflow pipe.

Column 4: Enter design discharge for outflow pipe.

Column 5: Enter length of outflow pipe.

Column 6: Enter friction slope in feet/foot of the outflow pipe using the formula:

$$S_f = \left[\frac{Qn}{1.486\ AR^{2/3}}\right]^2$$

Column 7: Enter friction loss by multiplying column 5 by column 6.

Column 8: Enter velocity of the outflow pipe.

Column 9: Enter design discharge (Q_1, Q_2, Q_3.....) for each pipe flowing into the junction.

Column 10: Enter velocity (V_1, V_2, V_3....) for each pipe flowing into the junction.

Column 11: Enter terminal junction losses for the upper reach of each storm sewer run using the formula $H_{tm} = \frac{V2}{2g}$

Column 12: Enter pipe entrance losses for the upper reach of each storm sewer run using the formula $H_e = 0.5\ \frac{V^2}{2g}$

assuming square-edge.

Column 13: Enter junction losses H_{j1} or H_{j2} for each junction using the formula:

$$H_{j1} = \frac{V^2\ \text{outflow}}{2g}$$

$$\text{OR} \quad H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

Column 14: Enter bend losses (changes in direction of flow) for each inflowing pipe to the outflow pipe using the formula $H_b = \frac{K V^2}{2g}$. Refer to Figure 5-6, page 5-38 for "K" values.

Column 15: Enter total head losses using the formula: $H_t = H_f + H_{tm} + H_e + H_{j1}$ or $H_{j2} + H_b$.

Column 16: Enter the new Hydraulic Grade by summing elevations in Column 2 and Column 15. This elevation is the potential water surface elevation for the junction under design conditions.

Column 17: Enter the top of junction cover or the gutter flow line, whichever is lowest and compare it with the HG in Column 16. The HG should be sufficiently below the gutter line or top of junction to provide an approximate freeboard of 9 inches.

5-800 ADDITIONAL DESIGN CHARTS

These charts are designed to further enable direct solutions to various flow characteristics in circular pipes.

5-801 RELATIVE VELOCITY, AREA AND DISCHARGE IN A CIRCULAR PIPE FOR ANY DEPTH OF FLOW CHART

Chart 5-15, page 5-76 is a nomograph for estimating steady uniform flows for pipes flowing only partly full. Velocity, hydraulic radius, quantity and area of flow vary with depth of flow. These values are proportionate to full flow values and for any depth of flow plotted. The capital letter subscript indicates full-flow condition. Both the discharge and velocity curves show maximum values, which occur at above 0.938d and 0.81d respectively.

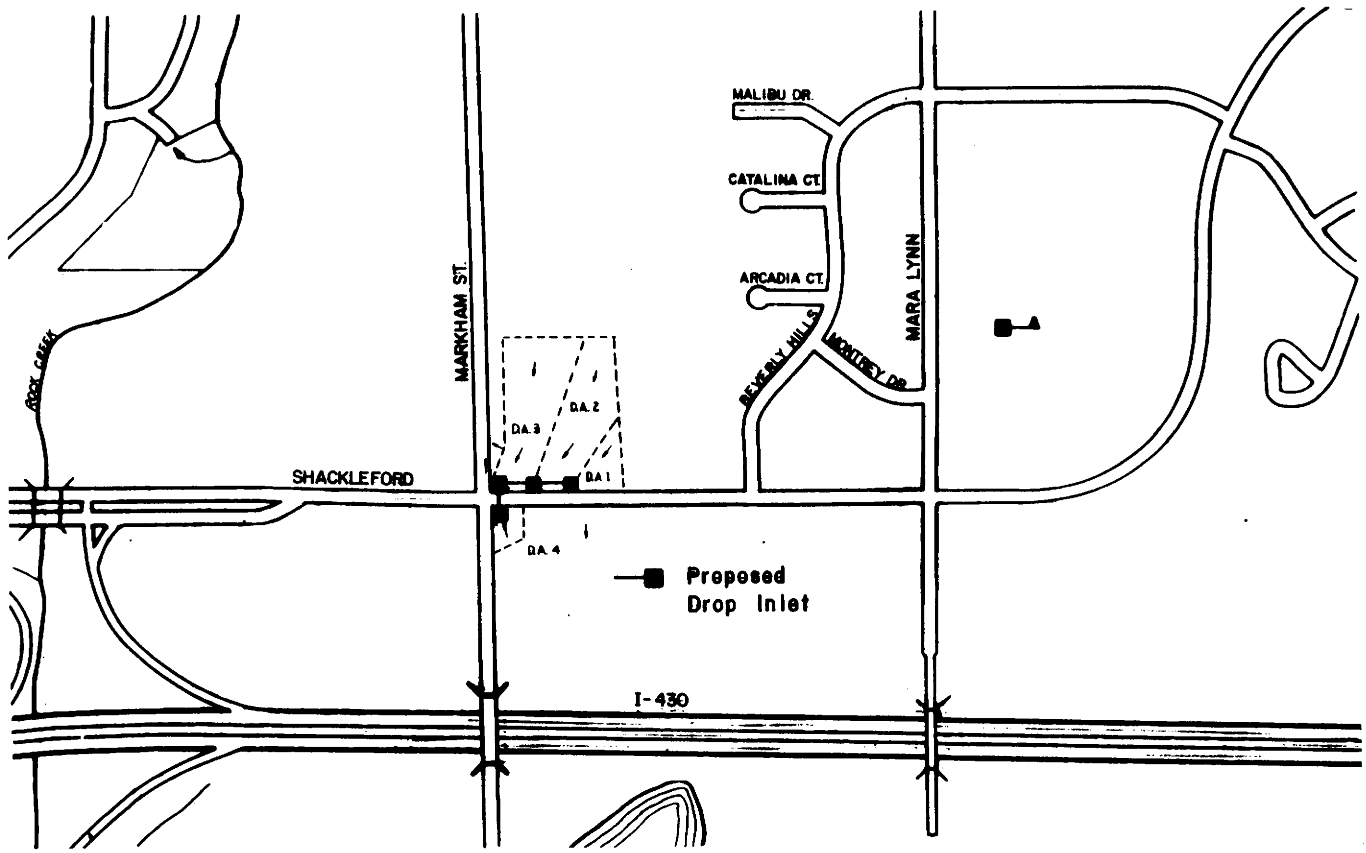
5-802 PIPE FLOW CHARTS

Charts 5-16 to 5-26, pages 77 to 87 are designed to enable direct solution of the Manning equation for flow in circular pipes. Each chart applies to a certain diameter pipe.

Depth of uniform flow for a given discharge "Q" in a given size pipe on a given slope S_o, and with roughness coefficients "n = 0.013, 0.015, or 0.024" may be determined directly from the chart for that size by entering on the appropriate slope line (or an interpolated slope). Normal velocity may be read on the appropriate "V-scale" opposite this same point. The procedure may be reversed to determine discharge at a given depth of flow.

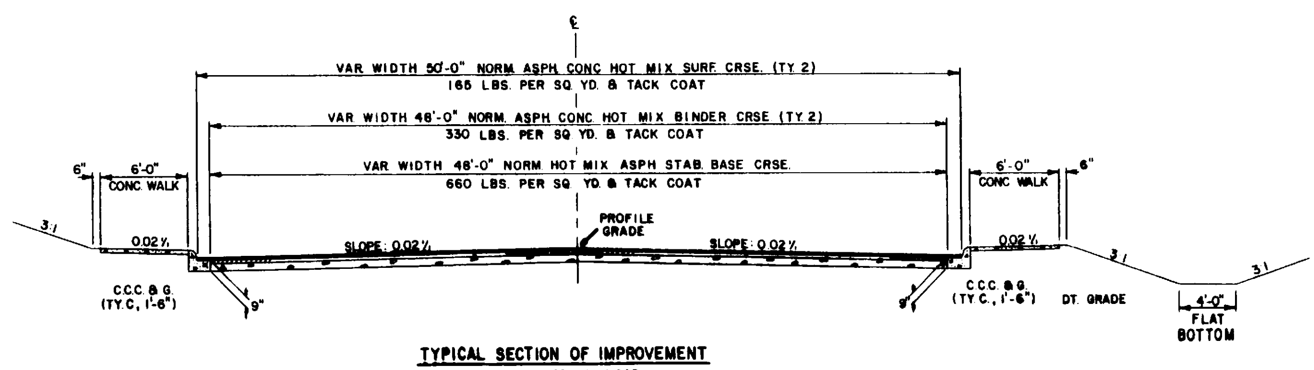
Where the "Q-ordinate" intersects a slope line, "S_o", in the area near its right terminus, two alternate depths will be indicated if "dn" is greater than 0.82 diameters. For these cases, flow will occur at the lesser of alternate depths.

For pipe roughness coefficients other than those of "n" = 0.013, 0.015 or 0.024 shown on the chart scales, enter the chart on the inner scale for "n" = 0.015 with an adjusted value of "Q" which is equal to the design "Q" x "n"/0.015 to determine depth and velocity. Read depth directly from the chart at the pipe slope line, "S_o" and obtain velocity by dividing the value read on the "V-scale" for "n" = 0.015 by the ratio "n"/0.015. In reversing the above procedure to determine "Q" for a given depth, read "Q" on the scale for "n" = 0.015 and divide by the ratio "n"/0.015 to obtain the flow rate "Q".



RUNOFF COMPUTATIONS

DRAINAGE AREA		ACRES - DRAINED			TOTAL CA
NO.	ACRES	PAV'T.	COMM.	UNIMPR	
		C = 0.95	C = 0.70	C = 0.30	
1	0.50	0.09	0.21	0.20	0.29
2	1.66	0.08	0.79	0.79	0.87
3	2.84	0.08	1.38	1.38	1.46
4	0.31	0.25	0.06		0.28



TYPICAL SECTION OF IMPROVEMENT
SHACKLEFORD ROAD

MAP OF DRAINAGE AREAS & RUNOFF COMPUTATIONS

TABLE 5 - 5

FORM HYD 5-1

RUNOFF COMPUTATIONS

DRAINAGE AREA		ACRES - DRAINED					TOTAL CA
NO.	ACRES	C=	C =	C =	C =	C =	

TABLE 5-6

INLET COMPUTATIONS

FORM HYD 5-2

DRAINAGE AREA NO.	INLET NUMBER	INLET LOCATION	RUNOFF AT INLET (CA PRODUCTS)	CARRY OVER FROM 18 (CA PRODUCTS)	TOTAL (CA PRODUCTS)	TIME OF CONCENTRATION "Tc" MIN.	INTENSITY "I" IN./HR.	TOTAL FLOW "Qt" CFS	GUTTER PROPERTIES "z"	GUTTER PROPERTIES "z/n" RATIO	GUTTER PROPERTIES GRADE "So" FT./FT.	GUTTER PROPERTIES DEPTH AT CURB "d" FEET	GUTTER PROPERTIES PONDED WIDTH "T" FEET	INLET SIZING TYPE	INLET SIZING LENGTH "Li" FEET	INLET SIZING INTERCEPTED "Qi" CFS	CARRY OVER CA = "Qt - Qi/I"	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
1	1	10 + 00 LT.	0.29	-	0.29	10.0	6.0	1.74	50	3846	0.055	0.12	6.0	"C"	5'+ 8' EXT.	1.31	0.07	
2	2	8 + 80 LT.	0.87	0.07	0.94	10.3	6.0	5.64	50	3846	0.09	0.17	8.5	comb.	5'+ 8' GRATE	1.3 3.3 (1)	0.17	Assume 50% Efficiency for Comb. Inlet
3	3	7 + 60 LT.	1.46	0.17	1.63	10.0	6.0	3.78	50	3846	0.042	0.24	12.0	comb.	5'+ 8' GRATE	2.15 3.9 (2)	0.62	Assume 50% efficiency for Comb. Inlet
4	4	7 + 60 RT.	0.28	-	0.28	5.0	7.3	2.04	50	3846	0.042	0.13	6.5	"C"	5'	0.75	0.18	

(1) 2.59/2 = 1.3
6.6/2 = 3.3

(2) 4.3/2 = 2.15
7.84/2 = 3.92

TABLE 5-7

FORM HYD 5-2

INLET COMPUTATIONS

DRAINAGE AREA NO.	INLET		RUNOFF AT INLET	CARRY OVER FROM 18	TOTAL	TIME OF CONCENTRATION "T_c"	INTENSITY "I"	TOTAL FLOW "Q_t"	GUTTER PROPERTIES					INLET SIZING			CARRY OVER CA = "$Q_t - Q_i/I$"	REMARKS
	NUMBER	LOCATION							"z"	"z/n" RATIO	GRADE "S_o"	DEPTH AT CURB "d"	PONDED WIDTH "T"	TYPE	LENGTH "L_i"	INTERCEPTED "Q_i"		
			CA PRODUCTS			MIN.	IN./HR.	CFS			FT./FT.	FEET			FEET	CFS		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	

TABLE 5-8

FORM HYD 5-3

STORM SEWER COMPUTATIONS

						"T_c"					PIPE DESIGN						PIPE INVERTS			DROP INLET ELEV. & HEIGHT		
INLET NO.	LOCATION	INLET CA	ADDITIONAL CA	FROM STRUCTURE NO.	TOTAL CA	OVERLAND FLOW	TIME IN PIPE	USED IN DESIGN	INTENSITY "I_p"	TOTAL "Q_p" IN PIPE	DIAMETER	SLOPE	CAPACITY	VELOCITY	LENGTH	T_c IN PIPE L/V/60	INLET ELEVATION	TOTAL FALL L x S	OUTLET ELEVATION	FLOW LINE	TOP ELEVATION	HEIGHT
						MINUTES			IN/HR.	CFS	IN.	FT./FT.	CFS.	FT/SEC	LIN. FT.	MIN.	FEET			FEET		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	10 + 00 LT.	0.22	-	-	0.22	10.00	-	10.00	6.0	1.32	18"	0.08	30	8.3	115	0.23	417.38	9.20	408.18	417.38	421.38	4.00
2	8 + 80 LT.	0.99	-	-	0.99	10.3	0.23	10.53	6.0	5.94	18"	0.07	28	13.0	115	0.15	407.67	7.53	400.14	407.67	412.31	4.64
3	7 + 60 LT	2.00	-	-	2.00	10.3	0.38	10.68	6.0	12.00	30"	0.002	18	3.7	60	0.27	398.31	0.12	398.19	398.31	403.31	5.00
4	7 + 60 RT.	2.10	-	-	2.10	10.3	0.65	10.65	5.9	12.39	30"	0.005	29	5.5	12	0.04	398.08	0.06	398.02	398.08	403.25	5.17

TABLE 5-9

FORM HYD 5-3

STORM SEWER COMPUTATIONS

						"T_c"					PIPE DESIGN						PIPE INVERTS			DROP INLET ELEV. & HEIGHT		
INLET NO.	LOCATION	INLET CA	ADDITIONAL CA	FROM STRUCTURE NO.	TOTAL CA	OVERLAND FLOW	TIME IN PIPE	USED IN DESIGN	INTENSITY "I_p"	TOTAL "Q_p" IN PIPE	DIAMETER	SLOPE	CAPACITY	VELOCITY	LENGTH	T_c IN PIPE L/V/60	INLET ELEVATION	TOTAL FALL L x S	OUTLET ELEVATION	FLOW LINE	TOP ELEVATION	HEIGHT
						MINUTES			IN/HR.	CFS	IN.	FT./FT.	CFS.	FT/SEC.	LIN. FT.	MIN.	FEET			FEET		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

TABLE 5-10

FORM HYD 5-4

HYDRAULIC GRADE LINE

INLET STATION	OUTLET WATER SURFACE ELEVATION	D_o	Q_o	L_o	S_{fo}	H_f	VELOCITY HEAD LOSSES V_o	Q_I	V_i	H_{tm}	H_e	H_j	H_b	TOTAL H_f & HEAD LOSSES	HYDRAULIC GRADE	TOP RING OR GUTTER
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
7+60 RT.	400.02	30"	12.4	12	.0009	0.01	5.5	12.0	3.7	–	–	0.26	–	0.27	400.29	402.25
7+60 LT.	400.18 (a)	30"	12.0	60	.0009	0.05	3.7	5.9	13.0	–	–	0.00	0.15	0.20	400.38	402.31
8+80 LT.	398.55 (a)	18"	5.9	115	.0032	0.37	13.0	1.3	8.3	–	–	2.39	–	2.76	401.31	411.31
											(b) (Lower than F.L. D.I.)					
10+00 LT.	408.87	18"	1.3	115	.0002	0.02	8.3	1.3	–	0.03	0.01	–	–	0.06	408.93	420.38

407.67 Outlet Invert
+1.2 (0.8) 18" Pipe
408.87 Estimated Water Surface

(a) Includes deduction for draw-down within drop inlet for in and out pipe invert elevations.
(b) Hydraulic grade cannot be < d_n.

WHERE
D_o = diameter of the outflow pipe
Q_o = discharge of the outflow pipe
L_o = length of the outflow pipe
S_{fo} = friction slope of the outflow pipe
H_f = friction loss of outflow pipe
V_o = velocity of the outflow pipe
V_I = velocity of the inflow pipe
H_{tm} = terminal junction loss
H_e = entrance loss
H_j = junction loss
H_b = bend loss

TABLE 5-11

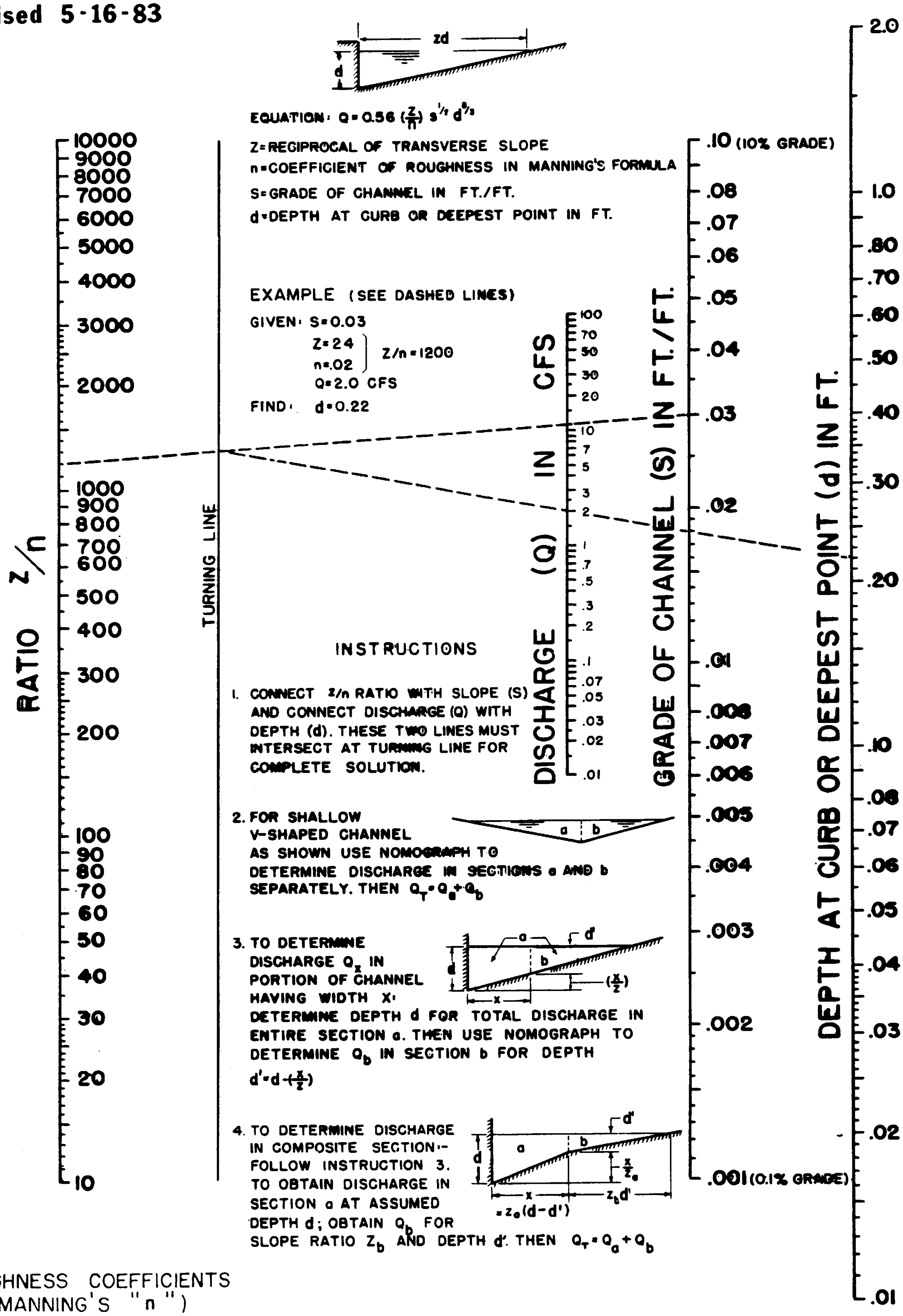
FORM HYD 5-4

HYDRAULIC GRADE LINE

INLET STATION	OUTLET WATER SURFACE ELEVATION	D_o	Q_o	L_o	S_{fo}	H_f	VELOCITY HEAD LOSSES							TOTAL H_f & HEAD LOSSES	HYDRAULIC GRADE	TOP RING OR GUTTER
							V_o	Q_l	V_i	H_{tm}	H_e	H_j	H_b			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

WHERE
D_o = diameter of the outflow pipe
Q_o = discharge of the outflow pipe
L_o = length of the outflow pipe
S_{fo} = friction slope of the outflow pipe
H_f = friction loss of outflow pipe
V_o = velocity of the outflow pipe
V_l = velocity of the inflow pipe
H_{tm} = terminal junction loss
H_e = entrance loss
H_j = junction loss
H_b = bend loss

TABLE 5-12



EQUATION: $Q = 0.56 \left(\frac{z}{n}\right) s^{1/2} d^{8/3}$

Z = RECIPROCAL OF TRANSVERSE SLOPE

n = COEFFICIENT OF ROUGHNESS IN MANNING'S FORMULA

S = GRADE OF CHANNEL IN FT./FT.

d = DEPTH AT CURB OR DEEPEST POINT IN FT.

EXAMPLE (SEE DASHED LINES)

GIVEN: S = 0.03
Z = 24, n = .02 } Z/n = 1200
Q = 2.0 CFS

FIND: d = 0.22

INSTRUCTIONS

1. CONNECT Z/n RATIO WITH SLOPE (S) AND CONNECT DISCHARGE (Q) WITH DEPTH (d). THESE TWO LINES MUST INTERSECT AT TURNING LINE FOR COMPLETE SOLUTION.

2. FOR SHALLOW V-SHAPED CHANNEL AS SHOWN USE NOMOGRAPH TO DETERMINE DISCHARGE IN SECTIONS a AND b SEPARATELY. THEN $Q_T = Q_a + Q_b$

3. TO DETERMINE DISCHARGE Q_x IN PORTION OF CHANNEL HAVING WIDTH X: DETERMINE DEPTH d FOR TOTAL DISCHARGE IN ENTIRE SECTION a. THEN USE NOMOGRAPH TO DETERMINE Q_b IN SECTION b FOR DEPTH $d' = d - \left(\frac{x}{z}\right)$

4. TO DETERMINE DISCHARGE IN COMPOSITE SECTION: FOLLOW INSTRUCTION 3. TO OBTAIN DISCHARGE IN SECTION a AT ASSUMED DEPTH d; OBTAIN Q_b FOR SLOPE RATIO Z_b AND DEPTH d'. THEN $Q_T = Q_a + Q_b$

ROUGHNESS COEFFICIENTS
(MANNING'S "n")

ASPHALT PAV'T . . . . 0.013 - 0.016
CONC. GUTTER WITH ASPH. PAV'T . . . 0.013 - 0.015
CONC. PAV'T . . . . 0.014 - 0.016

NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

CHART 5-1

STANDARD CURB-OPENING INLET CHART

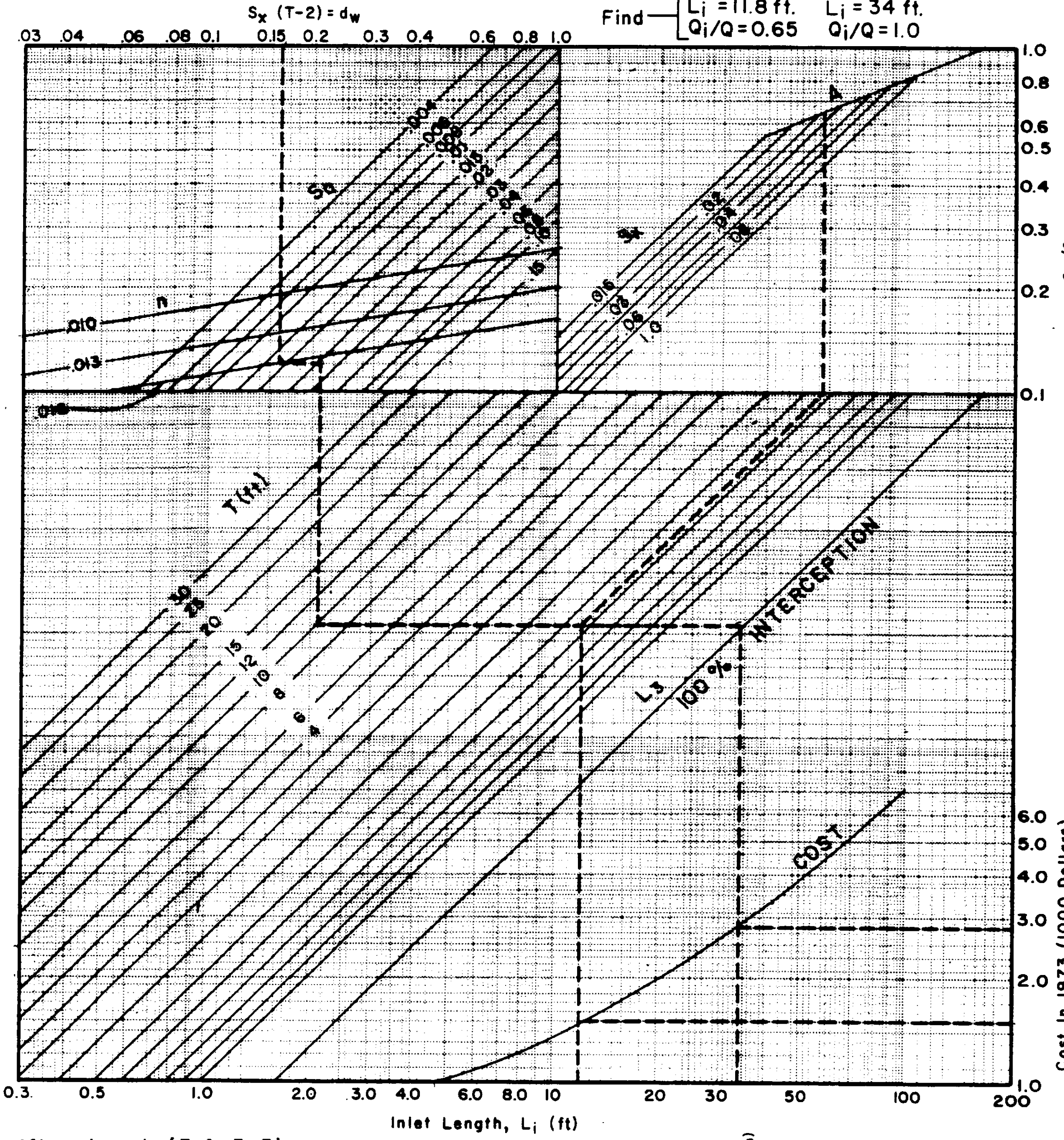
W = 2 ft.
a = 2 in.
h = 6 in.

One inch is 25.4mm
One foot is 0.3048m

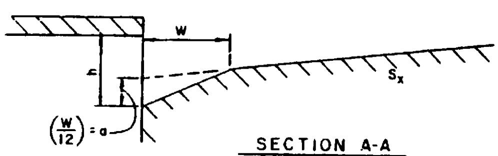
EXAMPLE

Given — S_x = 0.02 ft./ft., T = 10 ft., S_o = 0.03 ft./ft.

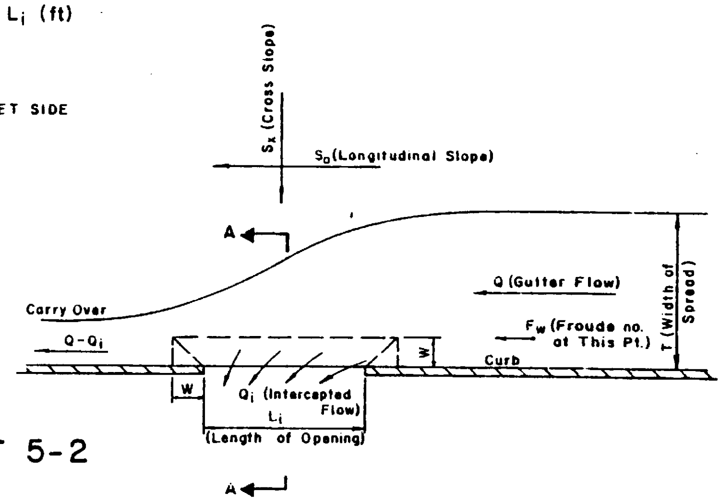
Find — L_i = 11.8 ft., Q_i/Q = 0.65; L_i = 34 ft., Q_i/Q = 1.0



After Izzard (Ref. 5-5)

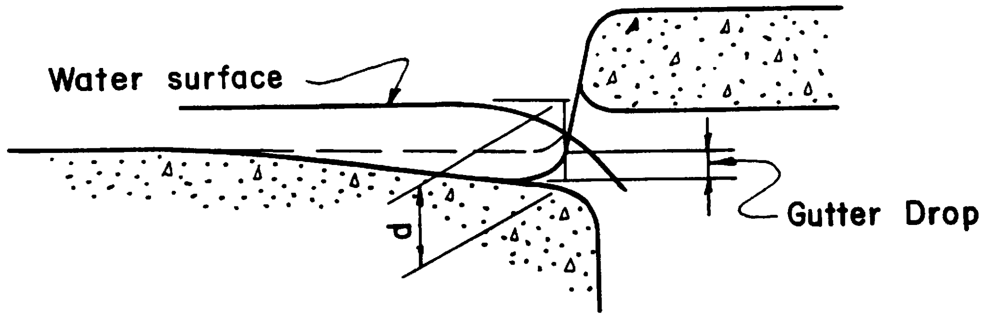


SECTION A-A

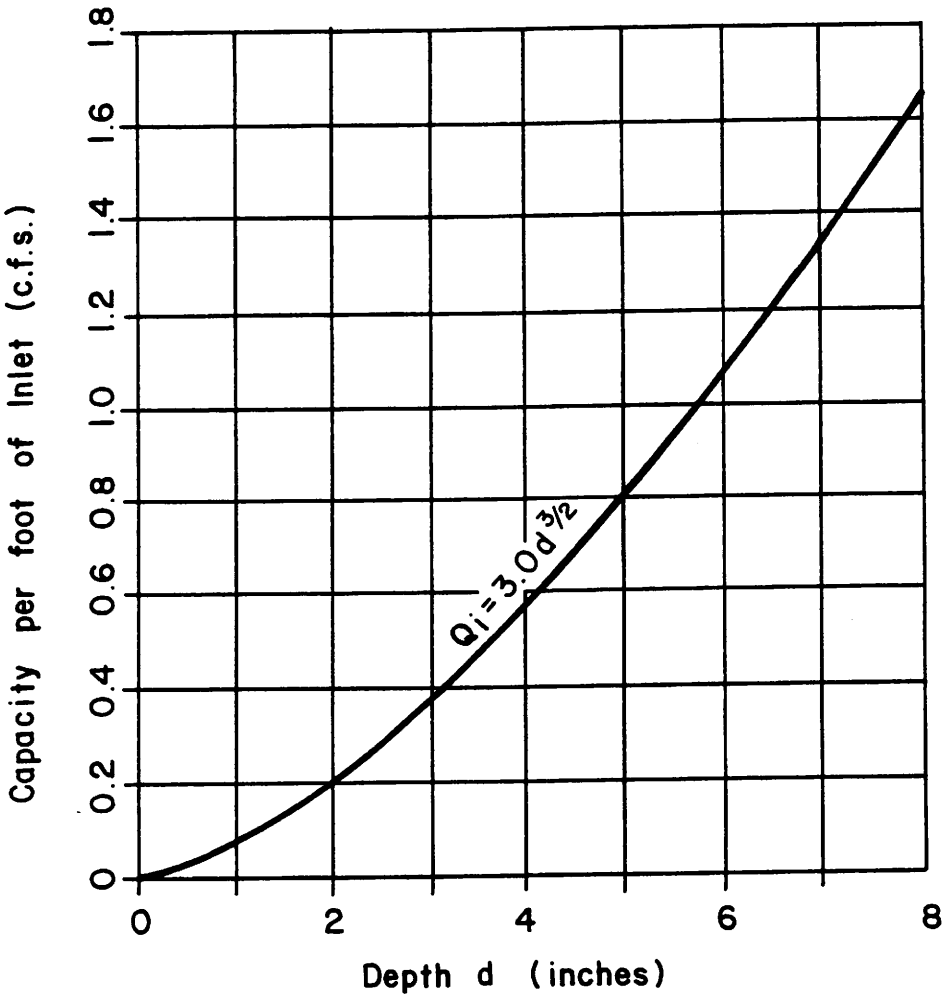


PLAN

CHART 5-2

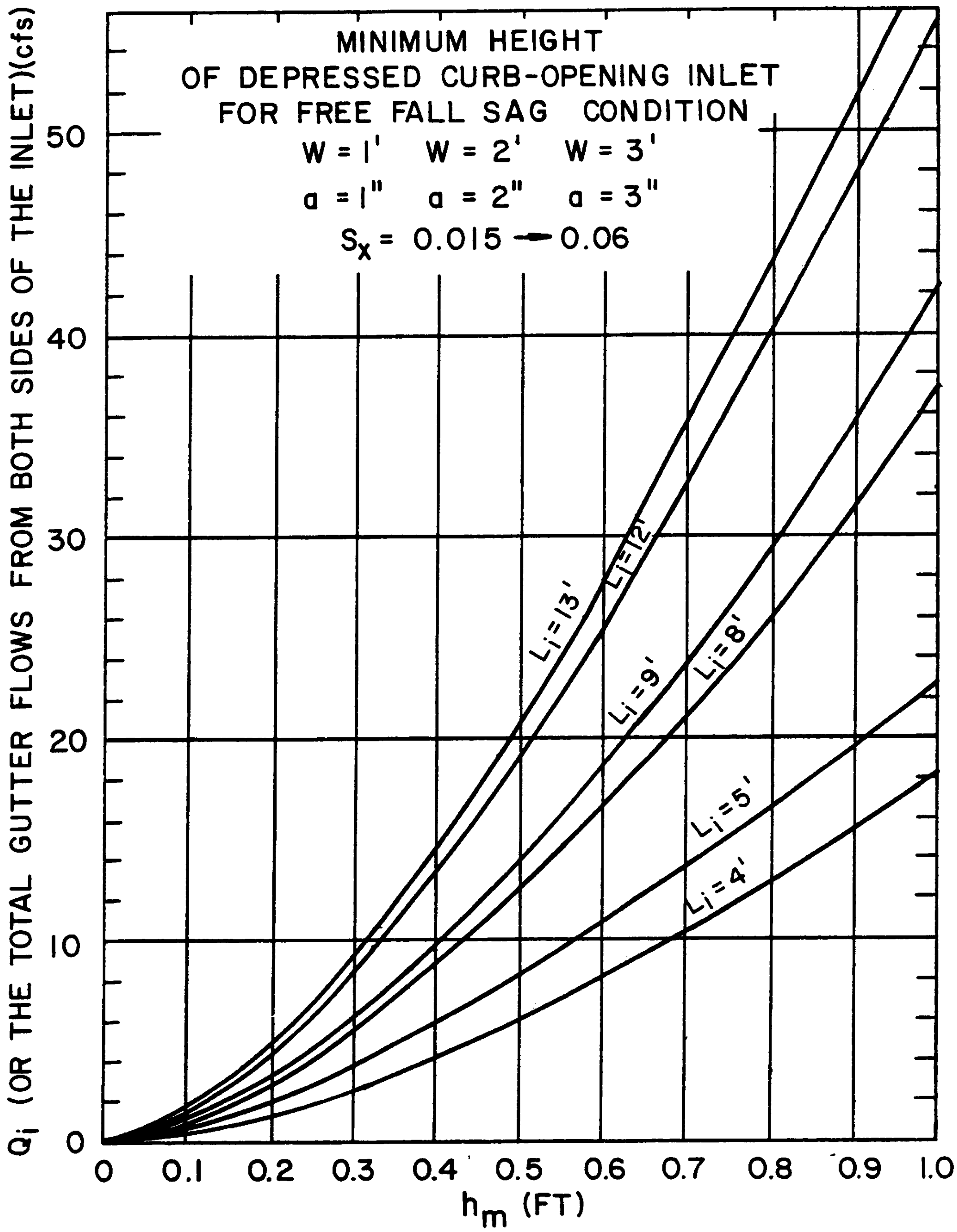


SECTION



INLET CAPACITY FOR SAG CURB INLETS

CHART 5-3



(After HEC-12)

CHART 5-4

MINIMUM HEIGHT OF DEPRESSED CURB-OPENING INLET FOR FREE FALL SAG CONDITION

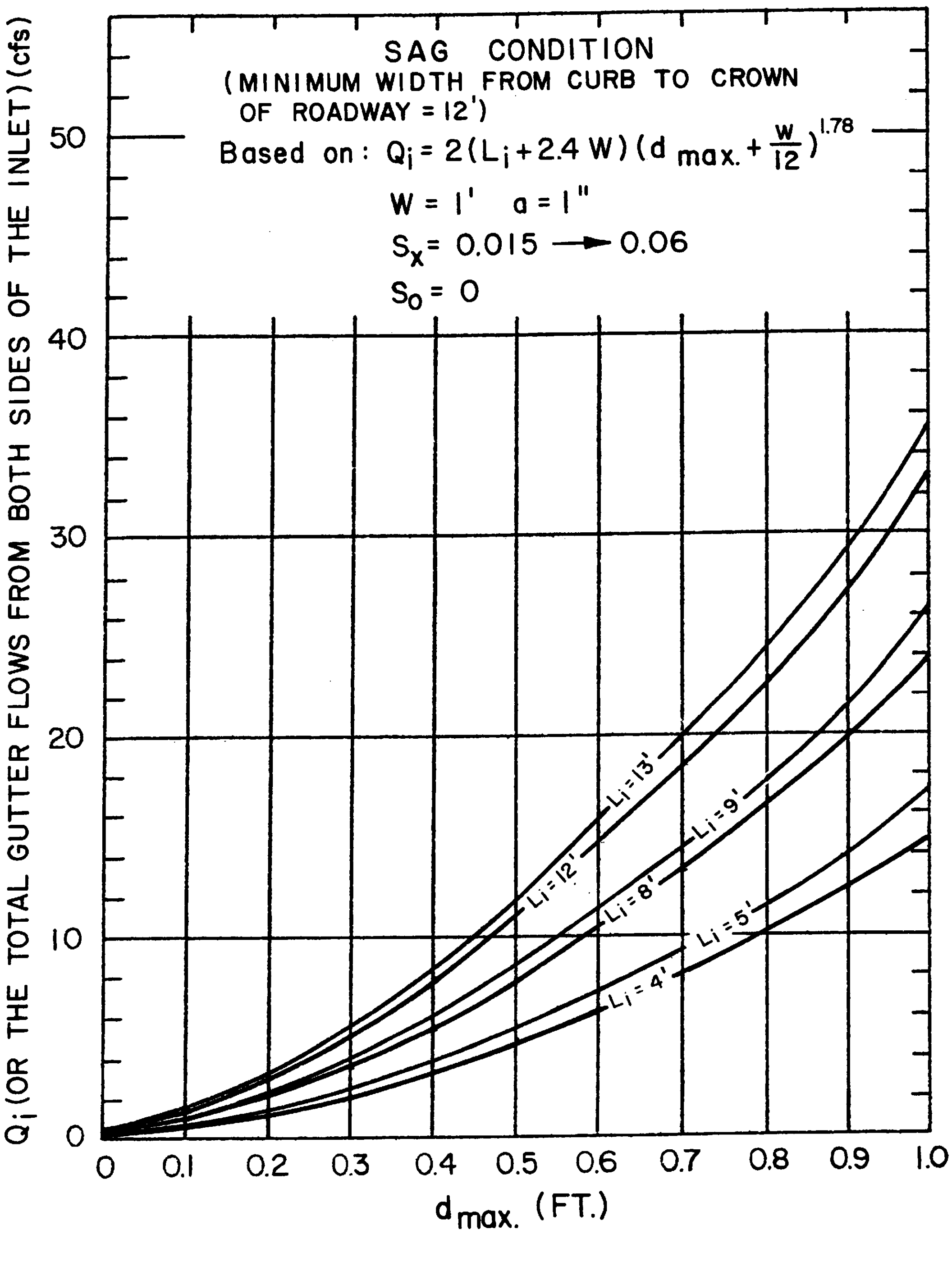


CHART 5-5

INLET CAPACITY FOR SAG CURB INLETS
(W = 1', a = 1")

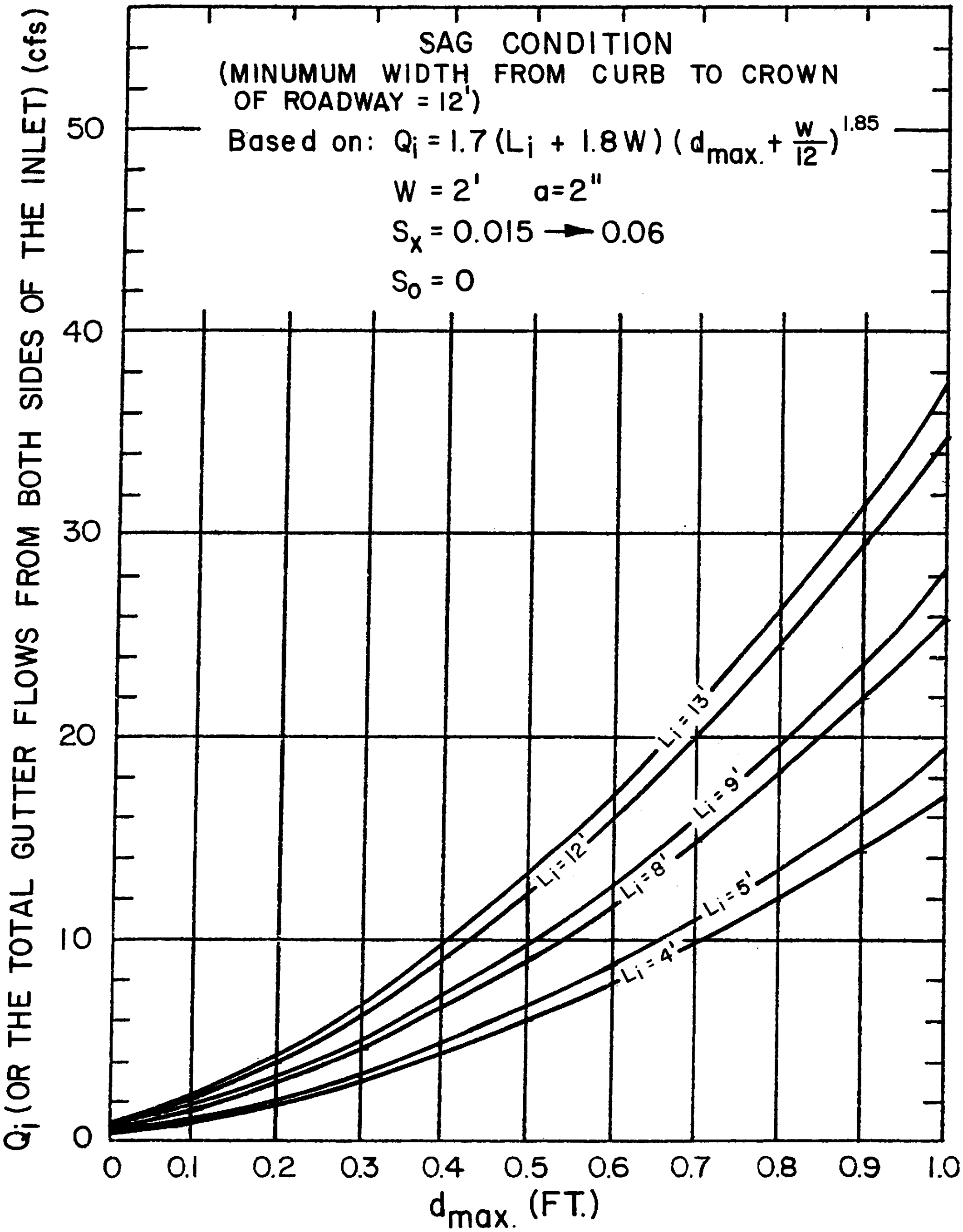
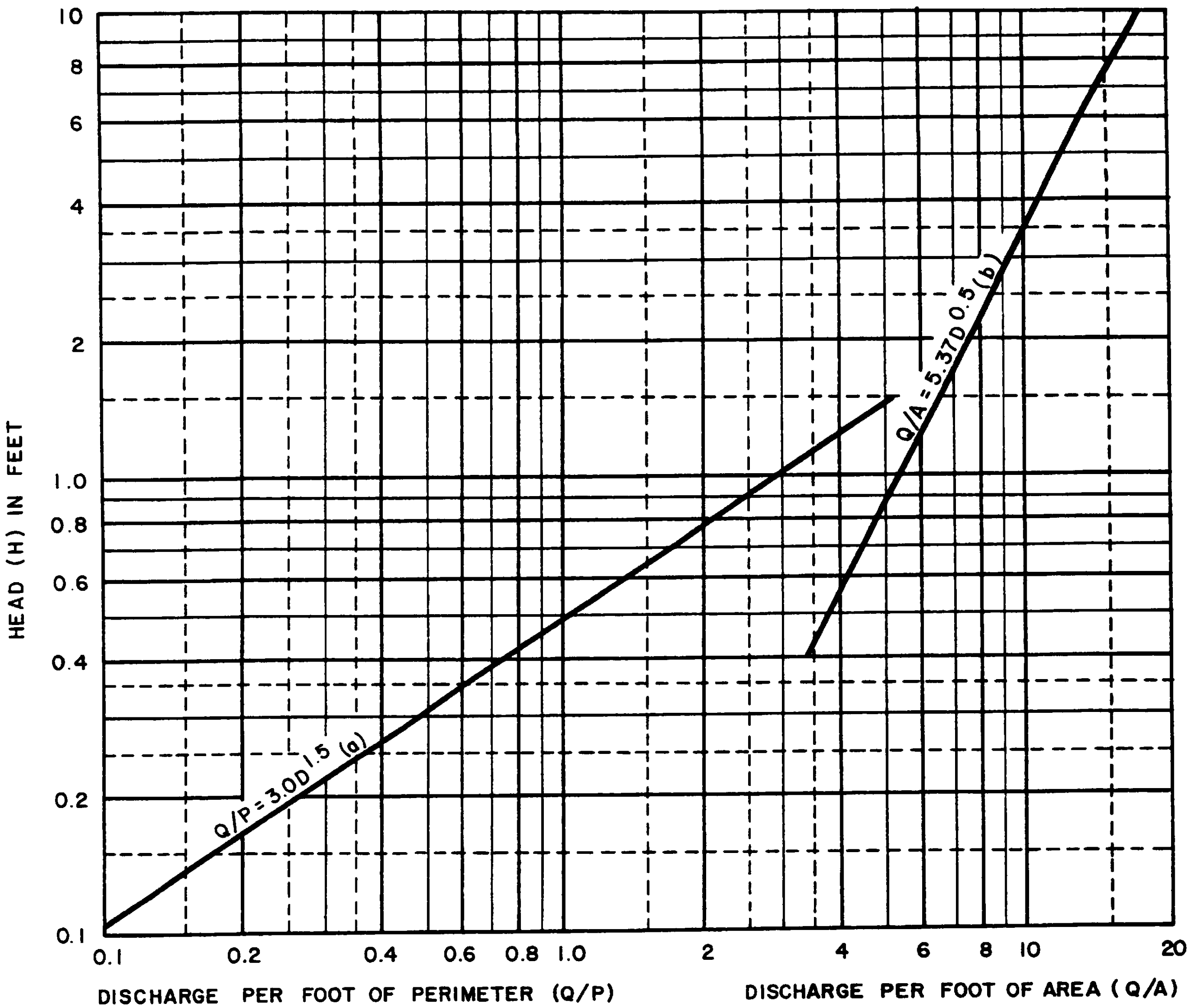
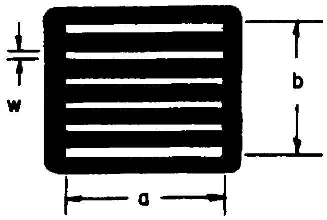


CHART 5-6

INLET CAPACITY FOR SAG CURB INLETS
(W = 2', a = 2")





P = 2 (a + b)
A = 6 aw

HEADS UP TO 0.4, CURVE (a) APPLIES
HEADS ABOVE 1.4, CURVE (b) APPLIES
HEADS BETWEEN 0.4 & 1.4, TRANSITION SECTOR, USE LESSOR VALUE OF DISCHARGE

CAPACITY OF GRATE INLET IN SAG

CHART 5-7

TABLE 5-13

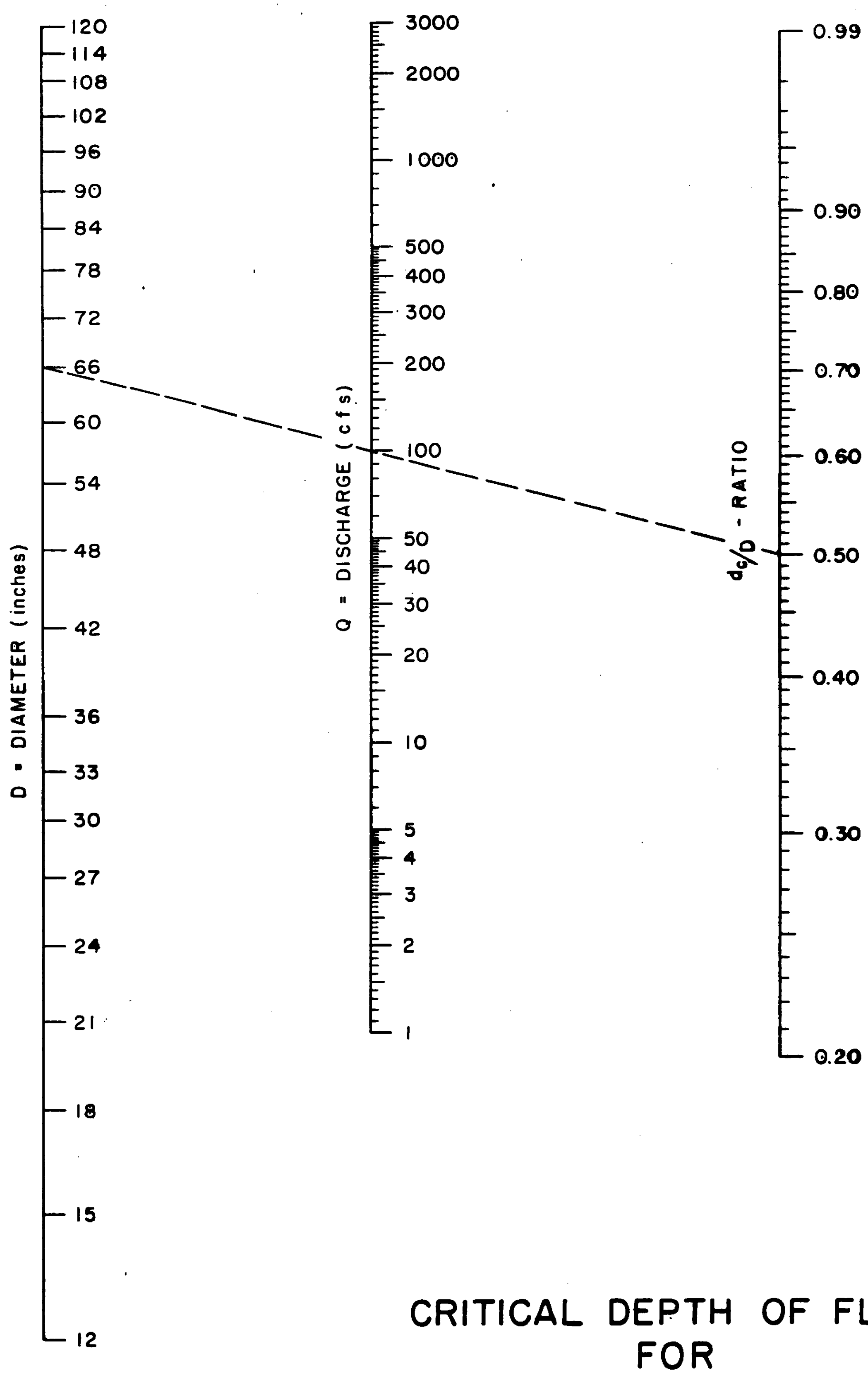
ROUGHNESS COEFFICIENTS
MANNING'S "n"
CORRUGATED METAL PIPE

DIAMETER	12"	24"	36"	48"	60"	72"	84"	96"	108"	120"
CORRUGATIONS	ANNULAR $2\frac{2}{3} \times \frac{1}{2}$"									
PARTLY FULL FLOW	0.026	0.025	0.024	0.024	0.024					
FULL FLOW	0.027	0.025	0.025	0.024	0.024					
CORRUGATIONS	3 x 1"									
PARTLY FULL FLOW				0.028	0.028	0.028	0.028	0.028	0.028	0.027
FULL FLOW				0.028	0.028	0.028	0.028	0.028	0.028	0.027
CORRUGATIONS	HELICAL $2\frac{2}{3} \times \frac{1}{2}$"									
PARTLY FULL FLOW	0.026	0.025	0.024	0.024	0.024	0.024	0.023	0.023		
FULL FLOW (1)	0.011	0.016	0.019	0.020	0.022	0.023	0.023	0.023		
CORRUGATIONS	3 x 1"									
PARTLY FULL FLOW				0.028	0.028	0.028	0.028	0.028	0.028	0.027
FULL FLOW (2)				0.023	0.024	0.026	0.027	0.027	0.027	0.027
CORRUGATIONS	ANNULAR STRUCTURAL PLATE 6 x 2"									
DIAMETER	10'	12'	14'	16'	18'	20'	21'			
PARTLY FULL FLOW	0.034	0.034	0.033	0.033	0.033	0.033	0.033			
FULL FLOW	0.034	0.034	0.034	0.034	0.034	0.033	0.033			

After FHWA-TS-80-216

(1) Lower resistance values for helical CMP occur only when fully developed spiral flow exists. In order to use the lower resistance factors the designer must assure himself that fully developed spiral flow can occur in his design situation. For conduits shorter than 20 diameters long, full development of spiral flow cannot be assured. Therefore, for short culverts, less than about 20 diameters long, the high resistance factors for an annular CMP of the same size and corrugation is recommended. Do not use lower resistance factors for pipe-arches.

(2) Manning's "n" values from AISI.



CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

(After Ref. 5-6)

CHART 5-8

FLOW FOR CIRCULAR PIPE FLOWING FULL

BASED ON MANNING'S EQUATION n=0.013

From Concrete Pipe Design Manual

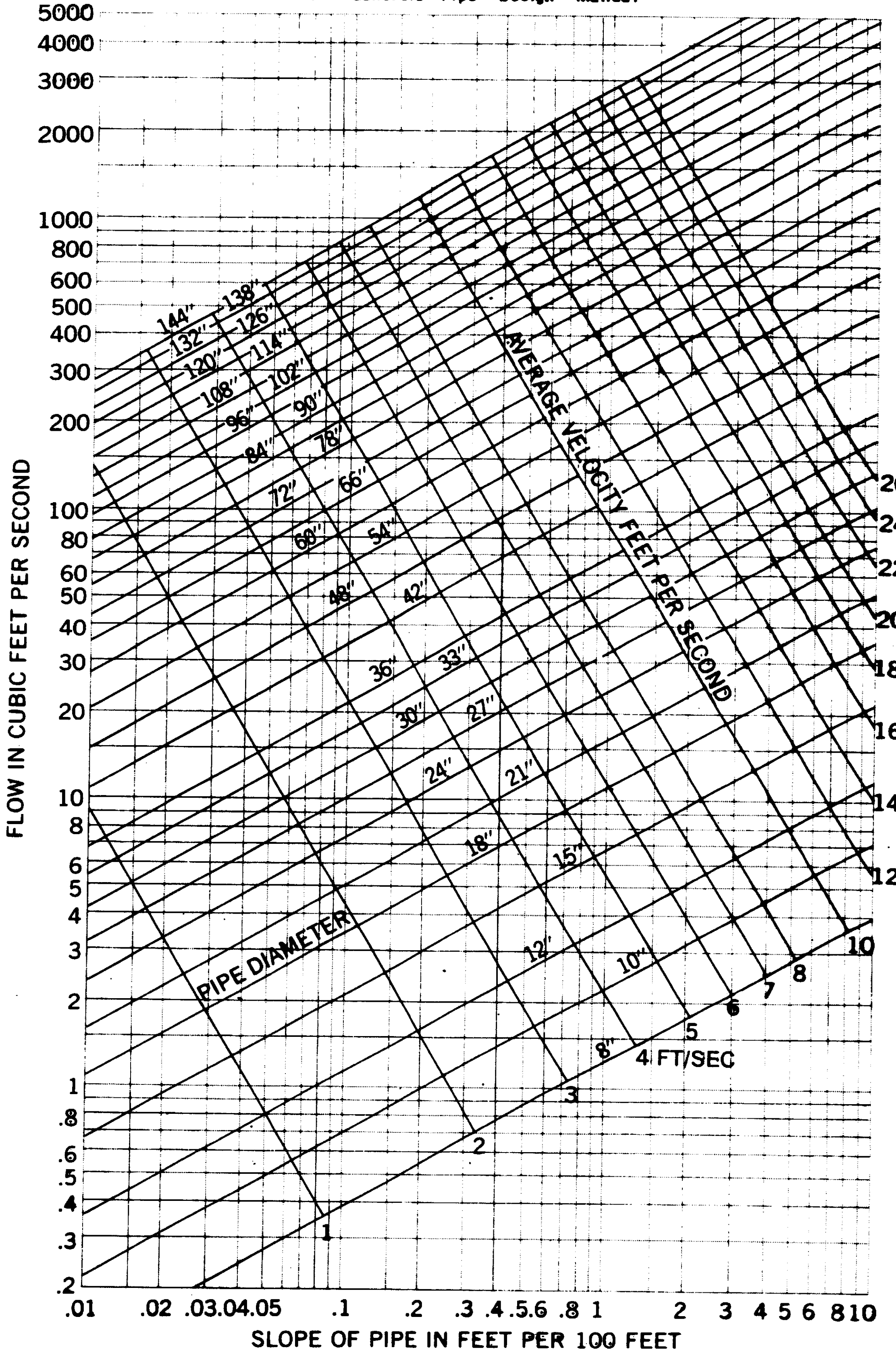
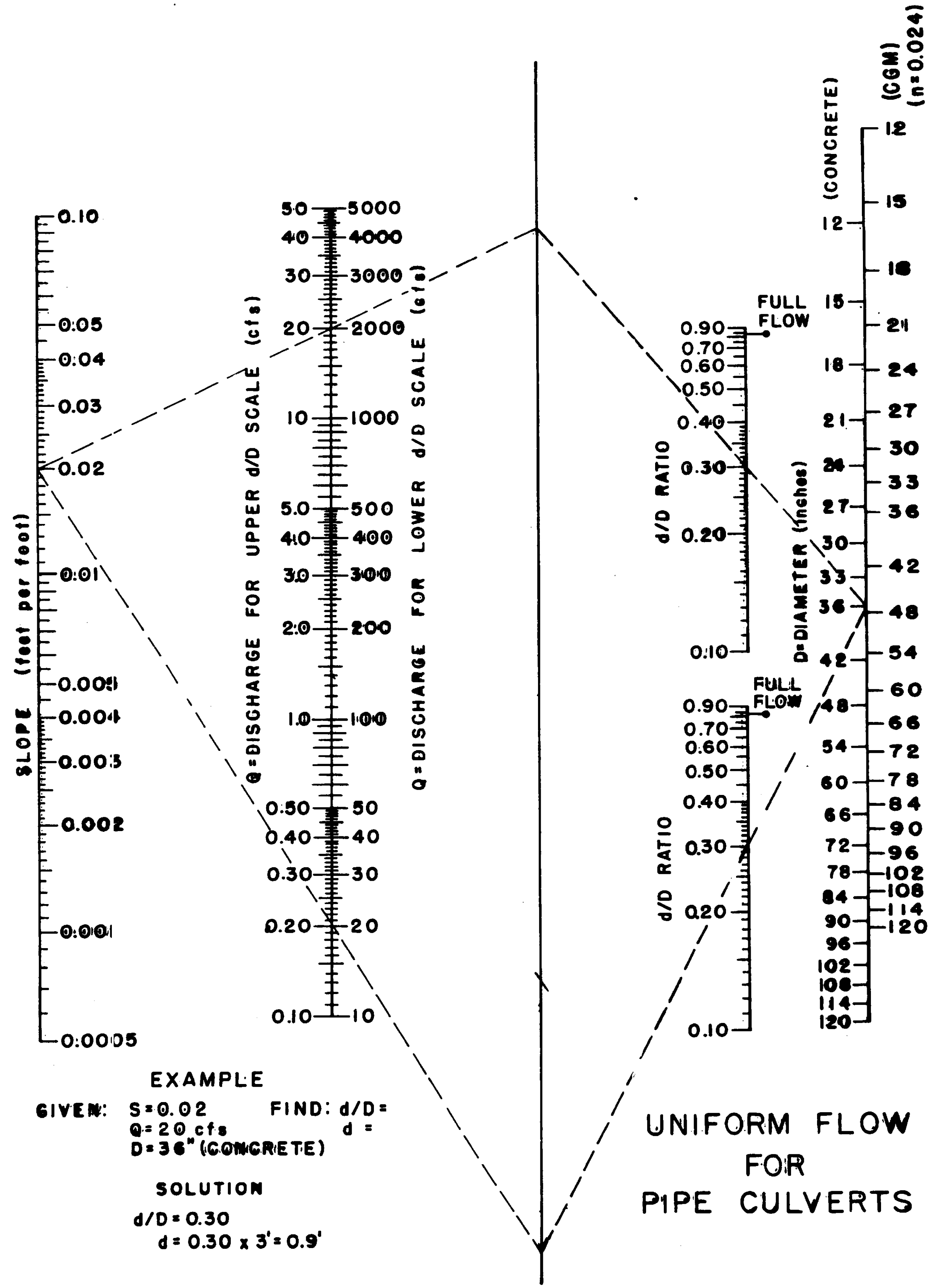
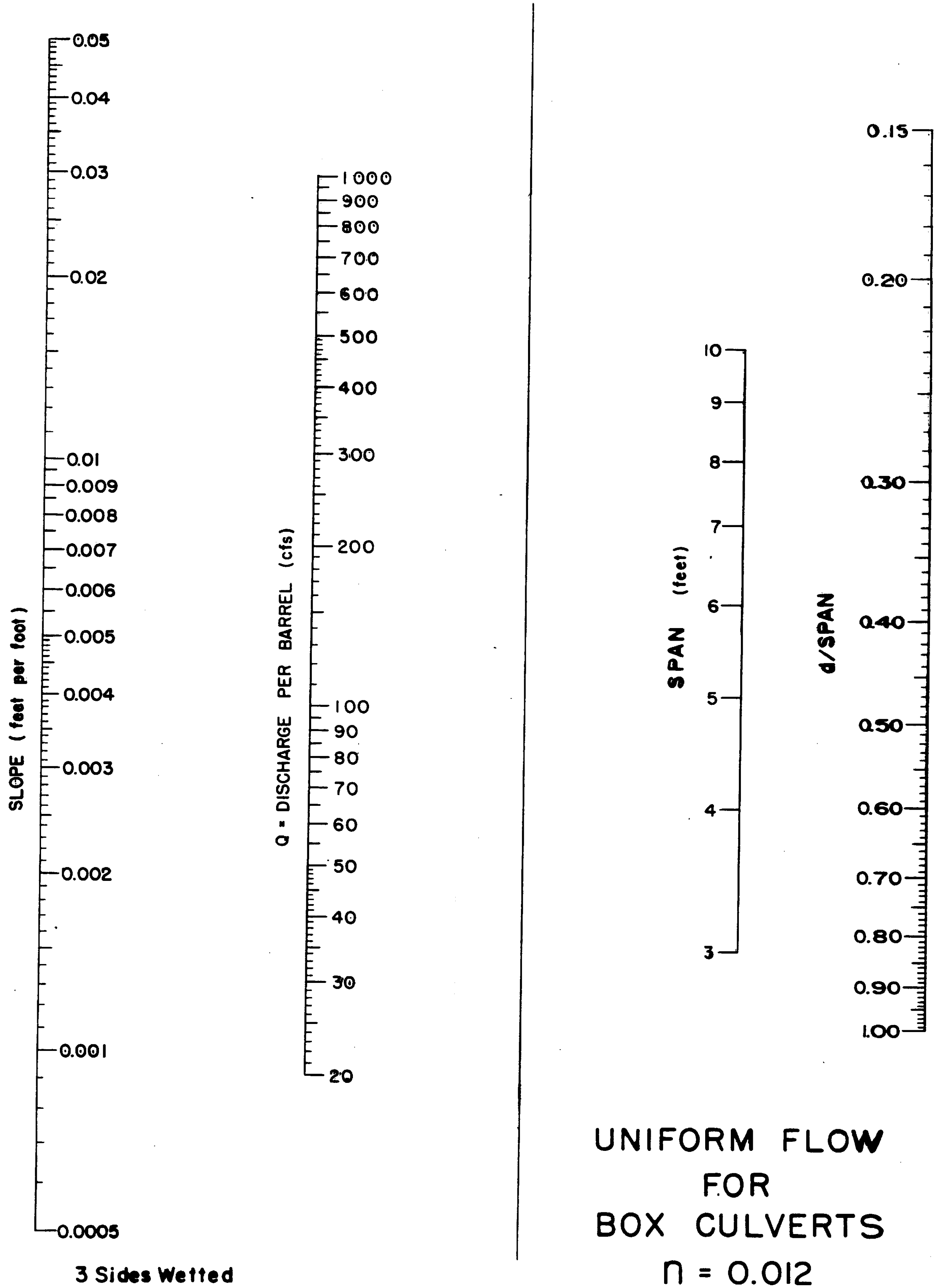


CHART 5-9



SLOPE (feet per foot)
0.10
0.05
0.04
0.03
0.02
0.01
0.005
0.004
0.003
0.002
0.001
0.0005
Q=DISCHARGE FOR UPPER d/D SCALE (cfs)
Q=DISCHARGE FOR LOWER d/D SCALE (cfs)
d/D RATIO
FULL FLOW
D=DIAMETER (inches)
(CONCRETE)
(CGM) (n=0.024)
EXAMPLE
GIVEN: S=0.02 Q=20 cfs D=36" (CONCRETE)
FIND: d/D= d =
SOLUTION
d/D=0.30
d=0.30 x 3'=0.9'
UNIFORM FLOW FOR PIPE CULVERTS

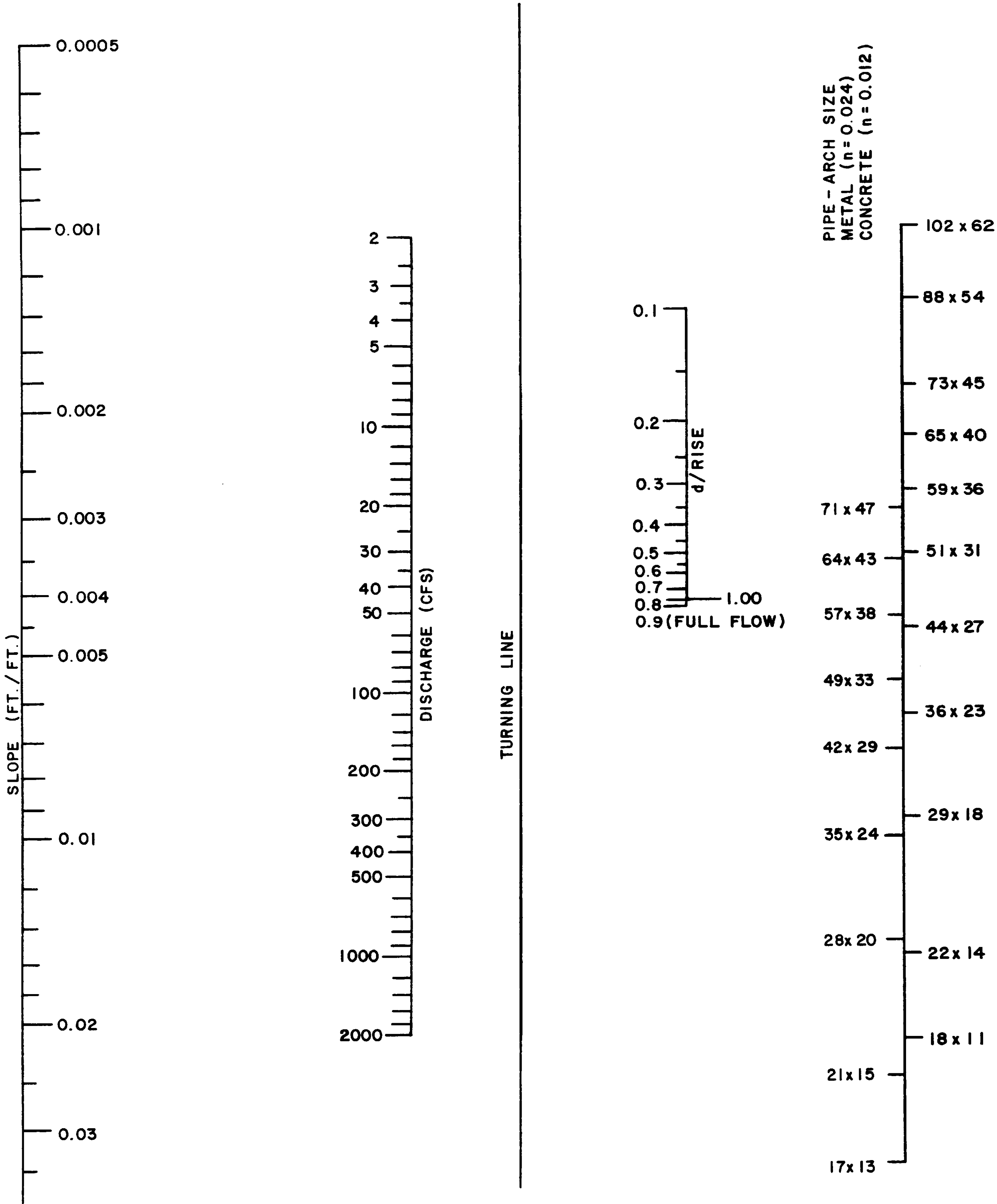
CHART 5-10



UNIFORM FLOW FOR BOX CULVERTS

$n = 0.012$

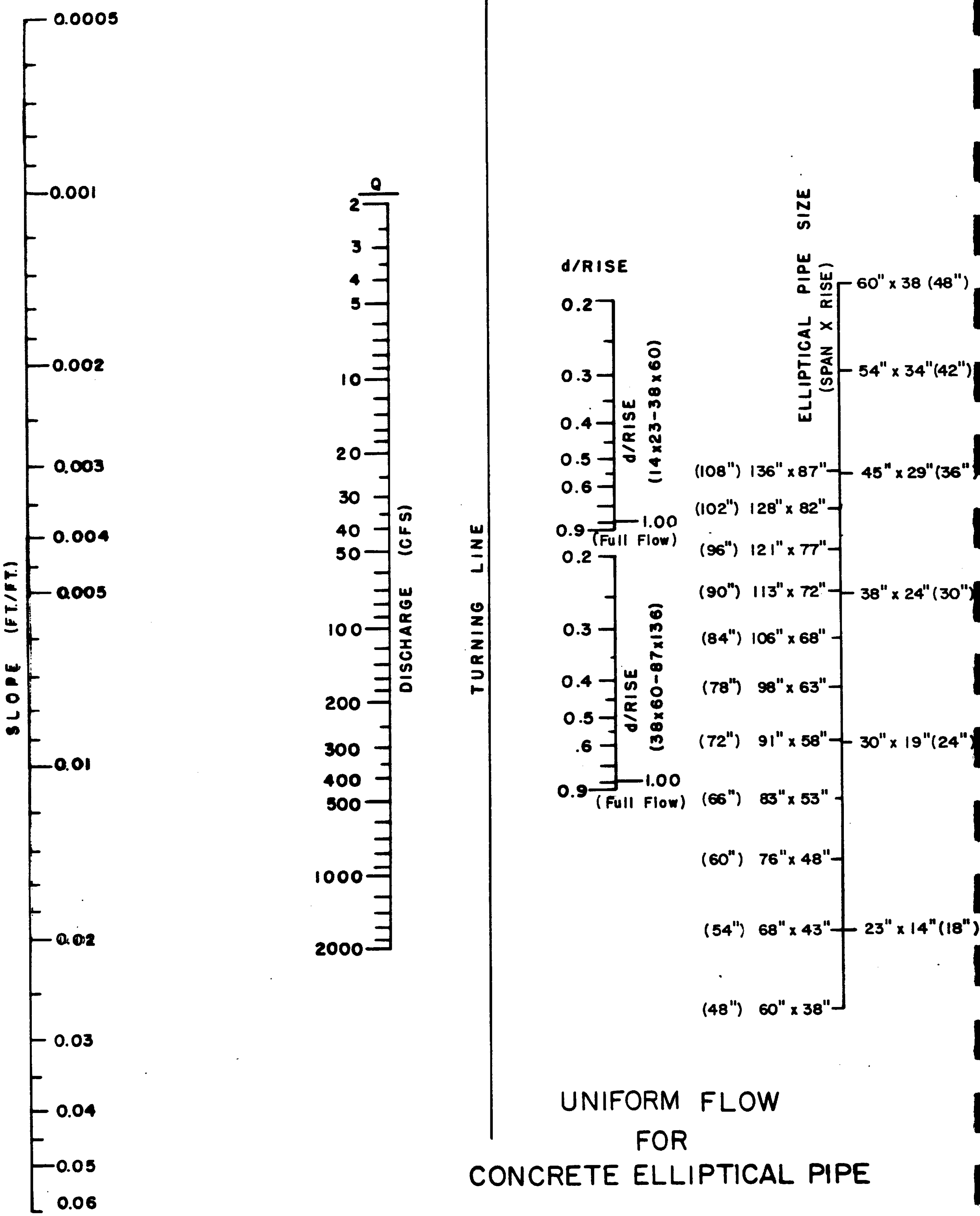
CHART 5-11



UNIFORM FLOW FOR PIPE-ARCH

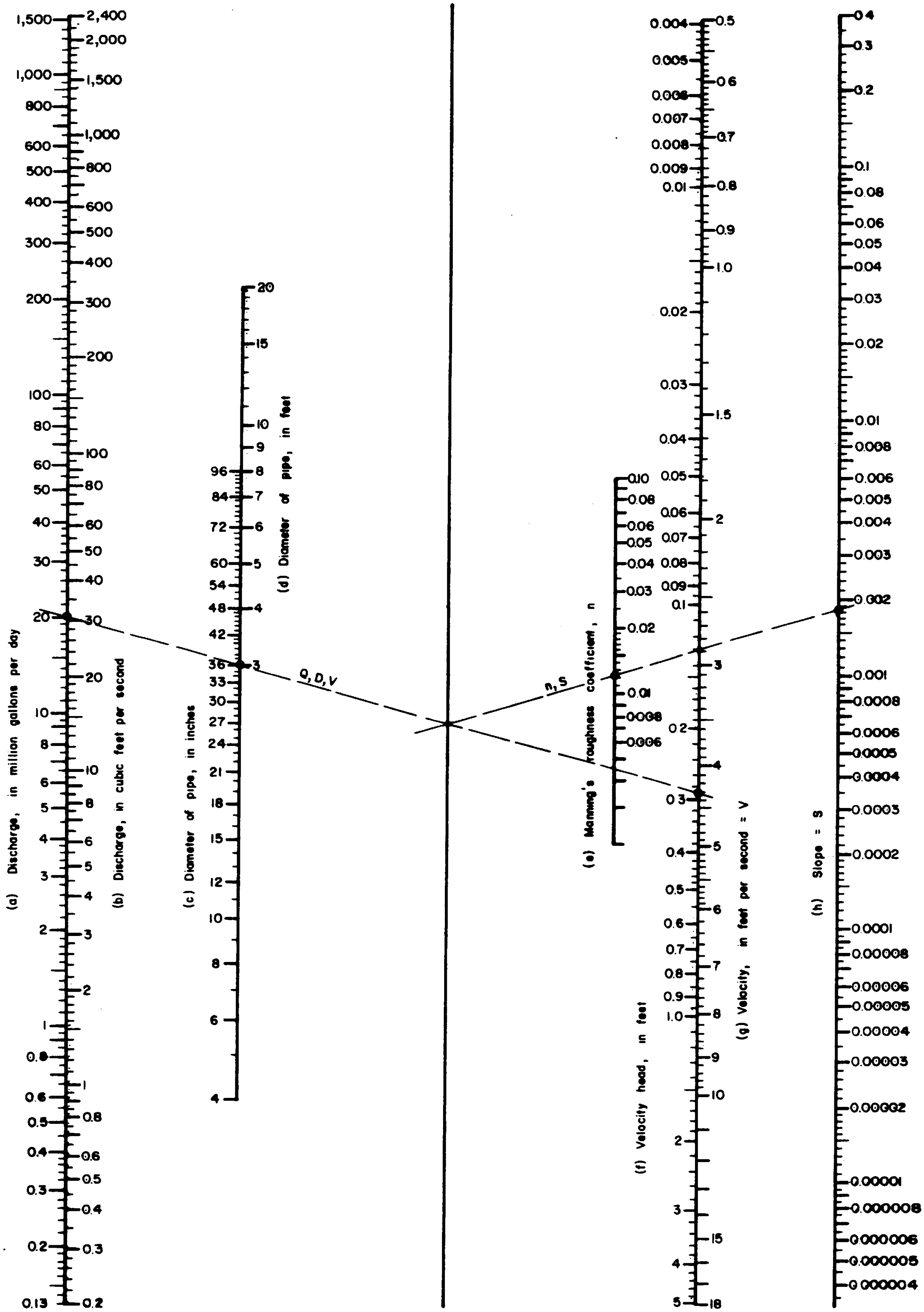
CHART 5-12

Revised 5-16-83



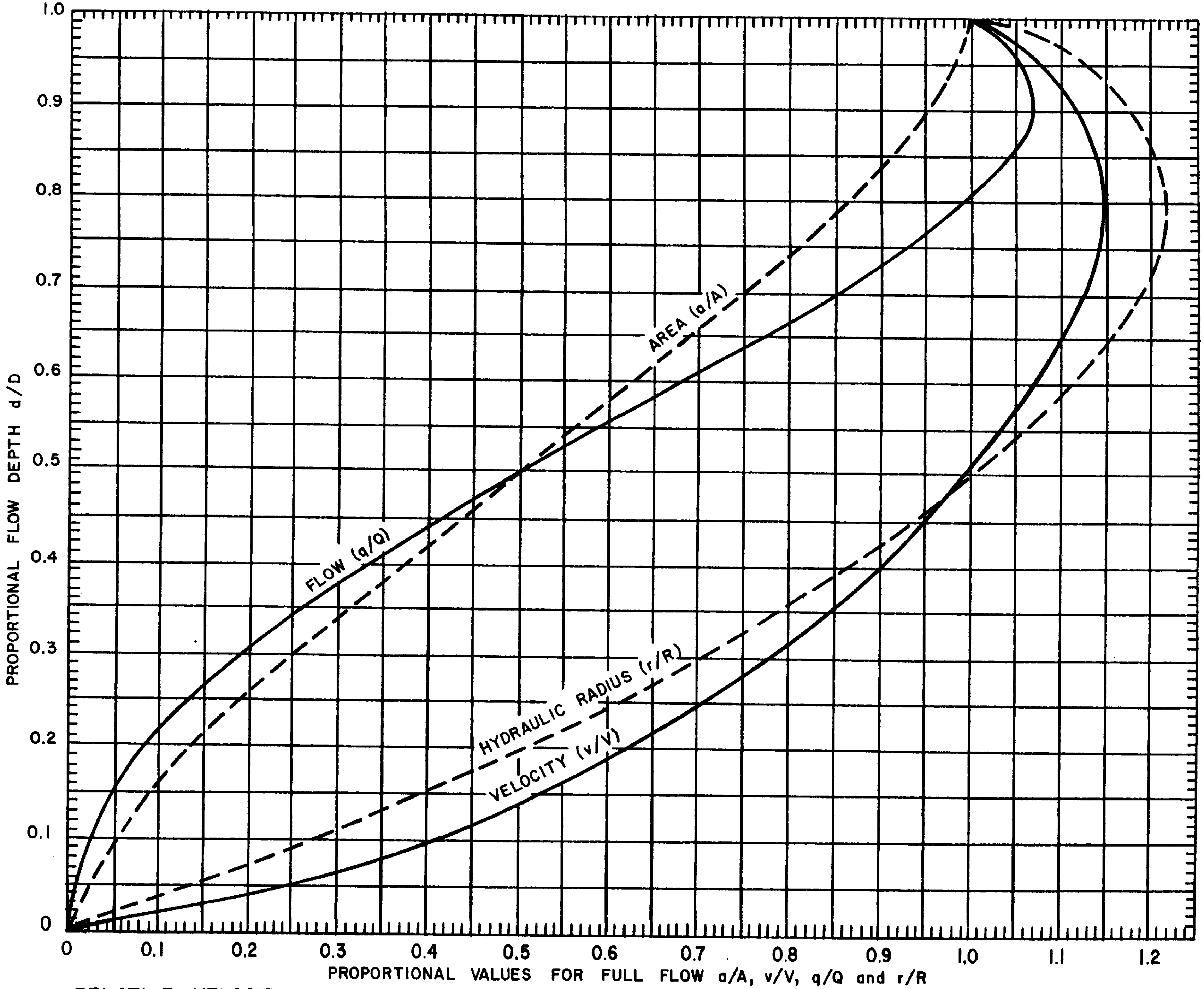
UNIFORM FLOW
FOR
CONCRETE ELLIPTICAL PIPE

CHART 5-13



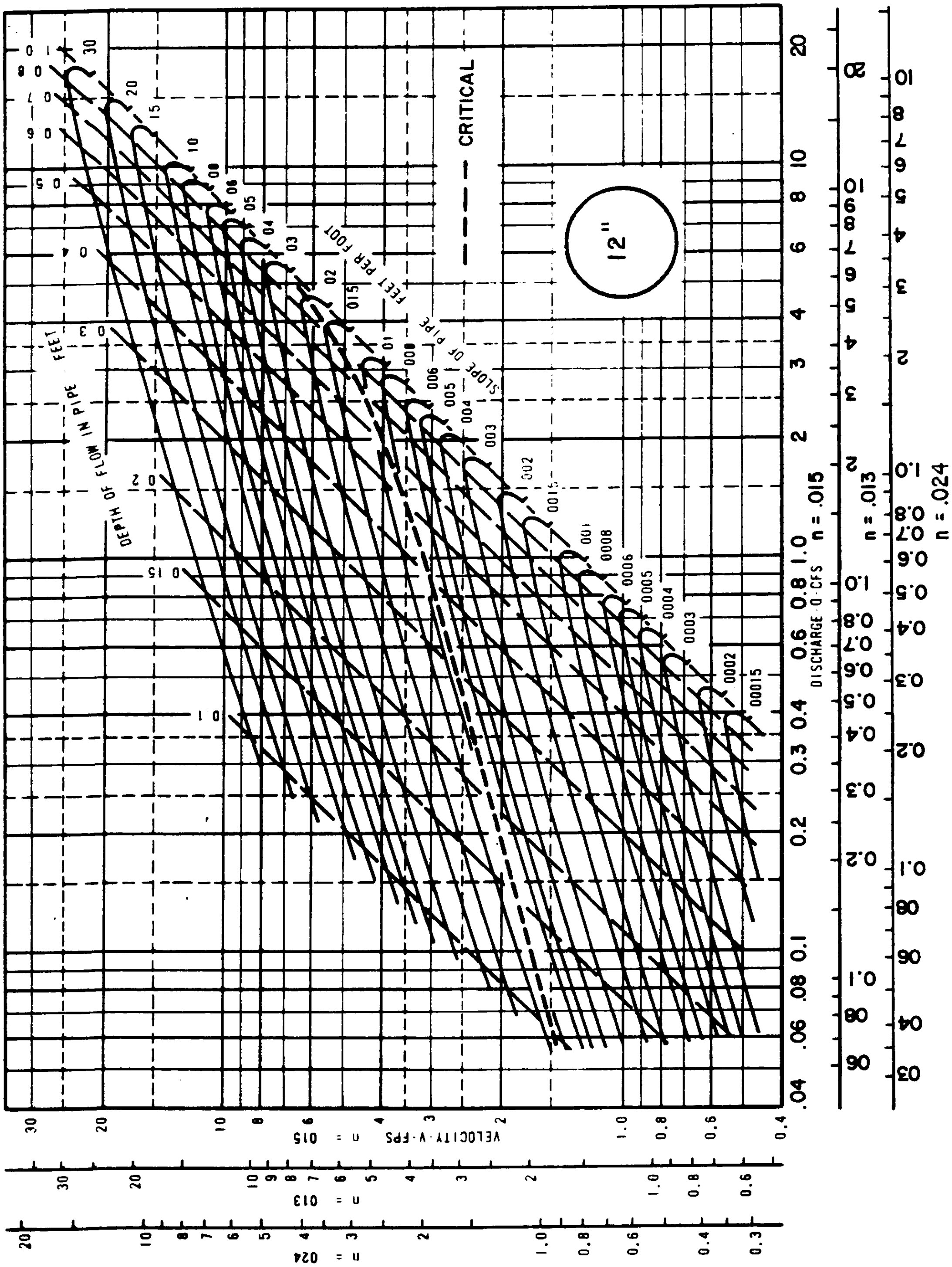
NOMOGRAPH for flow in round pipe - Manning's formula
(From Design and Construction of Concrete Sewers', Portland Cement Assoc.)

CHART 5-14

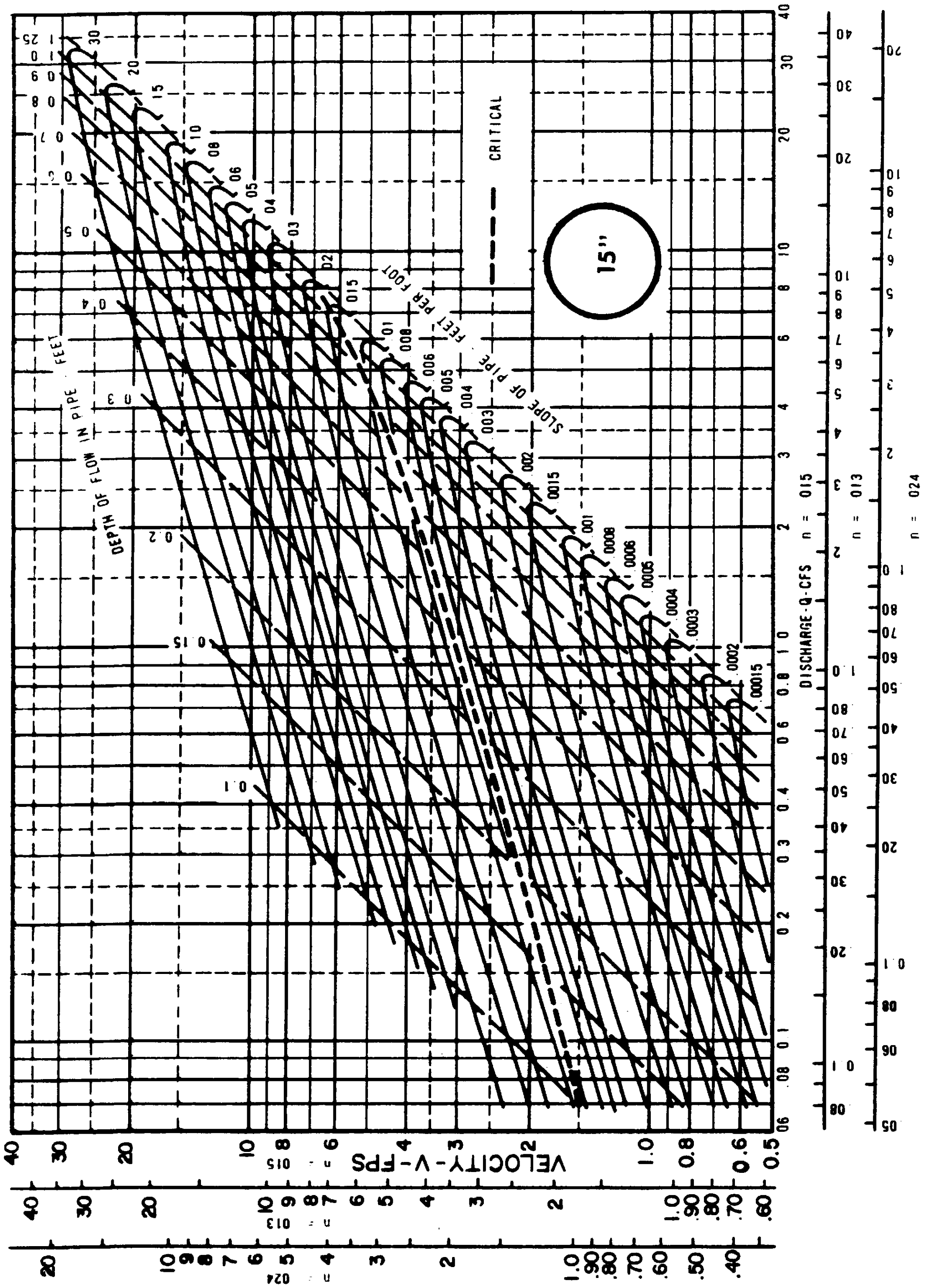


RELATIVE VELOCITY, AREA, AND DISCHARGE IN A CIRCULAR PIPE FOR ANY DEPTH OF FLOW

CHART 5-15

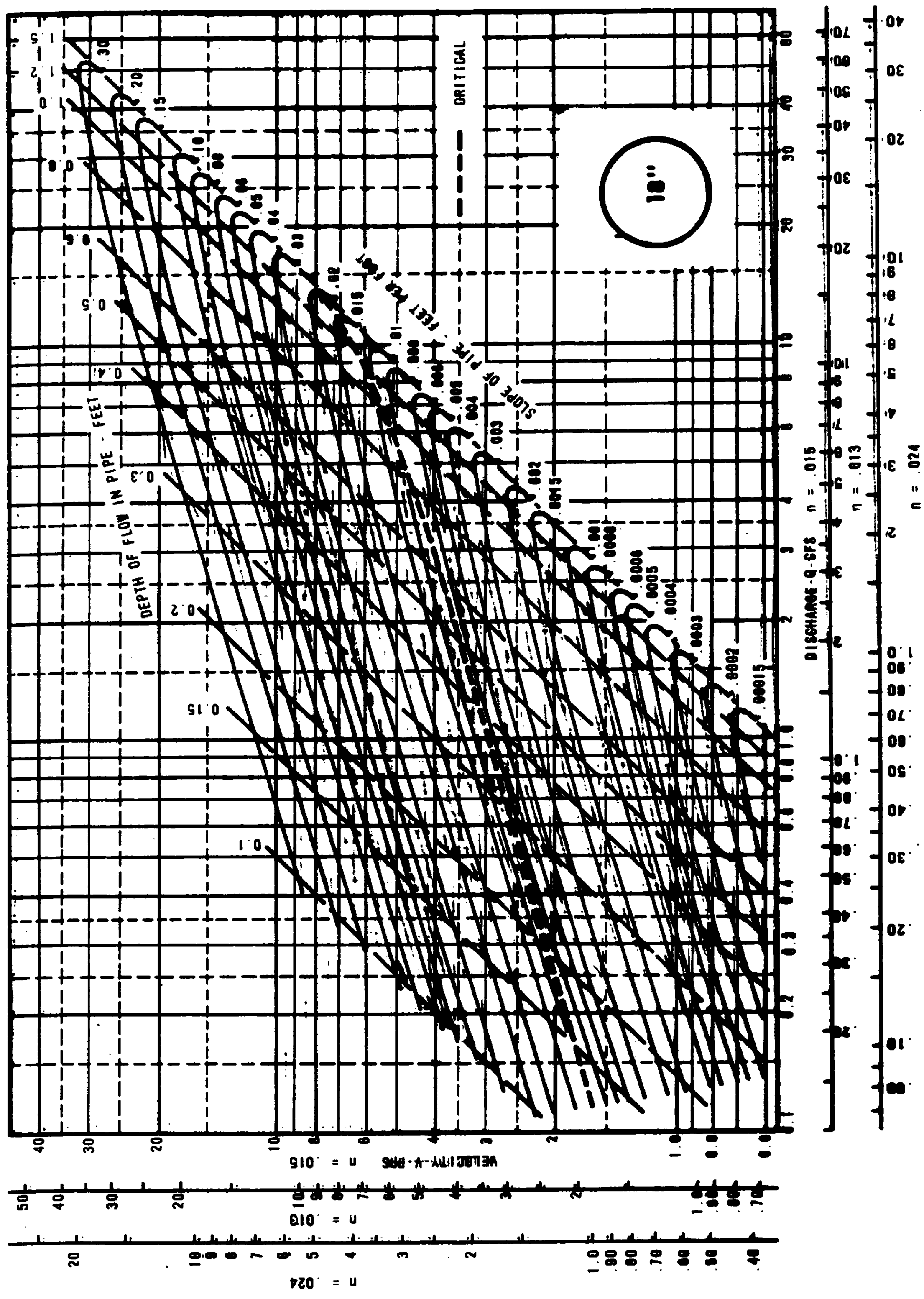


Pipe Flow Chart -12-Inch Diameter
CHART 5-15.1.

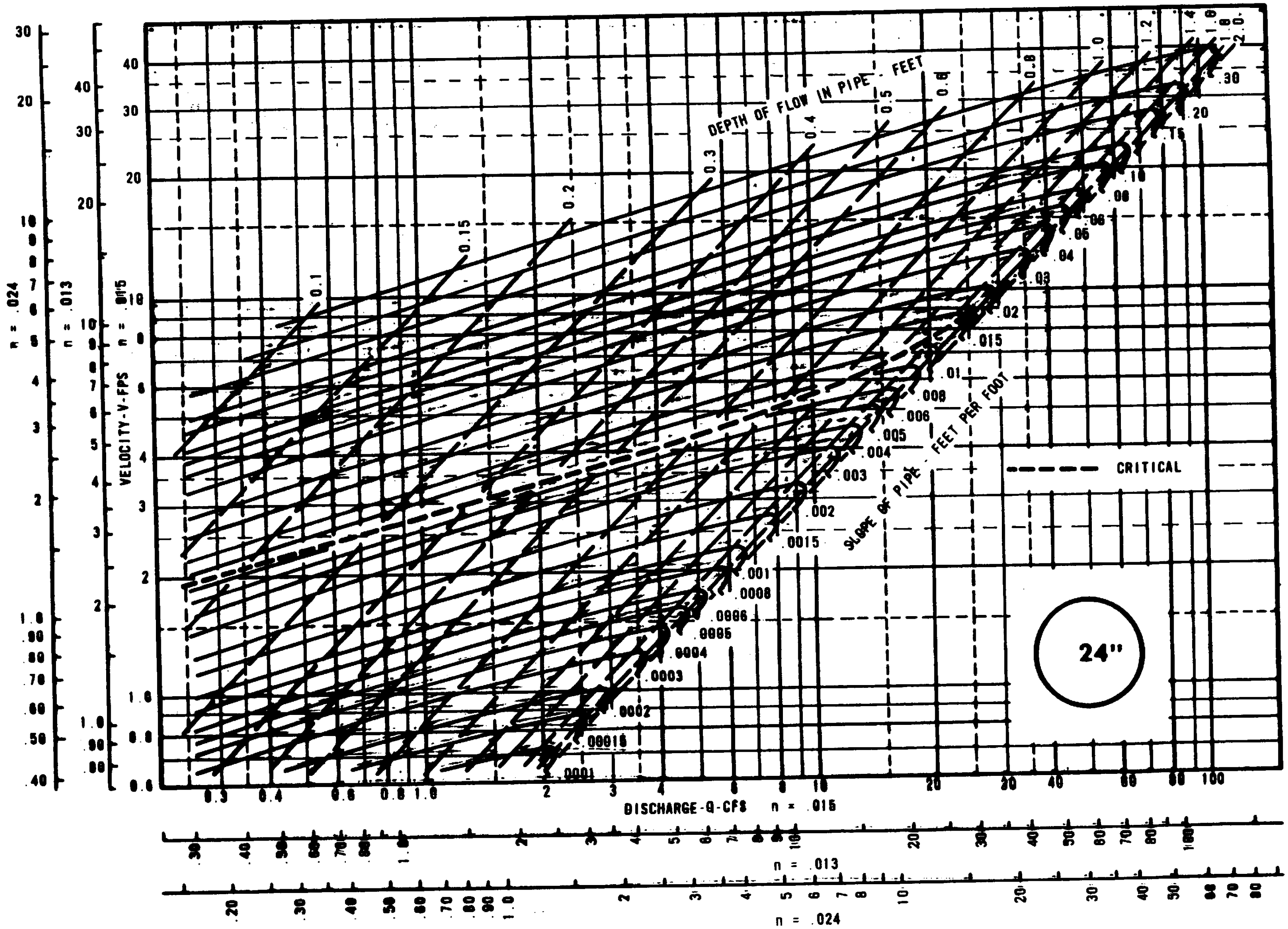


Pipe Flow Chart - 15 - Inch Diameter

CHART 5-15.2

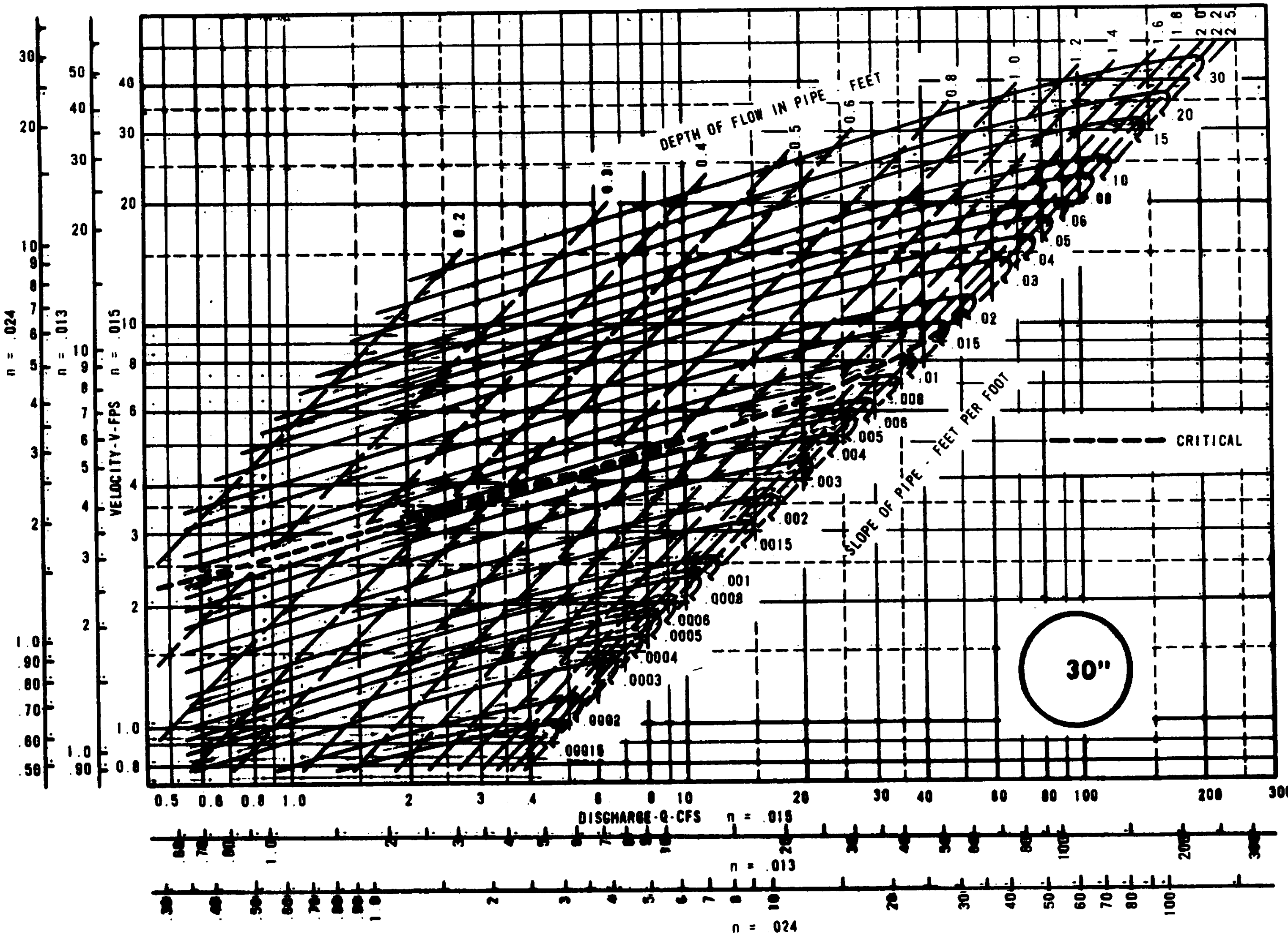


Pipe Flow Chart 18-Inch Diameter
CHART 5-16



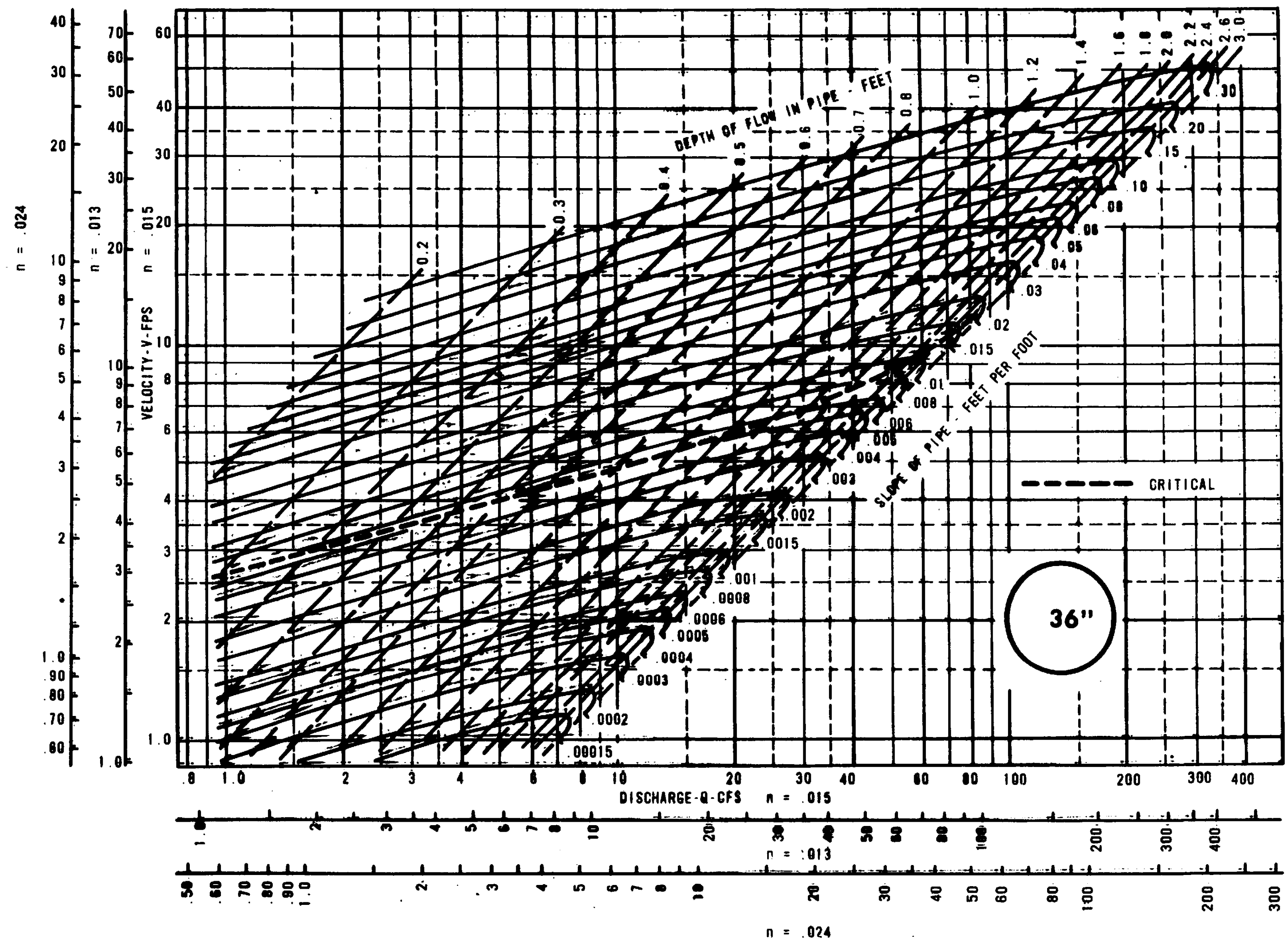
Pipe Flow Chart 24-Inch Diameter

CHART 5-17



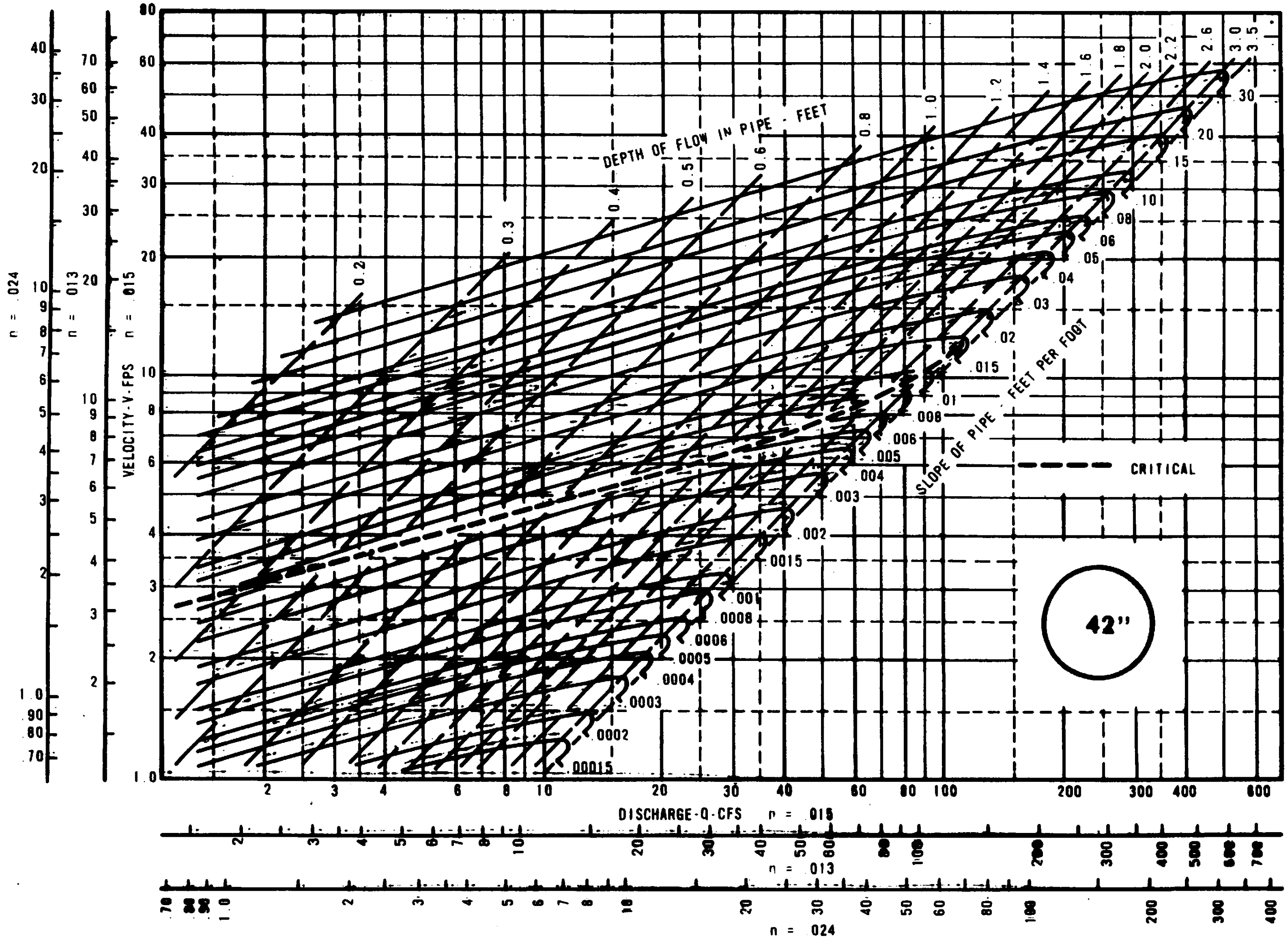
Pipe Flow Chart 30-Inch Diameter

CHART 5-18



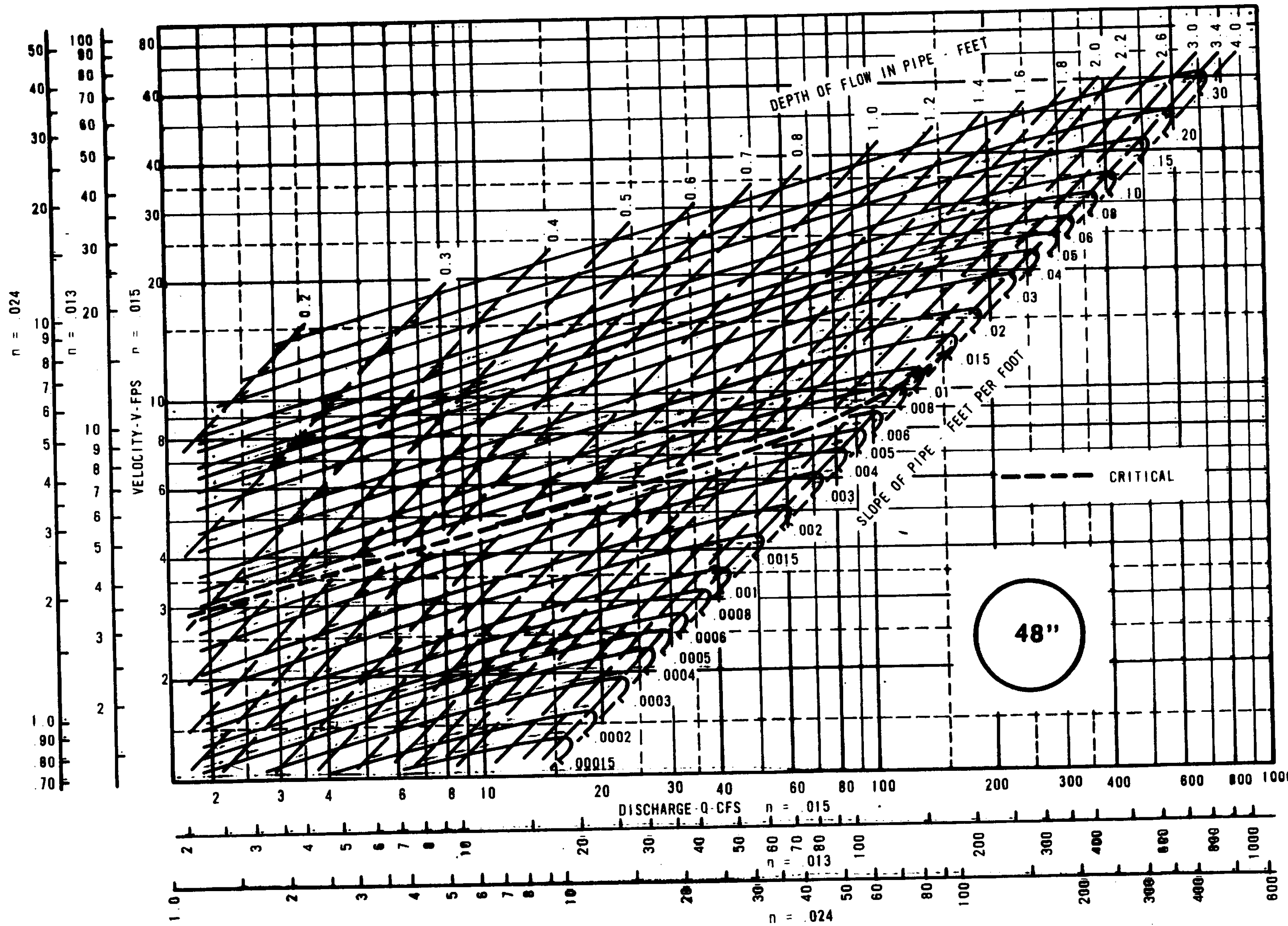
Pipe Flow Chart 36-Inch Diameter

CHART 5-15



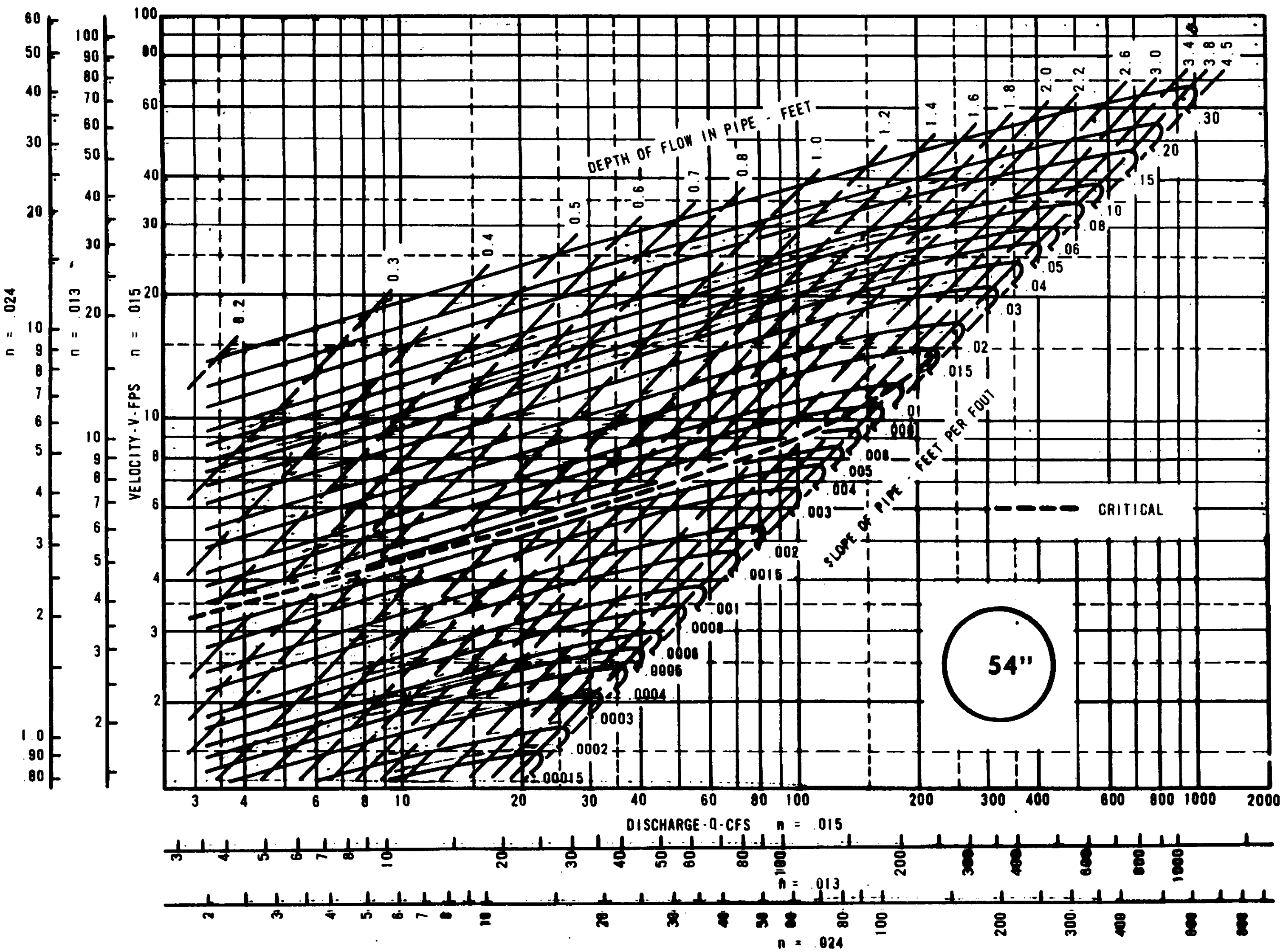
Pipe Flow Chart 42-Inch Diameter

CHART 5-20



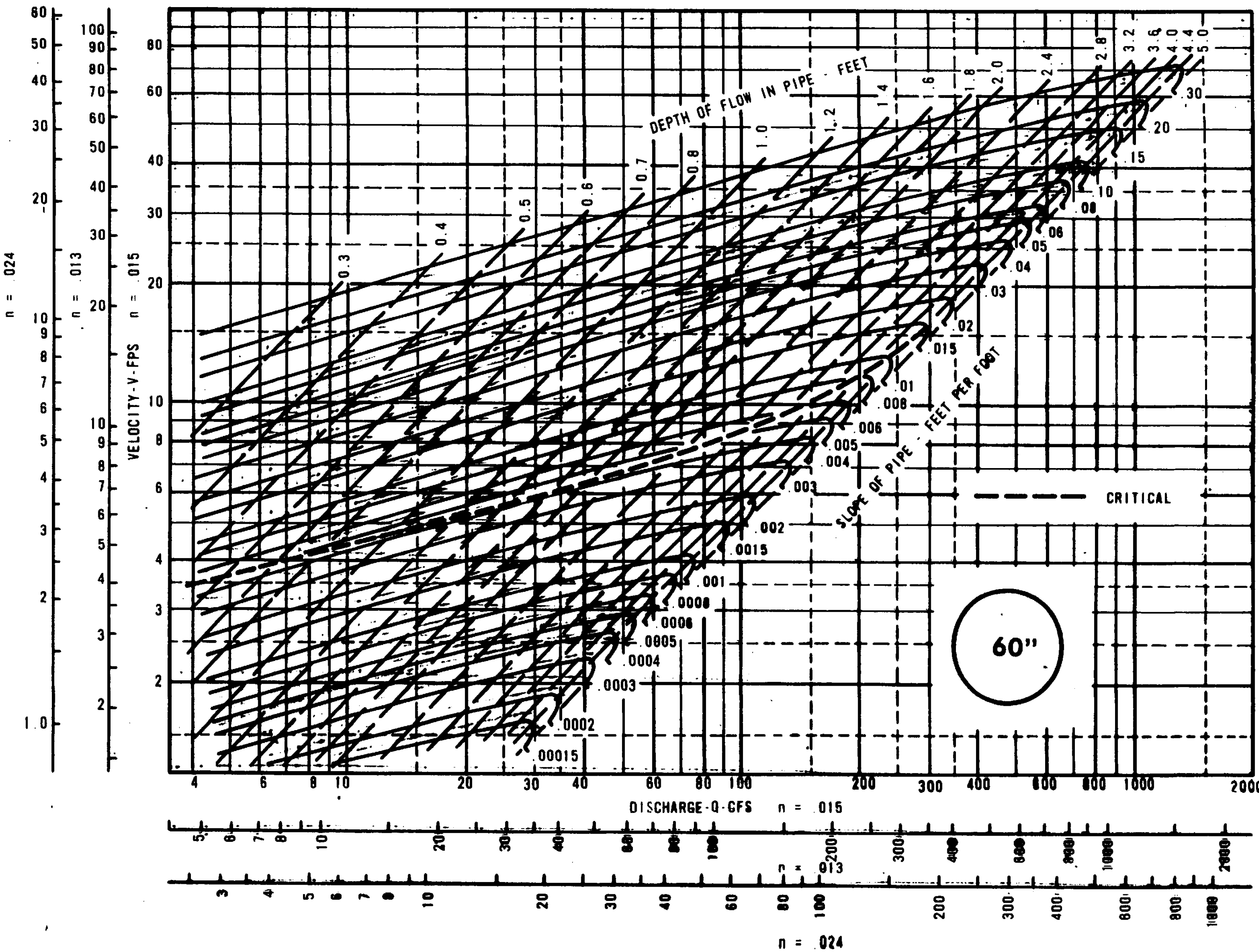
Pipe Flow Chart 48-Inch Diameter

CHART 5-21



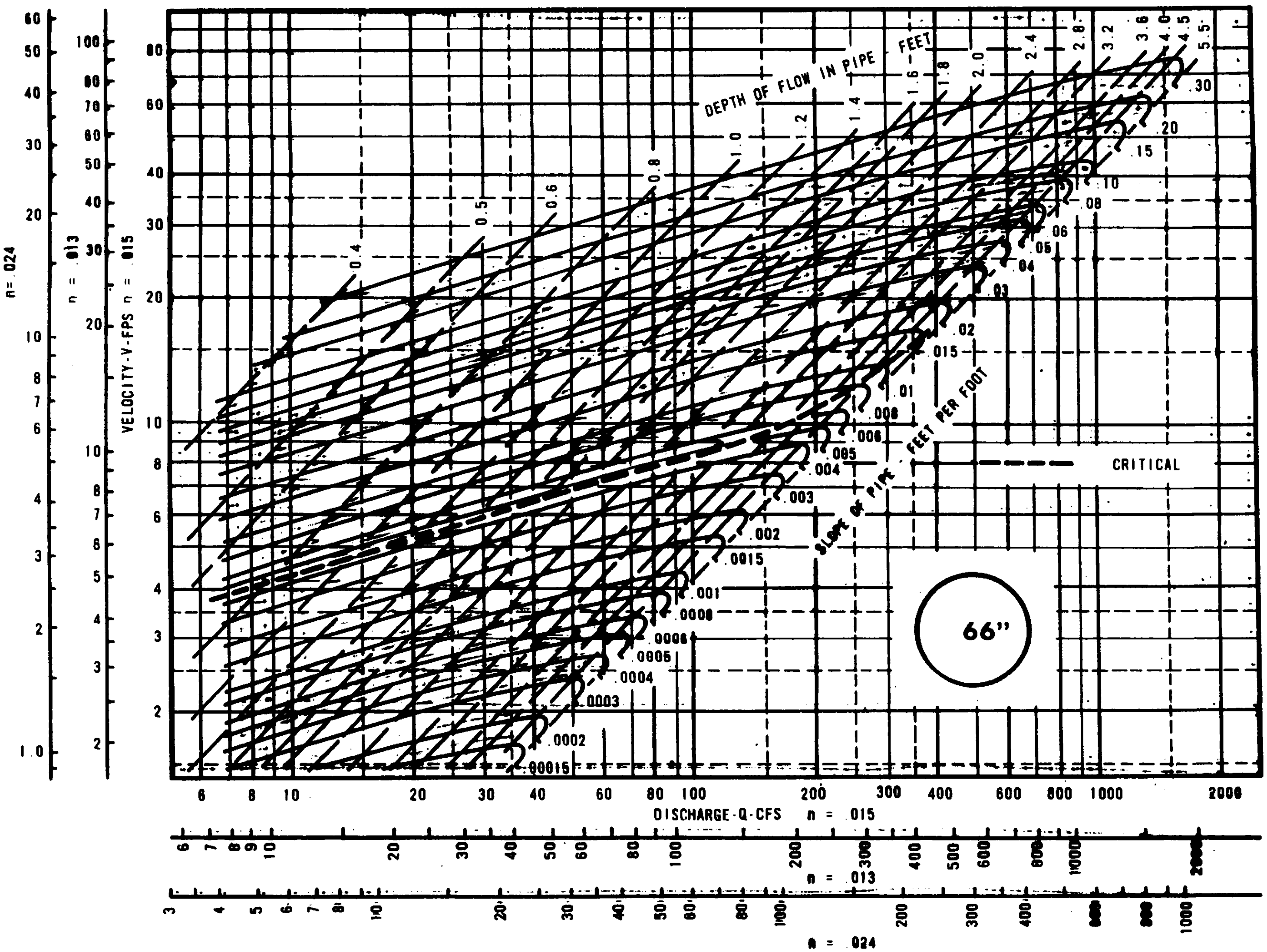
Pipe Flow Chart 54-Inch Diameter

CHART 5-22



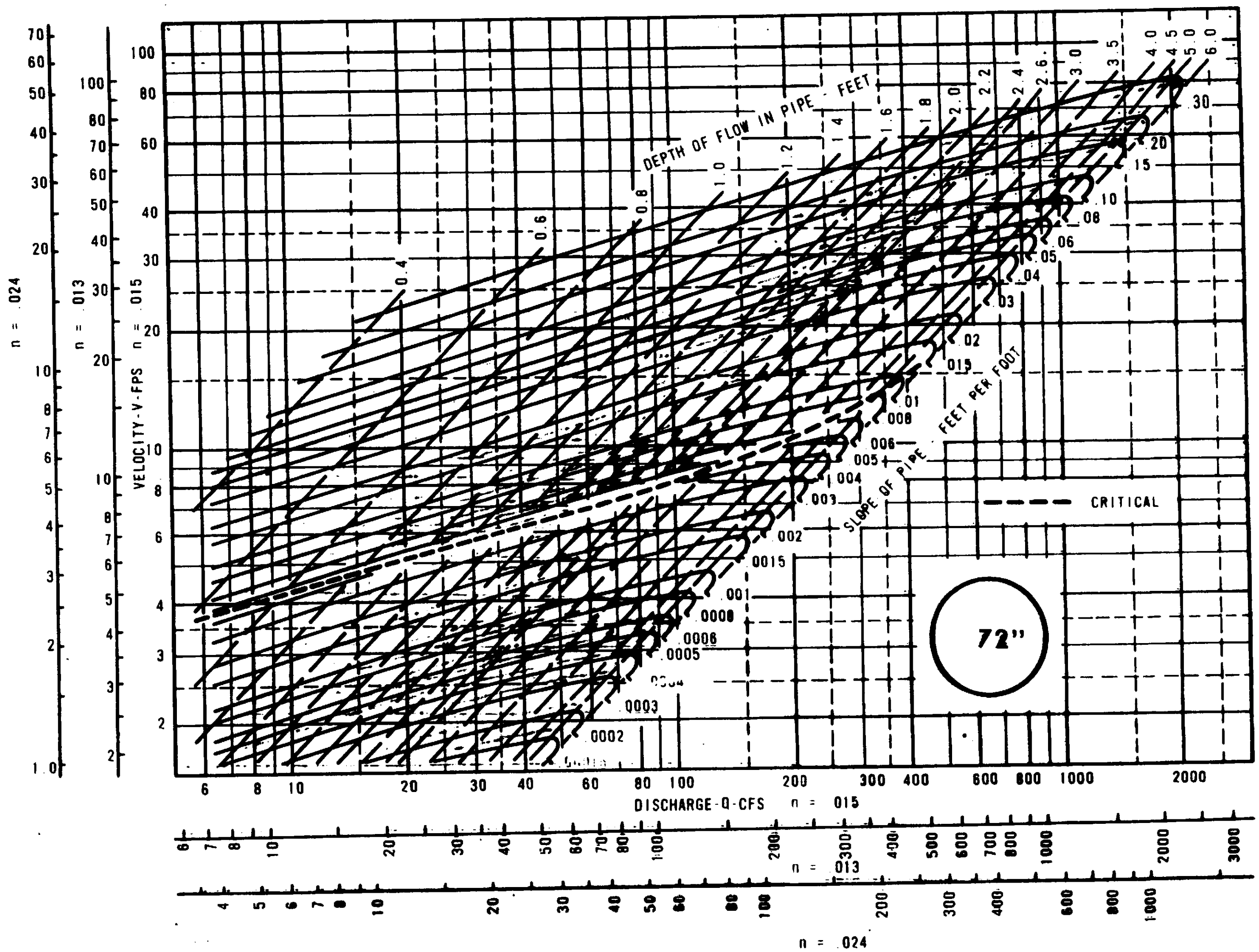
Pipe Flow Chart 60-Inch Diameter

CHART 5-23



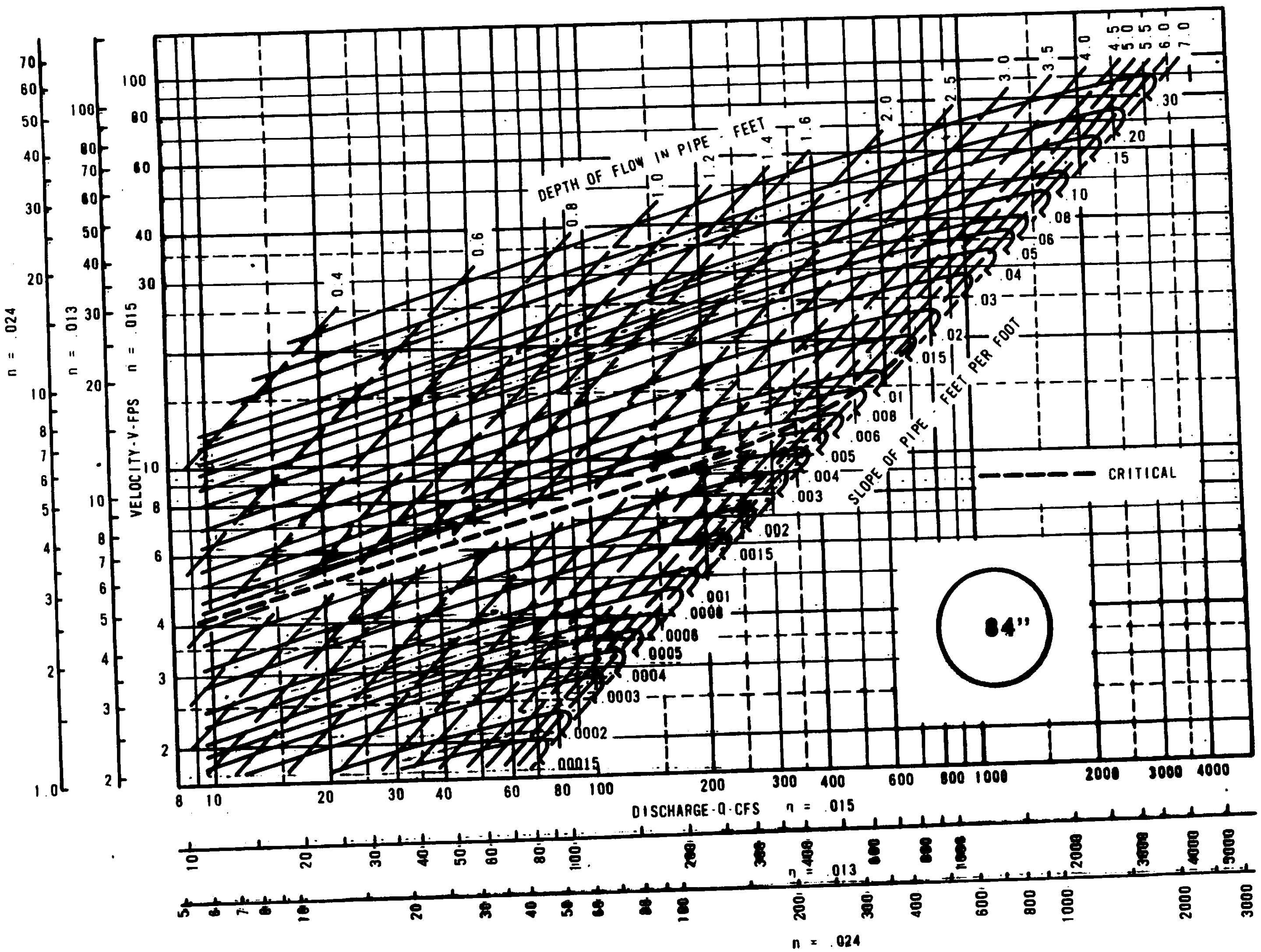
Pipe Flow Chart 66-Inch Diameter

CHART 5-24



Pipe Flow Chart 72-Inch Diameter

CHART 5-25



Pipe Flow Chart 84-Inch Diameter

CHART 5-26

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CHAPTER 5

STORM SEWERS

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5-2 URBAN DRAINAGE/FLOOD CONTROL CRITERIA MANUAL AND HANDBOOK, by Wright-McLaughlin Engineers, Engineering Consultants, Denver, Colorado, 1979.

5-3 MODERN SEWER DESIGN, American Iron and Steel Institute, Washington, D.C., 1980.

5-4 HANDBOOK OF APPLIED HYDROLOGY, by Ven Te Chow, McGraw-Hill Book Co., 1964.

5-5 SIMPLIFIED METHOD FOR DESIGN OF CURB-OPENING INLETS, by C.F. Izzard, Transportation Research Record 631: Geometrics, Water Treatment, Utility Practices, Safety Appurtenances and Outdoor Advertisement, 1977.

5-6 HYDRAULIC MANUAL, by the Bridge Division, Texas Highway Department, 1980.

5-7 BICYCLE-SAFE GRATE INLETS STUDY, Federal Highway Administration, Offices of Research and Development, Washington D.C., 1980.

5-8 OPEN - CHANNELS HYDRAULICS, by Ven Te Chow, McGraw-Hill Book Company, 1959.

5-9 <u>DRAINAGE DESIGN MANUAL</u>, by Missouri Highway and Transportation Commission, Jefferson, Missouri, 1978.

5-10 <u>CONCRETE PIPE DESIGN MANUAL</u>, by American Concrete Pipe Association, Vienna, Virginia, 1980.

5-11 <u>DRAINAGE DESIGN MANUAL</u>, by the State of Oklahoma Department of Transportation, Oklahoma City, Okla., 1980 Preliminary Draft.

5-12 <u>BICYCLE-SAFE GRATE INLETS DESIGN MANUAL</u>, Federal Highway Administration, Offices of Research and Development, Washington, D.C., 1980.

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OPEN CHANNEL

6-100 INTRODUCTION

Open channels are defined as any conduits in which the flow has a free surface exposed to atmospheric pressure. These conduits are sized to carry the design flow and to maintain the energy conditions of the existing natural channel whenever possible. Open-channel flow includes flow in irrigation ditches, flumes, channel changes, side ditches, rivers, streams, roadside ditches (our primary concern in this chapter) and storm sewers. Included in this chapter are charts for graphical solutions to open-channel problems using the Manning equation, principles of flow in open channels, types of channels, and other explanatory data which may be helpful in solving various problems pertaining to open-channel flow.

6-200 TYPES OF OPEN CHANNEL FLOW

Flow in open channels can be categorized by the following: Steady versus unsteady, uniform versus nonuniform, supercritical and subcritical. Flow is classifed as steady if the the rate of discharge is not varying with time. It can be further classified as nonuniform if velocity and depth of flow change from section to section. To have uniform flow the grade must be constant and all cross sections of flow must identical in form, roughness and area. Uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes in highway engineering, however, uniformity can be applied to most stream flow problems. Minor undulations in streambed or minor deviations from the average cross section can be ignored in highway drainage design as long as the standard deviation is not too large.

6-300 KINDS OF OPEN CHANNELS

There are two types of channels which will be discussed in this chapter - artificial and natural. Natural channels are water courses which are found in nature and are not formed by artificial construction methods. These channels include streams, creeks, tributaries, and other natural water courses. Artificial channels, conversely, are created by construction methods and are listed and defined below:

1. Gutters - gutters are the channels at the edges of the pavement or the shoulders formed by curbs or by shallow depressions. Gutters are usually paved with concrete, brick, stone blocks, or other structural material.

2. Chutes - chutes are steeply inclined open or closed channels which convey collected water to a lower level. Open chutes can be metal or be paved with portland cement concrete, bituminous material, stone or sod, depending upon the volume and velocity of the water to be removed.

3. Roadway channels - roadway channels are the channels provided in the cut section to remove the runoff from rain falling on the roadway and on the cut slopes. These channels are sometimes called gutters when paved.

4. Toe of slope channels - toe of slope channels are located when it is necessary to convey water collected by the roadway channel to the point of disposal. When the downhill side of the highway, this channel can often be laid on a mild slope and the lower end flared to spread the water over the hillside. Where this practice would cause erosion or permit water to drain into the highway embankment, the toe of slope channel must convey the storm water to a natural watercourse.

5. Intercepting channels - intercepting channels are located on the natural ground near the top edge of a cut slope or along the edge of the right-of-way, to intercept the runoff from a hillside before it reaches the roadway. Intercepting the surface flow reduces erosion of cut slopes, lessens silt deposition and infiltration in the roadbed area, and decreases the likelihood of flooding of the highway in severe storms.

6. Median swales - median swales are the shallow depressed areas at or near the center of medians used to drain the median area and portions of the roadway. The depressed area of swale is sloped longitudinally for drainage, and at intervals the water is intercepted by inlets and discharged from the roadway. It is not necessary that the longitudinal slope of the swale conform to the pavement grade, particularly on flat grades.

6-400 CONTINUITY AND ENERGY IN OPEN CHANNEL FLOW

When dealing with open-channel flow, two important factors to consider are continuity and energy. Continuity of flow must be satisfied in all flow problems and energy provides the basis for water to flow in an open channel and what it will do at certain points downstream.

6-401 CONTINUITY EQUATION

Continuity states the fact that no fluid is lost or gained and no cavities are formed or destroyed as the fluid passes through a conduit. When the fluid is essentially incompressible (as we assume water to be), continuity may be expressed as follows:

$$Q = A_1V_1 = A_2V_2 = A_nV_n$$

where

Q = is the discharge, A_1V_1, A_2V_2 and A_nV_n are cross sectional areas times average velocities of sections 1,2, and any section n, respectively.

Summarily,then,the flow is equal across any two sections in the conduit.

6-402 BERNOULLI ENERGY EQUATION

The basic principle which is used most often in hydraulics is the law of conservation of energy as expressed by the Bernoulli equation. In essence,the equation is a way of expressing the rate of energy dissipation in pipes and other open channels. When water flows in a channel or conduit it possesses kinetic and potential energy. The potential energy is due to the position of water above some datum line while the kinetic energy is due to velocity of flow. All flowing water has a velocity head which is equal to:

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

where

V = mean velocity in feet per second

g = acceleration of gravity (32.2 feet per second per second)

Bernoulli's equation states that the total head or total energy at any section is equal to the total head or total energy at any section downstream, plus any intervening head losses (h_L). Head losses are energy losses due to friction, channel contraction, changes in alignment, etc. The equation is as follows:

$$d_1 + \frac{V_1^2}{2g} + Z_1 = d_2 + \frac{V_2^2}{2g} + Z_2 + h_L$$

where

d_1, d_2 = depth of flow at upstream and downstream section, respectively

$\frac{V_1^2}{2g}$, $\frac{V_2^2}{2g}$ = velocity head of upstream and downstream section, respectively

Z_1, Z_2 = height above datum plane at upstream and downstream section, respectively

h_L = other head losses

Figure 6-1 diagrams the total energy head and lists specific head, which involves the depth of flow and velocity head terms. The total head line is also referred to as the energy gradient and when the velocity head is subtracted from this line you have the hydraulic gradient.

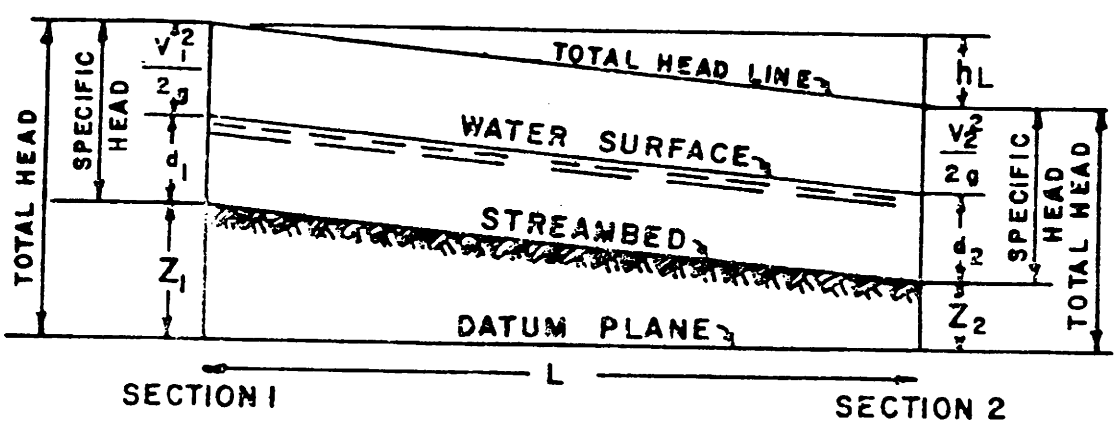


FIGURE 6-1 - Water surface profile of channel with uniform flow.

6-500 UNIFORM FLOW CALCULATIONS

In solving open-channel problems certain assumptions must be made in order to solve the problem as quickly as possible. It is always assumed that flow is uniform to make it possible to use Manning's equation, which can be used for uniform but not for nonuniform flow. Depth is constant, as is velocity.

6-501 - MANNING EQUATION

One of the earliest attempts to express energy loss in a pipe was developed by Chezy in 1775. The formula is as follows:

$$V = c\ (RS)^{\frac{1}{2}}$$

Manning concluded that the c in Chezy's formula should vary with $R^{1/6}$, as follows:

$$c = \frac{1.486}{n} R^{1/6}$$

where n is a roughness coefficient and R is the hydraulic radius. Substitution of this term into Chezy's formula yields Manning's equation:

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

where V is the average velocity and S is the average slope of the water surface. By combining the above equation with the continuity equation, discharge can be determined.

$$Q = \frac{1.486}{n} R^{2/3} S^{1/2} A$$

where

Q = discharge in cubic feet per second
n = coefficient of roughness (see Table 6-1, pages 6-15 and 6-16)
R = hydraulic radius in feet (flow area/ wetted perimeter)
A = cross-sectional flow area in square feet
S = slope of water surface in feet per foot

The "n" value to be used in the above equation is a function of the type of material from which the channel is constructed. The "n" values to be used in the design of most open channels and pipes are given in Table 6-1, pages 6-15 and 6-16.

A nomograph for the solution of the Manning Equation has been developed and is presented as Chart 6-1, page 6-17 .

A guide for maximum permissible velocities in erodible (unlined) channels are given in Table 6-2, page 6-18.

TABLE 6-I

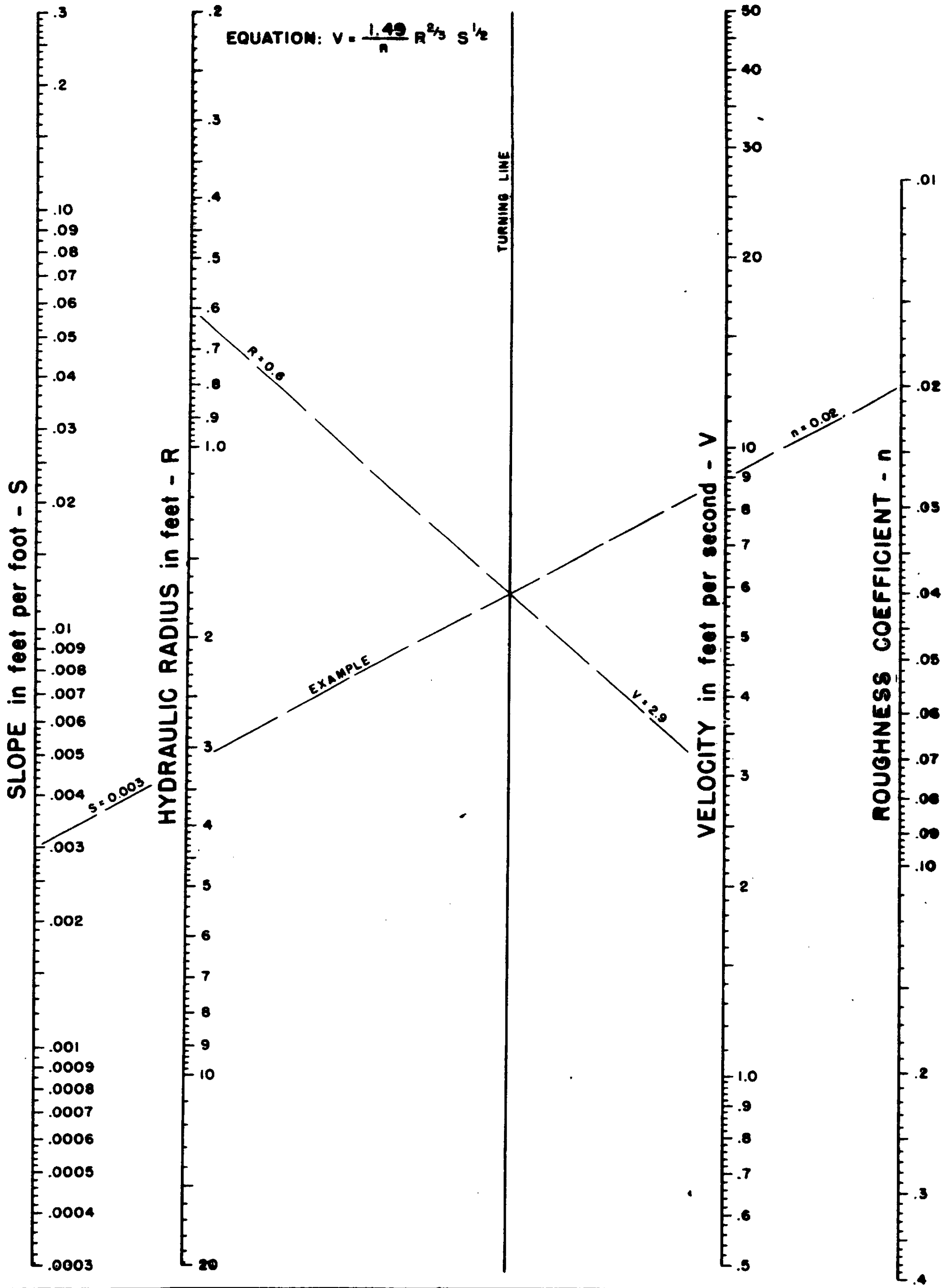
Mannings n For Channels, Natural Streams and Pipes

	Range	Ordinary Design Value
Open Channels - lined		
Asphalt		
Smooth		.013
Rough		.016
Concrete		
Smooth forms or troweled	.011-.015	.013
Float finish	.013-.016	.014
Unfinished	.014-.020	.017
Gunite, good		.019
Gunite, wavy		.022
Cemeted rubble masonry	.017-.030	.025
Dressed ashlar masonry	.013-.017	.015
Buck in Mortar	.012-.018	.015
Open Channels		
Excavated		
Earth, uniform and straight		
clean, neat-new to weathered	.016-.022	
short grass, few weeds	.022-.027	
gravelly soil, clean	.022-.030	
Earth, winding		
Clean	.023-.030	
Grass, some weeds	.025-.033	
Dense weeds or aquatic plants	.030-.040	
Stony bottom and weedy banks	.025-.040	
Dragline Excavated		
No vegetation	.025-.030	
Light brush on banks	.035-.060	
Channels not maintained, weeds and brush uncut		
Dense weeds to flow depth	.050-.120	
Clean bottom, brush on banks	.040-.080	
Dense brush, high stage	.080-.140	
Rock cuts		
Fairly smooth and uniform	.025-.040	
Rather irregular	.035-.050	
Natural Streams (top width at flood stage 100 ft)		
Streams on prairie, plain, or valley floor		
Fairly regular section		
Clean, straight, no rifts or deep pools	.025-.033	.030
Clean, winding, some pools and shoals	.033-.045	.040
Clean, winding, some weeds and stones	.035-.050	.045
Clean, winding, more weeds and stones	.045-.060	.050
Sluggish reaches, weedy deep pools	.050-.080	.070
Floodways with heavy stand timber underbrush	.075-.150	.100
Mountain Streams with trees and brush along banks submerged at high stages		
Bottom gravel, cobbles, and few boulders	.030-.050	.040
Bottom with large boulders	.040-.070	.050

	Range	Ordinary Design Values
Flood Plains (adjacent to natural streams)		
Pasture, short to high grass	.030-.050	
Cultivated areas		
no crop	.030-.040	
Mature row crops	.025-.045	
Mature field crops	.030-.050	
Heavy Weeds, scattered brush	.035-.070	
Light brush and trees		
winter to summer	.05 - .08	
Medium brush and trees		
winter to summer	.07 - .16	
Dense willows, summer, not bent over	.15 - 2.0	
Cleared land with tree stumps, 100-150		
no sprouts	.04 - .05	
heavy growth sprouts	.16 - .08	
Heavy stand timber, some sown		
little brush or undergrowth		
water below branches	.10 - .12	
water reaching branches	.12 - .16	
Major Streams (surface at flood stage 100 ft)		
n value usually less than for minor streams of like description because banks offer relatively less resistance		
Large stream, regular section, no branch or boulders	.025-.060	
Large stream, irregular with boulders and some brush	.035-.100	
Pipes		
Concrete	.011-.015	.013
CSP		
2 2/3" x 1/2" Corr.		
Unpaved		
25 percent paved		
Fully paved		
3" x 1" Corr.		
Unpaved		
25 Percent paved		
Fully paved		
Helical		
Unpaved		
25 percent paved		
Fully paved		
Structural Plate Pipe		

For corrugated metal pipe "n" values refer to Table 5-13, page 5-68.1.

* N. values Varies with size of Pipes, refer to Manufacturers Specifications.



NOMOGRAPH FOR SOLUTION OF MANNING EQUATION

CHART 6-1

Maximum permissible velocities in erodible channels 1/

Based on uniform flow in continuously wet aged channels

MATERIAL	Permissible Velocities for - Clear Water	Water Carrying Fine Silts (Colloidal)
	F.p.s.	F.p.s.
a. Fine sand (noncolloidal)	1.5	2.5
b. Sandy loam (noncolloidal)	1.75	2.5
c. Silt loam (noncolloidal)	2.0	3.0
d. Ordinary firm loam	2.5	3.5
e. Volcanic ash	2.5	3.5
f. Fine gravel	2.5	5.0
g. Stiff clay (very colloidal)	3.75	5.0
h. Graded, loam to cobbles (noncolloidal)	3.75	5.0
i. Graded, silt to cobbles (colloidal)	4.0	5.5
j. Alluvial silts (noncolloidal	2.0	3.5
k. Alluvial silts (colloidal)	3.75	5.0
l. Coarse gravel (noncolloidal)	4.0	6.0
m. Cobbles and shingles	5.0	5.5
n. Shales and hard pans	6.0	6.0

1/ Recommended in 1926 by Special Committee on Irrigation Research, A.S.C.E. for channels with straight alinement. For sinuous channels multiply allowable velocity by 0.95 for slightly sinuous, 0.9 for moderately sinuous channels, and by 0.8 for highly sinuous channels.

TABLE 6-2

6-502 - CONVEYANCE

Conveyance is the ability for any water course to transport flow. The conveyance factor for any cross-section of a water course is determined by the values of channel variables such as the cross-sectional area, type of cover, depth of flow, and slope of the channel. By surveying the channel at the stations where possible restrictions may occur, the variables can be defined. Once this is done, the conveyance factor for every cross-section can be defined, and when multiplied by the slope of the channel for that station, the flow (Q) can be calculated. The flow for a particular section of a channel will depend basically on the depth of flow, the slope, and plant growth on the channel bottom and sides. The channel conveyance (K) can be implemented into Manning's equation by substitution:

$$K = \frac{1.486}{n} AR^{2/3}$$

so then

$$Q = KS^{1/2}$$

6-600 UNIFORM FLOW REGIMES

To have uniform flow, the grade must be constant and all cross sections of flow must be identical in form, roughness, and area, necessitating a constant mean velocity. Under uniform flow conditions, the depth and the mean velocity for a particular discharge are said to be normal. Under these conditions, the water surface is parallel to the streambed (Figure 6-1 , page 6-12) . Normal depth is also defined as the depth at which uniform flow will occur when a given quantity of water flows through a long channel of uniform dimensions, roughness, and slope.

6-601 FROUDE NUMBER

The ratio of the inertial force of water to its gravitational force yields the Froude Number. It is a dimensionless number which can be determined from the following formula:

$$F^2 = \frac{V^2}{gd}$$

where:

- F = Froude Number
- V = velocity in feet per second
- d = depth of flow in feet

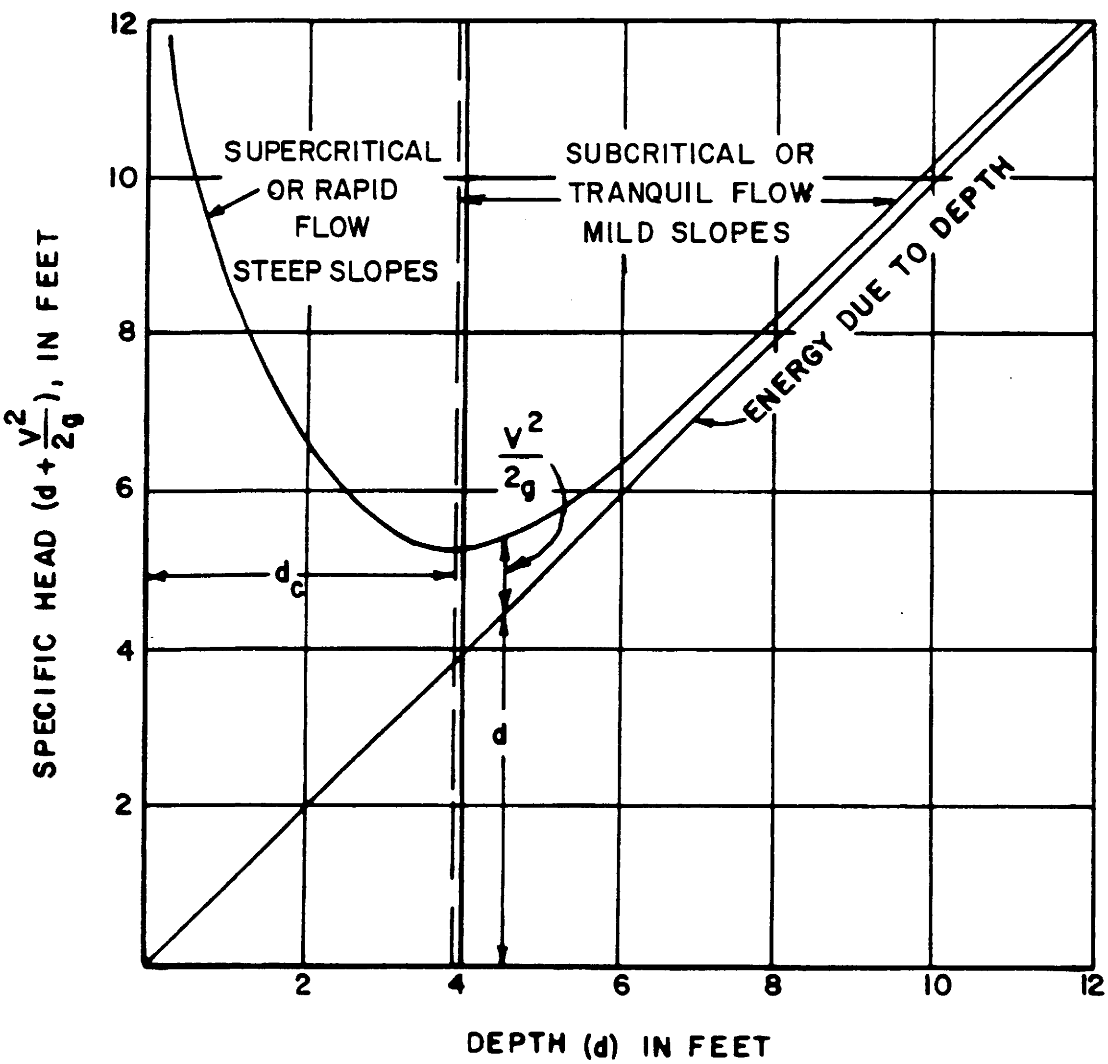
The Froude number is used to determine whether flow in an open channel is supercritical or subcritical. When $F > 1$ flow is supercritical and when $F < 1$ flow is subcritical.

6-602 SUBCRITICAL FLOW

To explain subcritical flow, critical depth must first be described. Shown in Figure 6-2, page 6-21, is the relationship between the water surface profile and energy profile of any free flowing channel. The specific head energy curve is asymptotic to the line representing the energy due to depth and to the vertical line of zero depth. At the curve's lowest point a horizontal distance is measured from the point to the vertical line of zero depth. This distance is known as critical depth. When the flow occurs at depths greater than critical depth, the flow is called subcritical. There are formulas which determine critical depth for a certain type geometric section. Some of these are listed below:

Rectangular section

$$d_c = 0.315\left[\frac{Q}{B}\right]^{2/3}$$



Specific head diagram for constant Q

FIGURE 6-2

Trapezoidal section

$$d_c = \frac{4zH_o - 3b + (16z^2H_o^2 + 16zH_ob + 9b^2)^{\frac{1}{2}}}{10z}$$

Triangular section

$$d_c = 0.57\frac{(Q)}{(z)}^{2/5}$$

where

- B = width of rectangular channel in feet
- b = bottom width of a trapezoidal channel in feet
- H_o = specific head at section in feet
- Q = rate of discharge in cfs
- z = slope of sides of a channel (horizontal to vertical)

6-603 CRITICAL

Flow is called critical when it occurs at the critical depth in an open channel.

6-604 SUPERCRITICAL FLOW

Just as flow is subcritical when it occurs above critical depth, flow is supercritical when it occurs below critical depth. The change or position where supercritical flow becomes subcritical is very abrupt and is known as a hydraulic jump.

6-700 DESIGN OF TRAPEZOIDAL CHANNELS

The following is a trial and error solution for designing a trapezoidal channel:

Example 1

Given: A trapezoidal channel in stiff clay, bottom width = 4 feet, side slopes 2:1, n = 0.03, slope 0.005 ft/ft., and discharge 100 cfs, water carries fine silt.

Find: Depth (d) and velocity (V).

Solution: General solution of area and hydraulic radius

$A = (4 + 2d)d$

WP (wetted perimeter) $= 4 + 2d\ (5)^{\frac{1}{2}} = 4 + 4.47d$

R (Hydraulic radius) $= \frac{A}{WP} = \frac{(4 + 2d)d}{4 + 4.47d}$

Try d = 3 feet

$A = (4 + 6)\ (3) = 30.0$ feet^2

$WP = 4 + 13.4 = 17.4$ feet

$R = 1.72$ feet

From Chart 6-1, page 6-11, V = 5.1 fps

$Q = 30\ (5.1) = 153$ cfs, which is greater than design discharge of 100 cfs.

Try d = 2.5 feet

$A = (4 + 5)\ (2.5) = 22.5$ feet^2

$WP = 4 + 4.47\ (2.5) = 15.2$ feet

$R = 1.48$ feet

From Chart 6-1, page 6-11 , V = 4.6 fps

$Q = 22.5\ (4.6) = 104$ cfs too high

Try d = 2.4 feet

$A = (4 + 4.8)\ (2.4) = 21.2$ feet^2

$WP = 4 + 4.47\ (2.4) = 14.7$ feet

$R = 1.44$ feet

From Chart 6-1, V = 4.5 fps

$Q = 4.5\ (21.1) = 95$ cfs too low

The last two trials (d = 2.5 and 2.4 feet) are about equally high and low, thus, the solution is d = 2.45 ft. V = 4.6 fps.

Chart 6-2, page 6-24, has been provided as an aid in solving design problems for trapezoidal channels. The following example problem outlines each step in using the chart.

Example 2

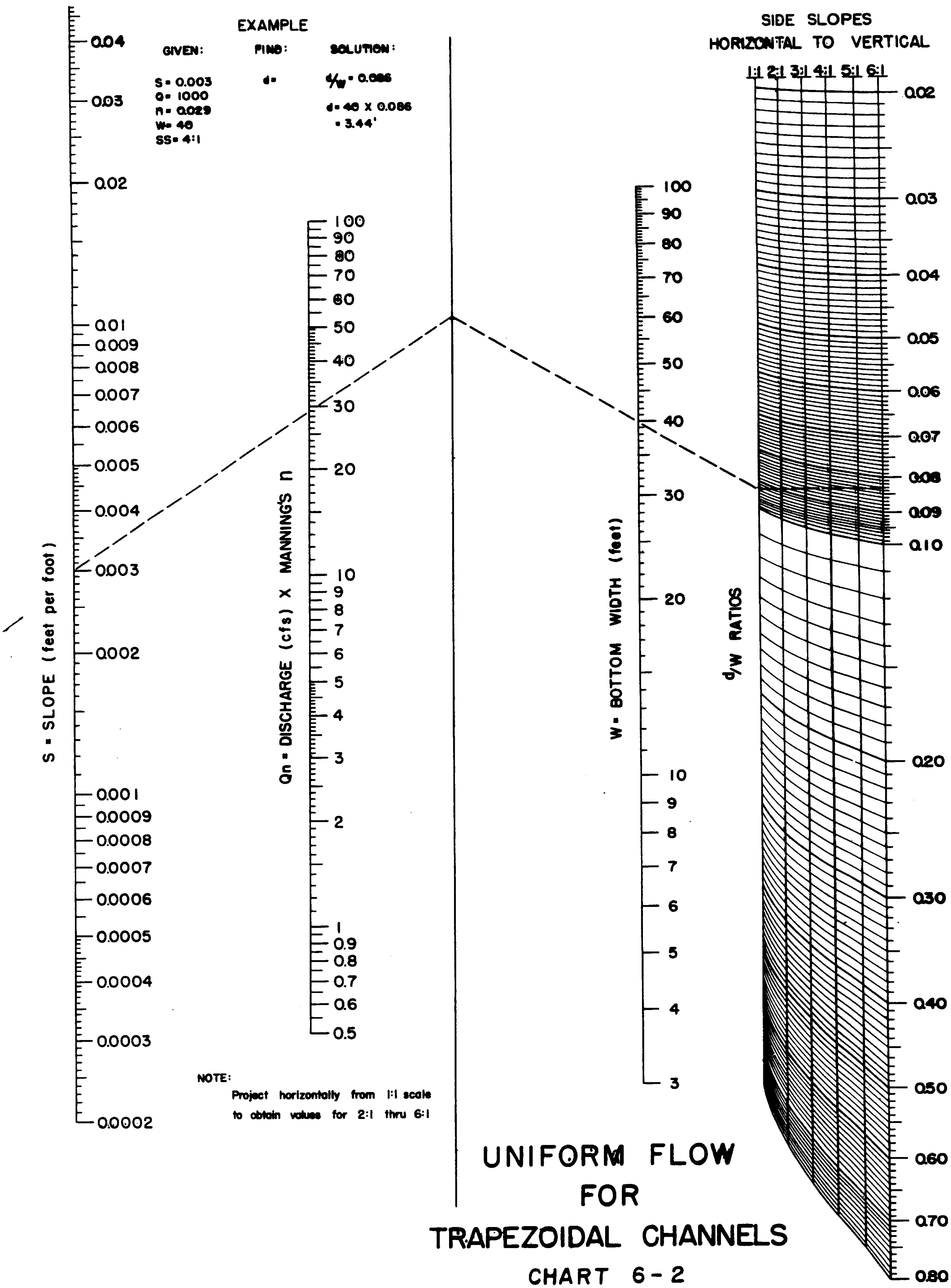
Given: A trapezoidal channel, bottom width 10 feet, side slopes 4:1, n = 0.03, slope = 0.005'/', and discharge 100 cfs.

Find: Depth (d)

Solution: Before using chart 6-2, page 6-18 , determine Q x n:

$$Q \times n = 100 \times 0.03 = 3.0$$

Proceed to the two scales positioned left of the pivot line which are marked slope (s) and discharge X Manning's



UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

CHART 6-2

n (Qn) and connect a line using S = 0.005 and Qn = 3.0

Using the intersection of the above line with the pivot line draw a line from said point intersecting the scale for bottom width (W) = 10.

Project this line onto the far right scale (d/w) intersecting the scale for side slope of 1:1. Proceed horizontally on this scale until the line representing 4:1 side slope is intersected. Read the approximate value for one of the horizontal lines which goes through this point. You should be reading approximately 0.16.

This value is called d/w. To get d simply multiply by 10 (W).

$$0.016 \times 10 = 1.6 = d$$

Therefore depth = 1.6 feet.

6-800 DESIGN OF NATURAL CHANNELS

A natural channel is any water course found in nature not formed by artificial construction methods. Natural channels are the final movers of stormwater runoff. When land development creates additional flows, the natural channels must handle the bulk of the additional runoff. It therefore becomes necessary to check the capacity of the natural channel and its ability to drain the area of any additional runoff load.

Generally, natural channels do not have uniform cross-sections throughout their length. Some cross-sections have larger areas than others, some are deeper, and some have structures crossing the flowline. The natural channel will have the capacity to carry a finite amount of flow for every incremental cross-section just before flooding occurs. Flooding at one cross-section is not important so long as the damage done is of little consequence. However, the threat of serious property damage at one location would be a design consideration when natural channels are planned as prime routes for runoff relief.

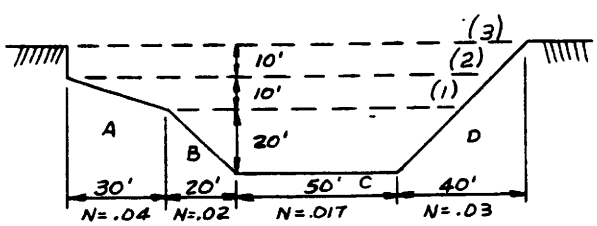
If a natural channel is to carry additional flows without being improved, then the most restrictive location and respective flowrate should be used as the design maximum capacity of the channel. Table 6-2, page 6-18, provides some maximum permissible velocities for various type soils in erodible channels. Personal property, livestock, and other roadways should not be endangered by the overflow of the natural channel, so checks should be made for all obvious cross-sections where damage might occur.

6-801 <u>CONVEYANCE</u>

Conveyance as defined in section 6-502, is the ability for any water course to transport flow. The conveyance factor for any cross-section of a water course is determined by the values of the channel variables, such as, the cross-sectional area, type of cover, depth of flow, and slope of the channel.

6-801.1 The following prodedure may be used for flowrate determination:

Given: Cross-section as shown below:



Required: Calculate the flowrate (Q) for stages 1, 2 and 3

Solution:

Stage 1 - Sections B, C, and D

Section B, $K = \frac{1.486}{n} AR^{2/3}$

$K = \frac{1.486}{0.02} (200 \text{ ft.}^2) (\frac{200 \text{ ft.}^2}{28.28})^{2/3}$

$K = 54,750.2$ cfs

Section C, $K = \frac{1.486}{0.017} (1000 \text{ ft.}^2) (\frac{1000 \text{ ft.}^2}{50 \text{ ft.}})^{2/3}$

$K = 644,055.3$ cfs

Section D, $K = \frac{1.486}{0.03} (200 \text{ ft.}^2) (\frac{200 \text{ ft.}^2}{28.28 \text{ ft.}})^{2/3}$

$K = 36,500$ cfs

Sum all section K's for Stage 1

K (sum) (Section B, C, and D) = 735,305.6 cfs

Slope at cross-section = 0.003 ft/ft.

$(\text{Slope})^{0.5}$ = 0.05477

Stage 1 flowrate $= K \text{ (sum) } (\text{Slope})^{0.5}$

$Q = 735,305.6 \ (0.05477)$

$Q = 40272.7$ cfs

Stage 2 - Sections A, B, C and D

Section A, $K = \frac{1.486}{0.04} (150 \text{ ft.}^2) (\frac{150 \text{ ft.}^2}{31.62 \text{ ft.}})^{2/3}$

$K = 15,732.7$ cfs

Section B, $K = \frac{1.486}{0.02} (400 \text{ ft.}^2) (\frac{400 \text{ ft.}^2}{28.28 \text{ ft.}})^{2/3}$

$K = 173,821.1$ cfs

Section C, $K = \frac{1.486}{0.017} (1500 \text{ ft.}^2) (\frac{1500 \text{ ft.}^2}{50 \text{ ft.}})^{2/3}$

$K = 1,265,926.7$ cfs

Section D, $K = \frac{1.486}{0.03} (450 \text{ ft.}^2) (\frac{450 \text{ ft.}^2}{42.43 \text{ ft.}})^{2/3}$

$K = 107,597.7$ cfs

Sum all section K's for Stage 2

K (sum)(section A, B, C, and D) = 1,563,078.2 cfs

Slope at cross-section = 0.003 ft/ft.

$(\text{Slope})^{0.5}$ = 0.05477

Stage 2 flowrate = $K\ (\text{Sum})(\text{Slope})^{0.5}$

$Q = 1{,}563{,}078.2(0.05477)$

$Q = 85609.8$ cfs

Stage 3 - Sections A, B, C and D

Section A, $K = \frac{1.486}{0.04}(450\ \text{ft.}^2)\left(\frac{450\ \text{ft.}^2}{41.62\ \text{ft.}}\right)^{2/3}$

$K = 81{,}741.0$ cfs

Section B, $K = \frac{1.486}{0.02}(600\ \text{ft.}^2)\left(\frac{600\ \text{ft.}^2}{28.28\ \text{ft.}}\right)^{2/3}$

$K = 342{,}655.1$ cfs

Section C, $K = \frac{1.486}{0.017}(2000\ \text{ft.}^2)\left(\frac{2000\ \text{ft.}^2}{50\ \text{ft.}}\right)^{2/3}$

$K = 2{,}044{,}747.9$ cfs

Section D, $K = \frac{1.486}{0.03}(800\ \text{ft.}^2)\left(\frac{800\ \text{ft.}^2}{56.57\ \text{ft.}}\right)^{2/3}$

$K = 23{,}734.1$ cfs

Sum all section K's for Stage 3

K (sum) (sections A, B, and D = 2,492,878.1

Slope at cross-section = 0.003 ft/ft.

$(\text{Slope})^{0.5}$ = $K\ (\text{sum})\ (\text{slope})^{0.5}$

$Q = 2{,}492{,}878.1\ (0.05477)$

$Q = 136{,}534.9$ cfs

Stage Analysis:

Stage 1: Stage 1 includes the channel bottom 50 feet wide and th depth within the channel of 20 feet. From the sketch, page 6-20, Section B, C, and part of D are included in this stage. Each section has a different Manning "n" factor and each factor is used to calculate each individual section Q.

Stage 2: Stage 2 includes the same channel section as in Stage 1 plus 10 feet more of depth. The additional depth of 10 feet adds a new section A, and more of Section D on the right. The total flowrate that Stage 2 will convey is based on the sections A through D, up to a depth of 30 total feet, and a slope of the channel of 0.003 ft/ft.

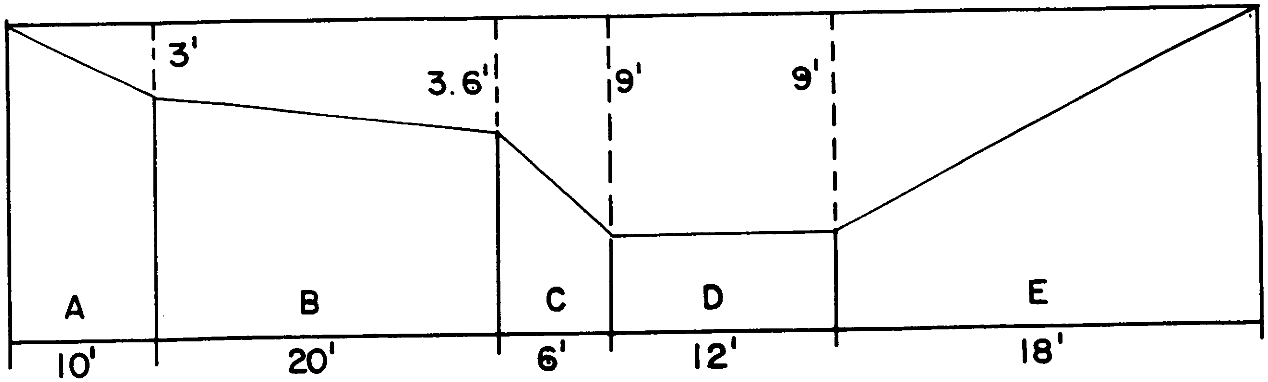
Stage 3: Stage 3 includes the entire channel width and depth. All channel sections from A through D and their respective Manning "n" factors are used in determining the conveyance factor K for stage 3. If one bank elevation is higher than another, then the lowest of the two should be used as a design maximum elevation, since the limits of flow may be undefinable above the lower of the two elevations.

Remarks: Some locations may be found to have restrictive conveyance factors. The designer may choose to alter the cross-section to allow a greater flowrate to pass. By knowing the proposed cross-section geometry, it is possible to determine the section conveyance K and the expected flowrate Q. The design can be altered until the flowrate Q fits the need.

6-801.2 EXAMPLE DATA SHEET

The following example data sheet is in a form recommended for use in organizing and calculating the conveyance properties of a natural channel section. The equations following the data summary are basic and will be valid for all channel sections.

VELOCITY - FLOWRATE - DEPTH
NATURAL CHANNEL
IRREGULAR SECTION



A	B	C	D	E
Heavy Weeds n = .05	Grass Some Weeds n = .035	Weeds n = .038	Gravel n = .032	Weeds n = .038

Slope Given As 0.68% or 0.0068 ft/ft

K -- CONVEYANCE TABLE						
Section	Area	W. Perimtr.	Hyd. Rad.	Hyd. Rad.$^{2/3}$	n	K
A	15.0	10.44	1.436	1.273	0.05	568
B	66.0	20.00	3.298	2.216	0.035	6,201
C	37.8	8.07	4.683	2.799	0.038	4,137
D	108.0	12.00	9.000	4.327	0.032	21,700
E	81.0	20.12	4.024	2.530	0.038	8,014
					Σ K	40,620

Equation: $K = \frac{1.486}{n} A R_h^{2/3}$

Equation: $Q = \Sigma K (S)^{1/2}$

$Q = \underline{40,620} \times \underline{0.0824}$

$Q = \underline{3,349.6}$ Ft. Cubed per Sec. $\frac{Ft^3}{Sec.}$

6-801.3 COMPUTER PROGRAMS FOR DETERMINING CONVEYANCE

Some of the more familiar computer programs which determine conveyance are as follows:

HEC-2:	Water surface profiles
HOW:	Conveyance and backwater
E431:	Step-backwater and floodway analyses
BPR:	Conveyance and backwater

Each of the above programs also determines the stage as well as conveyance.

6-900 EROSION CONTROL CHANNEL LININGS

The power of moving water is impressive; especially when erosion, such as in the Grand Canyon, is considered. The same erosive force, to a lesser extent, can damage or destroy unprotected soil surfaces. Water moving over a soil surface causes erosion that often results in two undersirable consequences - damage to the soil surface and silting or depositing of the eroded soil in another waterway, such as a stream or lake. Natural waterway channels are generally stabilized due to the action of erosion over long periods of time, and much of the earth's land area is protected from erosion by vegetative cover. When highway construction interferes with the natural flow of water, drainage channels must be designed and built to redirect the water to a natural waterway. Highway drainage channels should be made erosion resistant to prevent unsightly gullies, keep maintenance costs down, prevent silting of some other waterway or body of water, and to prevent damage to the highway.

This section deals with unlined channels and four other types of channel linings used for erosion control - temporary, grass, rock riprap, and concrete. One means of reducing erosion on the right-of-way during highway construction and operation is through the use of properly designed linings in drainage channels. Linings may be rigid, such as portland cement or asphaltic concrete, or flexible, such as vegetation or rock riprap.

Flexible linings of erosion-resistant vegetation and rock riprap should be used whenever feasible. When vegetation is chosen as the permanent channel lining, it may be established by seeding or sodding. Installation by seeding usually requires protection by one of a variety of temporary lining materials until the vegetation becomes established.

While vegetation and rock riprap linings have been used for many years, in most cases the success or failure of the lining has been a matter of chance; and design information has been limited or difficult to apply. This section presents design methods developed from recent research results for temporary linings, vegetative linings and rock riprap linings.

Flexible linings are generally less expensive to install than rigid linings, provide a safer roadside, and have self-healing qualities which reduce maintenance costs. They also permit infiltration and exfiltration; have a natural appearance, especially after vegetation is established; and provide a filtering medium for runoff contaminants. Vegetative and rock riprap liners provide fewer improvements in conveyance over natural conditions; and the resultant acceleration of flow velocity and peak is less than that for rigid liners.

Flexible linings have the disadvantage of being limited in the depth of flow which they can accommodate without erosion occurring. As a result, the channel may provide a low capacity for a given cross-sectional area when compared with rigid lining. Also, limited right-of-way, unavailability of rock, or the inability to establish vegetation may preclude the use of flexible linings. In these instances, rigid linings may be the only alternative.

Rigid linings are generally quite smooth, so that they have a high capacity for a given cross-sectional area due to low hydraulic resistance, and thus produce a high flow velocity. When properly designed and constructed, rigid linings will prevent erosion in steep or difficult channels where other linings cannot be used. They may also be used in areas where the channel width is restricted, since steep sidewall slopes may be constructed. So long as the rigid lining is intact, the underlying soil is completely protected upon construction of the lining.

Rigid linings have a number of inherent disadvantages. They are expensive to construct and maintain, have an unnatural appearance, prevent or reduce natural infiltration, and contribute to high velocities and scour at the downstream end of the lining unless roughness elements are added to slow the flow. Many rigid linings are destroyed due to flow's undercutting the lining, channel headcutting, or hydrostatic pressure behind the channel walls or floor.

Rigid linings will be discussed briefly as related to the flexible lining materials. However, the hydraulic design of rigid linings is covered in detail in Hydraulic Design Series No. 3, "Design Charts for Open Channel Flow."

The use of Hydraulic Engineering Circular (HEC) No. 11, "Use of Riprap for Bank Protection", for the design of dumped stone riprap channel linings should be discouraged since the methods contained in this section were based on more recent information. This recommendation is based on a detailed evaluation and comparison of the two methods. However, HEC No. 11 contains information and details on other rigid linings, such as handplaced riprap, sacked concrete, and grouted riprap. Hand-placed riprap is considered to be a rigid lining since it cannot accommodate even minor movement of the surface it protects.

6-901 UNLINED

The design charts for unlined channels (bare soils) are based on tests on ten different classes of soils, ranging from cohesive clays to noncohesive sands and gravels. These charts, 6-3 and 6-4 , are found on pages 6-35 and 6-36 . Generally, sandy, noncohesive soils tend to be very erodible, the large grained gravel clay-silt mixtures are erosion resistant, and the mixtures of sand, clay, and colloids are moderately erodible.

6-902 TEMPORARY

Temporary linings are flexible coverings used to protect a channel until permanent vegetation can be established using seeding. For the most part, the materials used are biodegradable. Listed below are some of the temporary linings that can be used, which are established in the charts for this section. Among the factors which should be known in order to use these charts are hydraulic radius, soil condition, and channel slope. When one or all of these factors are known then a flow velocity or maximum flow depth can be determined from these charts.

1. *Fiber Glass Roving

   Chart 6-5 , page 6-37
   Chart 6-6 , page 6-38
   Chart 6-7 , page 6-39

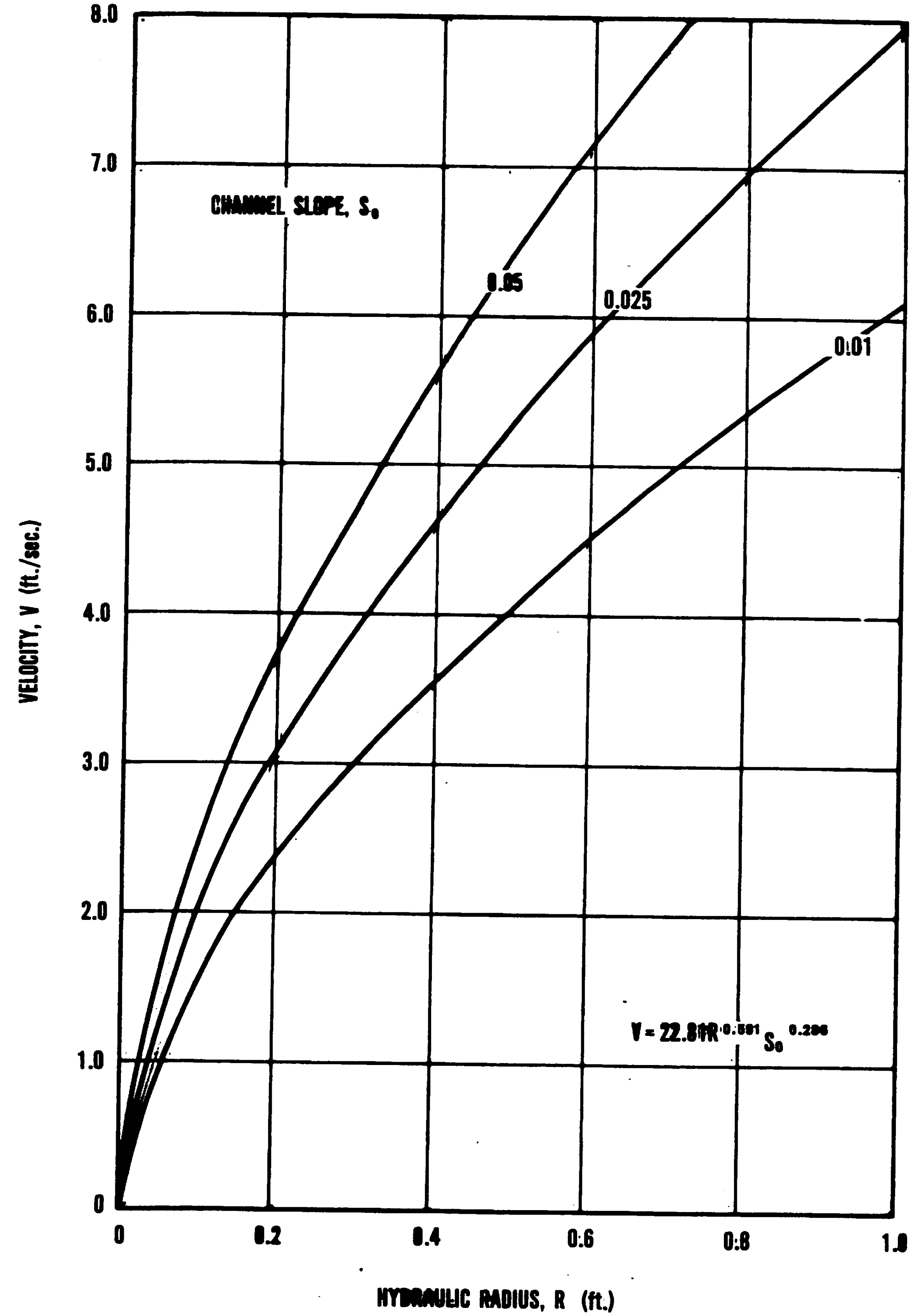
2. *Jute Matting

   Chart 6-8 , page 6-40
   Chart 6-9 , page 6-41

3. *Wood Fiber

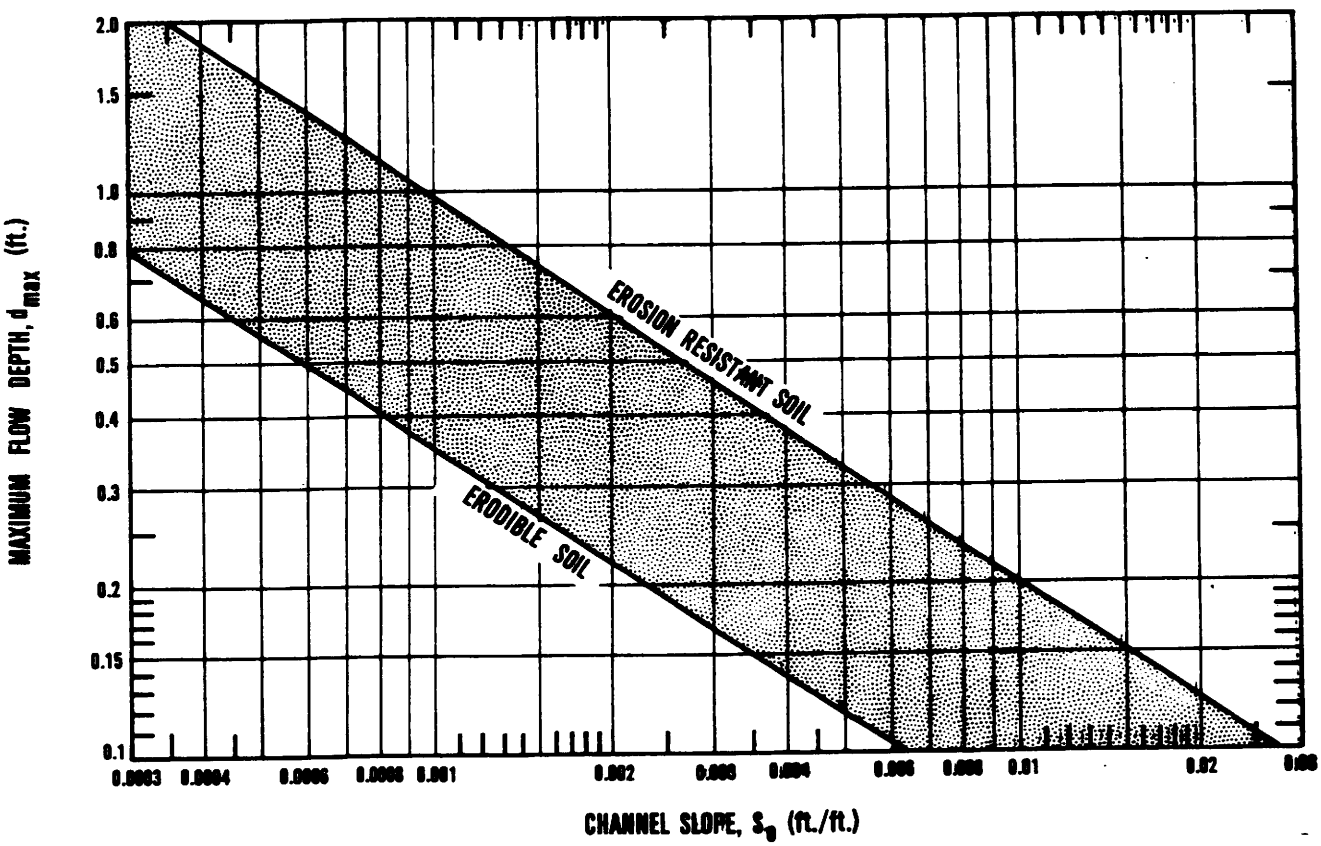
   Chart 6-10, page 6-42
   Chart 6-11, page 6-43
   Chart 6-12, page 6-44
   Chart 6-13, page 6-45

*Refer to the Department's Standard Specifications for material descriptions and construction methods, pages 319-322.



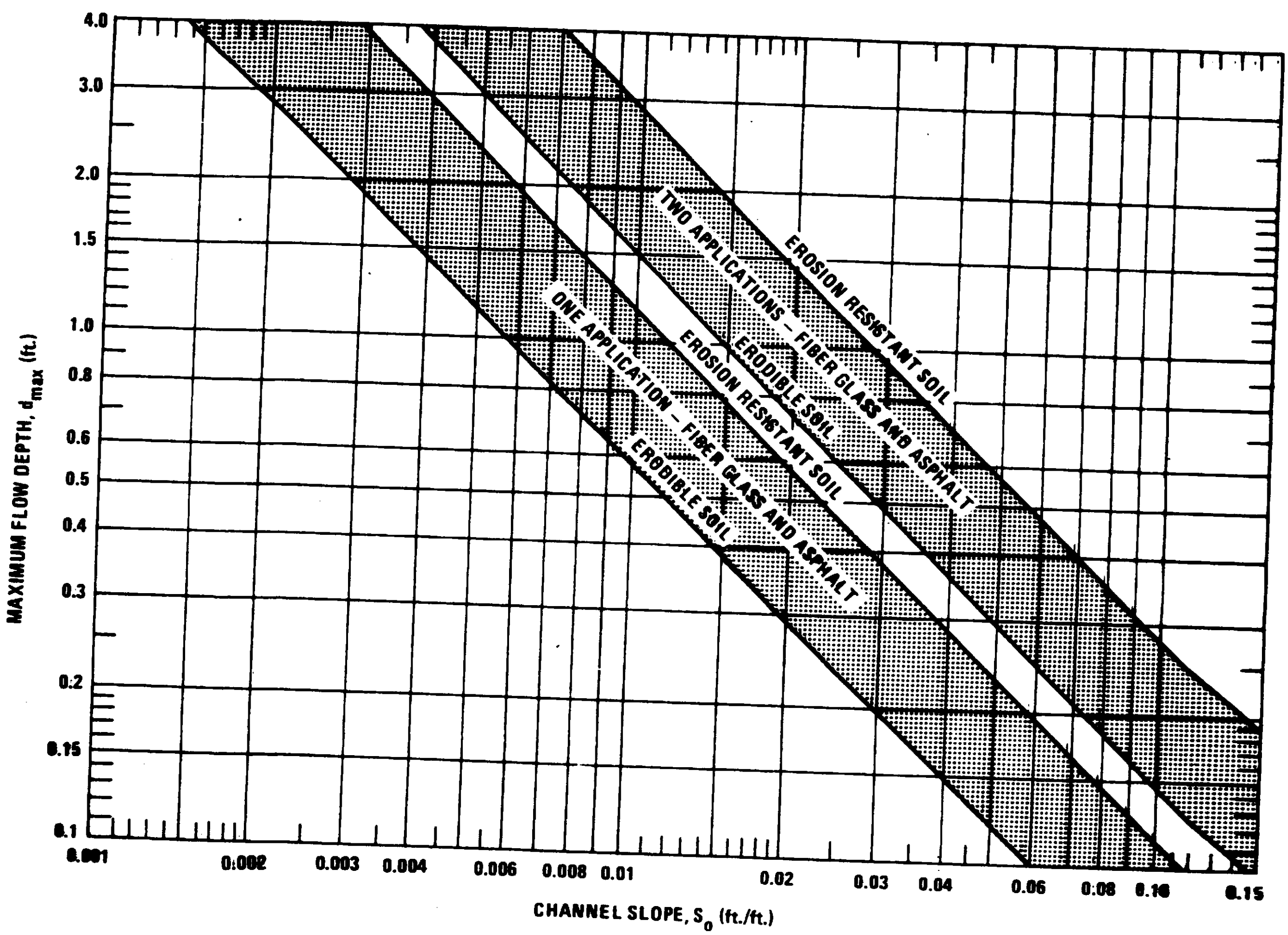
FLOW VELOCITY FOR UNLINED CHANNELS (BARE SOIL)

CHART 6-3



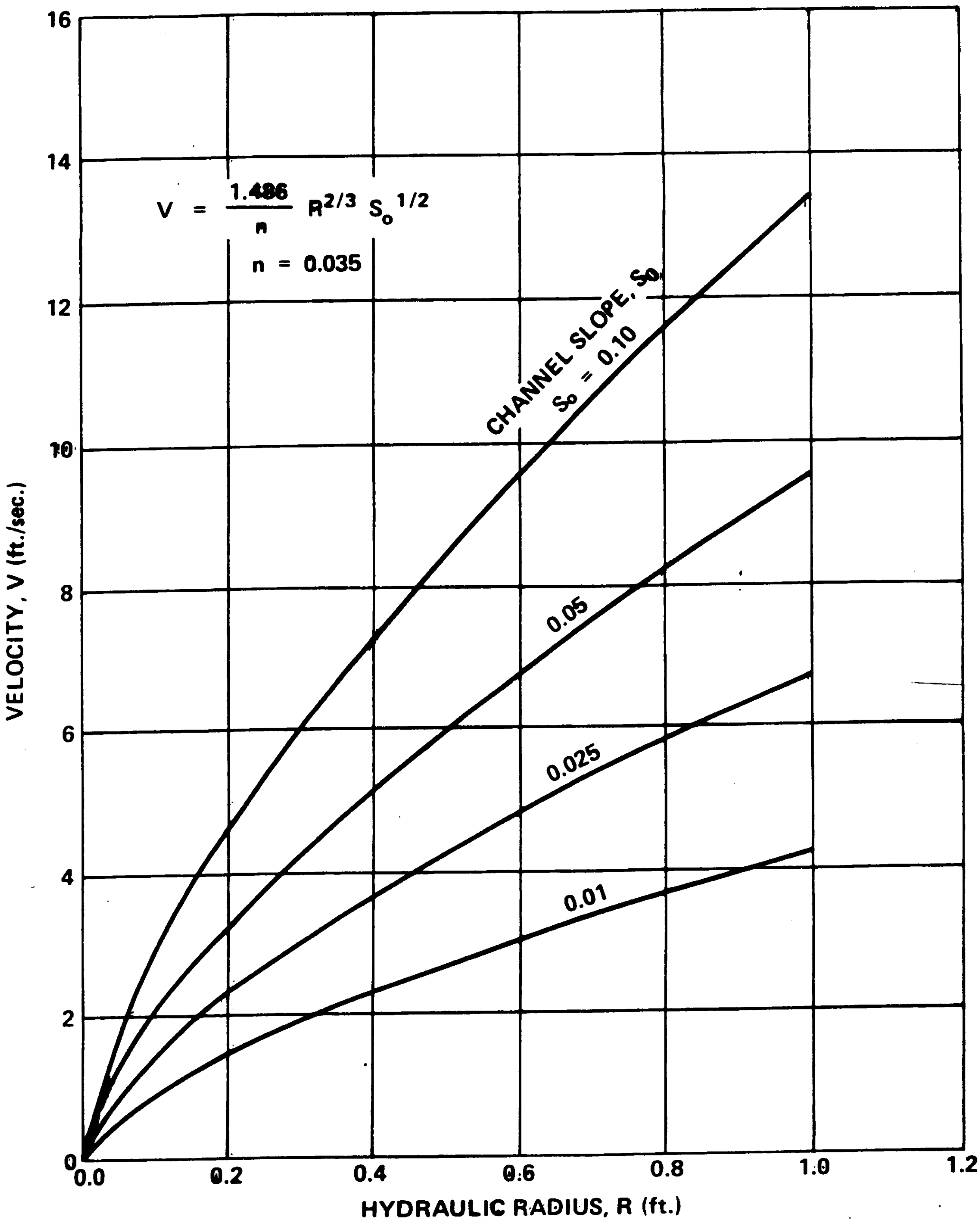
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR UNLINED CHANNELS (BARE SOIL)

CHART 6-4



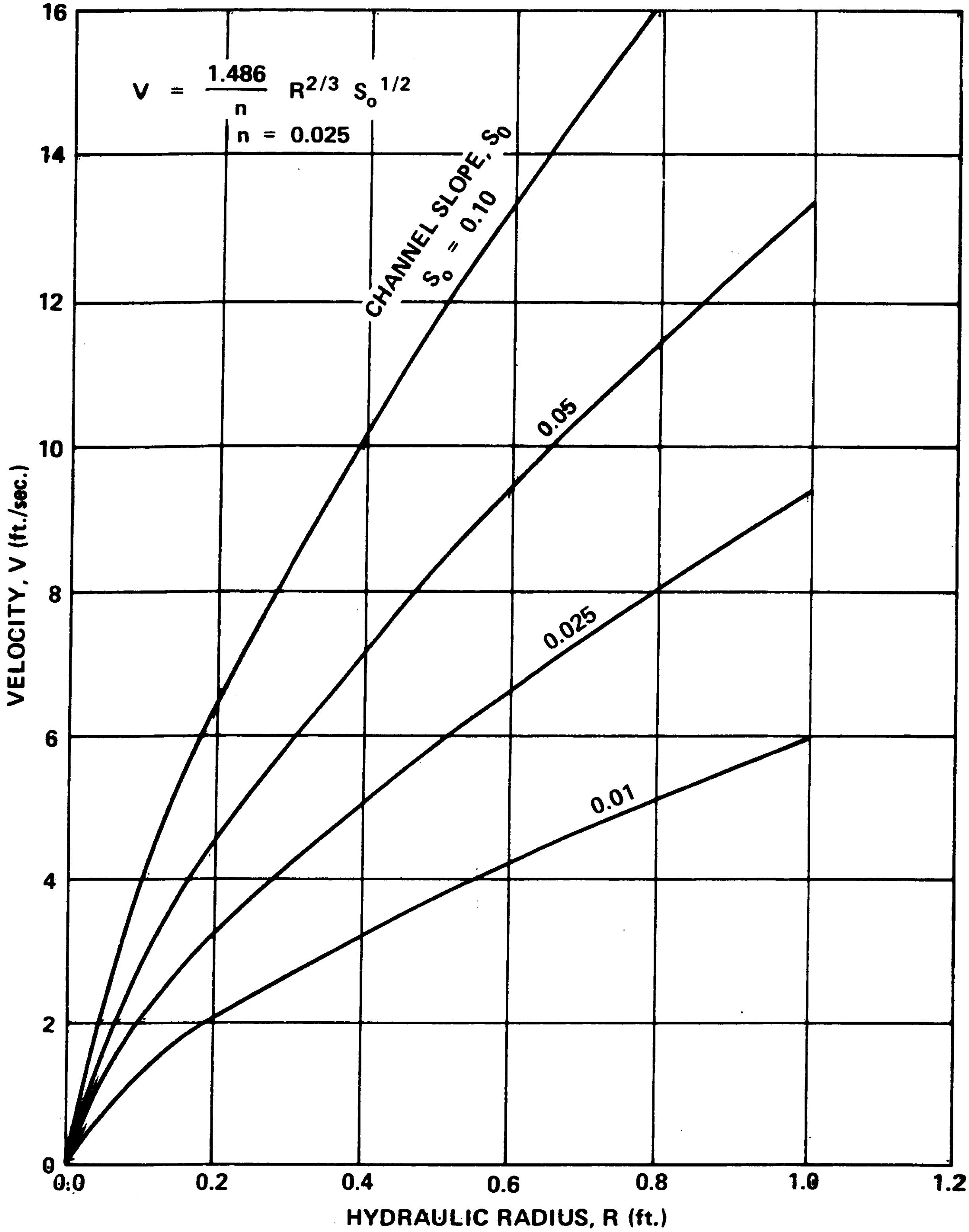
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH FIBER GLASS ROVING (SINGLE AND DOUBLE LAYER)

CHART 6-5



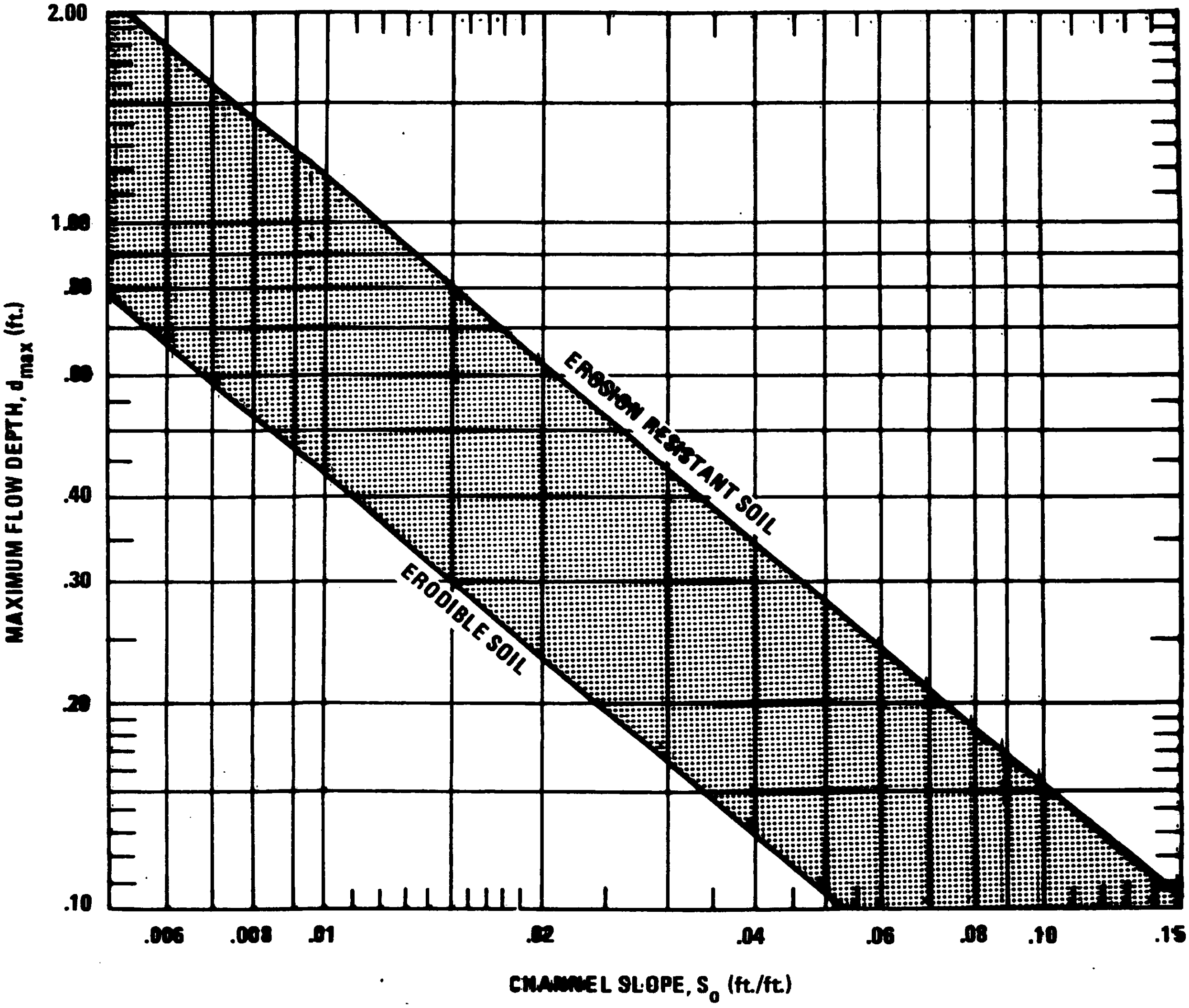
Flow Velocity for Channels Lined with Fiber Glass Roving Tacked with Asphalt, Single Layer

CHART 6-6



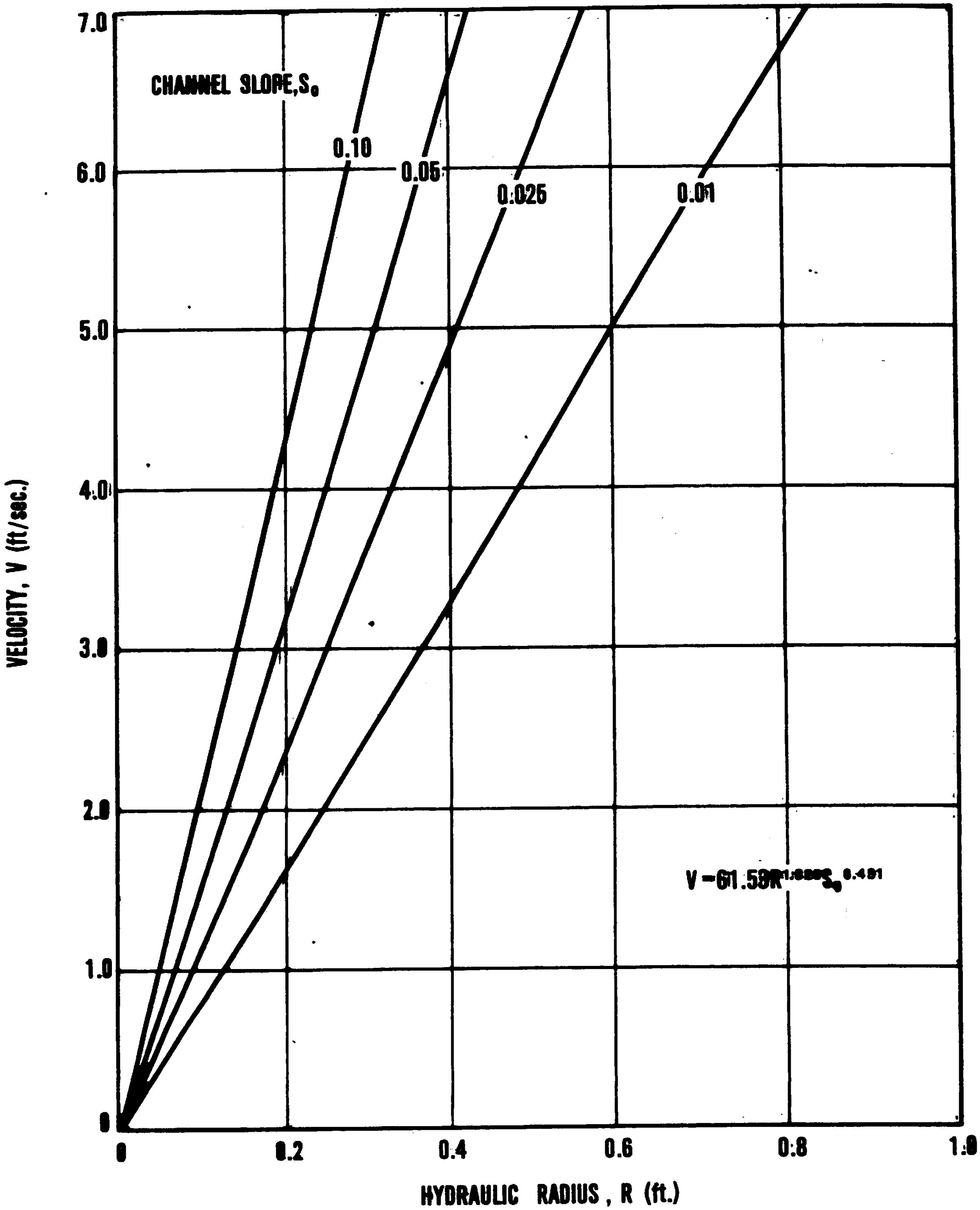
Flow Velocity for Channels Lined with Fiber Glass Roving Tacked with Asphalt, Double Layer

CHART 6-7



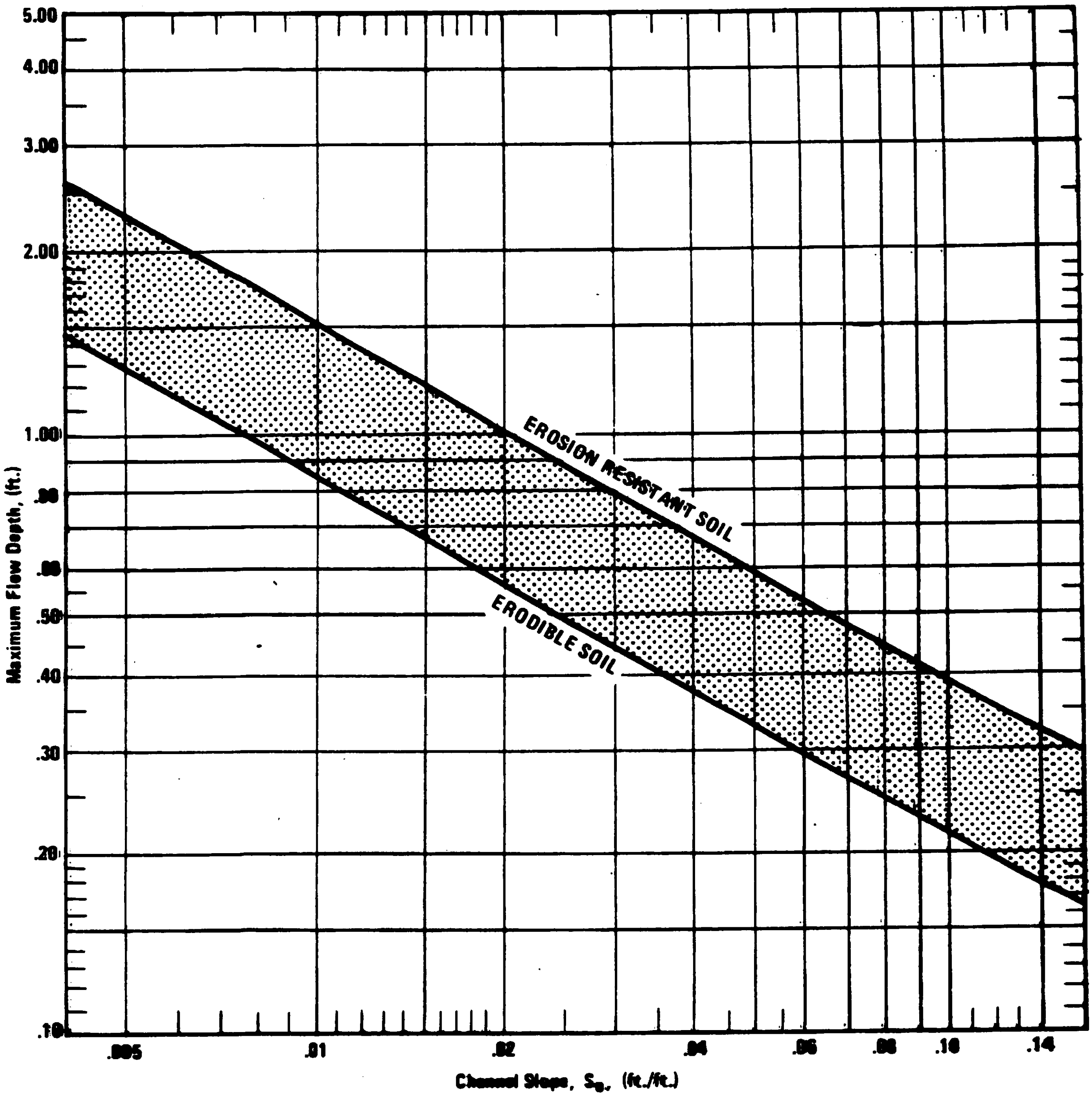
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH JUTE MESH

CHART 6-8



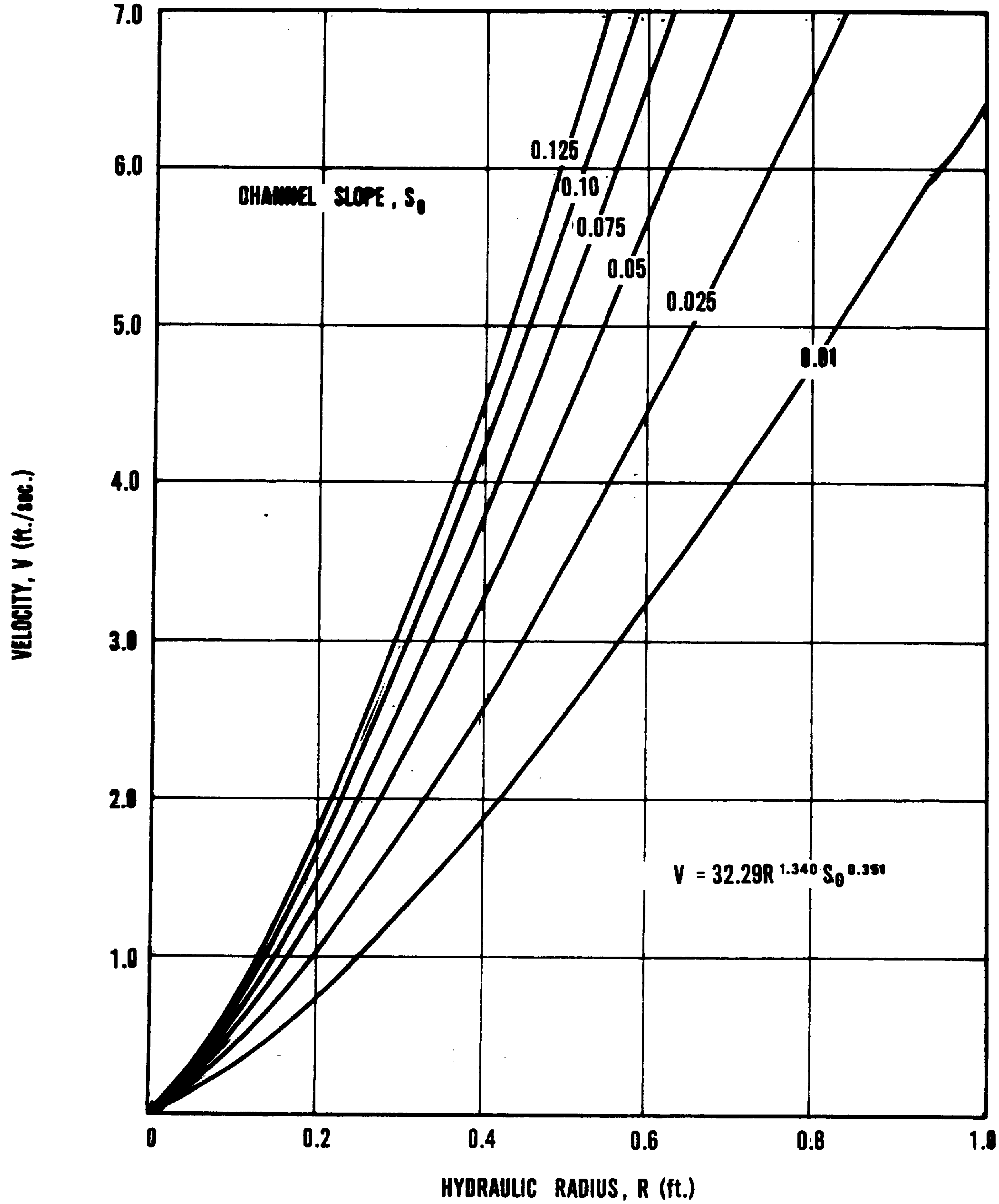
FLOW VELOCITY FOR CHANNELS LINED WITH JUTE MESH

CHART 6-9



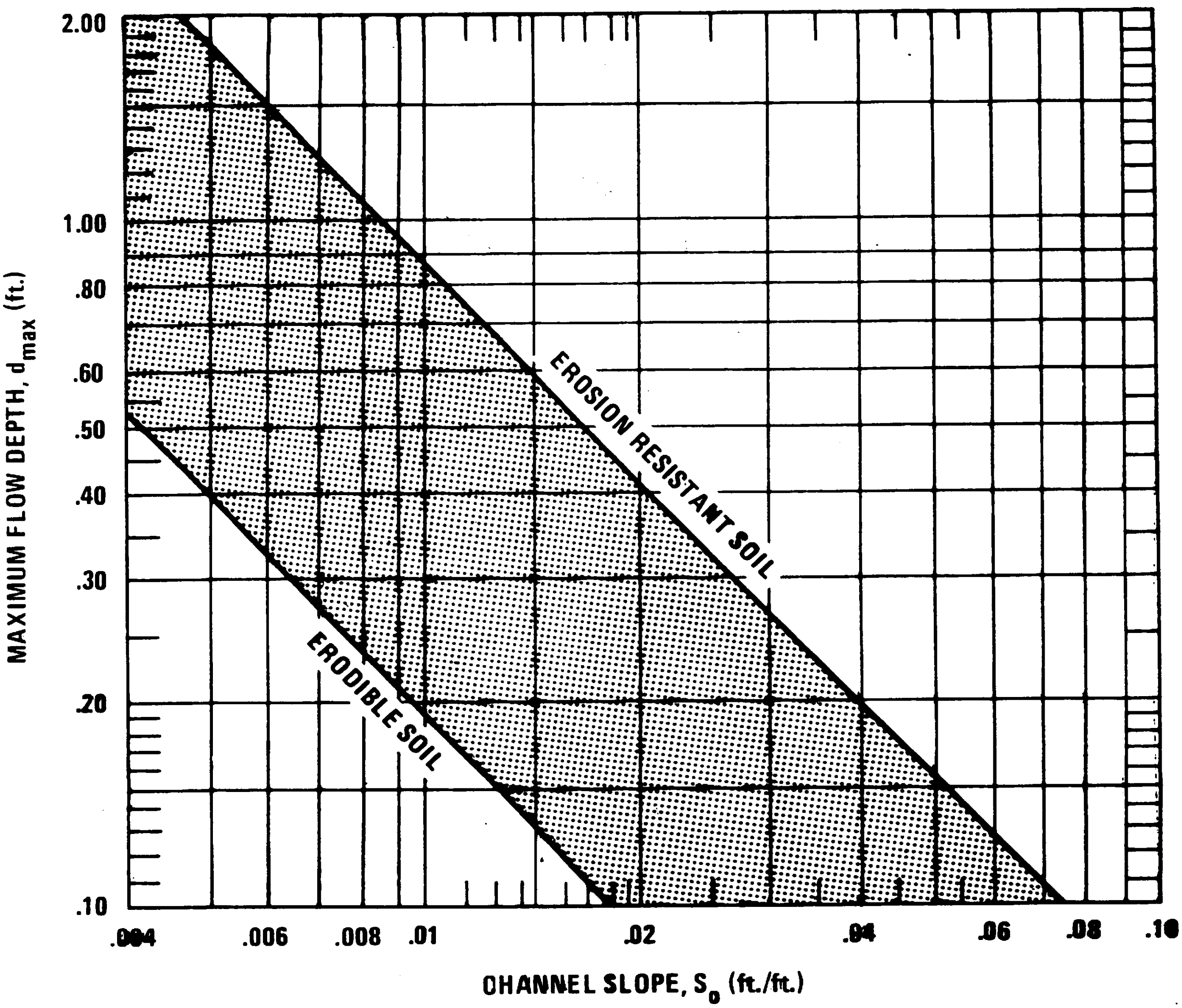
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH EXCELSIOR MAT

CHART 6-10



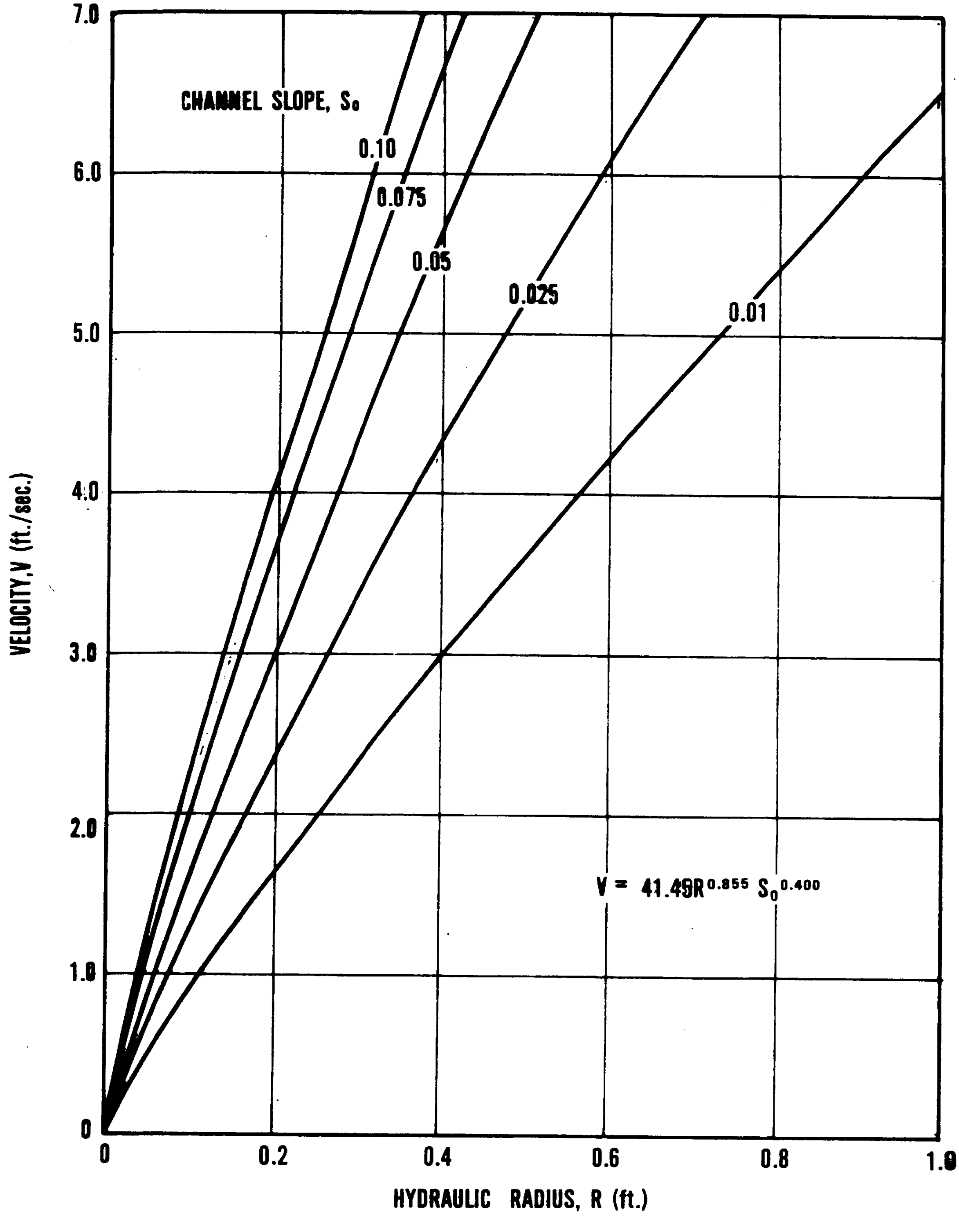
FLOW VELOCITY FOR CHANNELS LINED WITH EXCELSIOR MAT

CHART 6 - 11



MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH EROSIONET

CHART 6-12



FLOW VELOCITY FOR CHANNELS LINED WITH EROSIONET

CHART 6-13

6-903 GRASS

Several different types of vegetal covers are listed and grouped according to degree to retardance in Table 6-3, page 6-47. This table can be used in conjuction with seeding specifications in the Department's Standard Specifications, and Charts 6-14, thru 6-21, pages 6-49 thru 6-56, determine flow velocities or maximum flow depths given such factors as channel slope, hydraulic radius, and/or soil types. Table 6-4, page 6-48, is a relatively good source to check permissible velocities for different types of grass linings in channels.

6-904 ROCK RIPRAP

The resistance of random riprap to displacement by moving water depends upon:

1. Weight, size, shape and composition of the individual stones.
2. The gradation of the stone.
3. The depth of water over the stone blanket.
4. The steepness and stability of the protected slope and angle of repose of riprap.
5. The stability and effectiveness of the filter blanket on which the stone is placed.
6. The protection of toe and terminals of the stone blanket.

The size of stone needed to protect a streambank or highway embankment from erosion by a current moving parallel to the embankment is determined by the use of Charts 6-22, 6-23, and 6-24 on pages 6-57 through 6-59.

Classification of vegetel covers as to degree of retardance

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

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

From SCS "Handbook of Channel Design for Soil and Water Conservation"

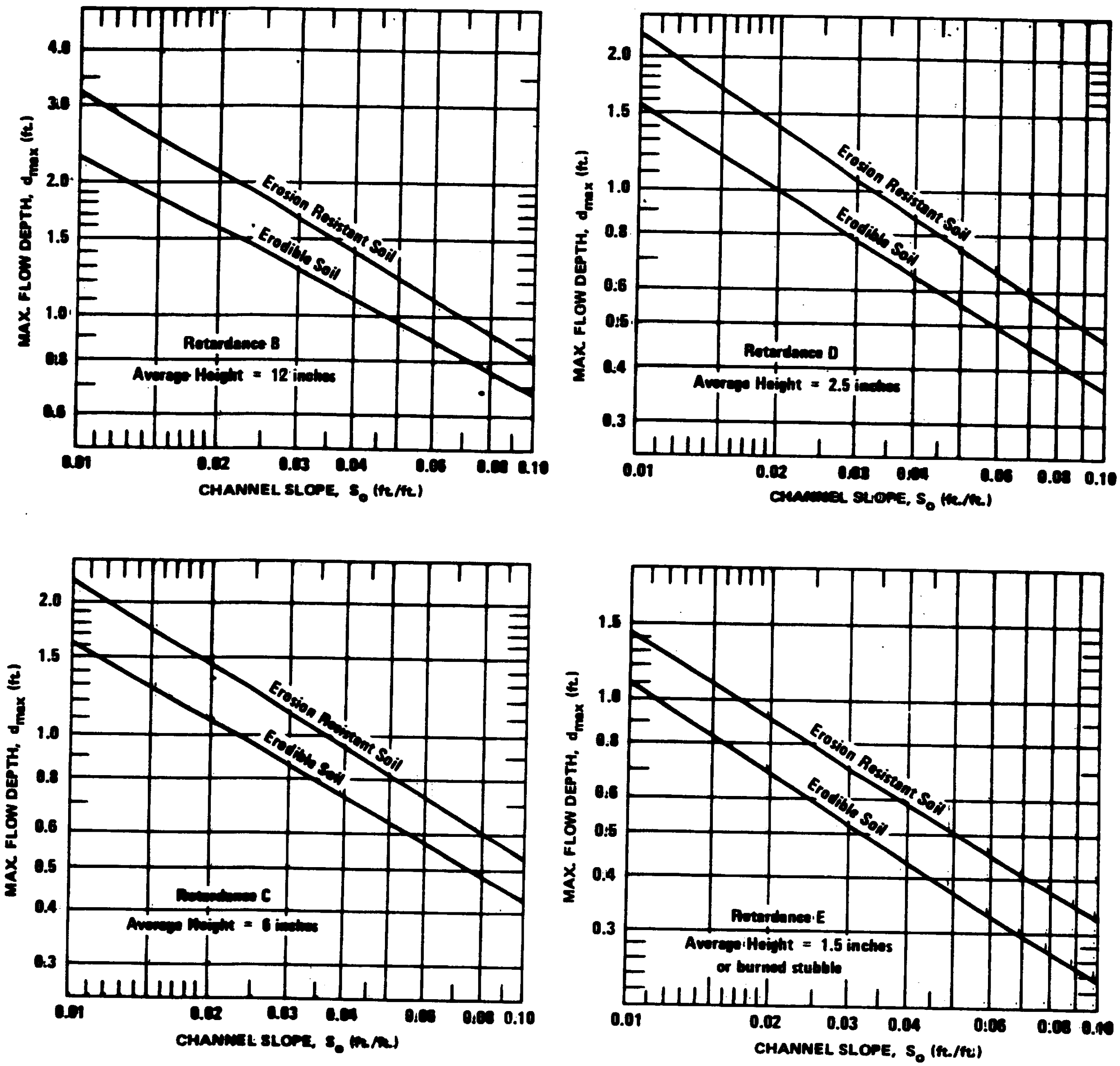
TABLE 6-3

PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH GRASS*

Cover	Slope range, %	Permissible velocity, fps: Erosion-resistant soils	Permissible velocity, fps: Easily eroded soils
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass, Kentucky bluegrass, smooth brome, blue grama	0-5	7	5
	5-10	6	4
	>10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
	Do not use on slopes steeper than 10%		
Lespedeza sericea, weeping love grass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass	0-5	3.5	2.5
	Do not use on slopes steeper than 5%; except for side slopes in a combinatuon channel		
Annuals—used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan grass	0-5	3.5	2.5
	Use on slopes steeper than 5% is not recommended		

REMARKS. The values apply to average, uniform stands of each type of cover. Use velocities exceeding 5 fps only where good covers and proper maintenance can be obtained.

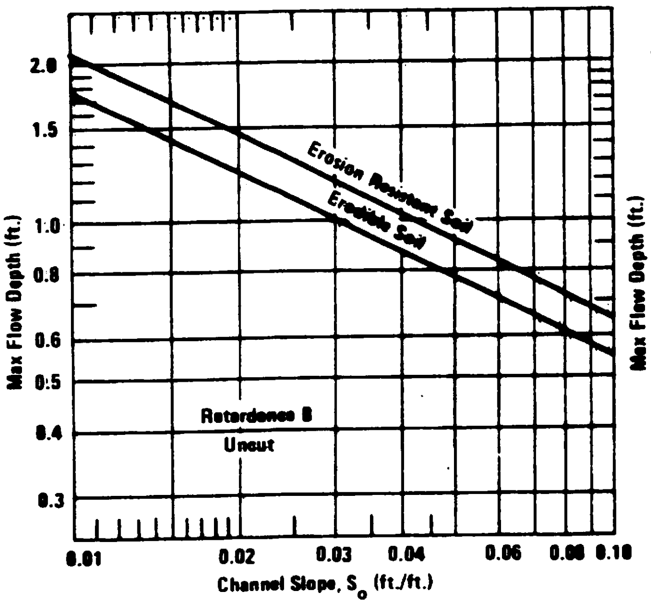
TABLE 6-4

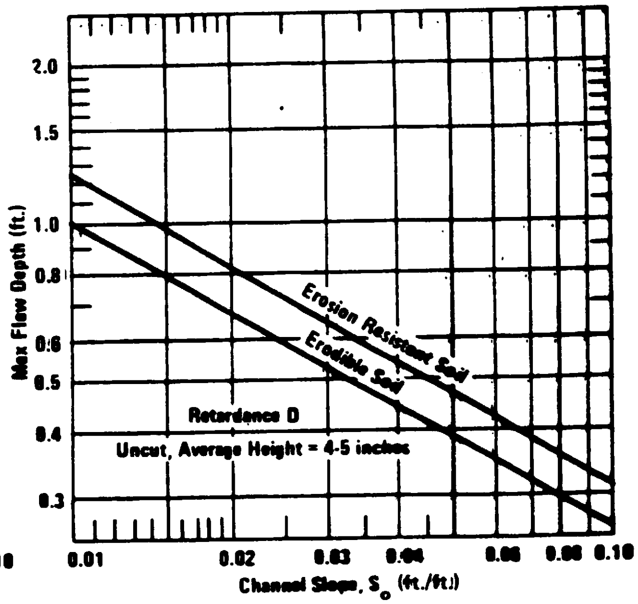


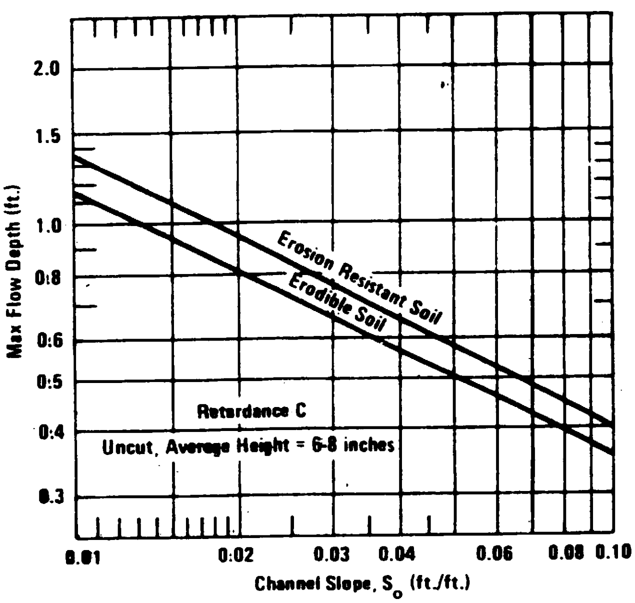
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH BERMUDA GRASS. GOOD STAND, CUT TO VARIOUS LENGTHS

Note: Use on slopes steeper than 10 percent is not recommended

CHART 6-14



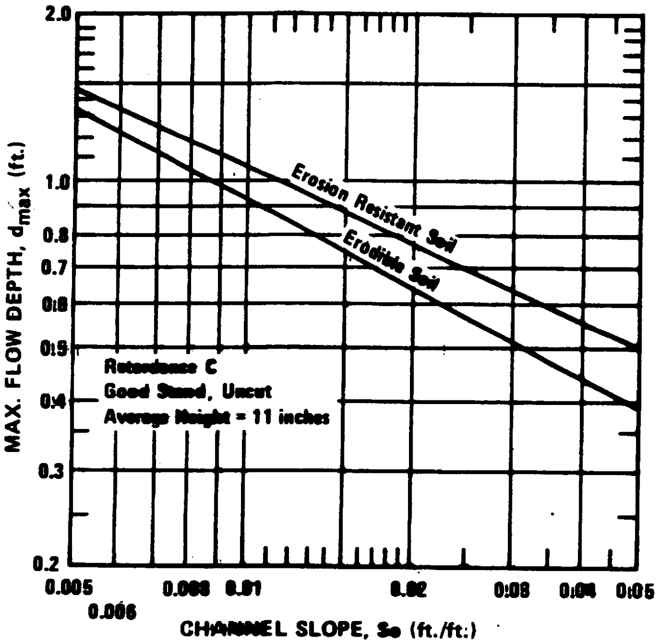


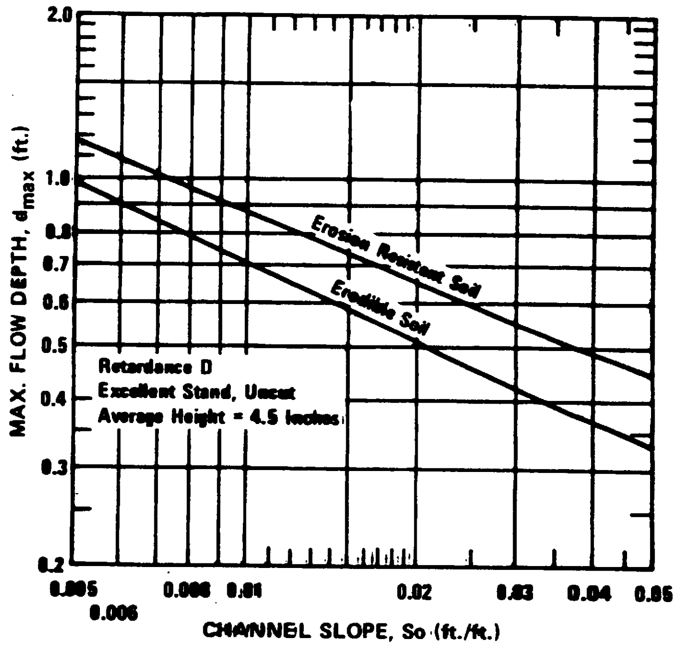


Retardance B: Native Grass Mixture
Little Bluestem, Blue Grama, Other Long and Short Midwest Grasses.
Retardance C: Grass-Legume Mixture
Summer-Orchard Grass, Redtop, Italian Ryegrass, Common Lespedeza
Retardance D: Grass-Legume Mixture
Fall, Spring – Orchard Grass, Redtop, Italian Ryegrass, Common Lespedeza

Maximum Permissible Depth of Flow (d_{max}) for Channels Lined with Grass Mixtures. Good Stand, Uncut

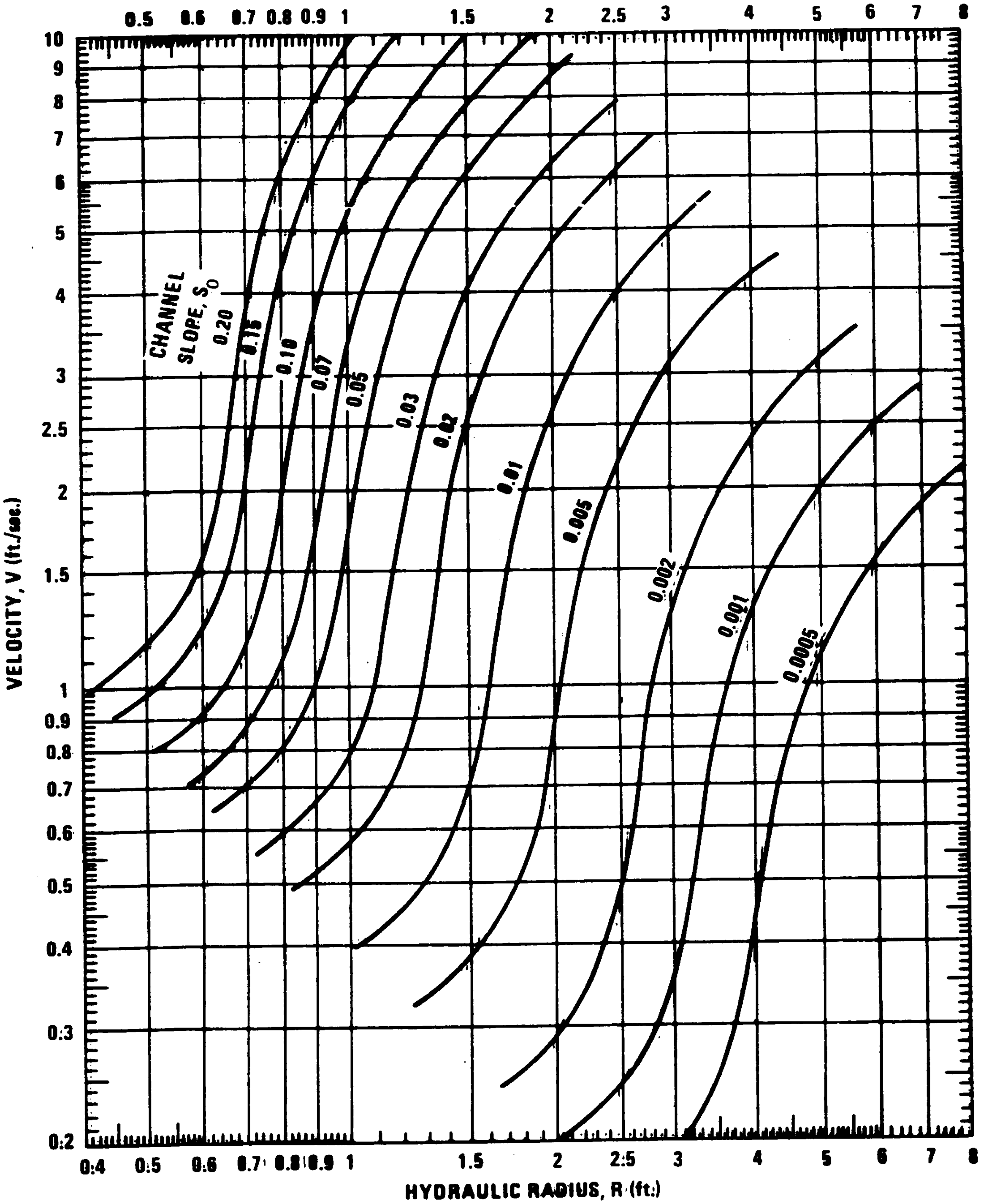
CHART 6 - 15





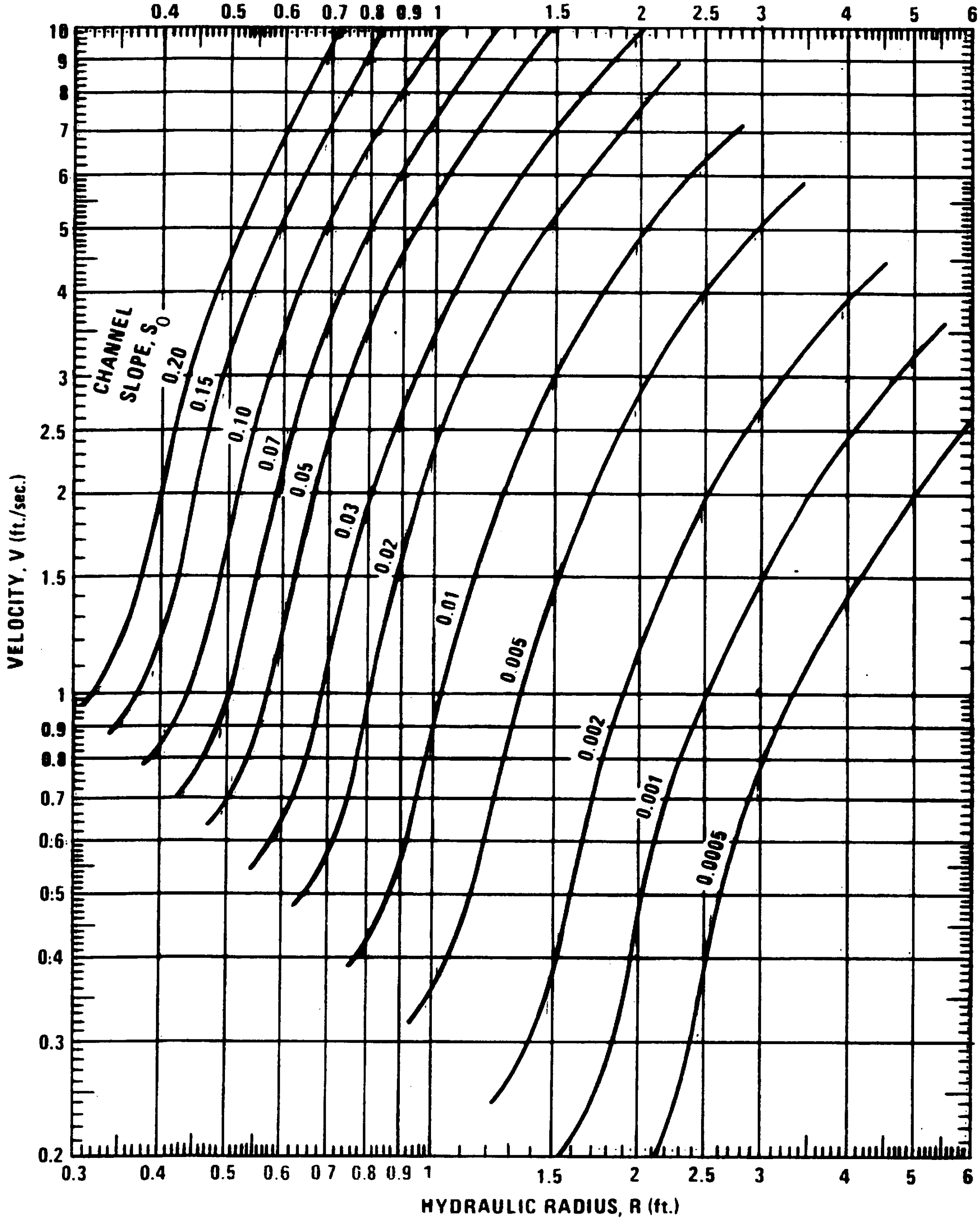
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH COMMON LESPEDEZA OF VARIOUS LENGTHS

CHART 6-16



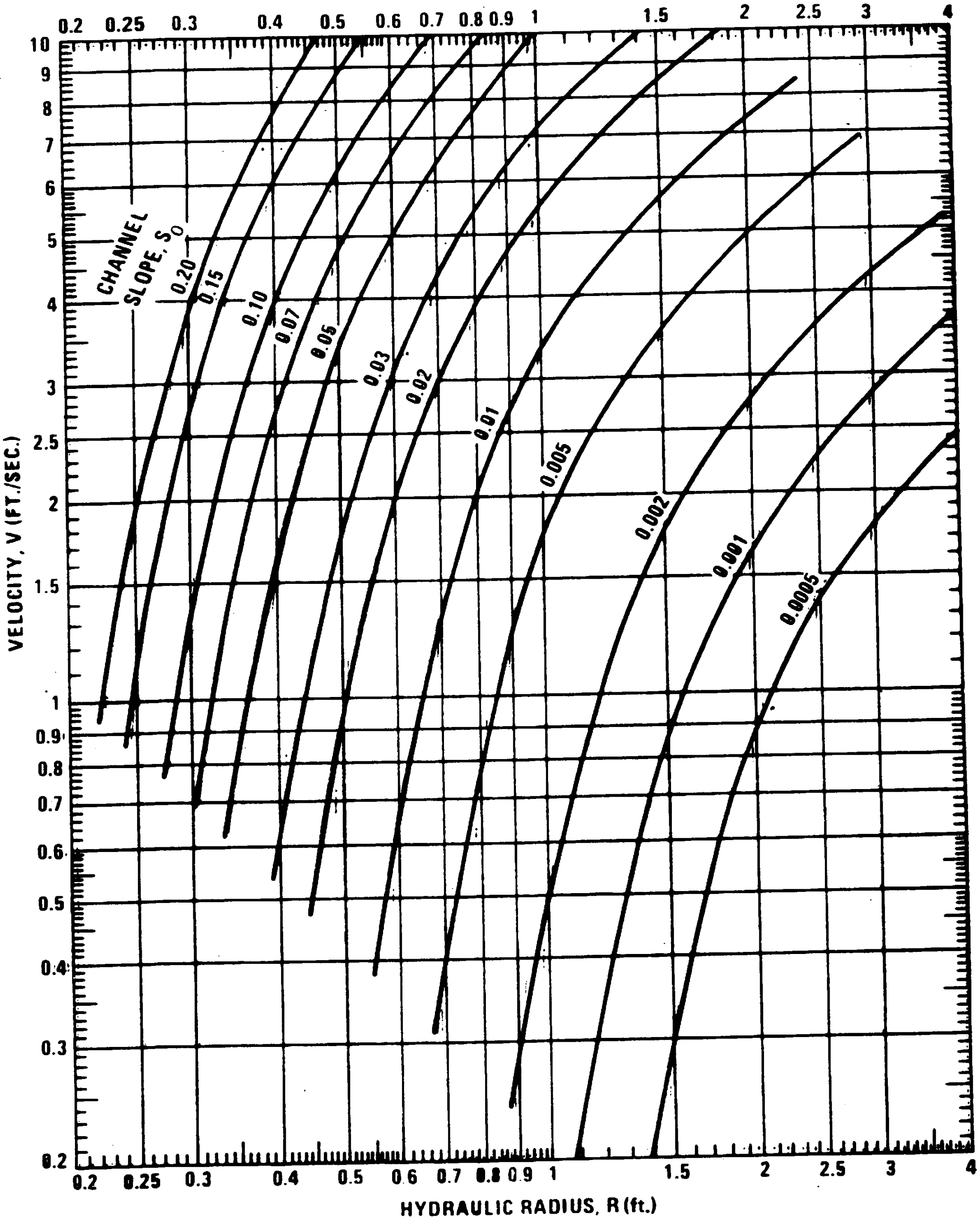
Flow Velocity for Channels Lined with Vegetation of Retardance A

CHART 6-17



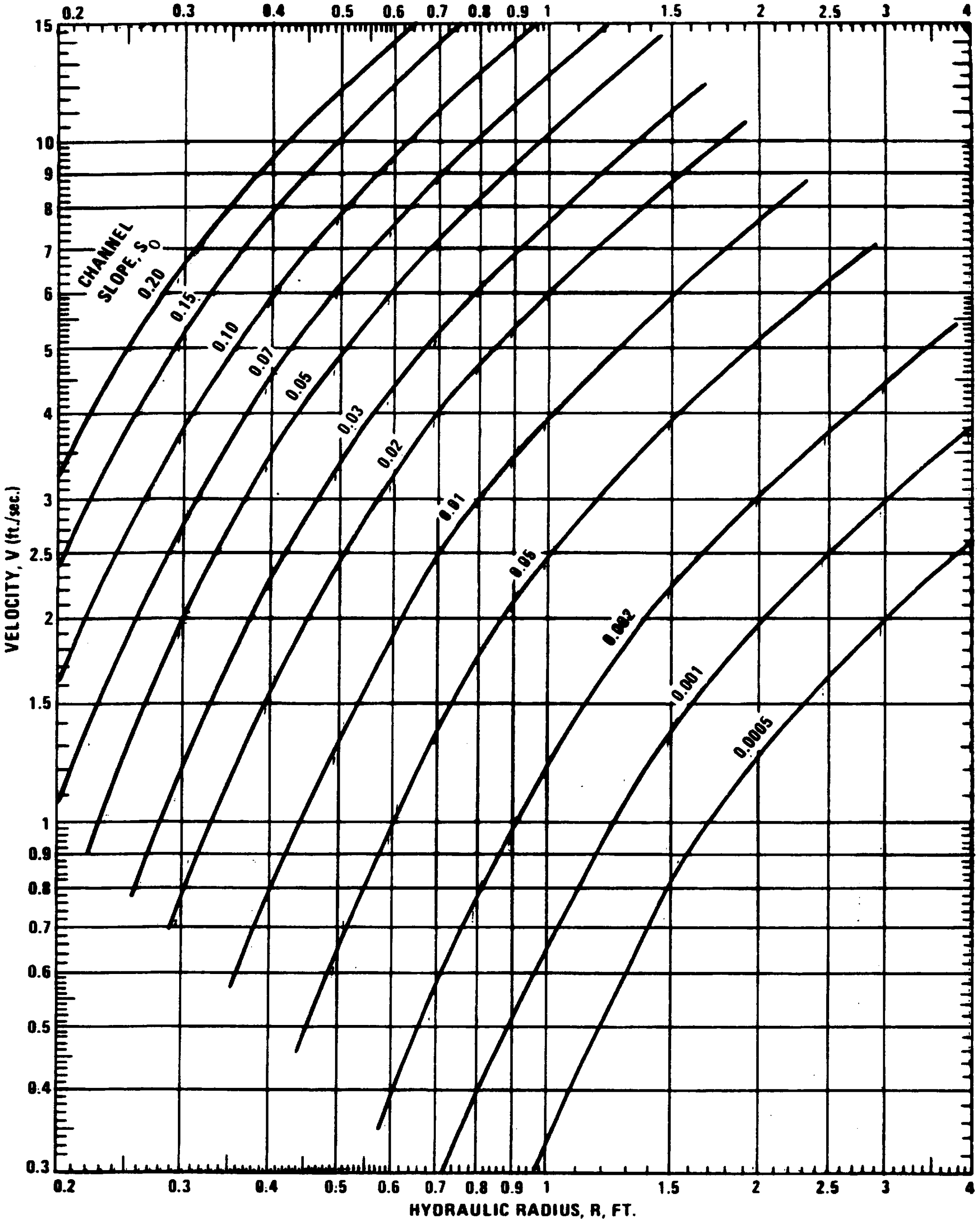
Flow Velocity for Channels Lined with Vegetation of Retardance B

CHART 6-18



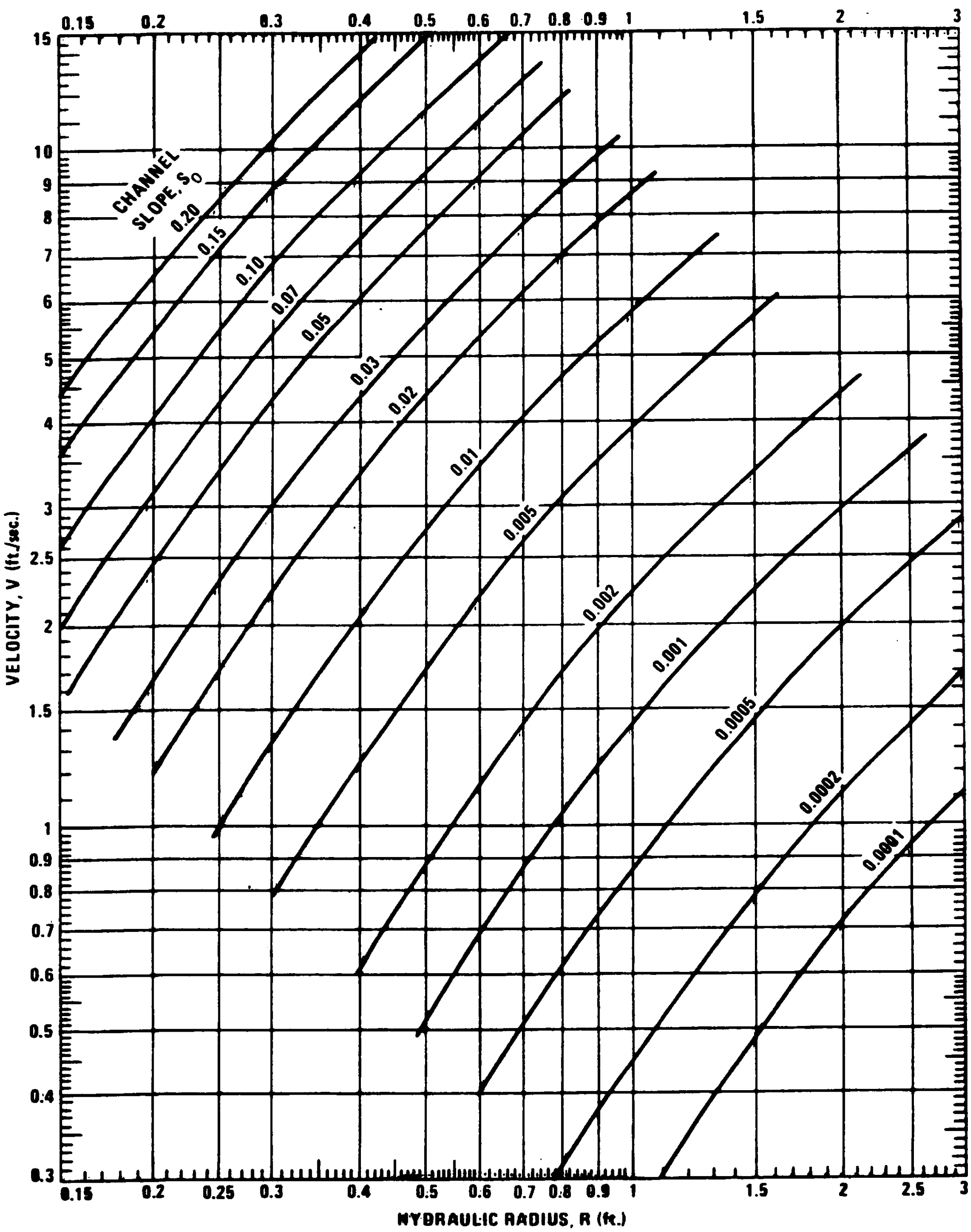
Flow Velocity for Channels Lined with Vegetation of Retardance C

CHART 6-19



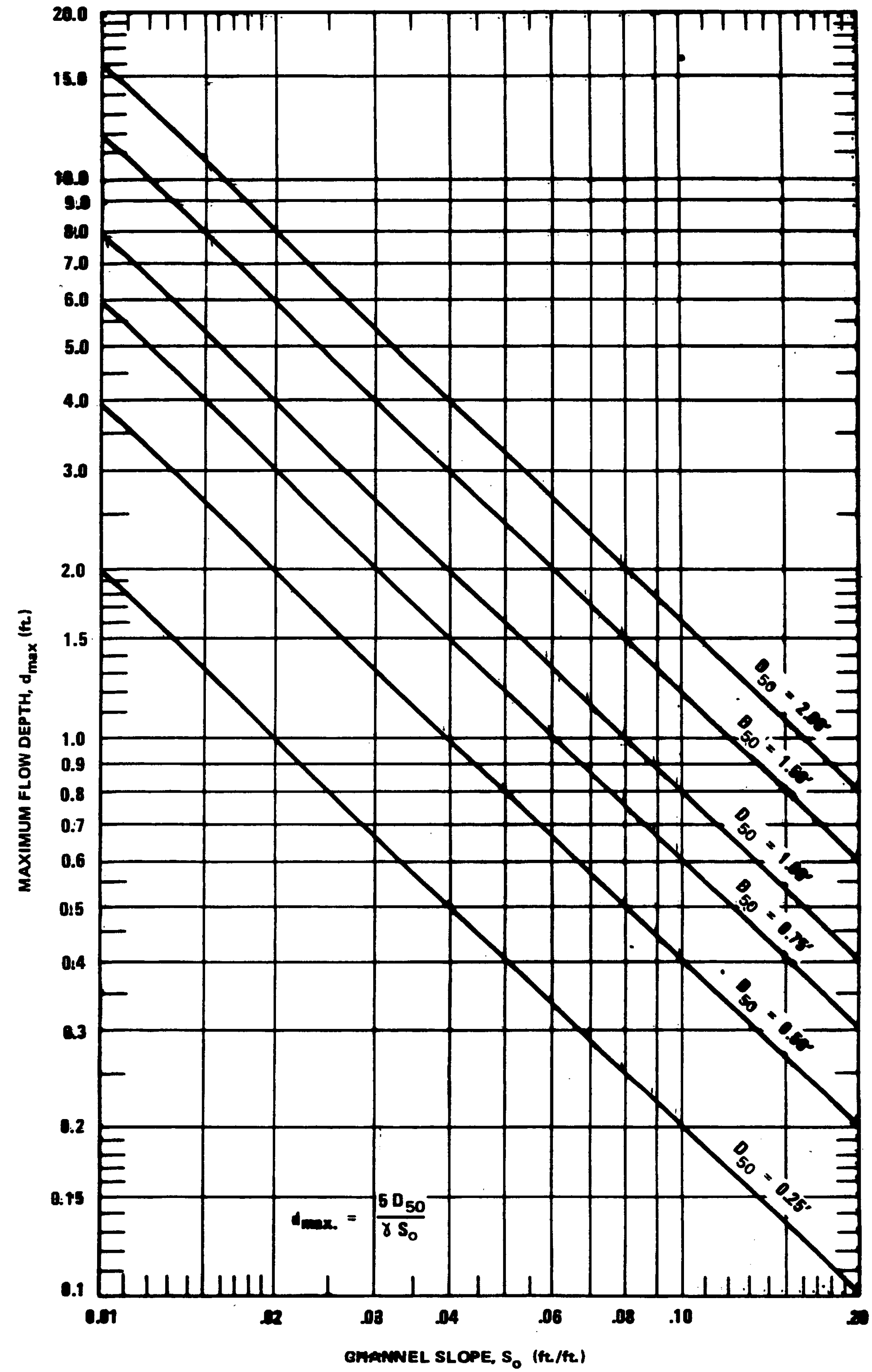
Flow Velocity for Channels Lined with Vegetation of Retardance D

CHART 6-20



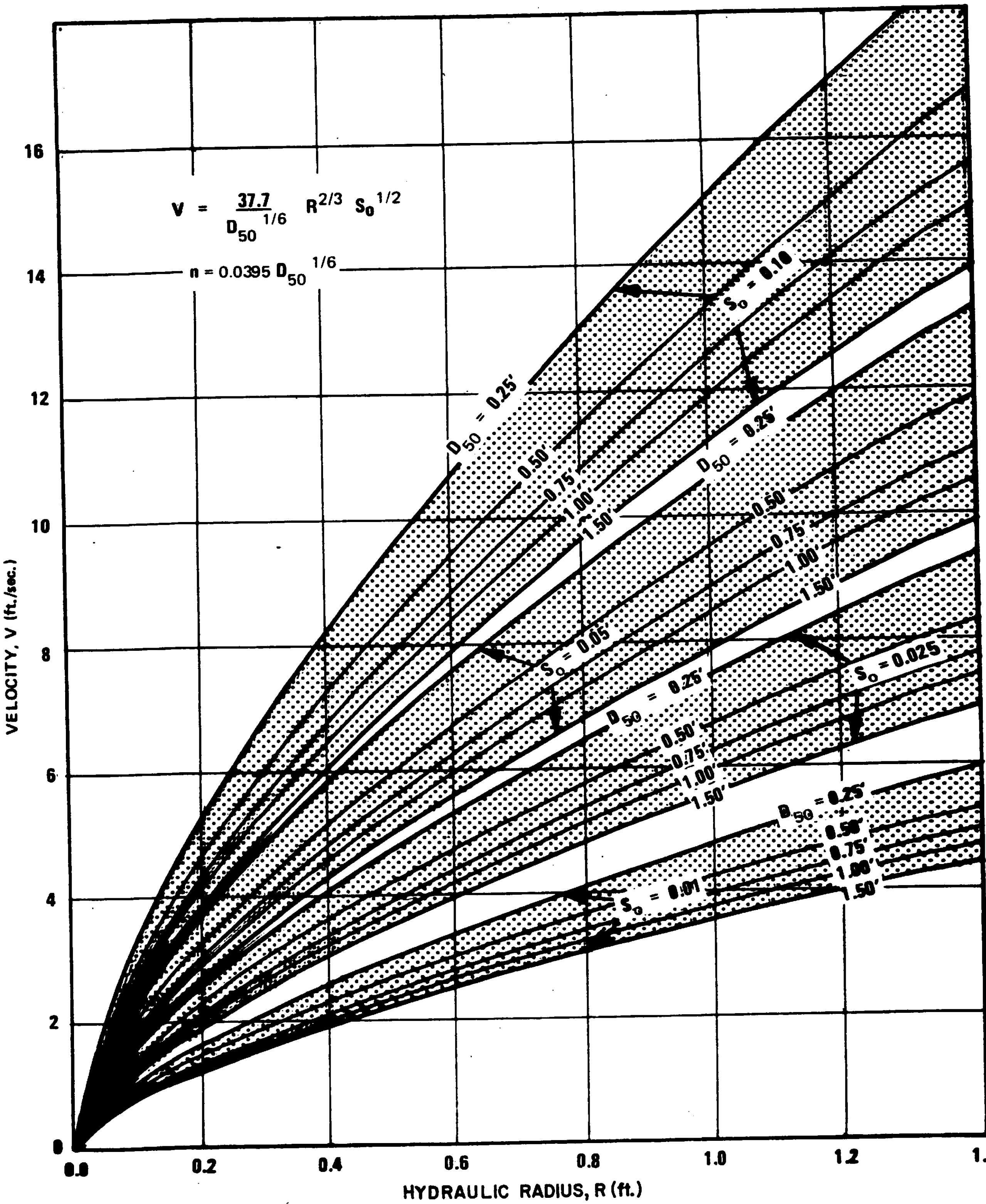
Flow Velocity for Channels Lined with Vegetation of Retardance E

CHART 6-21



MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max})
FOR CHANNELS LINED WITH ROCK RIPRAP

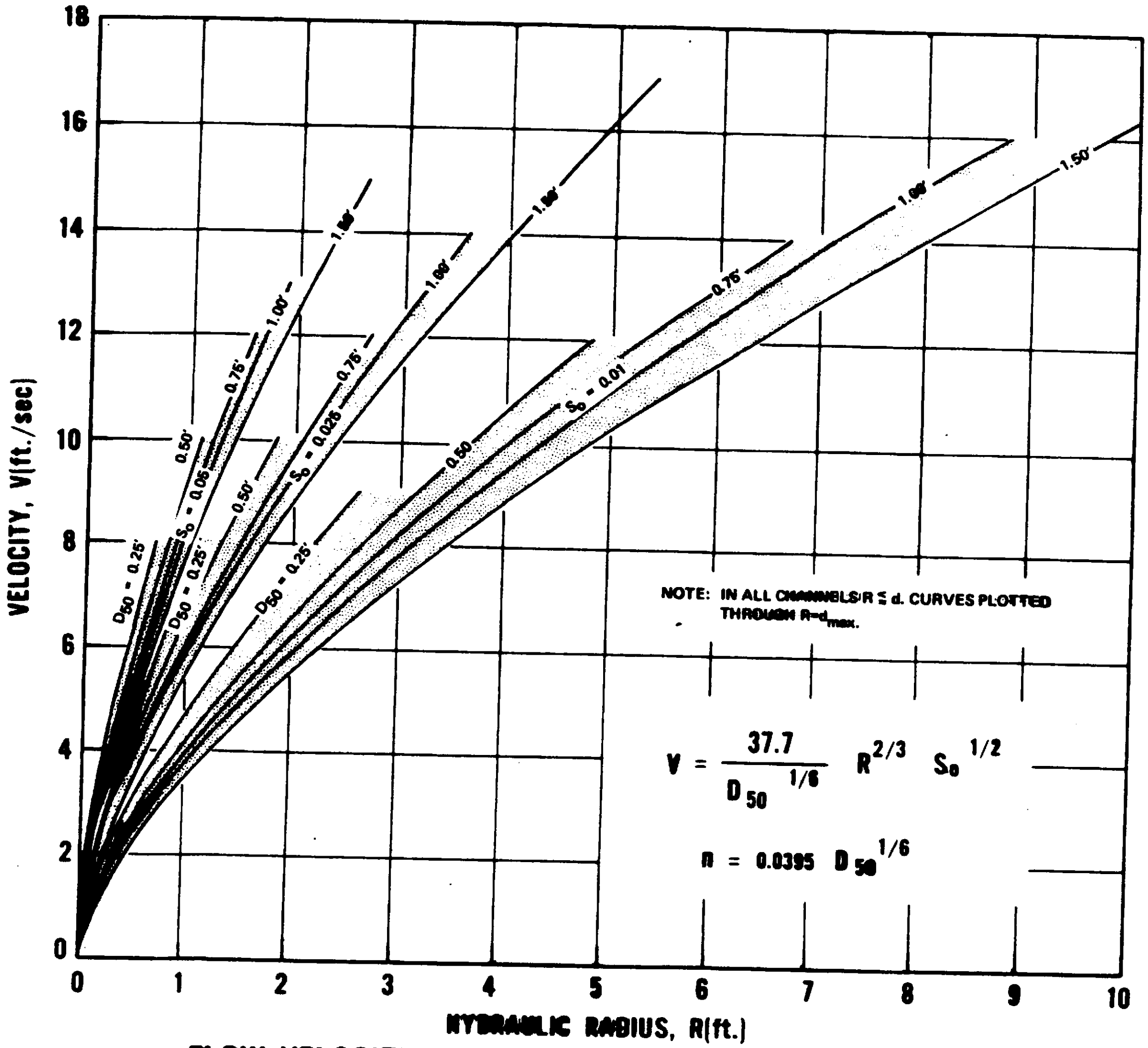
CHART 6-22



FLOW VELOCITY FOR CHANNELS LINED WITH ROCK RIPRAP

SLOPES=0.01 TO 0.10, D_{50} = 0.25' TO 1.50'

CHART 6-23



FLOW VELOCITY FOR CHANNELS LINED WITH ROCK RIPRAP
SLOPES = 0.01 TO 0.05, D_{50} = 0.25' TO 1.50'

CHART 6-24

When rock riprap is used, the need for an underlying filter material must be evaluated. The filter material may be either a granular filter blanket or plastic filter cloth.

Design of Granular Filter Blanket

For a granular filter blanket, the following criteria should be met:

$$\frac{D_{15}\text{ filter}}{D_{85}\text{ base}} \quad 5 \quad \frac{D_{15}\text{ filter}}{D_{15}\text{ base}} \quad 40$$

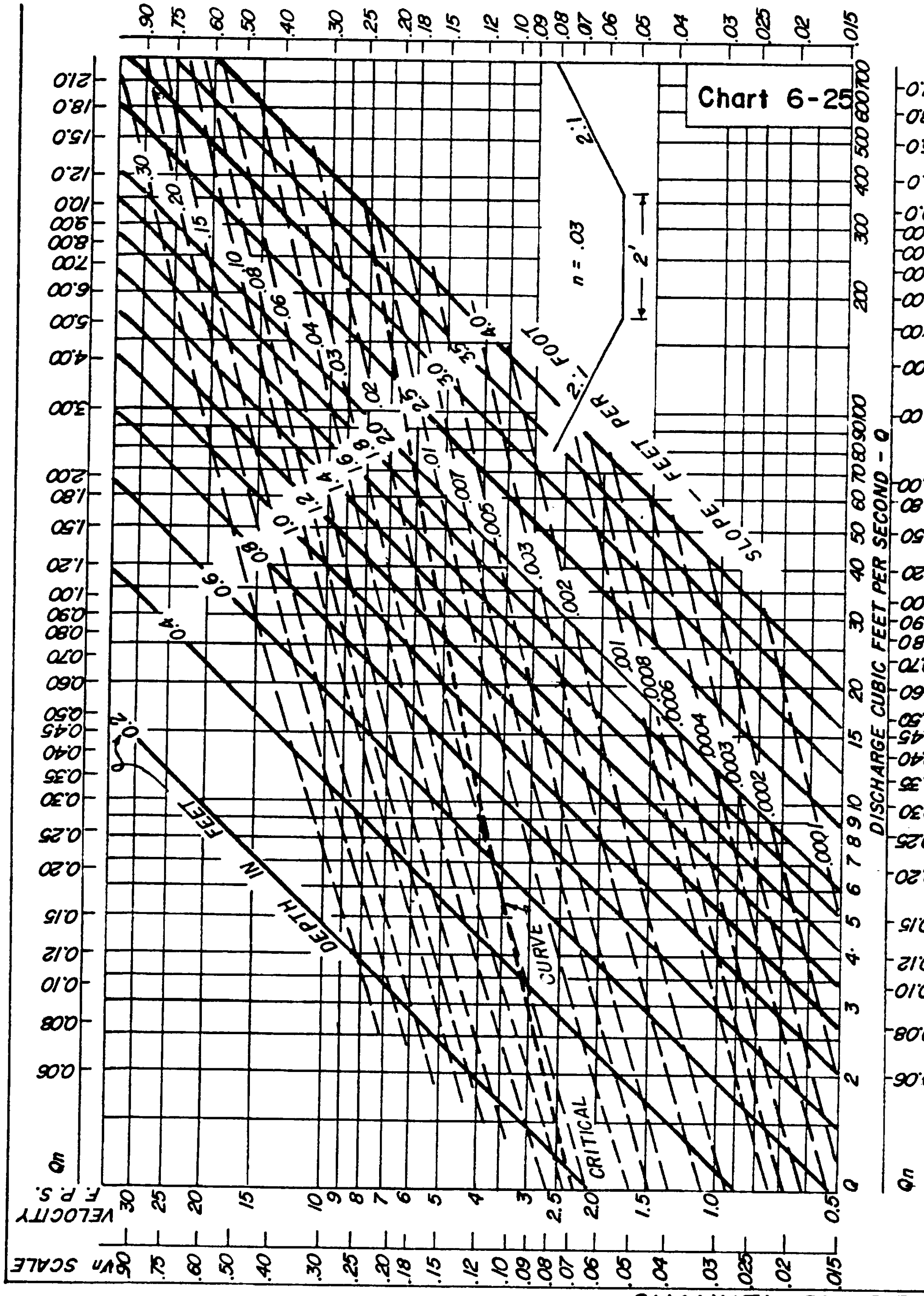
and

$$\frac{D_{50}\text{ filter}}{D_{50}\text{ base}} \quad 40$$

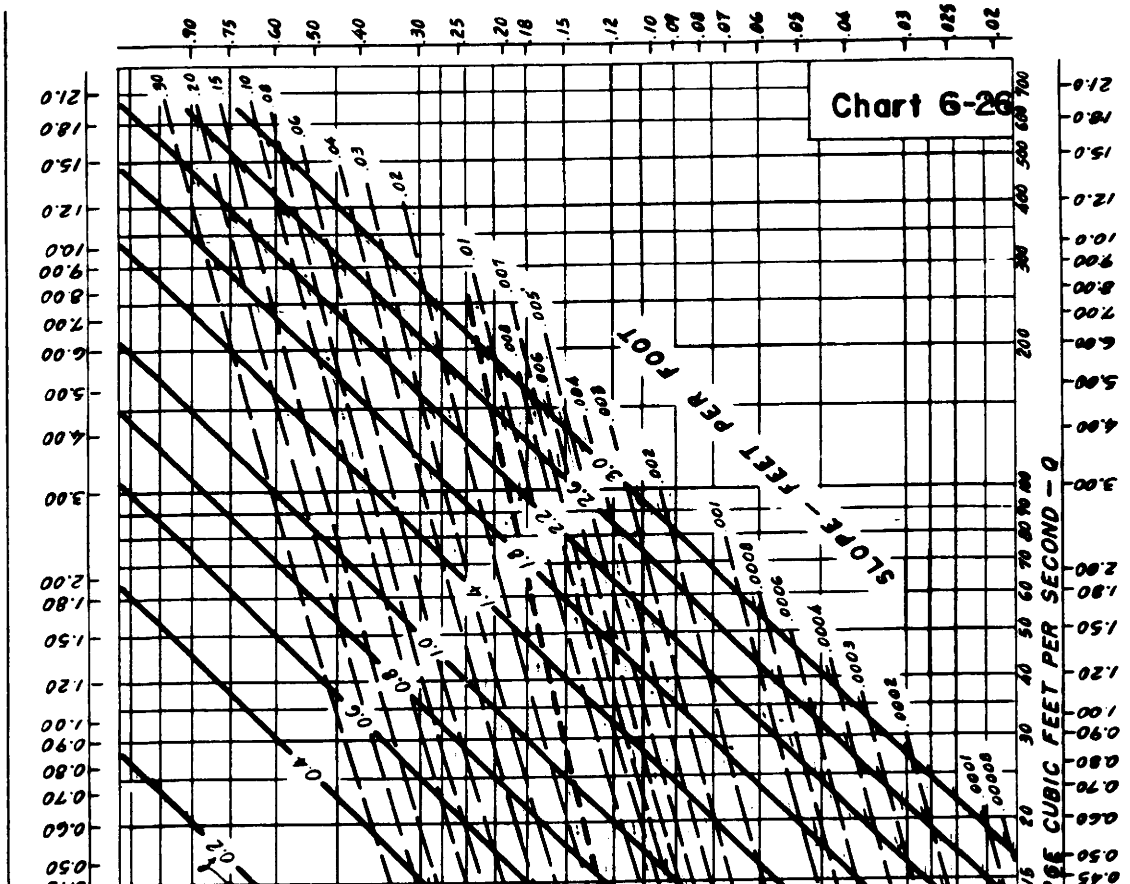
In the above relationships, filter refers to the overlying material and base refers to the underlying material. The relationships must hold between the filter blanket and base material and the riprap and filter blanket. Reference 12 contains a detailed procedure for the design of a filter blanket.

6-905 CONCRETE

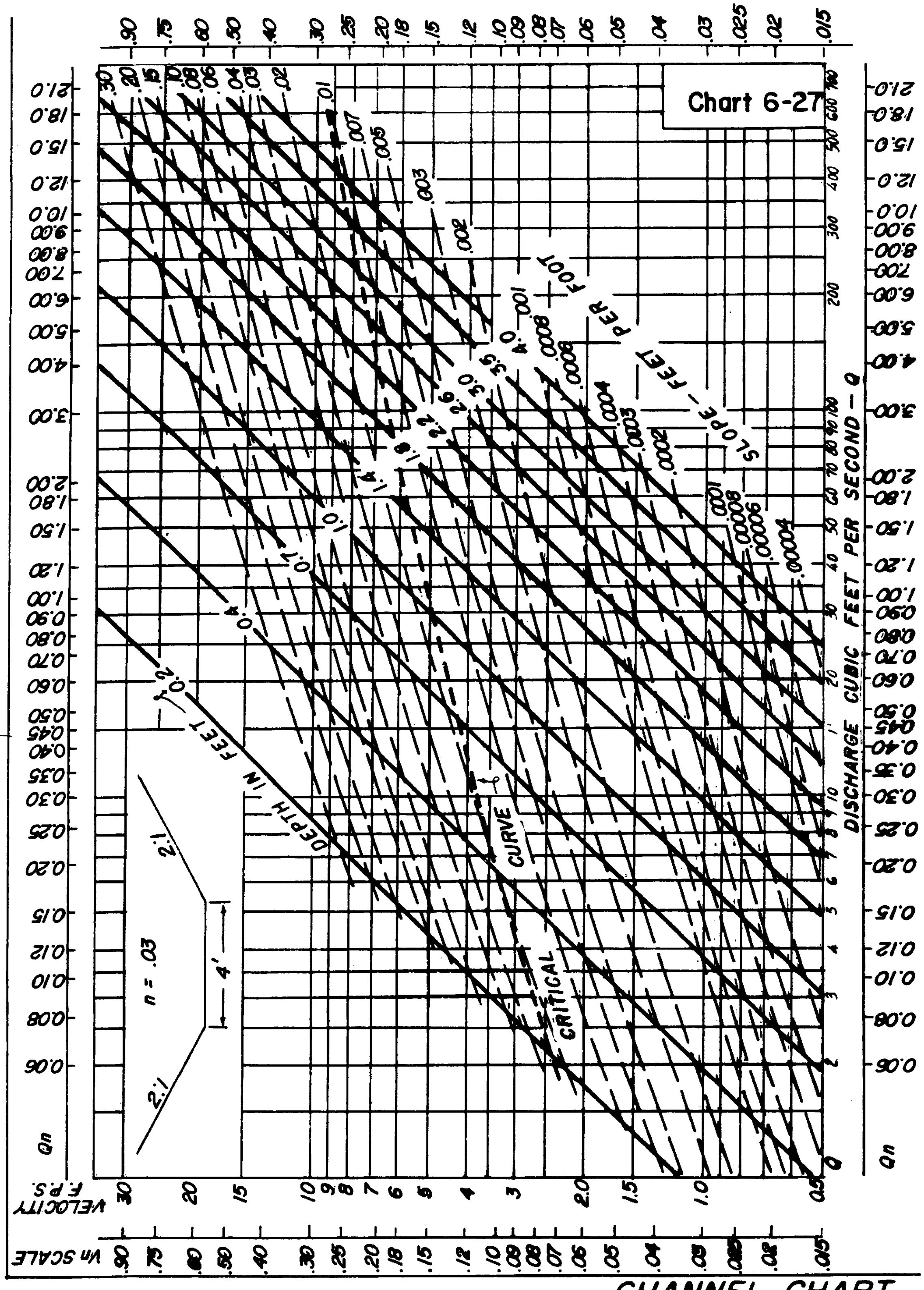
Concrete lined channels provide high capacities, but also have high outlet velocities so erosion problems become evident and must be dealt with. Since no scour occurs in rigid linings for the velocities normally encountered in drainage design, no d_{max} curves are necessary. Capacity charts 6-25 through 6-38, pages 6-61 thru 6-74, related velocity and discharge to the channel geometry, slope, and resistance. The Manning equation may be solved by trial and error by the trapezoidal channel charts.



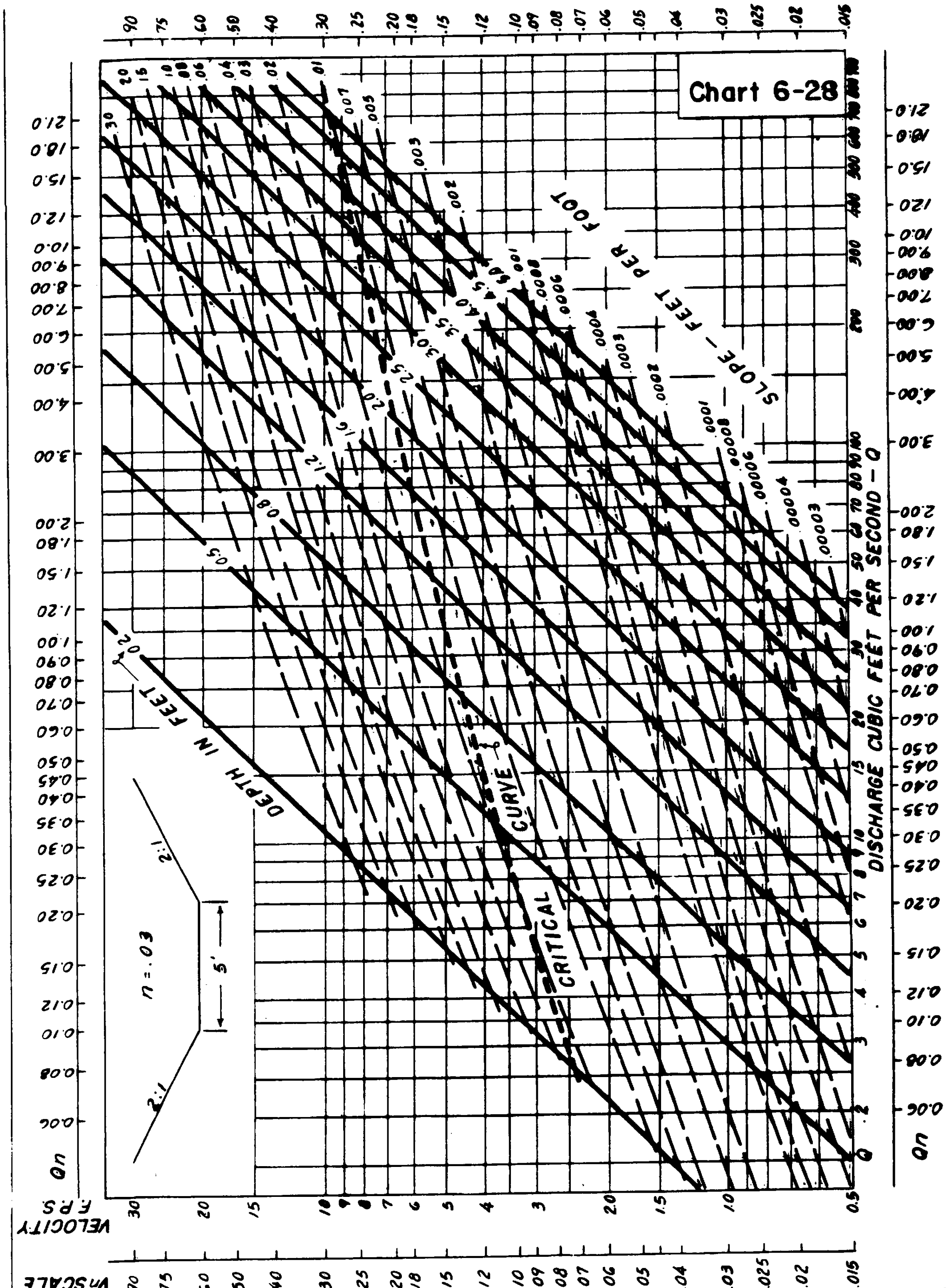
CHANNEL CHART
2:1 b = 2 FT.



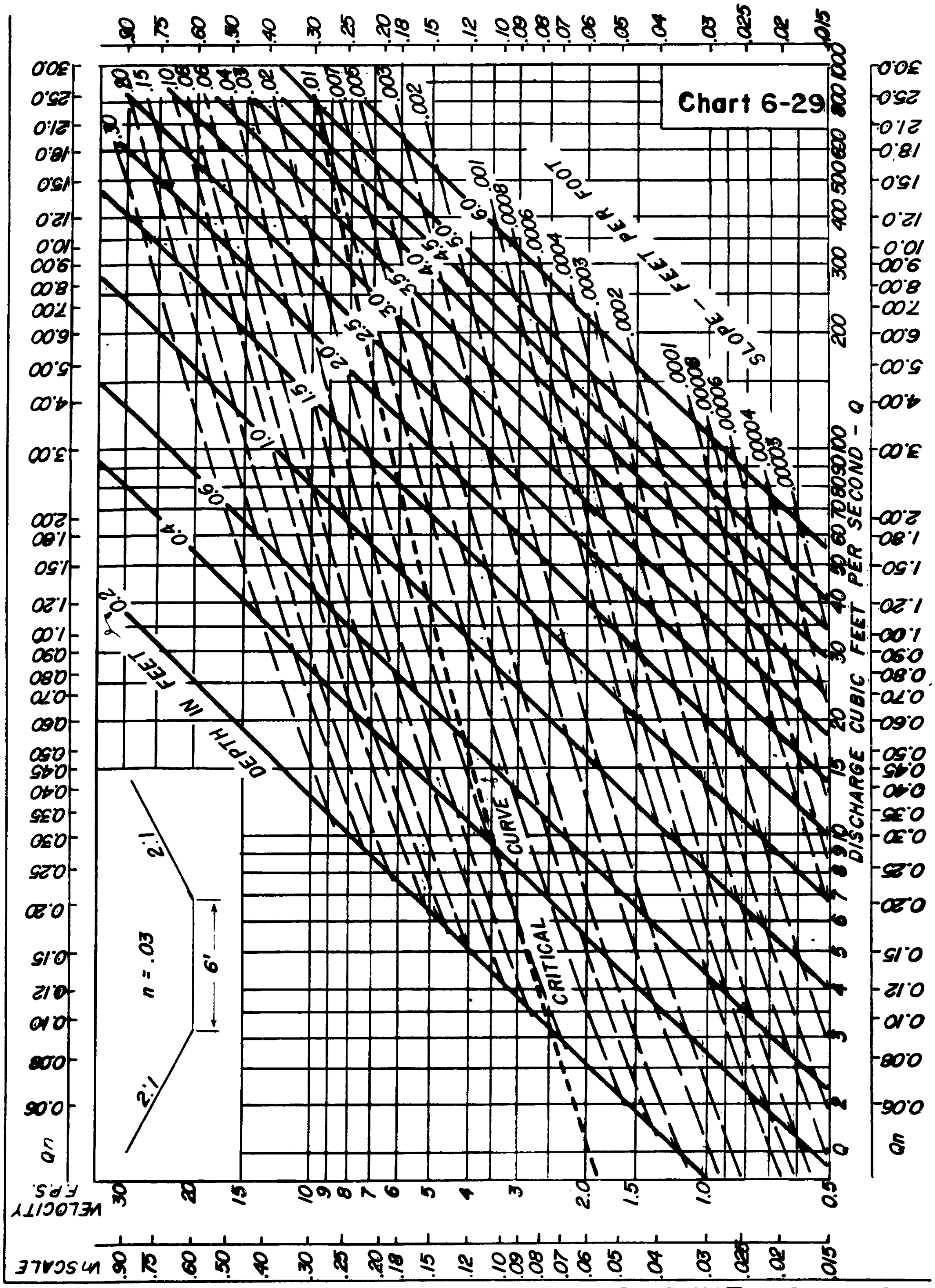
CHANNEL CHART
2:1 b = 3 FT.



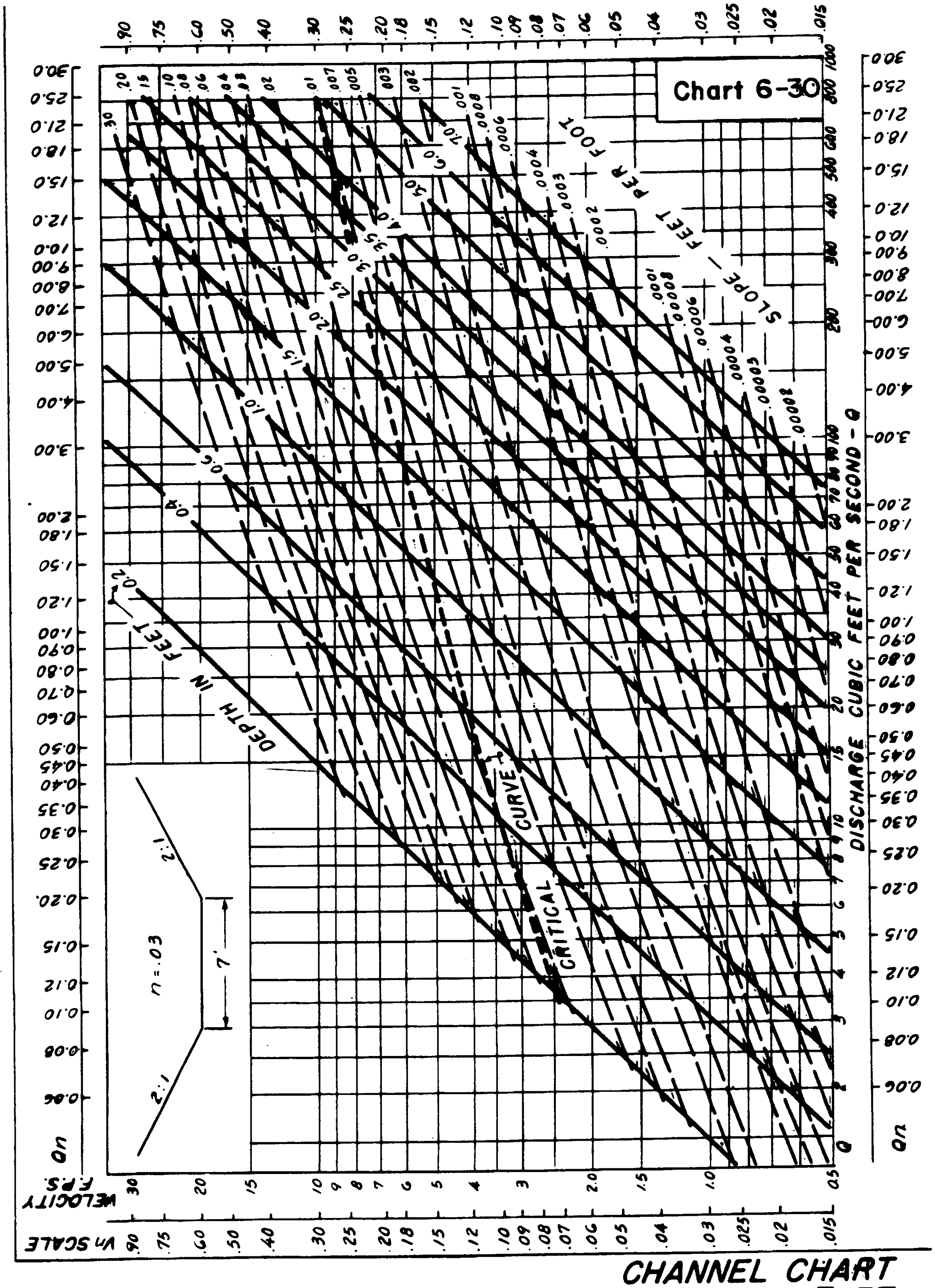
CHANNEL CHART
2:1 b = 4 FT.



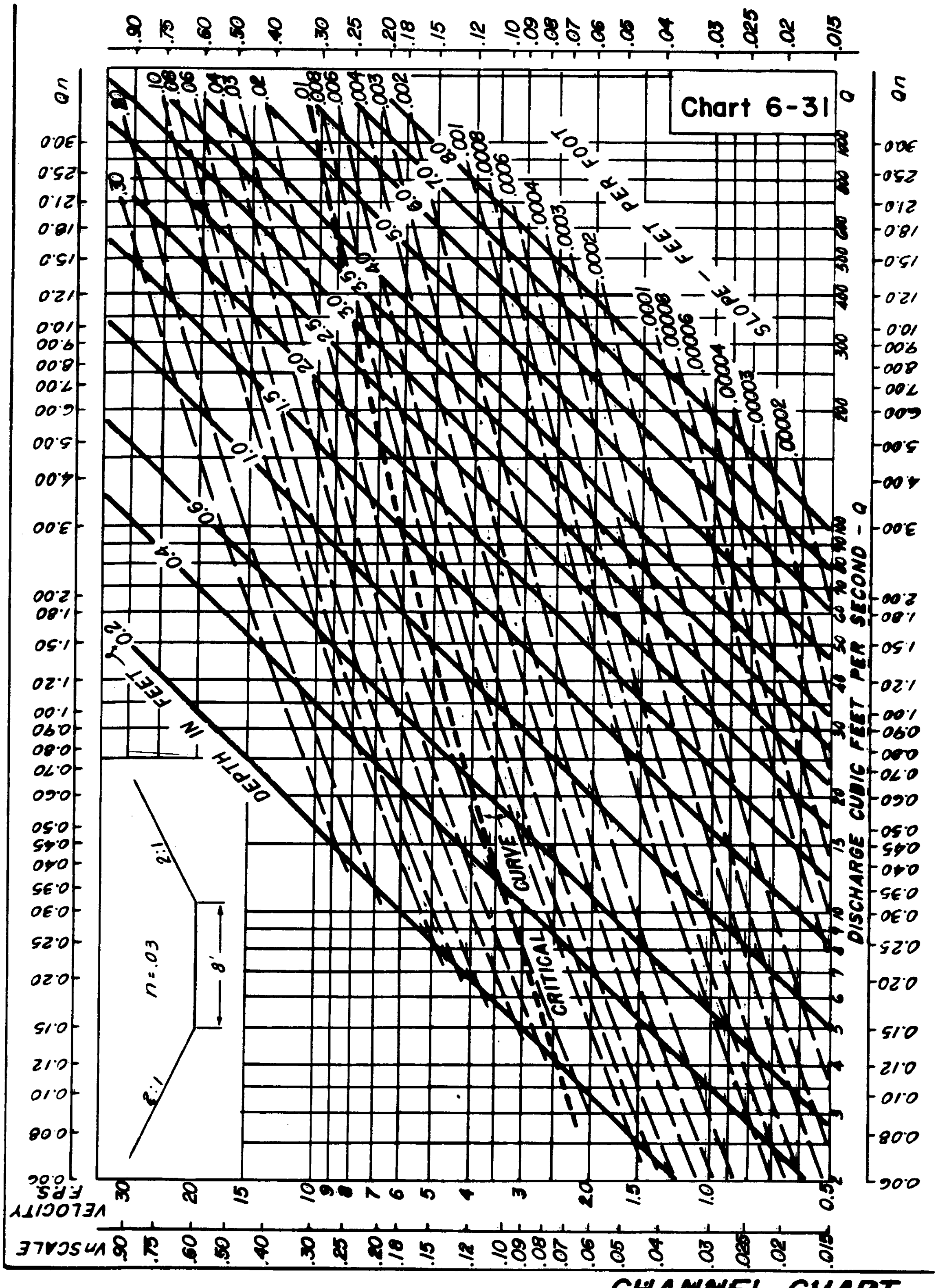
CHANNEL CHART
2:1 b = 5 FT.



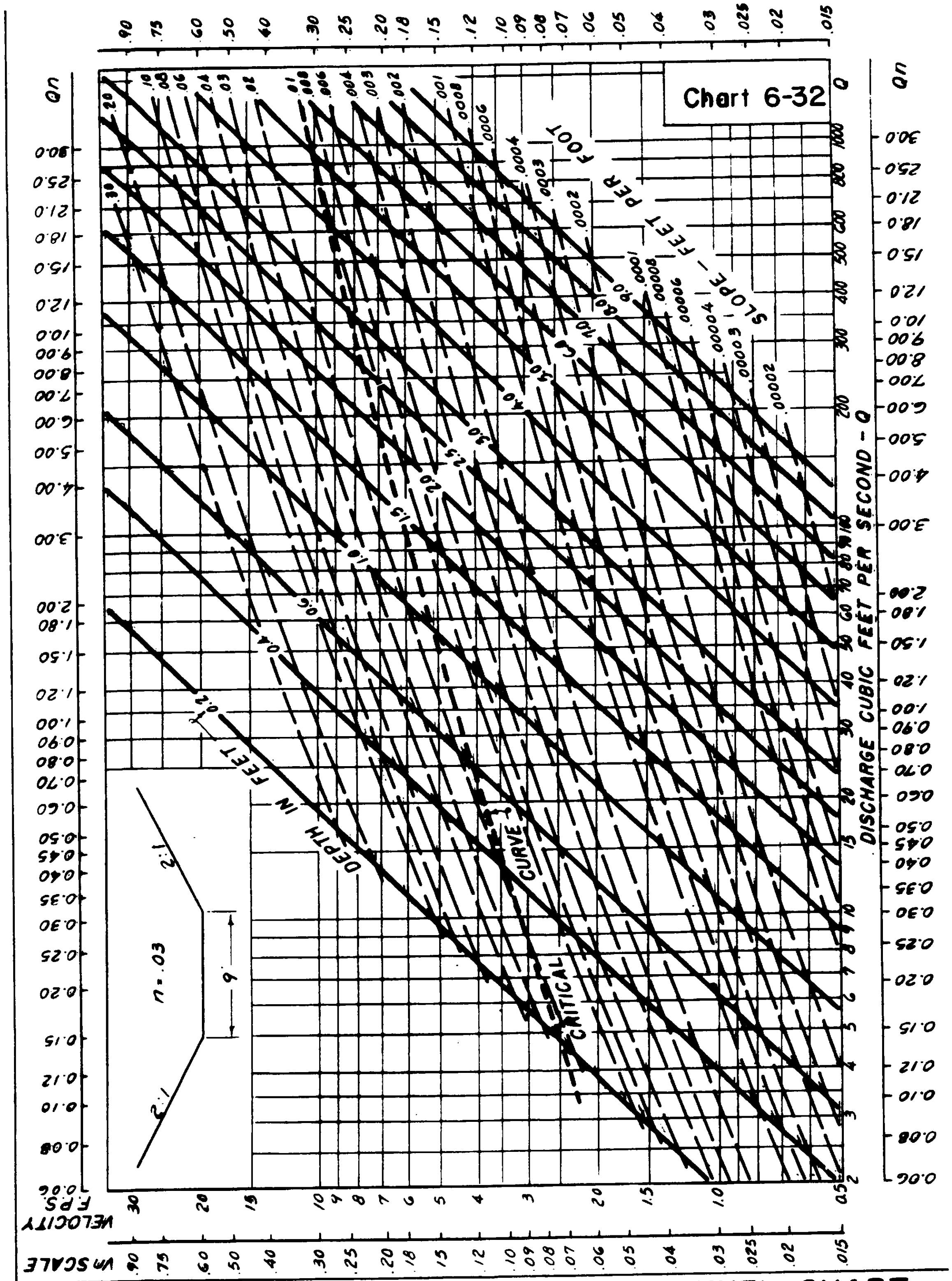
CHANNEL CHART
2:1 b = 6 FT.



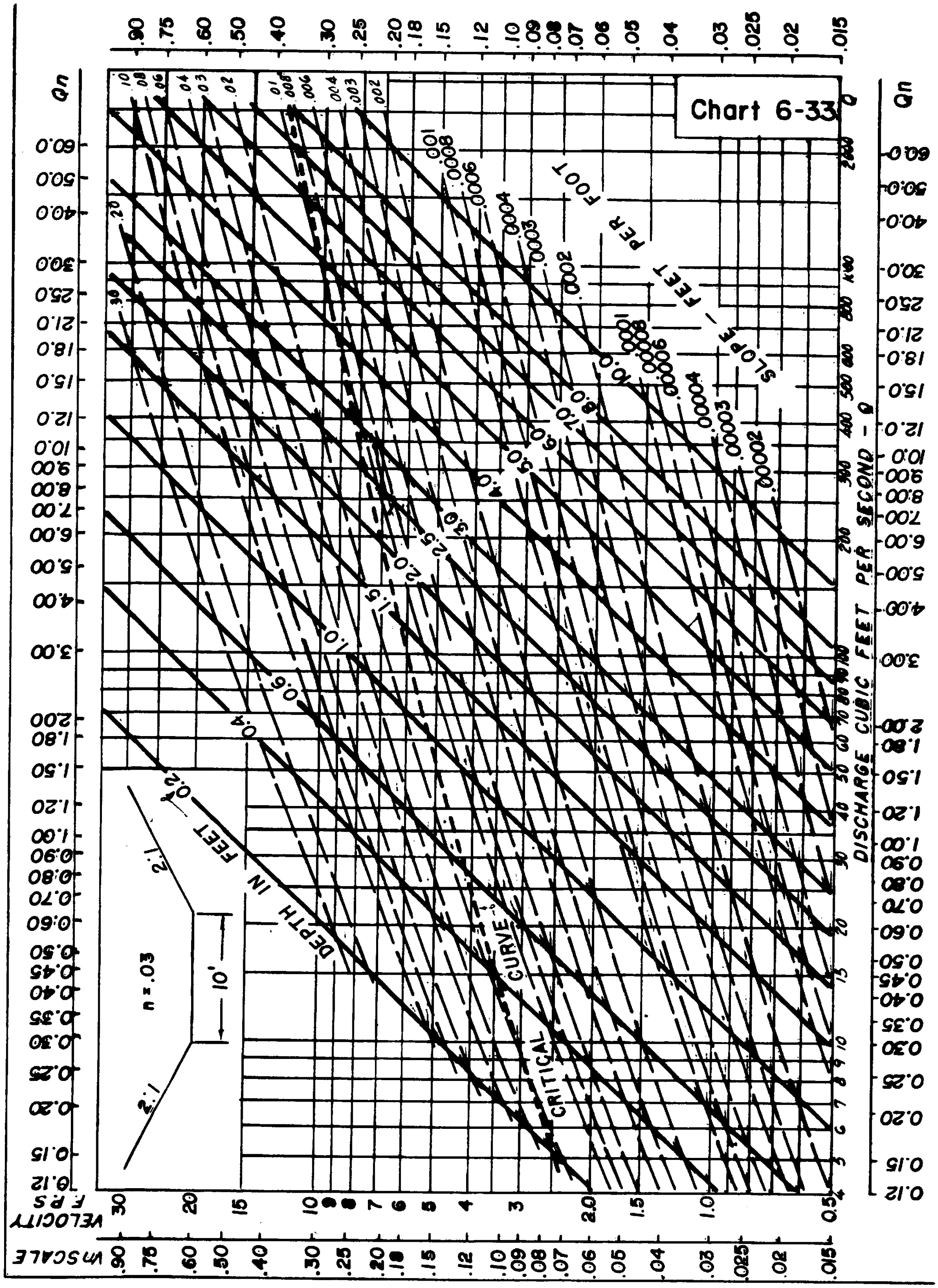
CHANNEL CHART
2:1 b = 7 FT.



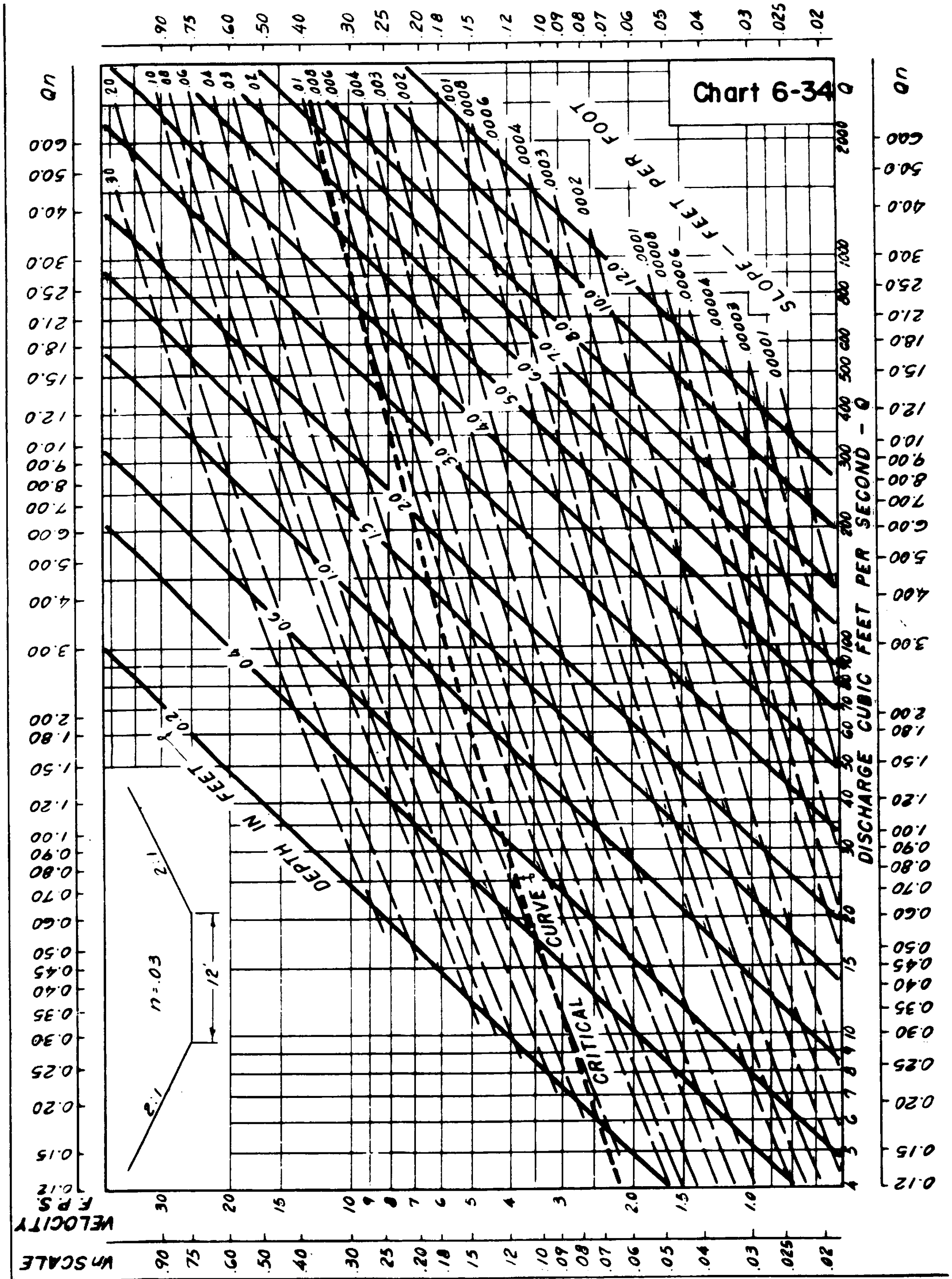
CHANNEL CHART
2:1 b = 8 FT.



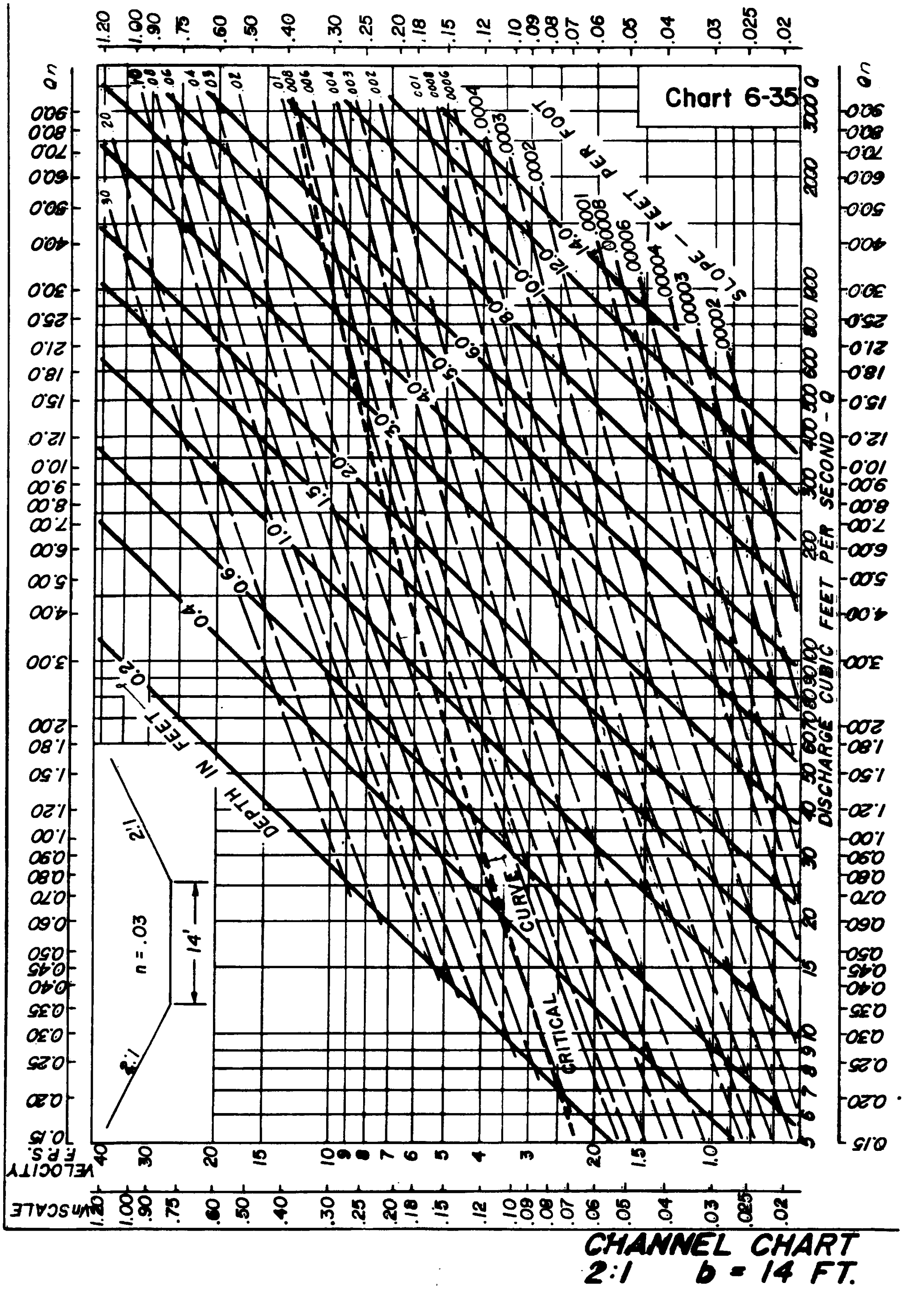
CHANNEL CHART
2:1 b = 9 FT.



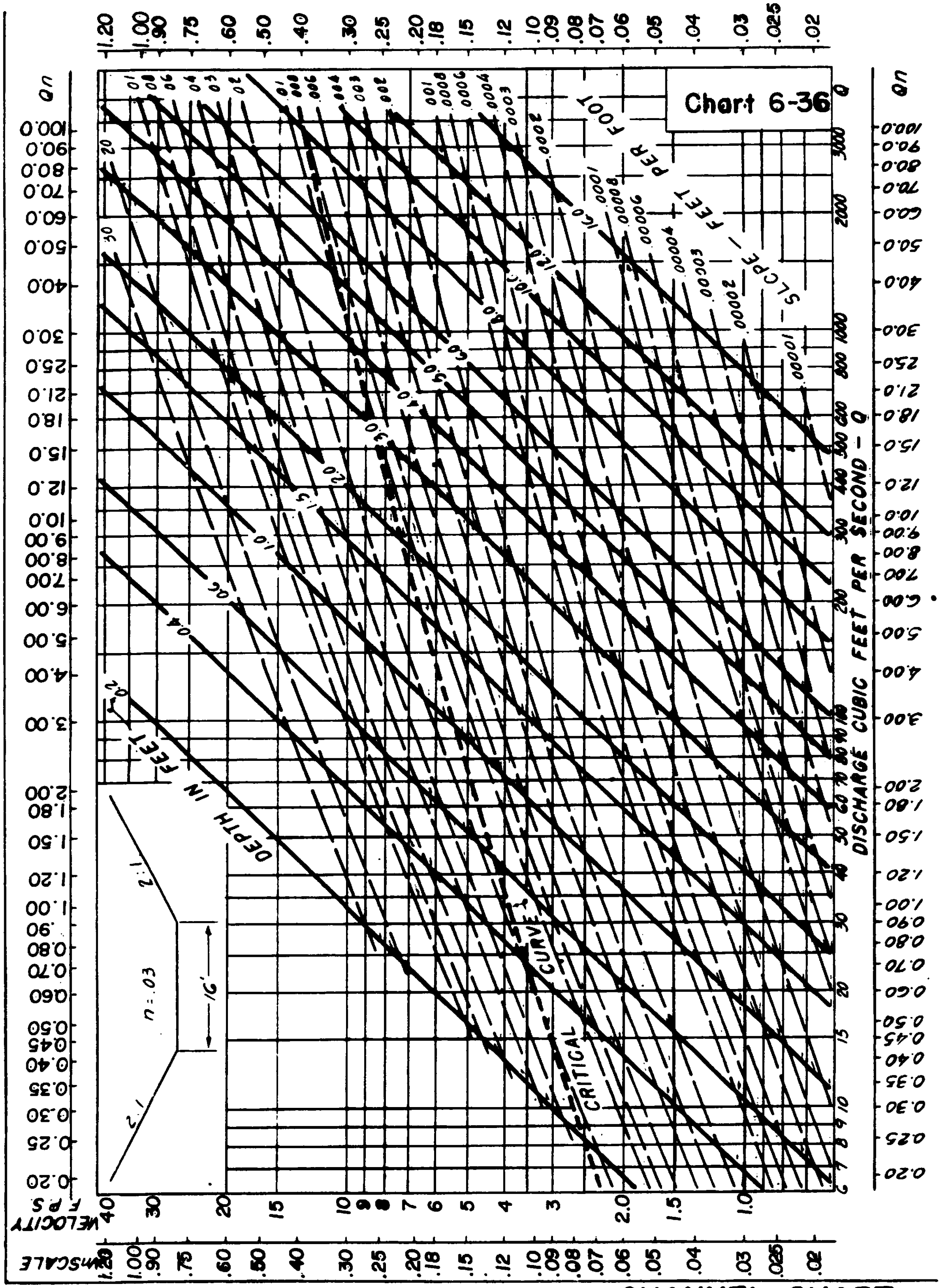
CHANNEL CHART
2:1 b = 10 FT.



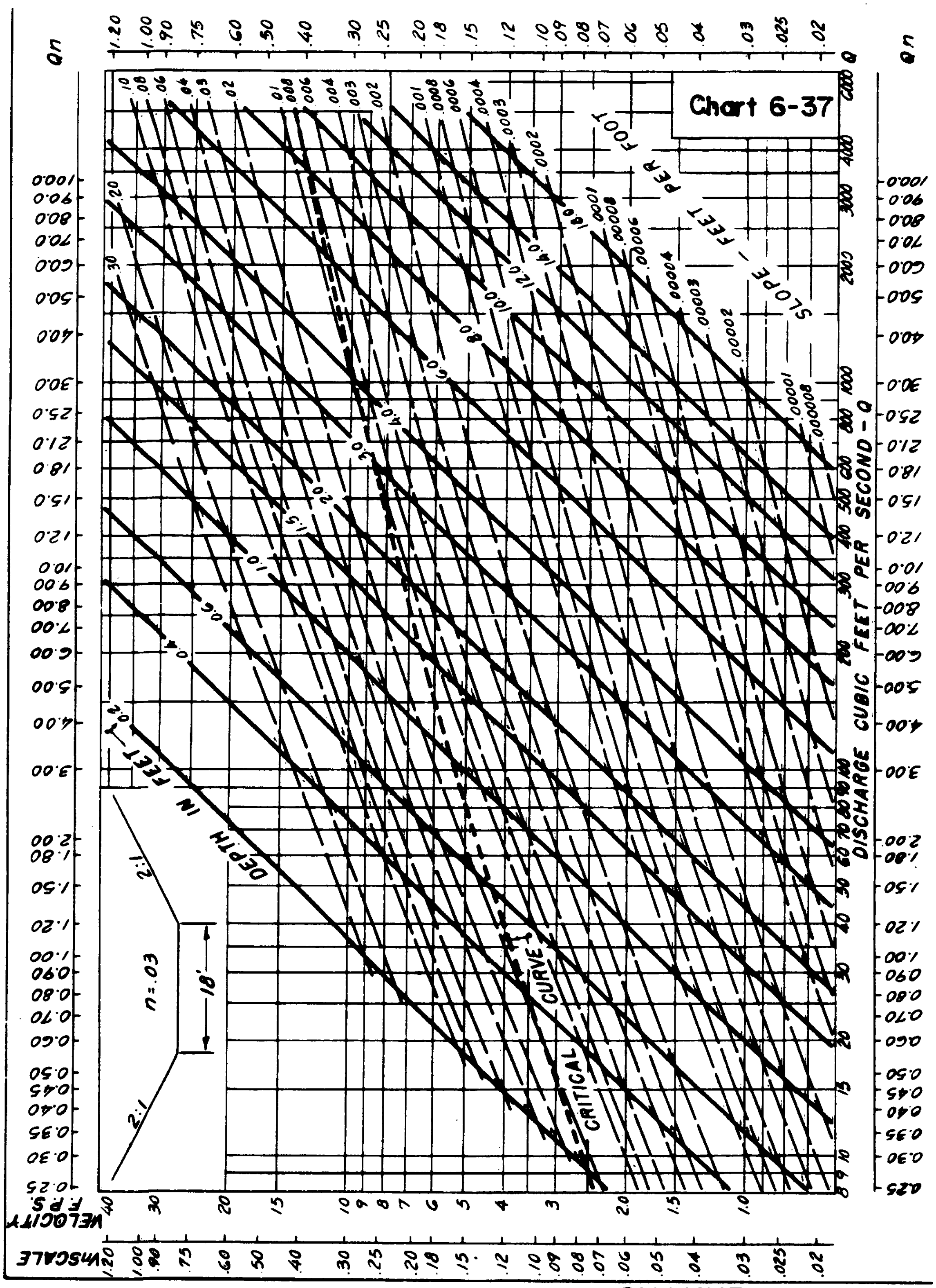
CHANNEL CHART
2:1 b = 12 FT.



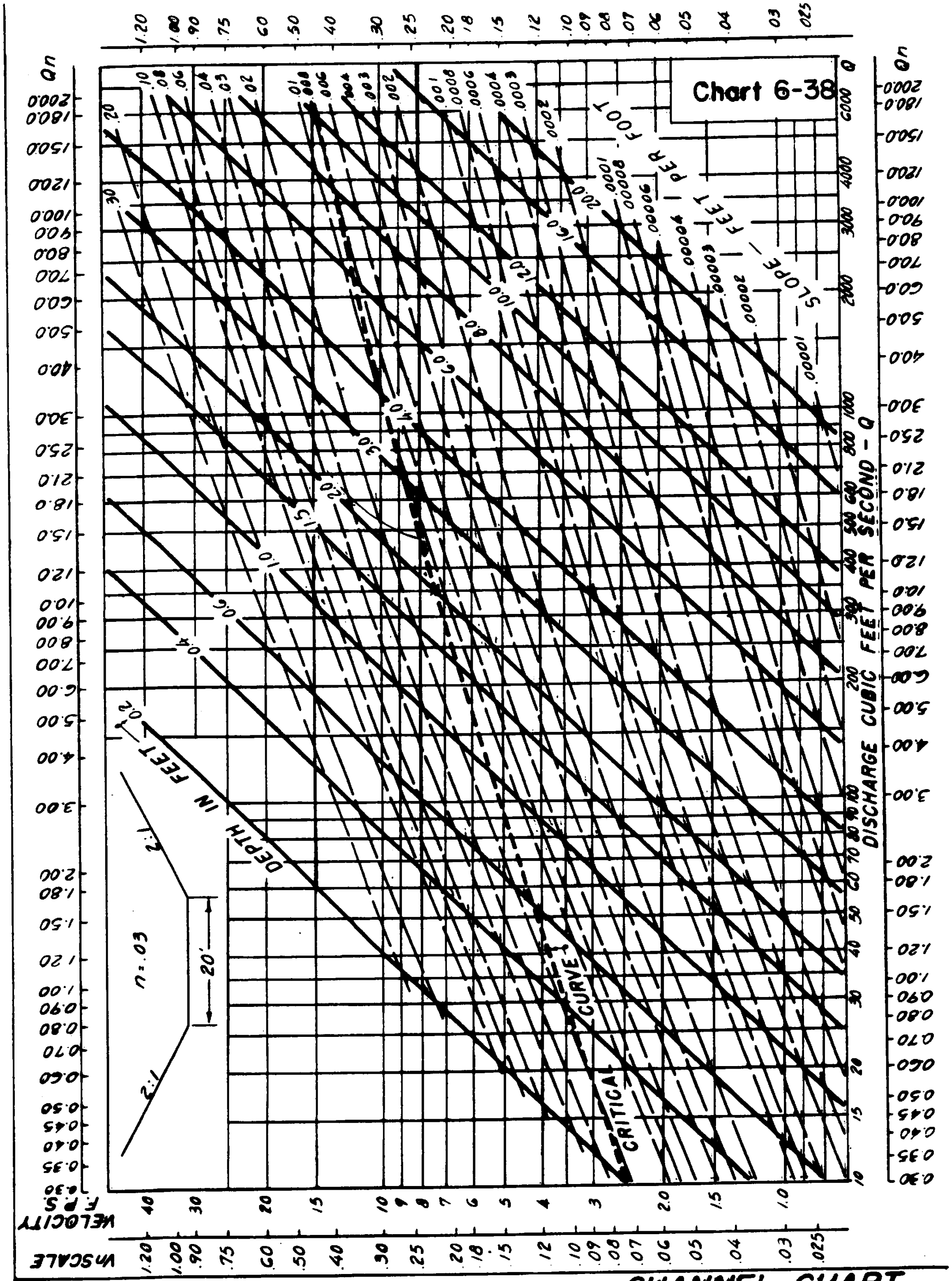
CHANNEL CHART
2:1 b = 14 FT.



CHANNEL CHART
2:1 b = 16 FT.



CHANNEL CHART
2:1 b = 18 FT.



CHANNEL CHART
2:1 b = 20 FT.

6-1000 DESIGN PROCEDURES FOR CHANNEL LININGS

The example problem in this section outlines a procedure for comparing different types of linings (permanent and temporary) to arrive at a hydraulically feasible answer. The table used in Chart 6-39, page 6-77, is a good form to use for channel lining design and can be implemented easily in form to check with each project.

This section outlines the design procedure for flexible channel linings and the design procedures for providing protection for channel bends and for channels with steep side slopes.

FLEXIBLE LINING DESIGN

The design consists of the following steps:

1. Perform hydrologic computations.
2. Select design flows for permanent lining material and for temporary linings.
3. Estimate soil erodibility.
4. Define channel shape, slope, and maximum top width.
5. Select least costly permanent lining material available.
6. Determine d_{max} for the selected lining, slope, and soil erodibility from the Maximum Permissible Depth Charts.
7. Determine hydraulic radius, R, and area, A, for the selected channel geometry and d_{max}. (Chart 6-1 or Calculations).
8. Determine velocity from R and slope, S_o, from the Flow Velocity vs. Hydraulic Radius Charts.
9. Allowable Q - AV.
10. If the allowable Q is much greater than the design Q, the channel is overdesigned. If Q is less than the design Q, the lining is inadequate. In either case, select another channel size and return to step 5 or select another lining material and return to step 6. Also, consider the feasibility of additional inlets to reduce the flow in the channel.

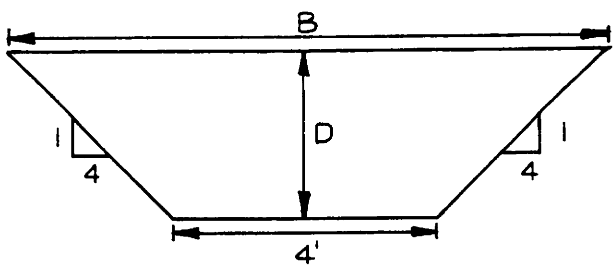
11. If a grass lining is the choice from the above computations, and it is desired to use a temporary lining material for channel protection during the period of grass establishment, perform steps 6, 7, 8 and 9 using the channel bottom width and side slopes for the grass lined channel with the selected temporary lining material and flow rate. The most stable temporary lining material is a double layer of fiber glass tacked with asphalt; however, this lining may retard vegetation germination and growth.

A computation sheet, shown in Chart 6-39, has been developed to facilitate the above design procedure using the charts of this section.

EXAMPLE PROBLEM

The objective in this problem is to design a channel lining for a trapezoidal channel with a 4 foot bottom width and 4:1 side slopes. Based on an analysis of the risks of channel failure, it is decided to design the permanent lining for a 10-year recurrence interval runoff and the temporary lining for the mean annual flood flow, with a recurrence interval of 2.33 years.

Refer to the sketch below for this example problem:



$S = 0.05$
T_R = 10 YRS. (PERMANENT LINING)
T_R = 2.33 YRS. (TEMPORARY LINING)
D.A. = 4.3 ACRE
$B \leq 12$ FEET
SOIL IS OF AVERAGE ERODIBILITY

DRAINAGE CHANNEL LINING DESIGN

DATE: 11/9/72

PROJECT: FAI - I(87)

DESIGNER: JMN

STATION 105 + 40 TO STATION 112 + 80

DRAINAGE AREA = 4.3 ACRES

HYDROLOGIC COMPUTATIONS:

10 year flow

$Q = CiA = (0.7)(5.0)(4.3) = 15$ cfs

Mean Annual flow

$Q = CiA = (0.7)(1.6)(4.3) = 4.8$ cfs

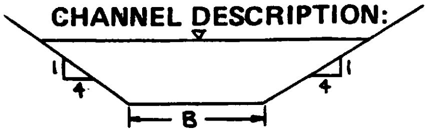
Use 5 cfs

DESIGN FLOW: Q_{10} = 15 cfs

DESIGN FLOW FOR TEMPORARY LINING: $Q_{2.33}$ = 5 cfs

SOIL ERODIBILITY: Average

CHANNEL DESCRIPTION:



MAX. TOP WIDTH = 12 ft. $T = B + (Z_1 + Z_2)d$

$= 4 + 8d$

S_o = 0.05

AVAILABLE LININGS:

Permanent
1. Bare Soil
2. Bermuda Grass (6")
3. Riprap (3", 6", 12")

Temporary
1. Fiber Glass Roving
2. Excelsior Mat

LINING	d_{max}	B	$\frac{d_{max}}{B}$	$\frac{A}{Bd}$	A	$\frac{R}{d}$	R	V	Q=AV	T	REMARKS
				Chart 1		Chart 1					
Bare Soil	0.04	4	0.01	—	0.17	—	0.038	1.40	0.24	4.3	No Good
6" Bermuda	0.71	4	0.18	1.7	4.83	0.69	0.49	3.3	15.9	9.7	OK
Grass (retard. C)											
3" Riprap	0.40	4	0.10	1.4	2.24	0.78	0.31	4.9	11.0	7.2	No Good
6" Riprap	0.80	4	0.20	1.8	5.76	0.68	0.54	6.3	36.3	10.4	OK - over designed
Therefore, use 6" Bermuda Grass											
Temporary Linings											
Fiber Glass											
1 Layer	0.16	4	0.04	1.16	0.74	0.87	0.14	2.5	1.85	5.3	No Good
2 Layers	0.40	4	0.10	1.35	2.16	0.77	0.31	6.0	13.0	7.2	OK
Excelsior Mat	0.45	4	0.11	1.4	2.52	0.76	0.34	2.7	6.8	7.6	OK
Check Capacity of 12" Bermuda Grass											
12" Berm. Grass	1.1	4	0.275	2.1	9.2	0.64	0.70	2.5	23.0	12.8	T>12', Use d=1
	1.0	4	0.25	2.0	8.0	0.66	0.66	1.9	15.2	12.0	
Concrete	1.0*							19	154		High Velocity
* No dmax											Rigid Channel

CHART 6-39

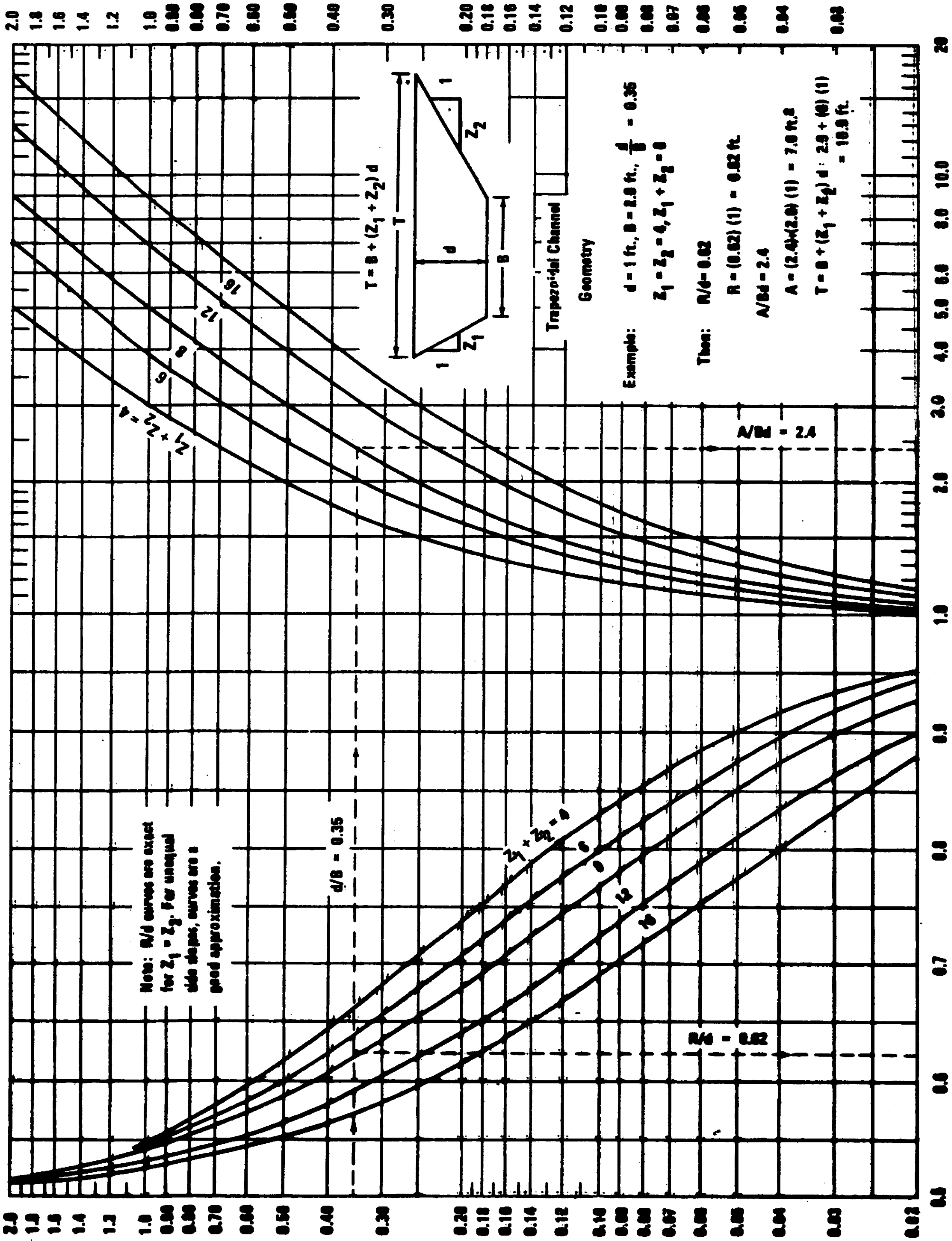
To determine the runoff rate, the Rational Equation is used for for the 4.3 acre drainage area. The soil is judged to have an average erodibility. Due to right of way constraints, the channel top width must be restricted to 12 feet. Channel slope is 4 percent. Several permanent and temporary channel lining materials are available.

Detailed calculations are shown in Chart 6-39, page 6-77. Note that the bare soild would convey very little flow on this 5 percent slope without severe erosion. Bermuda grass or rock riprap are adequate. Since 6-inch Bermuda grass is the lining chosen, temporary linings are evaluated and either a double layer of fiber glass roving and asphalt or excelsior mat is adequate to convey the mean annual flow rate of 5.0 cfs.

Should the grass be permitted to grow to a 12-inch length, the retardance of the channel would be increased. Then the channel may not convey the 10-year runoff without overtopping its banks. A check of the 12-inch Bermuda grass reveals that d_{max} is greater than 1.0 ft., so that the top width of the flow exceeds 12 feet. Therefore, a 1.0 ft. depth of flow is used to check the channel capacity, which is found to be 15.2 cfs.

The concrete lining has no d_{max}. From Chart 6-40, page 6-79, it is found that a 1.0 ft. depth of flow in the concrete lining at a 5-percent slope would convey 154 cfs at a velocity of 19 fps. This is the hydraulic advantage and disadvantage of a concrete lining (high capacity coupled with a high, erosive outlet velocity).

CHART 6-40



TRAPEZOIDAL CHANNEL GEOMETRY

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11. Anderson, A. G., Paintal, A. S., and Devenport, J.T., TENTATIVE DESIGN PROCEDURES FOR RIP RAP LINED CHANNELS, NCHRP Report 168, Highway Research Board, 1970.

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Chapter 7

BRIDGE HYDRAULICS

7-100 INTRODUCTION

Bridges are important and expensive highway-hydraulic structures that are vulnerable to failure from flood-related causes. In order to minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered in all phases of highway development, construction and maintenance.

With the spread of urbanization; with indefinite, unenforceable restrictions on the construction of housing and business establishments on floodplains of rivers and streams throughout the country, it has now become imperative that the backwater produced by new bridges be kept within very knowledgeable and reasonable limits. This has placed demands on the engineer to promote and develop a more scientific approach to the bridge waterway problem.

In fact, until recently, bridge lengths and clearances have been proportioned principally on rough calculations, individual judgement, and intuition. Because of increased economic pressures, the arbitary overdesign or making bridges longer than necessary cannot be justified anymore. The role of the engineer then is to size the most economical yet safe bridge. Good hydraulic engineering is necessary for good bridge design.

7-200 PLANNING AND LOCATION

The general stream crossing location is selected during the planning and location phase of highway project development. The final location should be selected only after detailed survey information has been obtained and preliminary hydraulic studies completed. Although not the sole consideration in bridge location, hydraulic aspects should receive major attention in the initial planning of a highway. The location and alignment of the highway can either exaggerate or ameliorate hydraulic problems at the crossing.

7-201 LOCATION OF STREAM CROSSING

Some of the main factors to be considered in the selection of a bridge location are listed below:

a) Safety of the highway user
b) User cost
c) Vertical and horizontal highway alignment
d) Construction and maintenance cost
e) Foundation condition
f) Availability
g) Traffic needs
h) Navigation requirements
i) Natural resources
j) Political considerations
k) Environmental considerations
l) Hazards from floods

Obviously, the most favorable hydraulic conditions cannot always be achieved when other considerations are in conflict. To illustrate, additional construction and maintenance costs are often accepted in order to achieve a satisfactory resolution of the conflict between other factors than a more desirable hydraulic condition. Other factors, however, should not be given such weight that the integrity of the hydraulic design and the safety of the bridge are compromised.

There are also national objectives which must be considered in site selection, some of these are:

a) Navigational clearance requirements must be met.
b) Highway stream crossings should be located and designed to aid in wise floodplain management of the nation's streams in efforts to achieve the objective of reducing flood losses.
c) The preservation of wetlands is also a national objective that must be considered in the selection of a stream crossing. The importance of wetlands is recognized because of their high productivity of food and fiber; beneficial effects of flooding, pollution, and sediment

control; and the wildlife habitat provided. Stream crossing locations must be selected so that important wetlands will not be destroyed or their value diminished unnecessarily.

Not all of the above will apply to each steam crossing or bridge location, but many of the more important site considerations are hydraulic or water-related. Location alternatives often do not include a desirable crossing site from the hydraulic design viewpoint, but the difficulties involved can often be reduced by giving attention to the hydraulic-related considerations.

7-202 ENVIRONMENTAL CONSIDERATIONS

The purpose of this chapter is not to include a definitive discussion of environmental considerations in site selection and bridge design. Because of many complex considerations, discussion will be general to environmental concerns that must be addressed in selection and design of a stream crossing.

The environmental effects of construction activities may be classified as hydrologic, physical, chemical, aesthetic, and biologic effects of water quality.

Hydrologic considerations, as used by ecologist, include both hydrology and hydraulics of the crossing. Hydrology at a crossing is unlikely to be a factor in site selection, but the hydraulics may be an overriding concern. The environmental considerations for hydraulic and physical aspects of water quality at alternate sites are the same concerns that the hydraulics engineer has historically addressed in evaluating the relative merits of alternate locations. These include the effects of the crossing on velocities, water surface profiles, velocity and flow distribution, scour, bank stability, sediment transport, aggradation, and degradation of the channel, and the supply of sediment to the stream or water body. The hydraulics engineer must evaluate the potential effect of these factors on the crossing as well as the potential physical effect of the crossing site on the environment.

Effects of a highway on the chemical quality of surface waters are not ordinarily a consideration in site selection, although it is possible that contaminants in the form of minerals or from a sanitary land fill would be exposed in one location and not at an alternate site. There is some concern for chemical quality at crossing sites, particularly near public water supply intakes, due to the risk of toxic material spills. The probability of such spills should be considered in weighing factors that influence site selection.

Aesthetic considerations include effects on the visual, odor and taste qualities of surface waters. The aesthetic quality of surface waters should be considered in site selection where potable water supplies, water contact sports, and fisheries are involved. The visual quality most often affected by highways is temporary turbidity during construction.

Biological considerations in site selection include the effects on habitat and ecosystems in the floodplain, and aquatic ecosystems in the stream and associated wetlands. It is advisable that biologists assess this aspect of site selection, but much of the information necessary for a valid assessment of the biological effects and the alternatives available for mitigation must come from the engineer. These include the economic viability of using a bridge rather than fill in wetland areas; the cost to replace lost marsh or wetland areas; circulation of fresh or brackish water in marshes and estuaries; the feasibility of providing mitigation measures for the loss of invertebate populations; and shade and nesting areas for fish.

7-203 COORDINATION WITH OTHER AGENCIES

Numerous local, State and Federal agencies have vested interests in surface waters. These agencies represent interests in water rights, flood control, drainage, conservation, navigation and maintenance of navigation channels, recreation, floodplain management and safety of floodplain occupancy, fish and wildlife, preservation of wetlands, and regulation of construction for the protection of environmental values. Other local, State and

Federal agencies have vested interests in historic bridge structures and archeological resources. Early coordination with other agencies will reveal areas of mutual interest and offer opportunities to conserve public funds by resolving conflicts between highway plans and water resources plans.

7-203.1 WATER RESOURCES AGENCIES

Water resources development projects often require the relocation or reconstruction of existing highways or interfere with the location or design of proposed highway stream crossings. It is necessary to coordinate with agencies responsible for proposed or existing water resources development projects in the planning and location phase of highway plan development. This allows for early agreement on cost proration for planned projects and the selection of optimal highway locations, considering water resources development projects.

Coordination with water resources agencies will, at times, provide opportunities to conserve public funds by each agency incorporating provisions in its plans to accommodate the needs of the other. Work which will be of mutual benefit can be undertaken by either the highway or water resources agency under an equitable cost sharing agreement and construction contract documents that meet the requirements of both agencies.

Many water resources development projects are planned or authorized for periods of years or even decades before construction begins. Others are never built and may even be deauthorized or permanently stopped by court decisions or regulatory agency actions. Where stream crossing locations are chosen to take advantage of or to accommodate planned water resources development projects, such as reservoirs or stream channel modifications, it should be recognized that the water resources agency plans may never be developed, and the highway facility must be designed for both existing and future site conditions.

7-203.2 PERMITS AND APPROVAL

Requirements for permits and approvals from local, State and Federal agencies having regulatory jurisdiction over streams should be considered early in the project plan development process in order to minimize later delays. Federal permits are required for construction in navigable waters for navigation clearances and the protection of water quality. Permits for bridges and causeways in waters that are navigable or have been used for commerce historically (except as provided in Section 124(a) of the Surface Transportation Assistance Act of 1978, PL 95-599) are under the jurisdiction of the Coast Guard, U. S. Department of Transportation (49 U.S.C. 1651 et seq.). Permits for other construction activities in navigable waters, including tunnels, are under the jurisdiction of the Corps of Engineers, U. S. Department of the Army (33 U.S. C. 403 et. seq.). The Corps of Engineers also has authority to issue permits for the discharge of dredged and fill material in all waters of the United States for the purpose of protecting water quality (33 U.S. C. 1344). The Act provides that States may undertake administration of this permit program. The courts have ruled that a public interest review may be made in evaluating applications for Federal permits. The National Environmental Policy Act of 1969 (42. U.S. C. 4321-437) requires an environmental assessment of every major Federal action, and some permit actions are considered major Federal actions.

Permits are discussed in more detail in Chapters 1 and 9.

7-300 BRIDGE SURVEYS

Complete and accurate survey information is necessary to design a crossing which will meet the requirements of the site. The individual in charge of the field drainage survey should have a general knowledge of drainage design and coordinate data collection with the hydraulics engineer. The amount of survey data collected and the detail of the data should be commensurate with the complexity of the hydraulics, stream stability problems, the importance and cost of the structure, and the risk of damage to the highway and of causing damage to other properties and values.

7-301 TOPOGRAPHIC FEATURES

The survey data collected should provide sufficient information for location, structural, and hydraulic engineers to select the location of the crossing, make trial layouts, and conduct foundation and hydraulic studies. All significant physical features and culture in the vicinity of the crossing site should be located, particularly those features which could be adversely affected. Features such as residences, commercial and industrial establishments, crop lands, wetlands, roadways, railroads, utilities, wells and other facilities can influence design and their locations and elevations should be established by the survey.

The extent of survey coverage required for the hydraulic design of the highway-stream crossing is related to topography and stream slope. Backwater above bridges may extend a considerable distance upstream in streams with relatively flat slopes and features which may be affected by backwater should be located and identified. For more detailed information on survey data required, Chapter 2 should be consulted.

7-302 HYDROLOGIC DATA

Data needs for hydrologic analysis are largely dependent upon the methods used to estimate flood flows. Information on flood flows, drainage basin characteristics, high water during past flood, flood history at existing structures, channel geometry, and precipitation are commonly needed hydrologic data. A more detailed and complete discussion of data needs for hydrologic analysis is contained in Chapter 3.

7-303 HIGHWATER INFORMATION

Reliable high water data can be invaluable information for establishing the stage and discharge of past floods, for locating existing hydraulic controls, and for establishing highway profiles. Several dependable highwater marks are required to compute flood discharge by the slope-area method.

It is extremely important that experienced personnel be used in identifying and evaluating high-water marks because the apparent quality of evidence of high-water can be deceiving to the uninitiated. High-water marks should be flagged and surveyed as soon as practicable after a flood because they may disappear within weeks in heavily vegetated areas. If an unusual flood has not occurred for several years, the high-water marks located by even experienced personnel are likely to represent a relatively small flood. High-water stages may be misleading since they are sometimes caused by ice, log jams, confluences, or land use which has subsequently changed. Such stages may be on the order of 5-10 feet above the normal stage for the same discharge. Examination of aerial photographs taken during the flood or more than one indirect measurement taken at reaches some distance apart can assist in identifying these stages as abnormal.

Information on high-water elevations can be obtained by observing seed and mud lines on tree trunks and bridge abutments, wash-lines and fine-debris lines on banks and bridge approach fills, whisps of grass or hay lodged in tree limbs and fences, and evidence of erosion and scour. Interviews with residents, commercial and school bus drivers, mail carriers, law enforcement officers, highway and railroad maintenance personnel, and others who might have opportunity to observe unusual floods will yield additional information. The date of the flood occurrence, the name and address of the observer and the stage and location of the observation should be recorded. The observed frequency of occurrence should be noted since reliable information that a stream reaches a certain elevation every 2 or 3 years provides important frequency information for the designer. A few hours spent in interviewing several people who are familiar with the flood history of a stream can result in substantial savings in construction, liability or future maintenance improvements in the design.

7-304 SITE PLAN

A general site plan should be prepared to show the physical features of the site. Although a contour map of a larger area is generally required for these purposes, a contour map of the site is helpful in defining flow directions, and in making decisions on span lengths, abutment and pier locations and orientation. The site plan should show the floodplain limits, stream banks and thalweg, vegetative cover, culture and land use, the location of surveyed cross sections, and the proposed roadway centerline, ramps, bridges, and channel modifications.

7-305 FIELD INSPECTION

A field inspection is mandatory in order for the engineer to become familiar with the site. The most complete survey data cannot adequately depict all site conditions, or substitute for personal inspection. A field review is also useful in confirming that additional site data is necessary. The selection of roughness coefficients, the evaluation of apparent flow directions and concentrations, and first-hand observation of land use and floodplain development are the factors that most often need to be confirmed by field inspection. Consultation with construction and maintenance employees regarding site conditions will often provide information regarding factors that they have found to be important to their responsibilities.

7-400 HYDROLOGIC ANALYSIS

For purposes of this chapter the hydrologic analysis consists of establishing peak flow frequency relationships for the crossing and such flow-duration hydrographs as may be necessary for the site analysis. Stage discharge relationships are a part of the hydraulic analysis and the hydraulic analysis for design includes an evaluation of the effects of the highway crossing for a range of flow rates.

It is customary to select an exceedance probability for design on the basis of highway system and traffic service requirements. The flood selected for design should also take into account economic, engineering, social, political, and environmental considerations. Because of uncertainties in the hydrologic analysis and the probability that the selected design flood will be exceeded, the hydrologic analysis should define peak flow frequency relationships for a wide range of events for use in the hydraulic analysis of the crossing.

Flood-frequency relationships are generally defined by highway agencies on the basis of a regional analysis, a gaging station analysis, or by consideration of both analyses. The U.S. Geological Survey has computed regional formulae for Arkansas by statistical analysis of gaging station records. Regional analyses is covered in more detail in Chapter 3. The results are generally applicable to watersheds which are unchanged by man, such as urbanization, channelization, and reservoir storage. Most of the analysis is based on regression analyses of factors which affect peak flows, such as watershed size, basin slopes, precipitation, ground cover, elevation, and other factors. Flood peaks can be estimated for various exceedance probabilities by substituting appropriate values in the resulting equations.

Flood-frequency relationships at gaged sites can be established from gaging station records which are of sufficient length to be representative of the total population of flood events on the particular stream. There are several methods of flow-frequency analysis generally considered acceptable. However, because of the great differences in results obtained by different analysts, the U.S. Water Resources Council has been charged with the responsibility of developing a uniform set of procedures for estimating flood potential. The Hydrology Committee, operating under the auspices of the Water Resouces Council, has published Bulletin #17B, "Guidelines for Determining Flood Flow Frequency", which "is the latest result of a continuing effort to develop a coherent set of procedures for accurately defining flood potential at gaged location." Bulletin #17B recommends the Pearson Type III distribution with log transformation of the flood data (log-Pearson III) as the basic distribution for defining the annual flood series. Recognizing that uncertainties in flood estimates exist for whatever method of analysis is employed, the methods of Bulletin #17B are recommended for analysis of gaging station records. Limitations on the applicability of the methods should be observed. The uncertainties of the estimate should be considered as an element of risk in the hydraulic analysis of crossings, just as the probability of exceedance should be considered.

Flood estimates made on the basis of a regional analysis will frequently not be in agreement with estimates made on the basis of gaging station records. Various factors such as length of gaging station records, storm distribution, and spatial variations in the occurrence of storm events may account for some discrepancies in the estimates. If the stream record is sufficiently long to give a good flood-frequency relationship, considerable weight should be given to the estimate made from the gaging station record. An examination of the factors that proved significant and insignificant in the regional flood-frequency analysis may reveal that the stream under consideration is not typical of the hydrologic region. In that event, the station analysis from even a short period of record should be given more weight, particularly where the station analysis indicates that the regional analysis underestimates floods for the stream.

High-water information, miscellaneous flood data, and information collected in the surveys at existing structures should be used in the hydrologic analyis to verify or improve confidence in the flood-frequency analysis. These miscellaneous data are particularly useful if flood estimates are based on a regional analysis of flood flows.

A hydrograph analysis is another useful tool that can be used when flood durations are required. For example, a hydrograph would be useful for the design of a low water crossing, for roadway overtopping design or for estimating the height of coffer-dams.

7-500 HYDRAULIC ANALYSIS OF THE STREAM

An analysis of the hydraulics of a stream involves a thorough study of the relationship between stage, or more precisely, water surface elevation, and discharge, and a study of flow directions, flow distribution and velocity distributions over a wide range of flow rates. The hydraulics of the stream with the highway crossing superimposed on the natural stream is discussed in this section.

7-501 ANALYSIS OF THE STREAM CROSSING SYSTEM

Analysis of the stream crossing system involves the use of hydraulic engineering principles, and techniques in engineering economics to select an alternative design which will provide traffic service required at a practicable minimum cost in public funds for construction and probable future costs. This includes appropriate consideration of criteria established for protection of the stream environment, as well as social, political, and other economic and engineering concerns.

The hydraulic analysis of a highway-stream crossing system involves determining the backwater associated with each alternative profile and waterway opening(s), the effects of flow distribution and velocities, and estimating the scour potential.

The hydraulic analyses must be of the total crossing facility including all roadway items, all waterway openings, and the environmental implication and risks associated with each alternative.

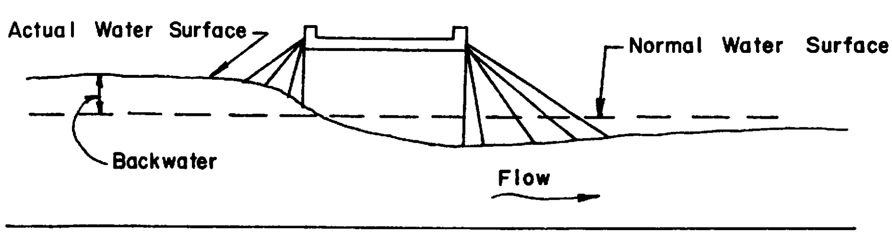
7-502 HYDRAULIC PERFORMANCE OF THE CROSSING SYSTEM

The hydraulic performance and backwater at various stream stages are the first measures generally used to judge the acceptability of an alternative highway-stream crossing system design.

The introduction of a constriction in a stream channel with supercritical flow conditions, such as a highway crossing, will not cause backwater above the constriction. Constriction results in the conversion of potential energy to kinetic energy and, therefore, there is no rise to the water surface elevation upstream of the introduced constriction.

A highway facility which constricts a stream with subcritical flow conditions will cause higher water surface elevations upstream of the crossing than prevailed during floods prior to construction of the highway facility. The higher water surface elevations represent the amount of kinetic energy converted to potential energy to overcome losses comprised principally of contraction and expansion losses. Other losses include those from piers, abutments, eccentricity, superstructures (if submerged) and friction from longer flow paths. Friction losses can be significant if the floodplain constriction is relatively severe and the resistance to flow in the floodplain is high, as in wooded areas (Reference 2).

The increase in water surface elevation upstream of the highway facility is referred to as backwater. It is measured upstream of the waterway opening above the theoretical water surface elevation (Figure 7-1) prior to construction of the crossing. Backwater above a highway crossing will cause incremental depth and duration of flooding and an increased area of inundation for a given flood magnitude. The incremental flooding associated with various flood magnitudes should be considered in evaluating the risks associated with alternative designs of the stream crossing system.



SECTION THROUGH BRIDGE

FIGURE 7-1

7-503 BACKWATER

Backwater or the increment of increased flood depth upstream of a highway crossing over a stream, is often used as the sole criteria for judging the adequacy of an alternative waterway opening. Many county and municipal governments within the State established maximum limits for increases in flood elevations without regard to risks which might be created at specific sites within the community. The National Flood Insurance Program administered by the Federal Emergency Management Agency is responsible for many jurisdictions adopting such limits. Since these limits are not site specific in many cases, they do not prevent the adoption of lesser increases in flood heights caused by backwater where there is extreme risk of property damage. Limits to increases of the base (100-year) flood vary from zero to one foot.

Backwater should not be used as the sole criterion for judging the acceptability of an alternative design. It is, instead, an aid that can be used in selecting the waterway opening, the crossing profile, and to assess the risk costs of incremental flooding caused by the crossing facility.

Backwater associated with each profile and opening alternative can be computed by methods in References 1 and 2. Overflow over the roadway can be computed using methods in References 1 and 3.

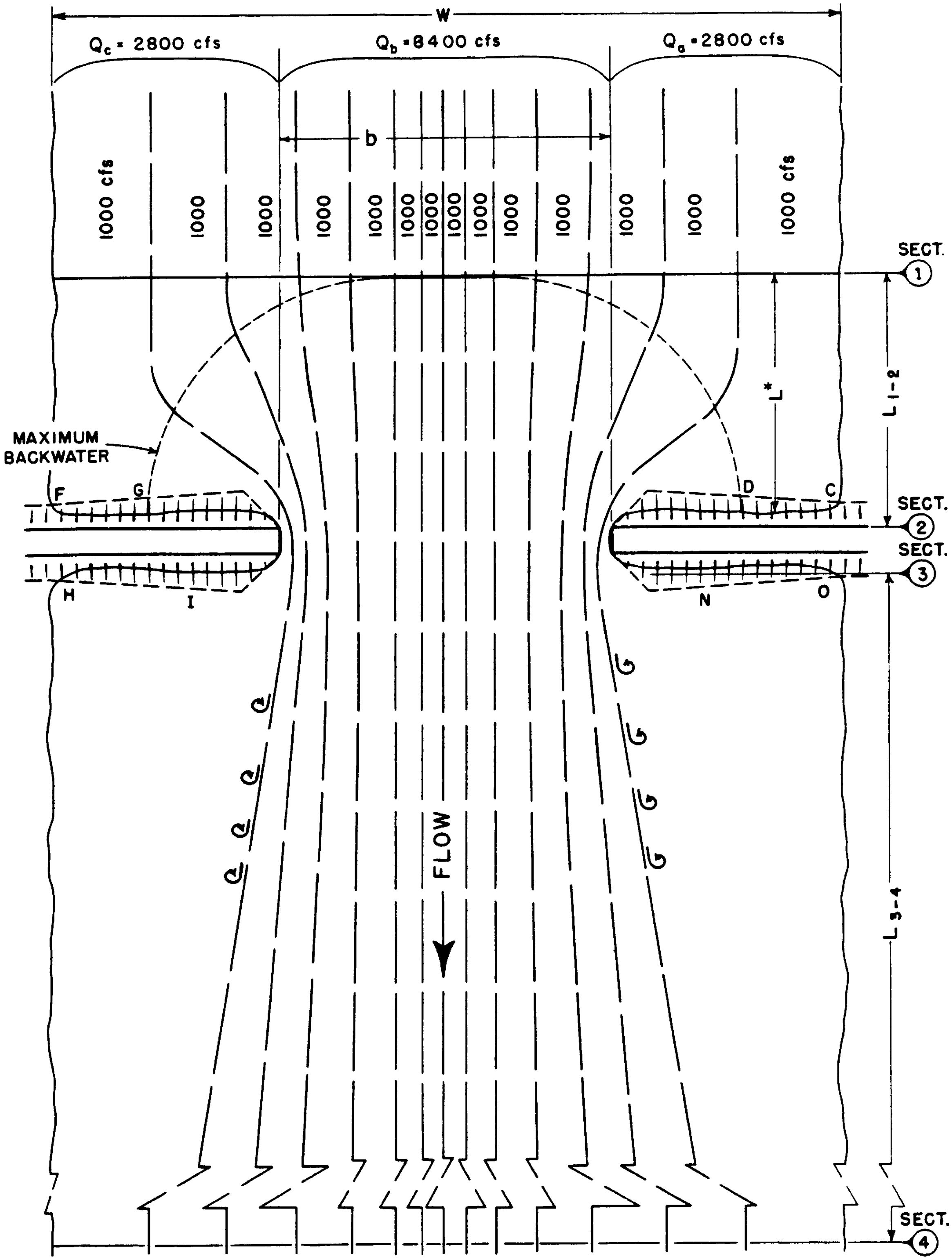
The following section will go into a detailed explanation of backwater.

7-504 EXPRESSION OF BACKWATER

The reader is referred to a publication by the Federal Highway Administration entitled "Hydraulics of Bridge Waterways" (Reference 1). The expression, figures, equations and symbols in this section are from Reference 1.

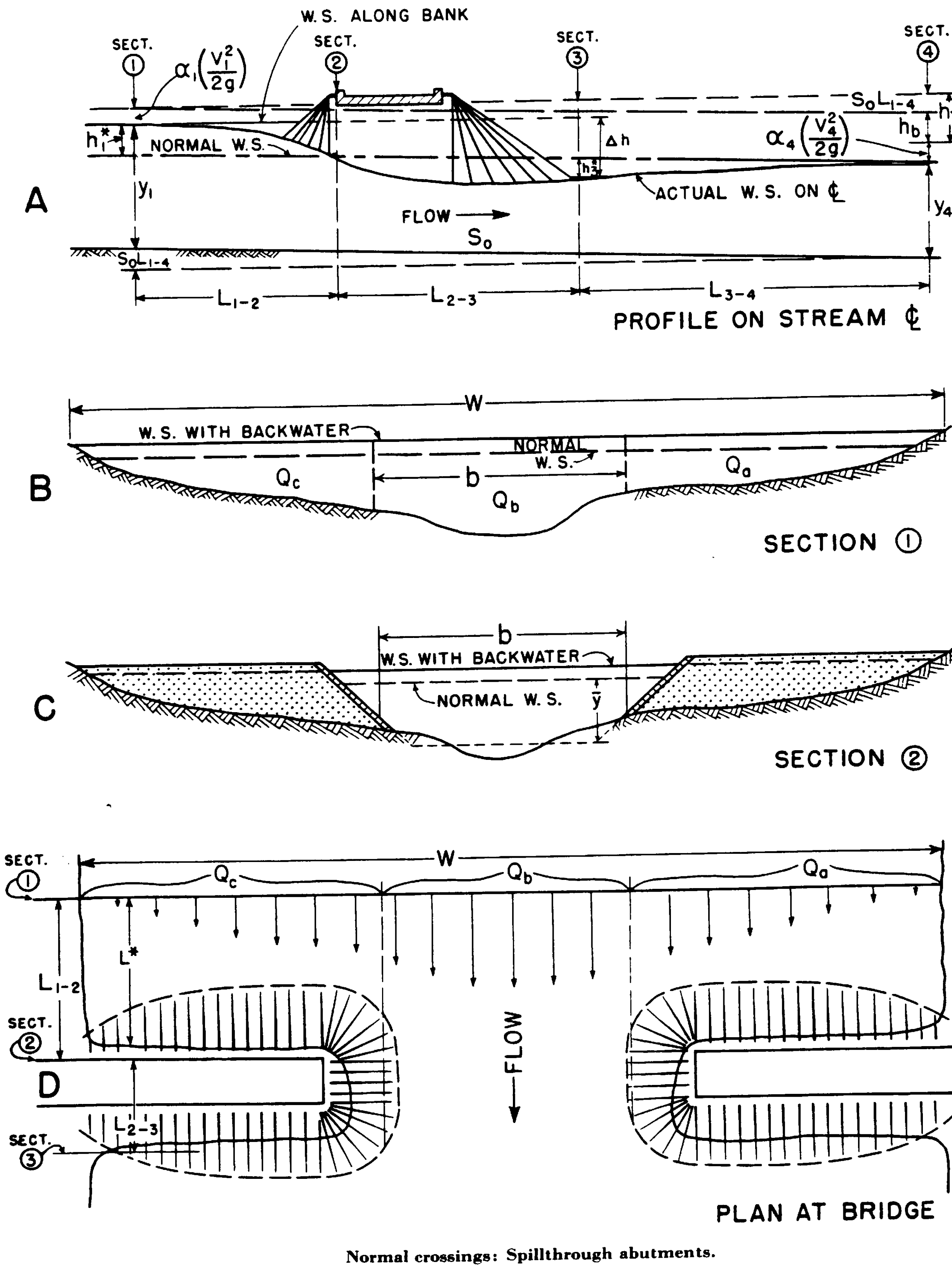
The manner in which flow is contracted in passing through a channel constriction is illustrated in Figure 7-2, page 7-19. The flow bounded by each adjacent pair of streamlines is the same (1,000 cfs). Note that the channel constriction appears to produce practically no alteration in the shape of the streamline near the center of the channel. A very marked change is evidenced near the abutments, however, since the momentum of the flow from both sides (or floodplains) must force the advancing central portion of the stream over to gain entry to the constriction. Upon leaving the constriction the flow gradually expands (5 to 6 degrees per side) until normal conditions in the stream are reestablished.

Constriction of the flow causes a loss of energy, the greater portion occurring in the reexpansion downstream. This loss of energy is reflected in a rise in the water surface and in the energy line upstream from the bridge. This is best illustrated by a profile along the center of the stream, as shown in Figure 7-3, page 7-20. The normal stage of the stream for a given discharge, before constricting the channel, is represented by the dashed line labeled "normal water surface". (Water surface is abbreviated as "W.S." in the figures.) The nature of the water surface after constriction of the channel is represented by the solid line, "actual water surface." Note that the water surface starts out above normal stage at Section 1, passes through normal stage close to Section 2, reaches minimum depth in the vicinity of Section 3, and then returns to normal stage a considerable distance downstream at Section 4. Determination of the rise in water surface at Section 1, denoted by the symbol h_1^* and referred to as bridge backwater, is the primary objective of this section. Attention is called to a common misunderstanding that the drop in water surface across the embankment, Δh, is the backwater caused by a bridge. This is not correct as an inspection of Figure 7-3, page 7-20 will show. The backwater is represented by the symbol h_1^* on the figure and is always less than Δh.



Flow lines for typical normal crossing.

Figure 7-2



Normal crossings: Spillthrough abutments.

Figure 7-3

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, Section 1, and a point downstream from the bridge at which normal stage has been reestablished, Section 4 (Figure 7-3A, page 7-20. The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constricting the flow, is as follows:

$$h_1^* = K^* a_2 \frac{V_{n2}^2}{2g} + a_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad \ldots\ (7\text{-}1)$$

Where

h_1^* = total backwater (ft.)

K^* = total backwater coefficient.

a_1 & a_2 = Kinetic energy coefficient as defined in Equations 7-2a and 7-2b, page 22

A_{n2} = gross water area in constriction measured below normal stage (sq. ft.).

V_{n2} = average velocity in constriction or Q/A_{n2} (f.p.s.).

A_4 = water area at Section 4 where normal stage is reestablished (sq. ft.).

A_1 = total water area at Section 1, including that produced by the backwater (sq. ft.).

As the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A_1)^2/2g$ for the stream at Section 1, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, a_1, defined as:

$$a_1 = \frac{\Sigma(qv^2)}{QV_1^2} \quad \ldots\ldots (7\text{-}2a)$$

Where

v = average velocity in a subsection.

q = discharge in same subsection.

Q = total discharge in river.

V_1 = average velocity in river at Section 1 or Q/A_1.

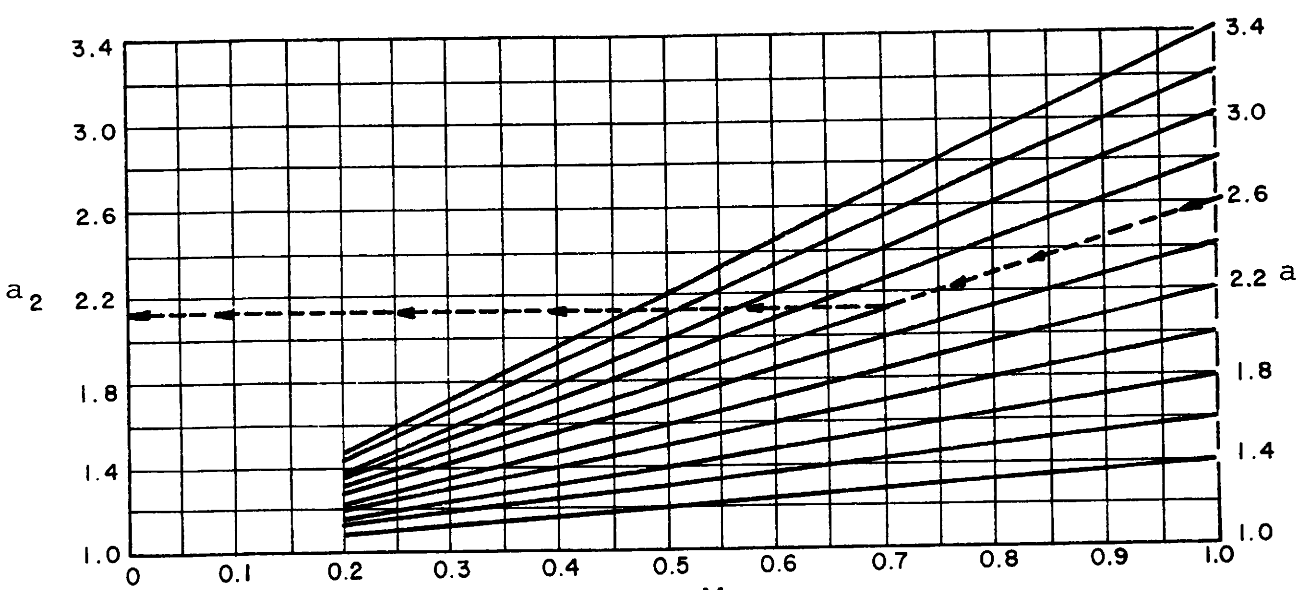
A second coefficient, a_2, is required to correct the velocity head for nonuniform velocity distribution under the bridge,

$$a_2 = \frac{\Sigma(qv^2)}{QV_2^2} \quad \ldots\ldots (7\text{-}2b)$$

where v, q and Q are defined as above but apply here to the constricted cross section and

V_2 = average velocity in constriction = Q/A_2.

Figure 7-4 can be used for estimating a_2.



AID FOR ESTIMATING a_2

Figure 7-4

To compute backwater, it is necessary to obtain the approximate value of h_1^* by using the first part of Equation 7-1 , page 7-21

$$h_1^* = K^* a_2 \frac{V_{n2}^2}{2g} \cdots\cdots\cdots\cdots (7\text{-}3a)$$

The value of A_1 in the second part of Equation 7-1, which depends on h_1^*, can then be determined and the second term of the expression evaluated:

$$a_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \cdots\cdots (7\text{-}3b)$$

This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head, $V_{n2}^2/2g$. Equation 7-1 may appear cumbersome, but this is not the case. It was set up as shown to permit ommission of the second part when the difference in kinetic energy between Section 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$$M > 0.7$$

$$V_{n2} < 7 \text{ f.p.s}$$

$$V_{n2}^2 < 0.5 \text{ foot}$$

If values in the problem at hand meet all three conditions, the backwater obtained from Equation 7-3a can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use Equation 7-1 in its entirety.

The bridge opening ratio, M, defines the degree of stream constriction involved, expressed as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river. Referring to Figure 7-2, page 7-19,

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \ldots (7\text{-}4)$$

or,

$$M = \frac{8,400}{14,000} = 0.60.$$

The irregular cross section common in natural streams and the variation in boundary roughness within any cross section result in a variation in velocity across a river as indicated by the stream tubes in Figure 7-2, page 7-19. The bridge opening ratio, M, is most easily explained in terms of discharges, but it is usually determined from conveyance relations. Since conveyance is proportional to discharge, assuming all subsections to have the same slope, M can be expressed also as:

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K_1} \ldots (7\text{-}5)$$

7-504.1 BACKWATER COEFFICIENT

The value of the overall backwater coefficient K*, which was determined experimentally, varies with:

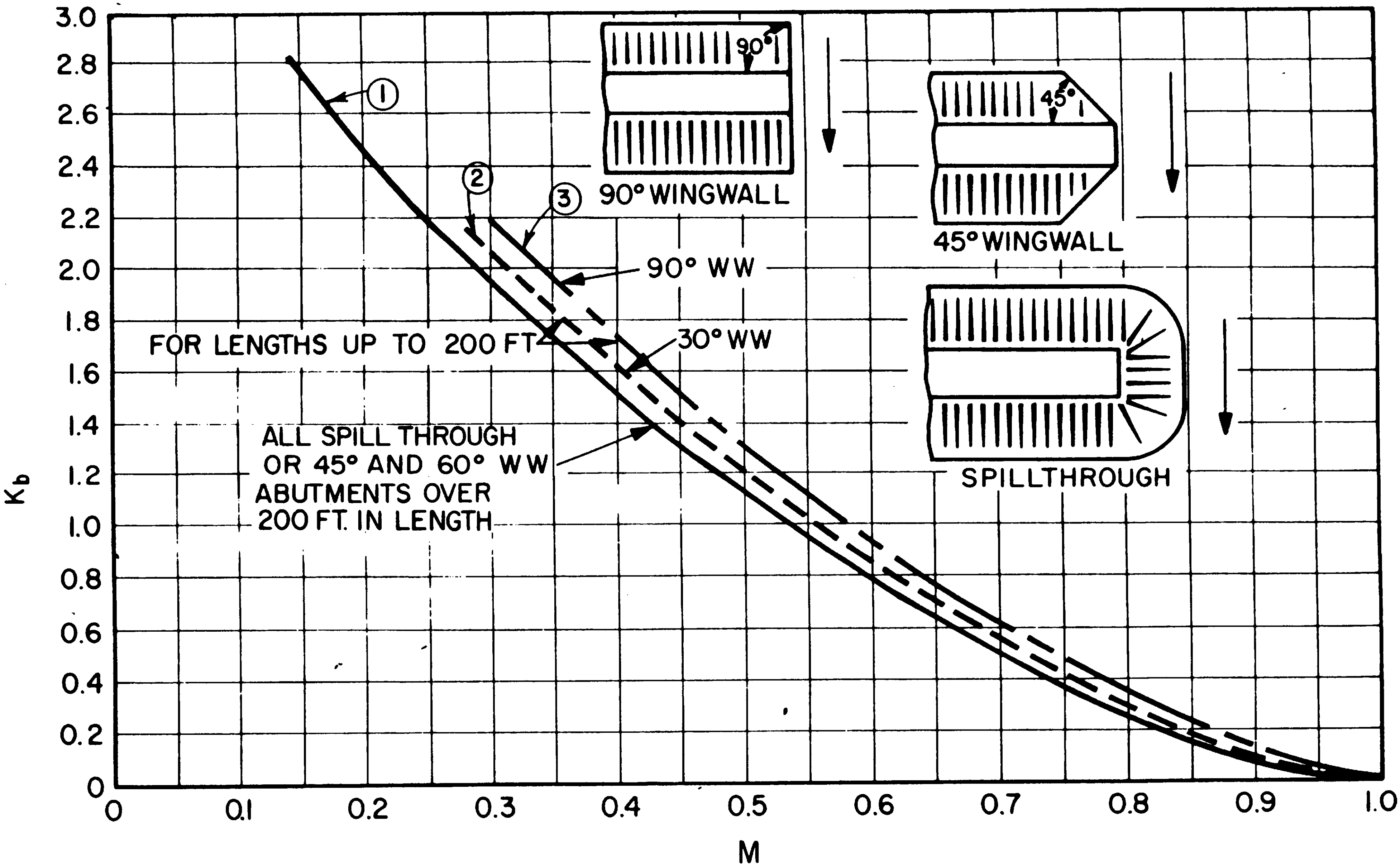
1. Stream constriction as measure by bridge opening ratio M;
2. Type of bridge abutment --wingwall, spillthrough, etc.;
3. Number, size, shape, and orientation of piers in the constriction;
4. Eccentricity, or asymmetric position of bridge with the floodplains; and
5. Skew (bridge crosses floodplain at other then 90° angle).

The overall backwater coefficient K* consists of a base curve coefficient K_b, to which is added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

Figure 7-5, page 7-26, shows the effect of M and abutment shape on the base curves for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for wingwall and spillthrough abutments. Note how the coefficient, K_b, increases with channel constriction. The lower curve applies for 45^o and 60^o wingwall abutments and all spillthrough types. Curves are also included for 30^o wingwall abutments and for 90^o vertical wall abutments for bridges up to 200 feet in length. These shapes can be identified from the sketches on Figure 7-5, page 7-26. For bridges exceeding 200 feet in length, regardless of abutment type, the lower curve is recommended. This is because abutment geometry becomes less important to backwater as a bridge is lengthened. The base curve coefficients of Figure 7-5, page 7-26 apply to crossings normal to flood flow and do not include the effect produced by piers, eccentricity and skew.

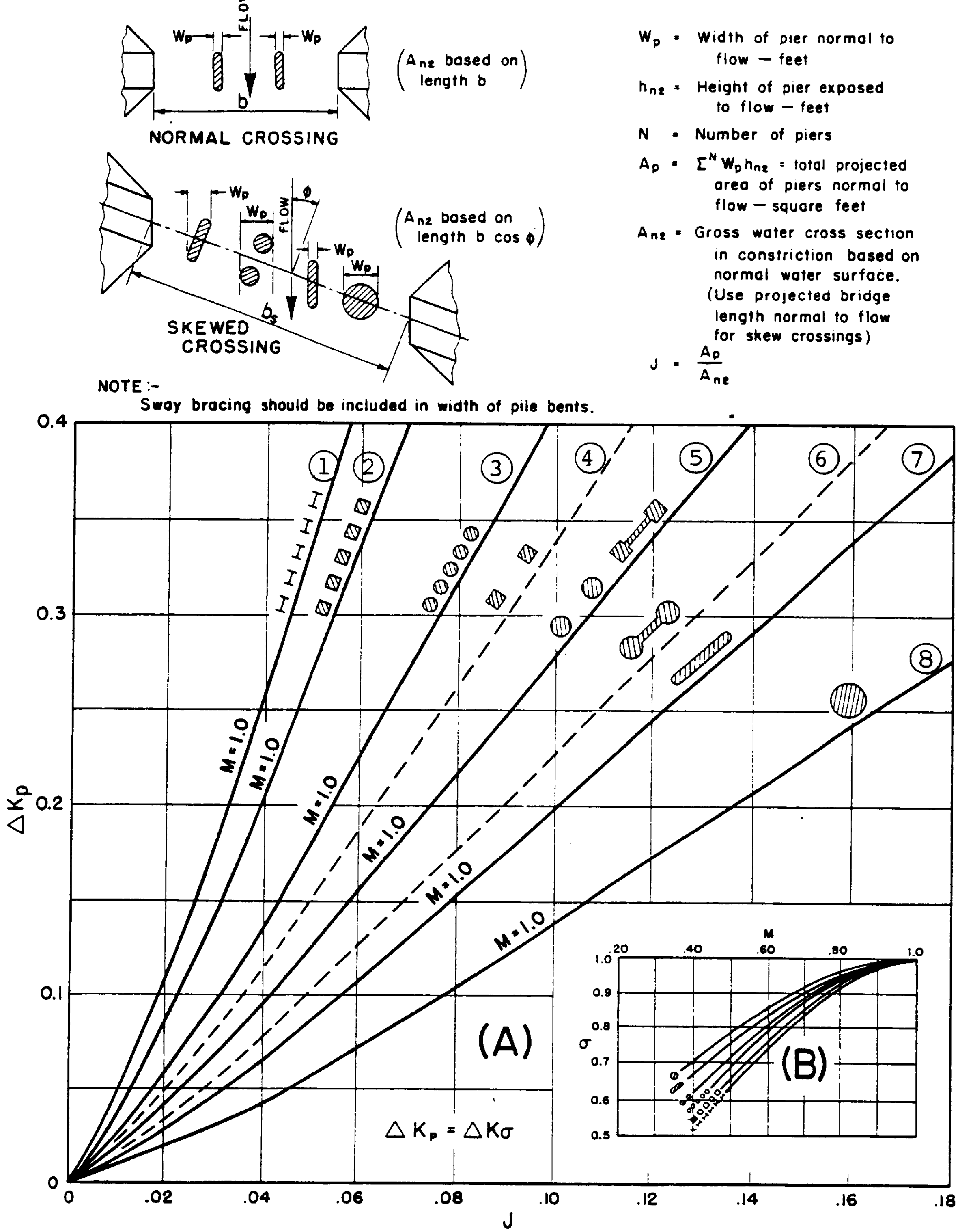
The effect of backwater caused by introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient K_b when piers are present in the waterway. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure 7-6, page 7-27. By entering Chart A, Figure 7-6, page 7-27, with the proper value of J and reading upward to the proper pier type, ΔK_p is read from the ordinate. Obtain the correction factor, σ, from Chart B, Figure 7-6, page 7-27, for opening ratios other then unity. The incremental backwater coefficient is then:

$$\Delta K_p = \sigma \Delta K$$



Backwater coefficient base curves (subcritical flow).

Figure 7-5



Incremental backwater coefficient for piers.

Figure 7-6

The incremental backwater coefficients for pile bents can, for all practical purposes, be considered independent of diameter, width, or spacing of piles but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20 percent higher than that shown for bents with 5 piles. If there is a possibility of trash collecting on the piers, or piles, it is advisable to use a larger value of J to compensate for the added obstruction. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figure 7-5)} + \Delta K_p \text{ (Figure 7-6)}$$

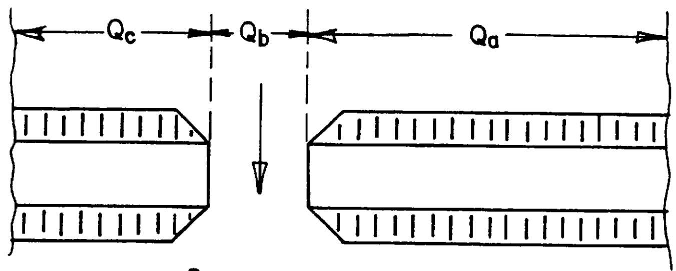
For the effect of eccentricity on backwater reference is made to Figure 7-7, page 7-29. It can be noted that the symbols Q_a and Q_c at Section 1 were used to represent the portion of the discharge obstructed by the approach embankments. If the cross section is extremely asymmetrical so that Q_a is less than 20 percent of Q_c or vice versa, the backwater coefficient will be somewhat larger than for comparable values of M shown on the base curve. The magnitude of the incremental backwater coefficient, ΔK_e, accounting for the effect of eccentricity, is shown in Figure 7-7, page 7-29. Eccentricity, e, is defined as 1 minus the ratio of the lesser to the greater discharge outside the projected length of the bridge, or:

$$e = 1 - \frac{Q_c}{Q_a} \quad \text{where } Q_c < Q_a$$

or:

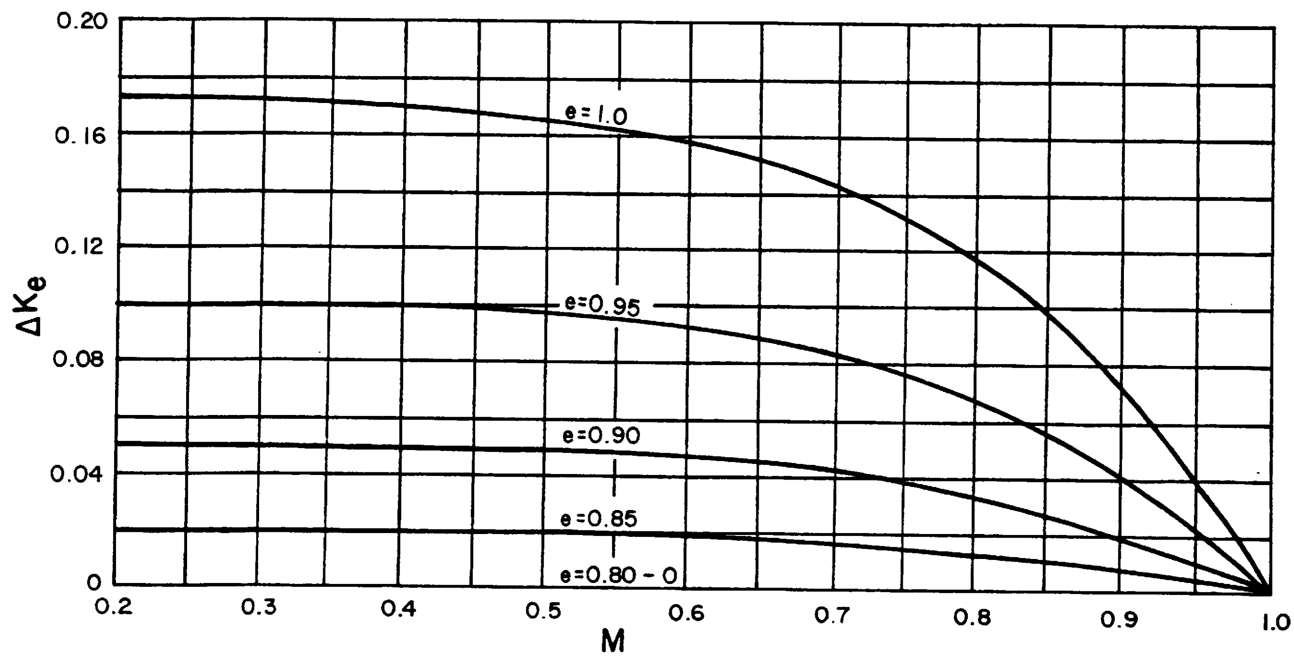
$$e = 1 - \frac{Q_a}{Q_c} \quad \text{where } Q_c > Q_a.$$

Reference to the sketch in Figure 7-7, page 7-29, will aid in clarifying the terminology. For instance, if $Q_c/Q_a = 0.05$, the eccentricty $e = (1 - 0.05)$ or 0.95 and the curve for $e = 0.95$ in Figure 7-7, page 7-29, would be used for obtaining ΔK_e. The largest influence on the backwater coefficient due to eccentricity will occur when a bridge is located adajcent to a bluff where a floodplain exists only on one side and the eccentricity



$$e = \left(1 - \frac{Q_c}{Q_a}\right) \quad \text{where} \quad Q_c < Q_a \quad \text{or}$$

$$e = \left(1 - \frac{Q_a}{Q_c}\right) \quad \text{where} \quad Q_a < Q_c$$



INCREMENTAL BACKWATER COEFFICIENT FOR ECCENTRICITY
Figure 7-7

is 1.0. The overall backwater coefficient for an extremely eccentric crossing with wingwall or spillthrough abutments and piers will be:

$K^* = K_b$ (Figure 7-5) $+ \Delta K_p$ (Figure 7-6) $+ \Delta K_e$ (Figure 7-7).

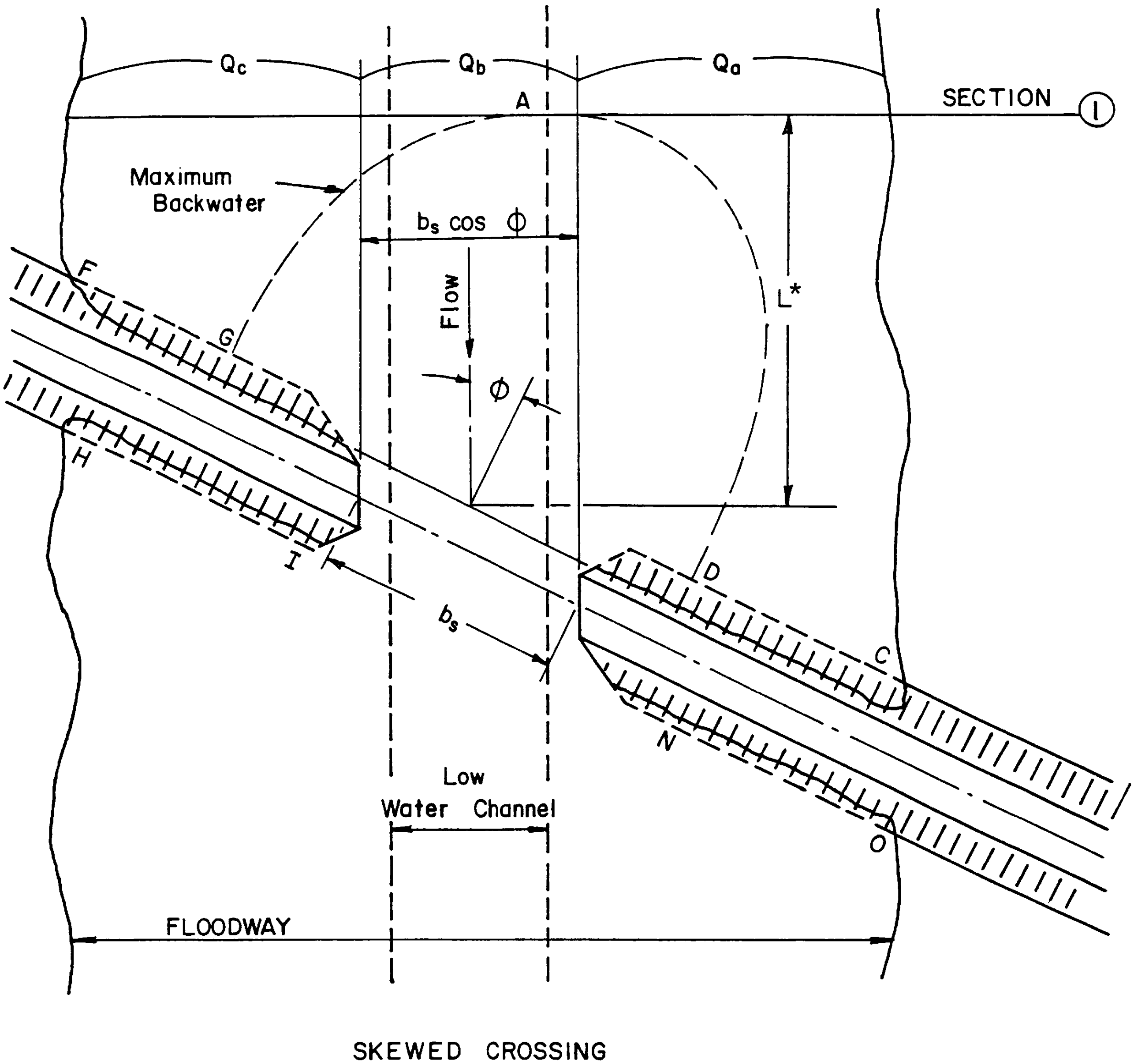
The method of computation for skewed crossings differs from that of normal crossings in the following respects: the bridge opening ratio, M, is computed on the projected length of bridge rather than on the length along the centerline. The length is obtained by projecting the bridge opening upstream parallel to the general direction of flood flow as illustrated in Figure 7-8, page 7-31. The general direction of flow means the direction of flood flow as it existed previous to the placement of embankments in the stream. The length of the constricted opening is $b_s \cos\phi$, and the area A_{n2} is based on this length. The velocity head, $\frac{V_{n2}^2}{2g}$, to be substituted in Equation 7-3a, page 7-23 is based on the projected area A_{n2}. The method is further illustrated in Example 3, Chapter XII (Reference 1). The total backwater coefficient for a skewed crossing with the abutment faces aligned with the flow and piers would be.

$K^* = K_b$ (Figure 7-5) $+ \Delta K_p$ (Figure 7-6) $+ \Delta K_s$ (Figure 7-9A).

Figure 7-10, page 7-33, was prepared from the same model information as Figure 7-9A, page 7-32. By entering Figure 7-10, page 7-33, with the angle of skew and the projected value of M, the ratio, $b_s \cos\phi/b$ can be read from the ordinate. Knowing b and h_1^* for a comparable normal crossing, one can solve for b_s, the length of opening needed for a skewed bridge to produce the same amount of backwater for the design discharge. The chart is especially helpful for estimating and checking and its use is demonstrated in Example 3, Chapter XII (Reference 1).

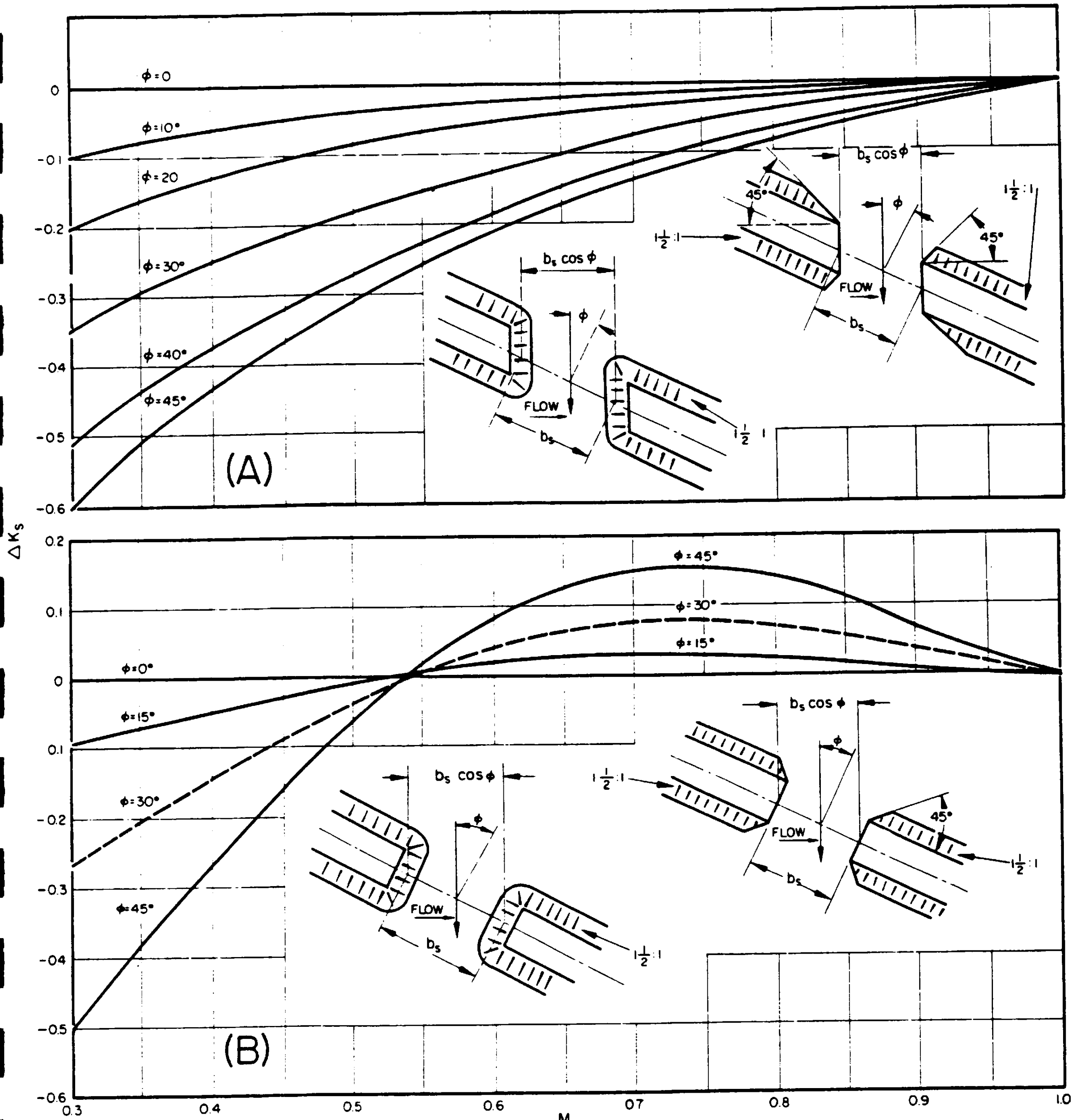
7-504.2 <u>EFFECT OF SCOUR ON BACKWATER</u>

Thus far the discussion of backwater has been limited to the case where the bed of a stream in the vicinity of a bridge is considered rigid or immovable and, thus, does not degrade with



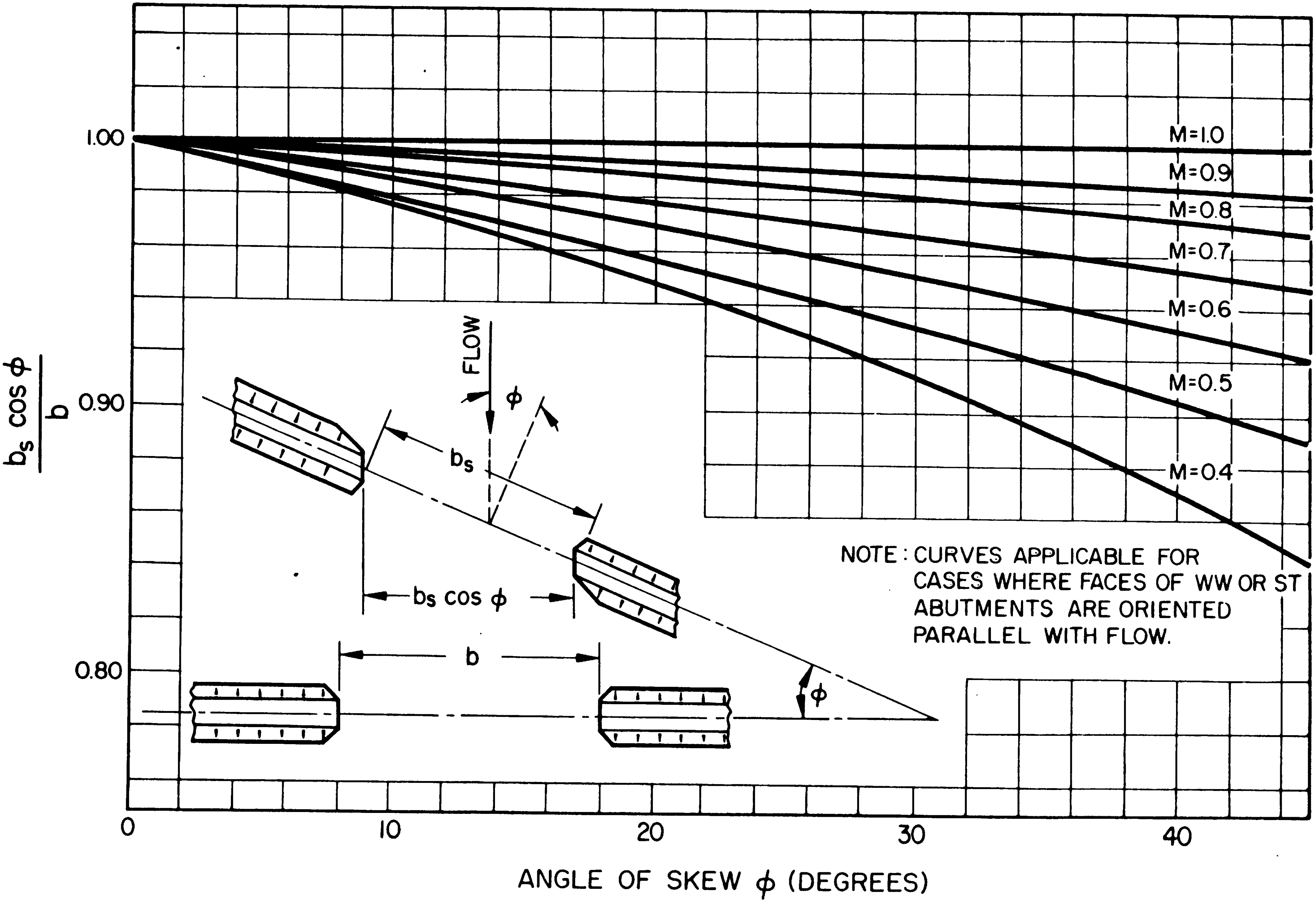
SKEWED CROSSING

Figure 7-8



Incremental backwater coefficient for skew.

Figure 7-9



$\frac{b_s \cos\phi}{b}$
1.00
0.90
0.80
M=1.0
M=0.9
M=0.8
M=0.7
M=0.6
M=0.5
M=0.4
FLOW
φ
b_s
$b_s \cos\phi$
b
NOTE: CURVES APPLICABLE FOR CASES WHERE FACES OF WW OR ST ABUTMENTS ARE ORIENTED PARALLEL WITH FLOW.
0
10
20
30
40
ANGLE OF SKEW φ (DEGREES)

Figure 7-10

introduction of embankments, abutments, and piers. In actuality the bed is usually composed of much loose material, some of which will move out of the constiction during flood flows. Nature wastes little time in attempting to restore the former regime, or the stage-discharge relation which existed prior to constriction of the stream. For within-bank flow little changes, but for flood flows there exists an altered regime, with a potential to enlarge the waterway area under the bridge.

Bearing in mind that during floods a stream is usually transporting sediment at its capacity, the process might be described as follows. Constriction of a stream produces backwater at flood flows; backwater is indicative of an increase in potential energy upstream. This makes possible higher velocities in the constriction, thus, increasing the transport capacity of the flow to above normal in this reach. The greater capacity for transport results in scoring of the bed in the vicinity of the constriction; the removed material is usually carried a short distance downstream and dropped as the stream again returns to full width. As the scouring action proceeds, the waterway area under the bridge enlarges, the velocity and resistance to flow decreases, and a reduction in the amount of backwater results.

In cases where abutments and piers can be keyed into bedrock, it may be advisable to encourage scour in the interest of utilizing a shorter bridge. This same objective is sometimes attained in another way by enlarging the waterway area under a bridge by excavation during construction. In such cases, it is desirable to be able to determine the amount of backwater to be expected after localized enlargement of the waterway.

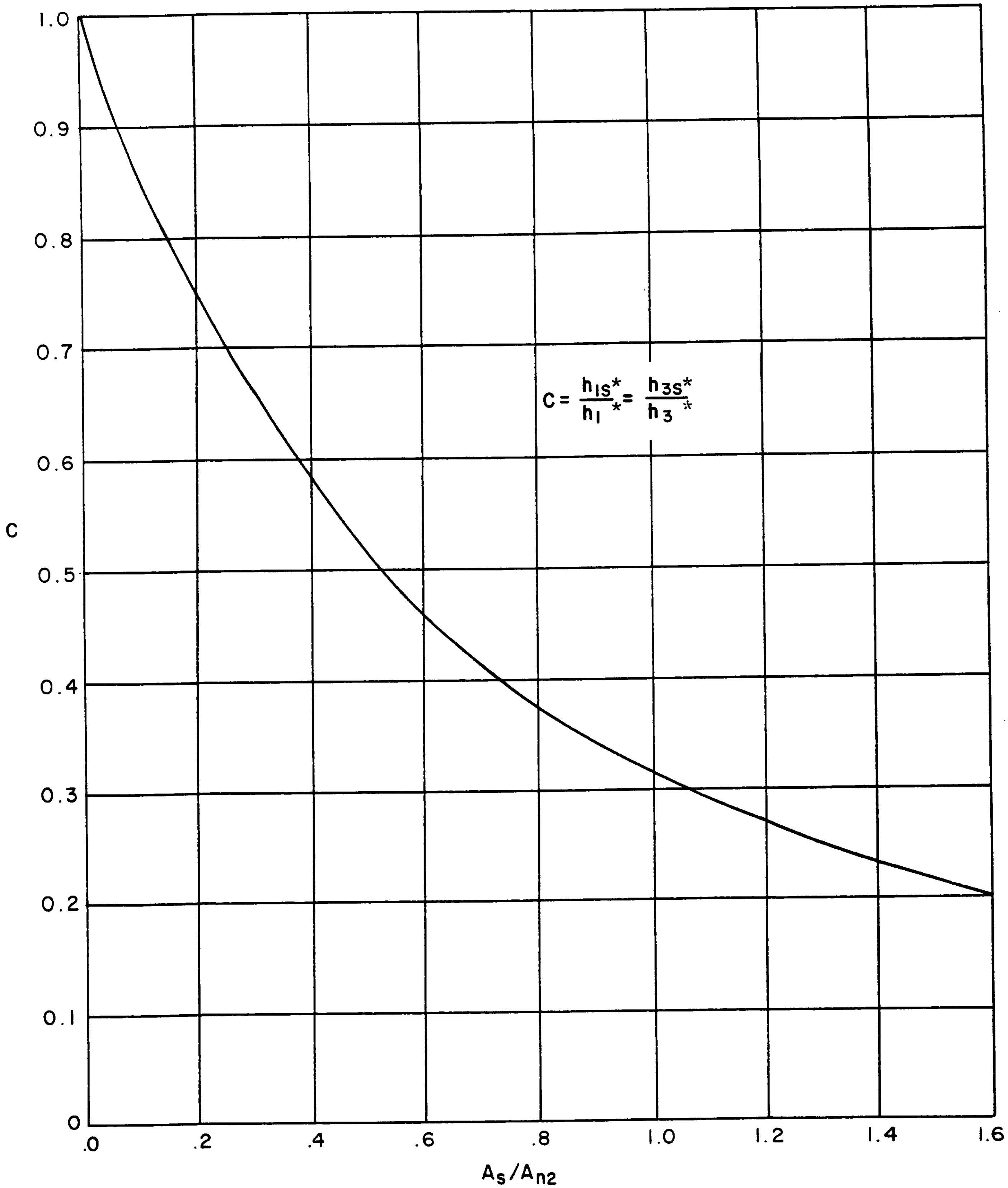
Any proposal to reduce backwater by excavation to enlarge a bridge waterway area should be carefully studied. If there is reason to believe that the enlarged area cannot be maintained or that the stability of the stream will be disturbed, alternate solutions such as additional spans should be considered.

From the backwater equations it can be seen that any means of increasing the waterway area under a bridge can be effective in reducing the backwater. A study completed by the FHWA derived a design curve, Figure 7-11, page 7-36, for the reduction of backwater with scour. The correction factor for backwater with scour ($c = h_{1s}^* = h_1^*$) is plotted with respect to A_s/A_{n2}, where the terms bearing the subscript s, designate values with scour; those not bearing this subscript represent the same values computed with a rigid bed. Supposing the backwater at a given bridge was 1 foot with no scour; it would be reduced to 0.52-foot were scour to enlarge the waterway area by 50 percent, or it would be reduced to 0.31-foot should the waterway area be doubled. Thus to obtain backwater and related information for bridge sites where scour is to be encouraged, where scour cannot be avoided, or where the waterway is to be enlarged during construction, it is first necessary to compute the backwater and other quantities desired according to the method outlined previously for a rigid bed, using the original cross section of the stream at the bridge site. These values are then multiplied by a common coefficient from Figure 7-11, page 7-36 as follows:

$$h_{1s}^* = Ch^*_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7\text{-}6)$$

$$h^*_{3a} = Ch_3^* \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7\text{-}7)$$

The engineer will probably be reluctant to depend on scour as a means of enlarging a waterway and thereby reducing backwater. If the waterway is enlarged by excavation, there is little to gain by excavating much beyond the limits (upstream or down stream) of the embankment as the downstream channel acts as the control. If additional volume is removed upstream or downstream, the channel may simply refill by deposition. Any enlargement of the cross section should be maintained to prevent reduction of area by the growth of willows and similar vegetation. Field surveys of existing bridges where channel enlargements have once been made should reveal worthwhile information on the question of permanence of enlarged waterways. Example 6, Chapter XII (Reference 1), which is based on an actual occurrence involving a flash flood on a stream with a bed consisting of noncohesive material, demonstrates how backwater is reduced by scour.



CORRECTION FACTOR FOR BACKWATER WITH SCOUR
Figure 7-11

7-504.3 OTHER CONDITIONS EFFECTING BACKWATER

(A) DUAL BRIDGES

With the advent of divided highways, dual bridges of essentially identical design, placed parallel and only a short distance apart, are now common. The backwater produced by dual bridges is naturally larger than that for a single bridge, yet less than the value which would result by considering the two bridges separately. The procedure is explained in Chapter V of (Reference 1), with an example included in Chapter XII of (Reference 1).

(B) ABNORMAL STAGE-DISCHARGE CONDITIONS

Sometimes the stage at a bridge site is not normal but is increased by unnatural backwater conditions from downstream. A general backwater curve may be produced, beginning at the confluence of tributary and main stream or at a dam, and may extend a considerable distance upstream if the stream gradient is flat. Where bridges are placed close to the confluence of two streams, abnormally high stage-discharge conditions can be of importance in design. For example, if a stream can always be counted on to flow at abnormally high stage during floods at a particular bridge site, the increased waterway area may permit a shorter bridge than would be possible under normal-stage conditions. To take advantage of the situation, the length of bridge would be determined on the basis of (1) the minimum abnormal stage expected which would produce the largest backwater increment, or (2) the maximum expected abnormal stage which may produce the highest stage upstream. Most problems in highway drainage do not require the accurate computation of water surface profiles; however, the engineer should know that the depth in a given channel may be influenced by conditions either upstream or downstream. Estimating the design stage at a bridge site under abnormal conditions can be a complicated process, requiring much individual judgment, as well as extensive survey data, and the use of the USGS E431, or the Corps of Engineers HEC-2 computer programs.

The approach to the computation of backwater in this case has been treated strictly as an approximate solution or a case where it is more important to understand the problem than to attempt precise computations.

The experimental backwater coefficients for abnormal stage discharge (without piers, eccentricity, and skew) were computed according to the expression:

$$K_{bA} = \frac{h^*_{1A}}{a_2 V^2_{2A}/2g} \;....(7\text{-}8)$$

where h^*_{1A} is backwater measured above abnormal stage at section 1 and $V_{2A} = Q/A_{2A}$, where A_{2A} is gross area of constriction based on abnormal stage, see Figure 7-12, page 7-39. The subscript A has been added throughout to signify that this is a special case, not to be confused with other expressions which precede or follow. Actually, Equation 7-8 is a modification of Equation 7-3a, page 7-23.

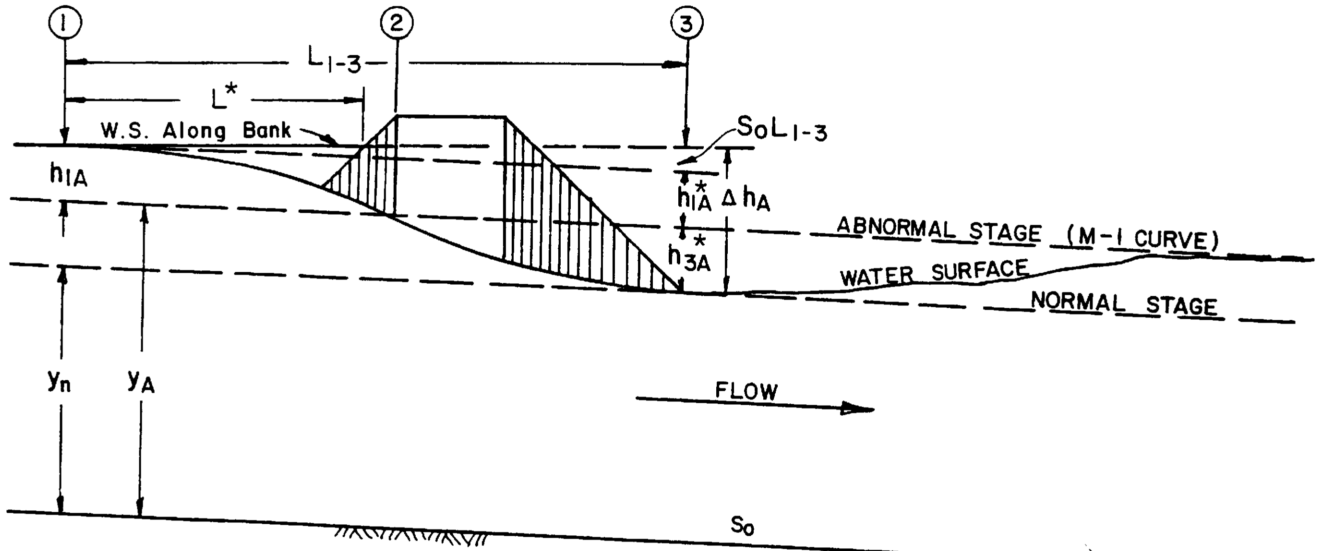
As the method of computation chosen results in backwater coefficients approximating those of the base curves, it is further assumed that the curves for incremental backwater coefficients, previously established for piers, eccentricity and skew, may be reasonably applicable to abnormal stage-discharge conditions. If this is permissible, the expression for the computation of backwater for abnormal stage discharge would then read:

$$h^*_{1A} = K^* a_2 \frac{V^2_{2A}}{2g} \qquad (7\text{-}9)$$

where $K^* = K_b$ (Figure 7-5) $+ \Delta K_p$ (Figure 7-6) $+ \Delta K_e$ (Figure 7-7) $+ \Delta K_s$ (Figure 7-9). Thus, the method and sources used to obtain the overall backwater coefficient remain unchanged. The one important difference in Equations 7-8 and 7-9 is insertion of the velocity head for abnormal stage rather than normal stage.

(C) SUPERSTRUCTURE PARTIALLY INUNDATED

Cases arise in which is it desirable to compute the backwater



A – PROFILE ON ℄ OF STREAM
BACKWATER EXPRESSION

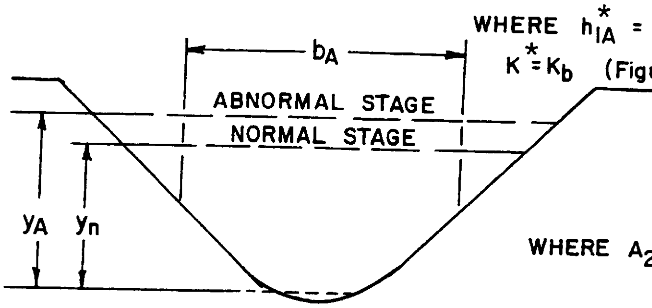
$$h^*_{1A} = K^*_{a2}\,\frac{V^2_{2A}}{2g}$$

WHERE h^*_{1A} = BACKWATER MEASURED ABOVE ABNORMAL STAGE

$K^* = K_b$ (Figure 4 or 5) $+ \Delta K_p$ (Figure 6) $+ \Delta K_e$ (Figure 7) $+ \Delta K_s$ (Figure 8 or 9)

$$V_{2A} = \frac{Q}{A_{2A}}$$

WHERE $A_{2A} = b_A h_A$ OR GROSS AREA OF CONSTRICTION BASED ON ABNORMAL STAGE



B – CONSTRICTED SECTION

BACKWATER WITH ABNORMAL STAGE – DISCHARGE CONDITION

FIGURE 7–12

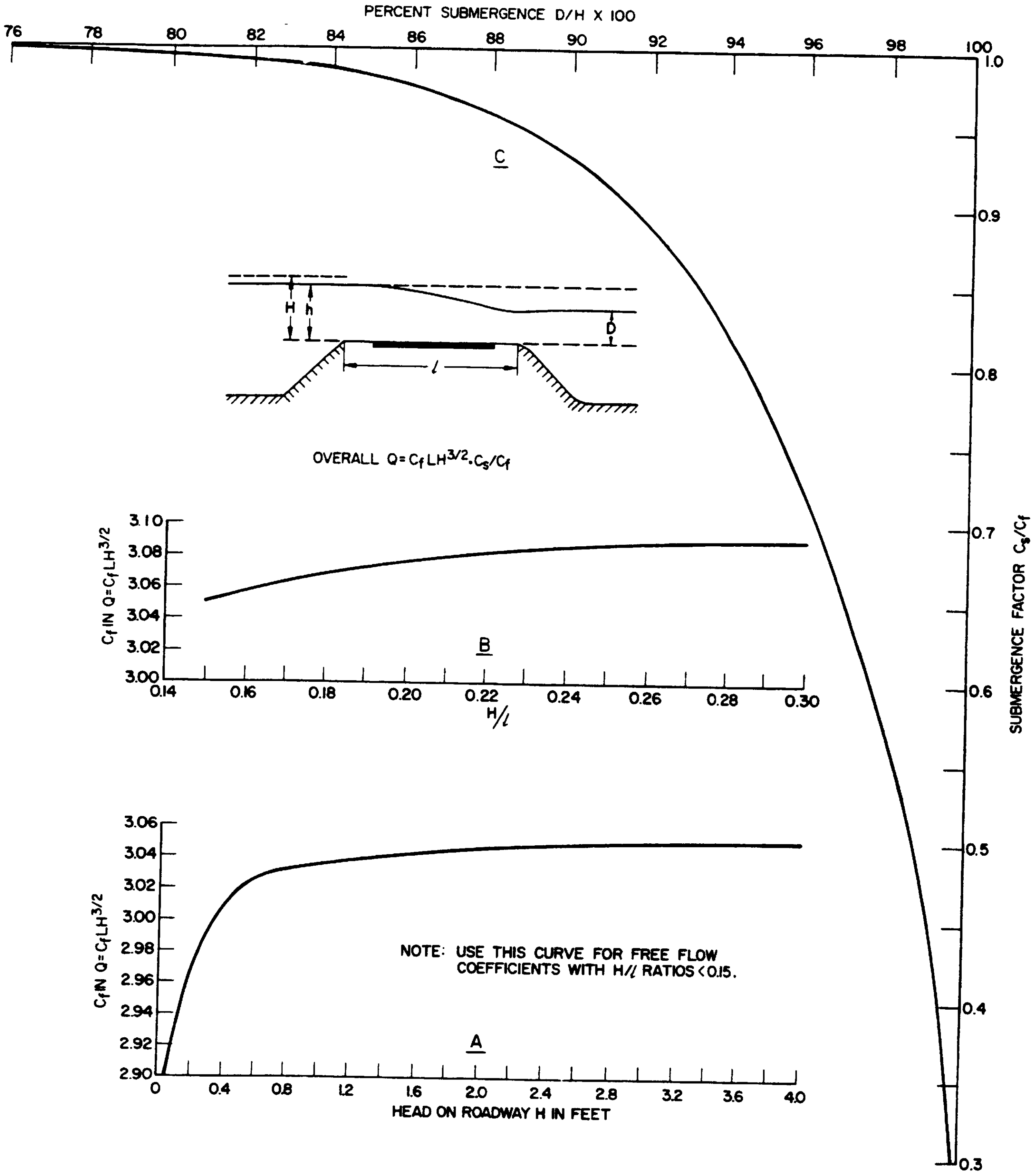
upstream from a bridge or the discharge under a bridge when flow is in contact with the girders. Once flow contacts the upstream girder of a bridge, orifice flow is established so the discharge then varies as the square root of the effective head. The result is a rather rapid increase in discharge for a moderate rise in upstream stage. The greater discharge, of course, increases the likelihood of scour under the bridge. Inundation of the bridge deck is a condition the engineer seldom contemplates in design but it occurs frequently on older bridges.

A rather common source of bridge failure results from the superstructure being virtually pushed or lifted off the abutments and piers by the combination of buoyancy and dynamic forces. Inundation reduces the effective weight of a concrete bridge to about 0.6 of its weight in air. Should air be trapped under the deck between girders, the effective weight can be further reduced to a dangerous limit so that only moderate horizontal forces are required to jar or slide bridge spans off the bents. The horizontal forces consist of unbalanced hydrostatic pressure, or ponding acting on the upstream face of the bridge, plus impact forces produced by large floating objects striking the bridge. The procedure for determining the upstream water surface or the discharge under the bridge is explained in Chapter VIII (Reference 1), with an example included in Chapter XII (Reference 1).

(D) Flow Over Roadway

In cases where bridge clearance is such that girders become inundated during floods, there is a good possibility that flow also occurs over portions of the approach roadway. Should it be desired to determine the discharge flowing over the roadway, a chart is included as Figure 7-13, page 7-41 (Reference 2 and 3).

To determine the discharge flowing over a roadway, first enter curve B, Figure 7-13, page 7-41, with H/l and obtain the free flow coefficient of discharge C_f. Should the value of H/l be less than 0.15, it is suggested that C_f be read for curve A of the



Discharge coefficients for flow over roadway embankments.

Figure 7-13

same figure. If submergence is present (e.g., if H/D is larger than 0.7) enter curve C with the proper value of submergence in percent and read off the submergence factor C_s/C_f. The resulting discharge is obtained by substituting values in the expression:

$$Q = C_f L H^{3/2} \cdot C_s/C_f, \ldots\ldots (7\text{-}10)$$

where L represents the length of inundated roadway, H is the total head upstream measured above the crown of the roadway and C_f and C_s are coefficients of discharge for free flow and with submergence, respectively. Where the depth of flow varies along the roadway, it is advisable to divide the inundated portion into reaches and compute the discharge over each reach separately. The process, of course, can be reversed to aid in determining backwater for a combination of bridges and roadways.

The overtopping of roadways bears a connotation of the past but this sort of thinking should not be discarded; it has far reaching possibilities in present and future design. The present tendency, for Interstate and primary roads, is to construct approach embankments well above the 50-year flood, or highest flood level of record, and depend on the bridge to pass all flood waters, including the super flood. A limit must be set on the length of bridge for economic reasons, which is usually proportioned for about a 50-year flood, but where topography is favorable, this same bridge with embankments set at a lower predetermined level may handle a 100-year flood safely. This safety value should be considered whenever possible, and can be used to reduce backwater and provide protection to the bridge. The roadway in this case would be located slightly above the flood level for which the bridge is design, and the lowest part of the superstructure should be high enough to clear the design high water. The importance of this safety value is that embankments that are damaged by overtopping can be restored in a few days, while bridges only after many months and thousands of dollars.

(E) SPUR DIKES

Where approach embankments encroach on wide floodplains and constrict the normal flood flow, special attention should be given to scour, particularly in the vicinity of bridge abutments. Flow from the floodplain travels along the embankment, and enters the constriction as a concentrated jet at an angle with the direction of flow in the main channel. In so doing the severity of the contraction is increased at the abutment, the effective length of the bridge opening is reduced, and the possibility of scour at the junction of the two jets is great. This situation also leads to higher backwater stages than are usually obtained by analysis. Section 7-504.3 (F) explain the increased backwater effect.

Where borrow pits and ditches exist along the upstream side of a bridge approach embankment, flow from the floodplain favors this path of least resistance; the result is often an unusually high flow concentration along the embankment. This condition can be alleviated to some extent on new bridges by prohibiting borrow pits on the upstream side of embankments and forbidding the cutting of trees back of the toe of the fill slope. For cases where channeling along an embankment is already present or cannot be avoided, the situation can usually be remedied by constructing a spur dike, the procedure is explained in Chapter IX of (Reference 1).

(F) BACKWATER AT WIDTH CONSTRICTIONS OF HEAVILY VEGETATED FLOODPLAINS

Until the completion of a study by the U.S.G.S., "Computations of Backwater and Discharges of Width Constrictions of Heavily Vegetated Floodplains", it had generally been assumed that backwater computed by the U.S.G.S. method or by the Federal Highway Administration method gave a high or conservative answer, where wide floodplains are involved. The above mentioned study proved this assumption to be in error; the following statement was taken from the study. "Backwater computed by the present Geological Survey method averaged 27 percent less than the measured, and that computed by the currently used Federal Highway Administration method averaged 47 percent less than the measured. Discharge computed by the Survey method averaged 21 percent more than the measured. Analysis of data showed that the floodplain widths and the Manning's roughness coefficient are larger than those used to develop the standard methods in current use and the accurate computation of backwater and discharge depends on improving the method of computing the energy loss".

It has been suggested that the reason for the increase in back water is because the flow moving laterally toward the bridge accumulates upstream of the bridge and the accumulating flows causes backwater. Furthermore, heavy vegetation downstream of the bridge causes downstream backwater because of flcw moving back onto the floodplain. The effect this study has is that in floodplains that are over 5 times the bridge length or have a Mannings roughness coefficient over 0.07, and there are backwater considerations or concerns, the method developed by the U.S.G.S. in the above mentioned manual should be considered for computing backwater.

7-505 BRIDGE BACKWATER DESIGN PROCEDURE

The following is a brief step-by-step outline for determining the backwater produced by a bridge constriction.

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.
2. Determine the stage of the stream at the bridge site for the design discharge.
3. Plot representative cross-section of stream for design discharge at Section 1 (upstream section). If the stream channel is essentially straight and the cross section is substantially uniform in the vicinity of the bridge, the natural cross-section of the stream at the bridge site may be used for this purpose.
4. Subdivide above cross-section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient "n" to each subsection. Careful judgment is necessary in selecting these values.

5. Compute conveyance and then discharge in each subsection.

6. Determine value of kinetic energy coefficient (Equation 7-2a, and 7-2b, page 7-22).

7. Plot natural cross section under proposed bridge based on normal water surface for design discharge, and compute gross water area (including area occupied by piers).

8. Compute bridge opening ratio M, observing modified procedure for skewed crossings.

9. Obtain value of K_b from appropriate base curve. (Figure 7-5, page 7-26).

10. If piers are involved, compute value of J and obtain incremental coefficient ΔK_p (Figure 7-6, page 7-27).

11. If eccentricity is severe, compute value of eccentricity and obtain incremental coefficient ΔK_e (Figure 7-7, page 7-29).

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain incremental coefficient ΔK_s for proper abutment type (Figure 7-9, page 7-32).

13. Determine total backwater coefficient K* by adding incremental coefficients to base curve coefficient K_b.

14. Compute backwater by Equation 7-1, page 7-21.

15. Determine distance upstream to maximum backwater from Figure 7-14, page 7-49 and convert backwater to water surface elevation at Section 1 if computations are based on normal stage at bridge.

Detailed steps illustrated by examples are presented in "Hydraulics of Bridge Waterways" (Reference 1).

An example problem is included at the end of this Chapter in Appendix A. A computer solution to this problem is in Appendix B.

7-506 BACKWATER COMPUTATION FORMS

The following three pages are forms that can be used in the computation of backwater elevations.

BACKWATER COMPUTATION FORM SHEET 1

Q = cfs

Elevation = feet (either normal depth or abnormal depth)

$a_1 = \dfrac{\Sigma qv^2}{Q_2 V_1^{\,2}} =$

$qv^2 =$ from Column 11 - Sheet 2

Q = Q_{Design}

$A_n =$ feet square (at Section 1) Sheet 2

$V_2 = \dfrac{Q_D}{A_b} =$

$A_b =$ feet square (gross water area under bridge)

$A_p =$ feet square (area of piers)

Qb = cfs (flow approaching bridge) Sheet 2

$M = \dfrac{Q_b}{Q_D} =$ $J = \dfrac{A_p}{A_b} =$

$k_b =$ Figure 7-5, page 7-26

$\Delta K_p =$ Figure 7-6, page 7-27 = Figure 7-6

$\Delta K_p =$ $\Delta K \sigma$ =

$\Delta K_e =$

$\Delta K_s =$

$K^*_{total} = K_b + \Delta K_p + \Delta K_e + \Delta K_s =$

$a_2 =$ Figure 7-4, page 7-22

$$\text{Backwater} = h_1^* = K^* a_2 \frac{V_2^{\,2}}{2g} + a_1 \left[\left(\frac{A_b}{A_4}\right)^2 - \left(\frac{A_b}{A_1}\right)^2 \right] \frac{V_2^{\,2}}{2g}$$

If $M > 0.7$, $V_2 < 7$ fps, $\frac{K V_2^2}{2g} < .5$

Second part of equation can be ignored.

Backwater = h_1 =

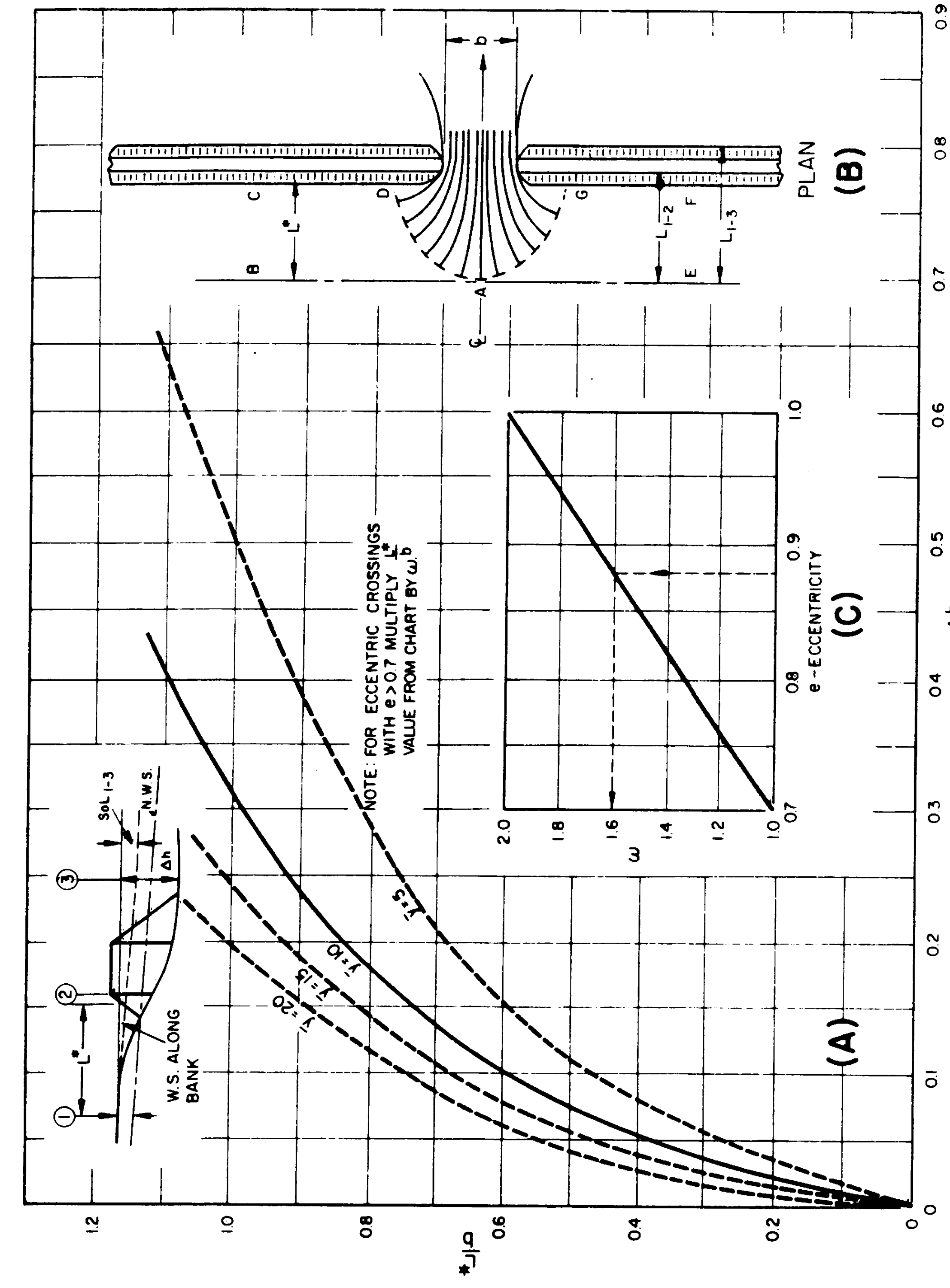
Distance to point of maximum backwater = L*, Figure 7-14, Page 7-49.

S = local slope at bridge

Backwater elevation = Elevation + h_1 + S (L*) = feet

BACKWATER COMPUTATION FORM SHEET 2

Sub-Section	n	$\frac{1.49}{n}$	a	p	$r = \frac{a}{p}$	$r^{2/3}$	$k = \frac{1.49}{n} ar^{2/3}$	$q = Q \frac{k}{K_1}$	$v = \frac{q}{a}$	qv^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
		$A_1 =$					$K_1 =$ ______	$Q =$ ______		$\Sigma qv^2 =$ ___



NOTE: FOR ECCENTRIC CROSSINGS WITH e > 0.7 MULTIPLY L*/b VALUE FROM CHART BY ω
W.S. ALONG BANK
N.W.S.
Δh
(A)
PLAN
(B)
e - ECCENTRICITY
(C)
ω
b/L*
Δh/y

FIGURE 7-14

7-600 CHANNEL MODIFICATIONS

A primary objective in the design of a highway-stream crossing system should be to disturb the stream as little as practicable. Channel modifications should be made only where modification is necessary to achieve compatibility between the highway and stream and to accommodate stream flow with a minimum of interruption to the stream and its environment.

Highway associated channel modifications involve only short reaches of a stream in almost all instances. However, environmental concerns for stream velocity, flow depth, shade, temperature of the water, pool and riffle ratio, overhanging banks, and other factors important to the stream ecosystem, and hydraulic concerns for stream bed and bank stability, make it normally inadvisable to undertake channel modification.

Modifications for the purpose of increasing channel capacity are not commonly encountered at highway stream crossings because of the limited effectiveness of such modifications. Channel capacity improvements usually must be carried long distances downstream if the stage-discharge relationship at the site is to be altered. Because of flow controls downstream, the stage-discharge relationship at a crossing will not be altered a discernable amount by a short reach of modified channel.

Stream channels are sometimes widened through the waterway opening of a bridge. As indicated, this will have essentially no effect on the natural stage-discharge relationship of the stream but by increasing the waterway opening under the bridge, backwater at high flood stages may be decreased. However, short reaches of stream channels which have been enlarged may return to the natural cross section by deposition within the enlarged reach unless it is regularly maintained. At many locations, it may be advisable to consider additional bridge length rather than to rely on an enlarged channel section under the bridge.

An enlarged section through the bridge may be more successful if excavated so that the bedload will not be deposited in the

excavated area and the enlarged area will be available when needed to convey flow through the bridge.
Concerns for the stream and floodplain environment and for bank stability may preclude the use of such an enlarged section on some streams, however.

7-700 HYDRAULIC RELATED CONSTRUCTION CONSIDERATIONS

Numerous considerations that are within the purview of the construction engineer and/or the contractor can significantly affect the integrity of the hydraulic design and the highway stream crossing system. Also, features of the design may be rational insofar as the hydraulics engineer is concerned but ill-conceived from the viewpoint of the construction engineer. Lines of communication between the designer and construction personnel should be established to ensure that designs are not unneccessarily complex or difficult to construct and the construction methods and measures do not invalidate design assumptions or create conditions which will adversely affect the stream crossing.

7-701 VERIFICATION OF PLANS

Plans should be checked to verify that site conditions at the stream crossing have not changed from those that existed at the time design plans were completed. Meander migration, bank caving, aggradation, headcutting or other natural or man-induced changes in the channel may have occurred which would require that the designer reconsider decisions made on the basis of conditions which were different from those which exist at the beginning of construction. The changed conditions may require river control works, revisions to pier locations and orientation, rearrangement of spans, or other modifications of the design to accommodate the changes that have occurred.

Dependent upon the time that has elapsed between completion of the design plans and the beginning of construction, changes in land use could significantly affect the validity of design considerations. Commercial mining of materials for construction is a rather common practice that can change flow velocities, volume and character of bedload, and flow direction and distribution at the crossing site. Land clearing for agricultural purposes may create a need to reconsider the location and size of waterway openings and the need for spur dikes. Land development near the site could change damage risk considerations for the crossing.

The hydraulic and structural engineers should be consulted regarding the need to modify the design at any stream crossing which has changed significantly from the conditions which existed during design.

7-702 BORROW AREAS

Borrow areas should be located so that they will not contribute to the hazards of the stream crossing. Stream bed and bank borrow can induce changes in the stream that may cause active meandering, a new channel to be formed, or result in deposition of the bedload which will cause clear water scour in the waterway. Borrow areas established along the embankment can cause concentrated flow along the embankment and contribute to serious scour at the bridge abutment.

7-703 DETOURS

Stream crossings for detours are built to much lesser standards than the crossing designed for the highway. This is good practice from both hydraulic engineering and economic points of view.

Figure 7-15, page 7-53, indicates that during a 1-year construction period, the odds are 4 to 1 against a flood as large as a 5-year flood and there is an even chance that the mean annual event will not be exceeded. The odds that a 10-year flood will occur during a 1-year construction period are 9 to 1 against and 2.7 to 1 against occurrence in a 3-year construction period. It follows that the criteria used for the hydraulic design of detour stream crossings should be based on risk factors which should be evaluated considering the probability of flood exceedance during the anticipated service life of the detour (the construction period for the highway crossing), the risk to life and property and traffic service requirements.

FLOOD FREQUENCY (T) – YEARS

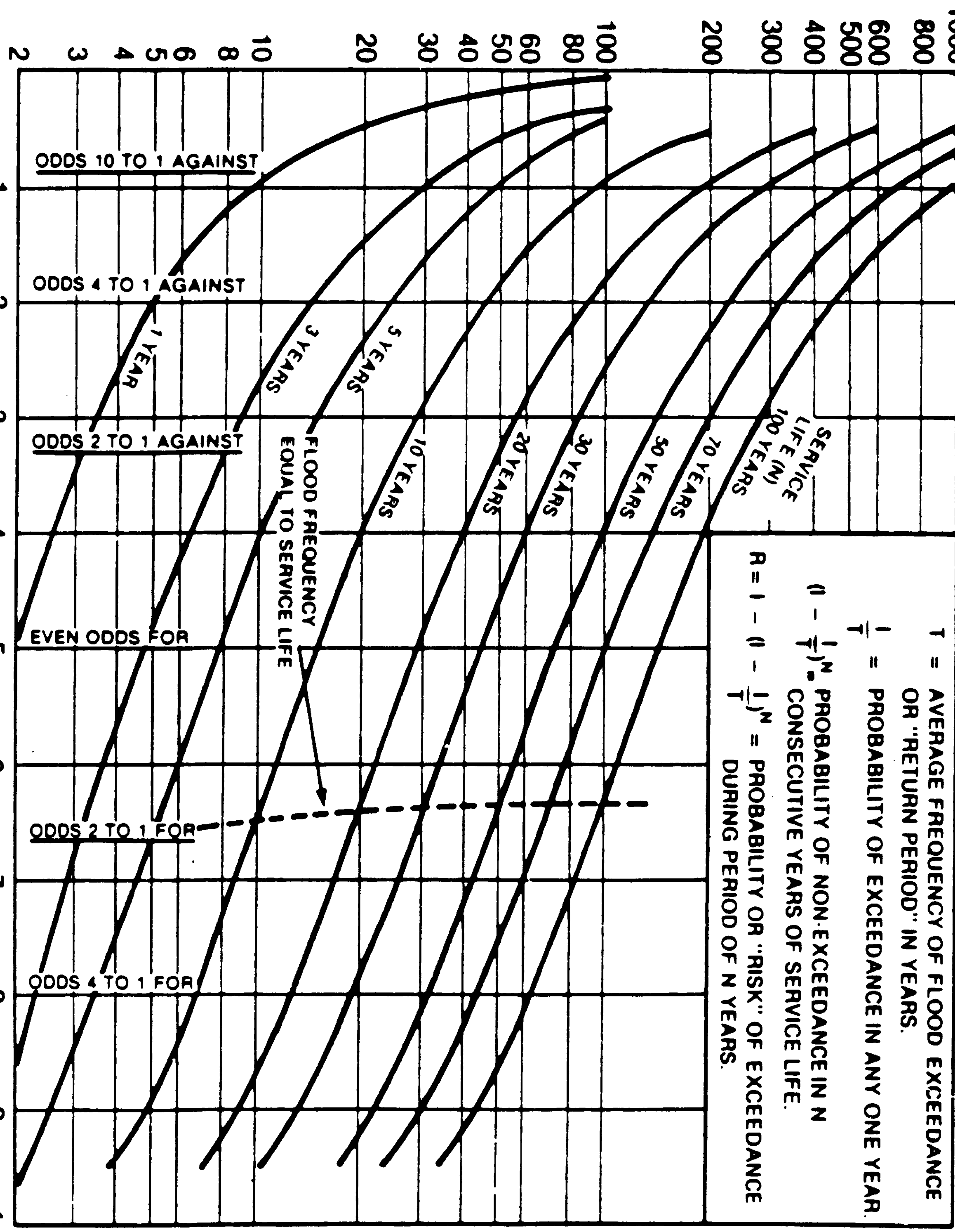


FIGURE 7-15: RISK OF FLOOD EXCEEDANCE

As in the design for highway stream crossings, detour designs should accommodate floods larger than the event for which they are designed in order to avoid undue liability for damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile which allows overflow without creating excessive velocities or backwater.

Temporary stream crossings necessary for the construction of highways are usually the responsibility of the contractor. It may be desirable in some instances, however, for the Department to design such crossings in order to minimize or mitigate the adverse effects on the stream environment, to facilitate securing permits, or to reduce the risk assumed by the contractor and thereby reduce construction costs.

7-704 HYDROLOGIC INFORMATION

A hydrograph of superimposed mean daily flows and a plot of the rating table for a stream gaging station near the crossing site are useful for the design of cofferdams, falsework and temporary crossings, in Department scheduling of the work, and in selecting the location of work and material storage areas. If the hydrograph is furnished to the contractor or included in the plans for the contractor's use in planning and scheduling operations, the contract documents should indicate that the plot is for information only and that the Department assumes no responsiblity for conclusions or interpretations made from the records.

7-705 FEEDBACKS

Most design engineers do not have an opportunity to participate in the construction of the works they have created. For this reason, designs that could be improved upon for construction purposes tend to be perpetuated simply because the design engineer is not informed of the deficiencies.

Construction engineering personnel are encouraged to invite design engineers, including hydraulic engineers, to visit construction sites to discuss problems with designs and possible improvements in future designs. Upon completion of a project, a design critique conducted jointly be designers and field personnel can be a very useful learning experience for both.

7-800 POST-CONSTRUCTION DATA

Possibly the most valuable experience of any design engineer is gained by observing and analyzing the performance of a design under test conditions. Hydraulic engineers, as compared with other engineers, are in an almost unique position in this regard because their designs are often tested by nature and may suffer damage from relatively small flow rates as well as from larger flows. In common with other engineers, however, performance data is rarely available for hydraulic engineers to accomplish more than a qualitative analysis of performance.

Hydraulic engineers should take advantage of their unique opportunities to gain experience from tests provided by nature. The following data are examples of the types of information which are of value in reviewing and analyzing designs to assess the validity of practices, procedures, assumptions and decisions.

1. High water elevations and flow rates.
2. Ice and drift conditions.
3. Erosion of approach overflow sections, embankment and spur dikes.
4. Stream aggradation or degradation .
5. Scour location, depth and extent
6. Performance of scour and erosion preventive measures
7. Meander and bend migration.
8. Performance of stream bank protection and river training measures.
9. Costs of maintenance, repair, and corrective measures.

While the emphasis in this section is on the knowledge and experience that can be gained from field tests of designs, the above data is also useful in detecting potential or existing problems at the crossing which can be resolved by providing corrective measures.

7-900 DESIGN DOCUMENTATION

Design data should be assembled in an orderly fashion and retained for future reference. The amount and detail of documentation for each highway stream crossing system should be commensurate with the risk and the importance of the crossing. For example, a small stream in a rural area would not ordinarily require the same degree of documentation as a small stream in a developed area.

Design data and documentation are important in the post-construction period for the following reasons:

1. The performance of structures over a period of time, as compared with information developed for the design, is very helpful in evaluation design policies and procedures and the validity of design assumptions.
2. In the event of failure, contributing causes can be identified, compared with design assumptions and computations, and considered in the design of a replacement structure(s).
3. Documentation of data for existing structures is a valuable source of information when structures are replaced, repaired, or rehabilitated, and for the design of other structures in the vicinity.
4. Information collected and analyzed for purposes of highway design can be valuable to others considering plans for the vicinity.
5. File documentation of data, analyses and decision-making is essential to the competent handling of subsequent complaints and litigation.

Project plans are the most permanent of all highway agency records and thus are a convenient and appropriate document for recording the results of analyses and decisions.

7-1000 DESIGN DATA FOR PERMANENT RECORDS

The permanent records should include all material used in arriving at the selected design. This should include the results, studies of alternative and reasons for rejection, as well as:

1. Copies of all pertinent correspondence, agreements, and minutes of conferences and, in particular, those with public involvement
2. Topography of site
3. Drainage area map, if used
4. Stream profile and cross sections
5. Historical high water documentation
6. Information on existing structures in the vicinity
7. Hydrologic design computations
8 Hydraulic design calculations
9. Scour investigation
10. Economic analysis of structure and profile selection
11. Other field notes.

7-1100 CHECKLISTS

The following checklists are available to help in the completion of a job in an orderly fashion.

a) Field inspection checklist (Form HYD 2-1), Page 2-11
b) Encroachment decision model (Form HYD 2-2), Page 2-12
c) Hydrology checklist (Form 102), Page 7-58
d) Interoffice memorandum checklist (Form 103), Page 7-59

FORM 102

Job Number ____________________

Job Name ____________________

Date ____________________

Initials ____________________

HYDROLOGY CHECK LIST

1. Maps of study area
   - ☐ a. Quad maps
   - ☐ b. Aerials
   - ☐ c. County and/or city maps

2. Regulations
   - ☐ a. FHPM 6-7-3-2
   - ☐ b. 404 requirements

3. Contact other agencies
   - ☐ a. Corps of Engineers
   - ☐ b. SCS
   - ☐ c. USGS
   - ☐ d. Local Government

4. ☐ Figure stream miles
5. ☐ Determine D.A. and Centroids
6. ☐ Compute stream slopes
7. ☐ Compute rainfall
8. ☐ Site Inspection
9. ☐ Request additional field information and x-sections
10. ☐ Regional analysis
11. ☐ Determine hydrograph coefficients
12. ☐ Perform hydrologic computations
13. ☐ Perform necessary hydraulic computations
14. ☐ Revise hydrologic computations
15. ☐ Perform final hydraulics
16. ☐ Compare flood elevations to historical H.W.
17. ☐ Memo to approporiate AHTD Department or Agency

FORM 103

INTER-OFFICE MEMORANDUM CHECKLIST

JOB NUMBER ____________________________

JOB NAME ____________________________

DATE ____________________________

INITALS ____________________________

1. ☐ Specify reason for memo
2. ☐ Explain job
3. ☐ Specify permit requirements
4. ☐ Explain flood regulations if applicable
5. Give pertinent information on basin
   - ☐ a. Drainage area
   - ☐ b. Stream slopes
   - ☐ c. Historical high water
   - ☐ d. Floodplain specifications (floodway widths, etc.)
   - ☐ e. Frequency flood flows
   - ☐ f. Frequency flood elevations
6. ☐ Explain analysis methodology
7. ☐ Certify reasonableness of solution (flood comparison, reasons for methods, etc.).
8. ☐ Explain any further analysis to be performed by Hydraulics
9. ☐ a. Possibly significant encroachment
   - ☐ b. Structures in and/or near floodplain
   - ☐ c. Risk analysis recommended
   - ☐ d. Risk assessment recommended
10. ☐ Sum up results and make recommendation(s)
11. ☐ Sign-off
12. ☐ Specify copies

LIST OF SYMBOLS

Chapter 7

Bridge Hydraulics

A_1 = Area of flow including backwater at Section 1 (sq. ft.).

A_{n1} = Area of flow below normal water surface at Section 1 (sq. ft.)

A_{n2} = Gross area of flow in constriction below normal water surface at Section 2 (sq. ft.)

A_4 = Area of flow at Section 4 at which normal water surface is reestablished (sq. ft.)

A_p = Projected area of piers normal to flow (between normal water surface and streambed) (sq. ft.).

A_s = Area of scour measured on downstream side of bridge (sq. ft.).

b = Width of constriction (ft.).

b_s = Width of constriction of a skew crossing measured along centerline of roadway (ft.).

C = h^*_{1s}/h^*_1 = Correction factor for backwater with scour.

C_f = Freeflow coefficient for flow over roadway embankment.

C_s = Submergence factor for flow over roadway.

e = Eccentricity = $(1-Q_c/Q_a)$ where

$$Q_c < Q_a,$$

or $(1-Q_a/Q_c)$ where

$$Q_c > Q_a.$$

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

h_T = Total energy loss between Sections 1 and 4 (ft.).

h_b = $h_T - S_oL_{1-4}$ = Energy loss caused by constriction (ft.).

$h_1{}^*$ = Total backwater or rise above normal stage at Section 1 (ft.).

$h_{1s}{}^*$ = Backwater with scour (ft.).

$h_3{}^*$ = Vertical distance from water surface on downstream side of embankment to normal water surface at Section 3 (ft.).

Δh = $h_1{}^*+h_3{}^*+S_oL_{1-3}$ = Difference in water surface elevation across roadway embankment (ft.).

J = A_p/A_{n2} = Ratio of area obstructed by piers to gross area of bridge waterway below normal water surface at Section 2.

K_b = Backwater coefficient from base curve.

ΔK_p = Incremental backwater coefficient for piers.

ΔK_e = Incremental backwater coefficient for eccentricity.

ΔK_s = Incremental backwater coefficient for skew.

K^* = $K_b+\Delta K_p+\Delta K_e+\Delta K_s$ = Total backwater coefficient for subcritical flow.

k = Conveyance in subsection of approach channel.

K_1 = Total conveyance at Section 1.

L_{1-4} = Distance from point of maximum back water to reestablishment of normal water surface downstream, measured along centerline of stream (ft.).

L_{1-3} = Distance from point of maximum backwater to water surface on downstream side of roadway embankment (ft.).

L_{1-2} = Distance from point to maximum backwater to upstream face to bridge deck (ft.).

L^* = Distance from point of maximum backwater to water surface on upstream side of roadway embankment, measured parallel to centerline of stream (ft.).

M = Bridge opening ratio. (Page 7-23 & Figure 7-5, page 7-26).

n = Manning roughness coefficient.

p = Wetted perimeter of a subsection of a channel (ft.).

Q_b = Flow in portion of channel within projected length of bridge at Section 1 (c.f.s.).

Q_a, Q_c = Flow over that portion of the natural floodplain obstructed by the roadway embankments (c.f.s.).

Q = $Q_a+Q_b+Q_c$ = Total discharge (c.f.s.).

r = a/p = Hydraulic radius of a subsection of floodplain or main channel (ft.).

S_o = Slope of channel bottom or normal water surface.

V_1 = Q/A_1 = Average velocity at Section 1 (ft./sec.).

V_4 = Q/A_4 = Average velocity at Section 4 (ft./sec.).

V_{n2} = Q/A_{n2} = Average velocity in constriction for flow at normal stage (ft./sec.).

V_{2c} = Critical velocity in constriction (ft./sec.).

W_p = Width of pier normal to direction of flow (ft.).

y_1 = Depth of flow at Section 1 (ft.).

y_4 = Depth of flow at Section 4 (ft.).

y_n = Normal depth of flow in model (ft.).

y = A_{n2}/b = Mean depth of flow under bridge, referenced to normal stage, (ft.).

a_1 = Velocity head coefficient at Section 1 (Greek letter alpha).

a_2 = Velocity head coefficient for constriction (Greek letter alpha).

σ = Multiplication factor for influence of M on incremental backwater coefficient for piers (Greek letter sigma).

w = Correction factor for eccentricity (Greek letter omega).

ϕ = Angle of skew - degrees (Greek letter phi.).

REFERENCES

Chapter 7

BRIDGE HYDRAULICS

7-1 HYDRAULICS OF BRIDGE WATERWAYS, J. N. Bradley, Hydraulic Design Series No. 1, Federal Highway Administration, 1970.

7-2 COMPUTATIONS OF BACKWATER AND DISCHARGE AT WIDTH CONSTRICTIONS OF HEAVILY VEGETATED FLOODPLAINS, U. S. Geological Survey, Water-Resources Investigation, 1977.

7-3 DISCHARGE CHARACTERISTICS OF EMBANKMENT SHAPED WEIRS, C. E. Kindsvater, U. S. Geological Survey WSP 1607-A, 1964.

7-4 HYDRAULIC ANALYSIS FOR THE LOCATION AND DESIGN OF BRIDGES, AASHTO Highway Drainage Guidelines, Washington, D.C., 1982.

APPENDIX A

EXAMPLE PROBLEM

Given:

The channel crossing shown in Figure 7-16, page 7-65, with the following information: Cross section of river at bridge site showing areas, wetted perimeters, and values of Manning n; normal water surface for design =El. 37.0 ft. at bridge; average slope of river in vicinity of bridge S_o = 10.0 ft./mi. or 0.00189 ft./ft.; cross section under bridge showing area below normal water surface and width of roadway = 40 ft.

The stream is essentially straight, the cross section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.

Find.--

(a) Conveyance at Section 1.

(b) Discharge of stream at El. 37.0 ft.

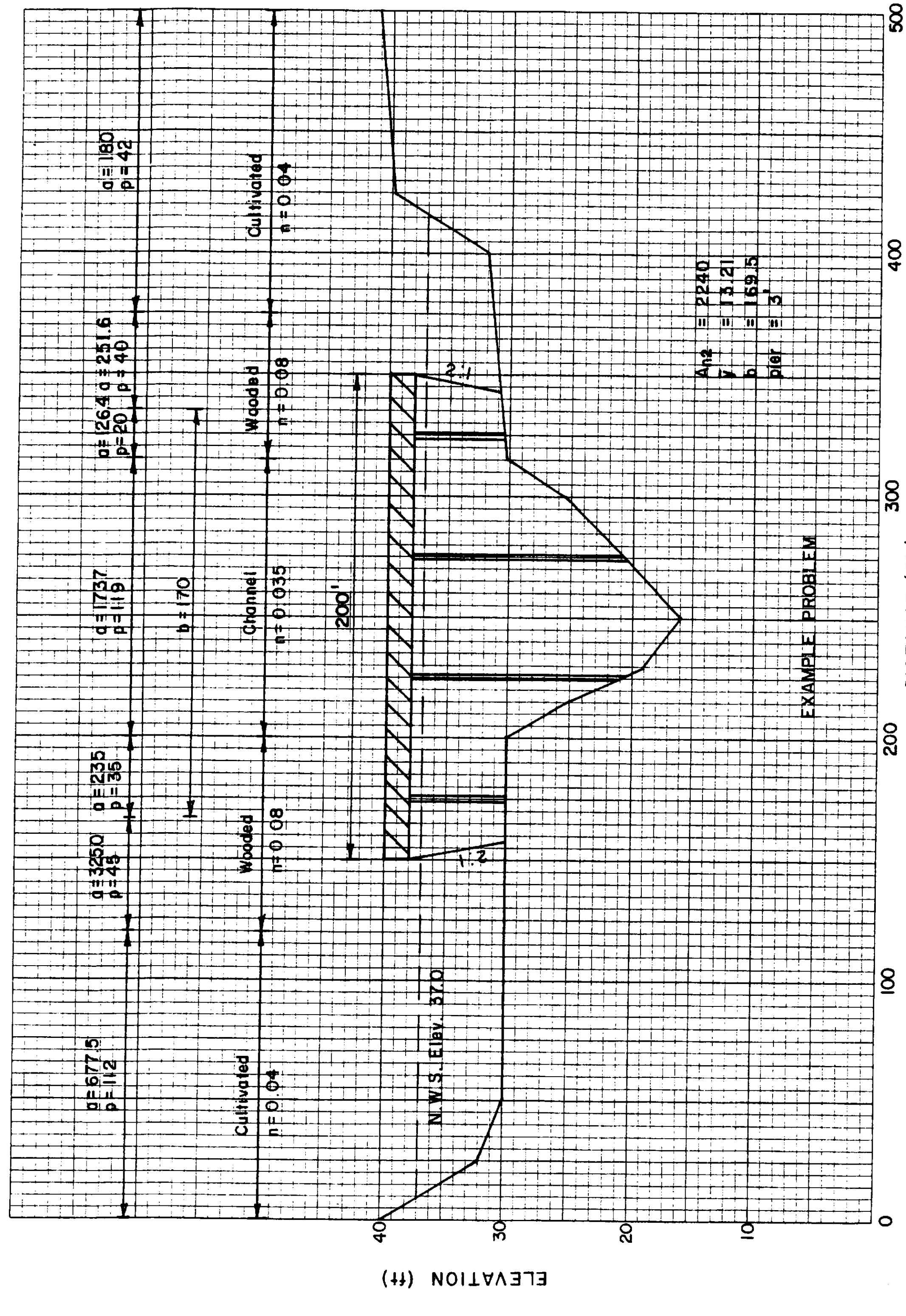
(c) Velocity head correction coefficient, a_1.

(d) Bridge opening ratio, M.

(e) Backwater produced by the bridge.

(f) Water surface elevation on upstream side of roadway embankment.

Assumptions. Under the conditions stated, it is permissible to assume that the cross sectional area of the stream at Section 1 is the same as that at the bridge. The approach section is then divided into subsections at abrupt changes in depth or channel roughness as shown in Figure 7-16. The computations are then completed using computation Sheets 1 and 2, pages 7-66, 7-67 and 7-68.



a = 677.5
p = 112
a = 325.0
p = 45
a = 235
p = 35
a = 1737
p = 119
a = 126.4
p = 20
a = 251.6
p = 40
a = 180
p = 42
b = 170
Cultivated
n = 0.04
Wooded
n = 0.08
Channel
n = 0.035
Wooded
n = 0.08
Cultivated
n = 0.04
200'
N.W.S. Elev. 37.0
1:2
2:1
A_n2 = 2240
V = 13.21
b = 169.5
pier = 3'
EXAMPLE PROBLEM
DISTANCE (FT.)
ELEVATION (ft)
0
100
200
300
400
500
10
20
30
40

FIGURE 7-16

BACKWATER COMPUTATION FORM SHEET 1

Q = 26310 cfs

Elevation = 37 feet (either normal depth or abnormal depth)

$$a_1 = \frac{\Sigma qv^2}{Q_2 V_1^{\,2}} = \frac{2484400}{26310\ (26310/35325)^2} = 1.70$$

qv^2 = 2484400 from Column 11 - Sheet 2, page 7-68.

Q = 26310 Q_{Design}

A_n = 3532.5 feet square (at Section 1) Sheet 2, page 7-68.

$$V_2 = \frac{Q_D}{A_b} = \frac{26310}{2240} = 11.74 \text{ fps}$$

A_b = 2240 feet square (gross water area under bridge)

A_p = 141 feet square (area of piers)

Qb = 20218 cfs (flow approaching bridge) Sheet 2, page 7-68

$$M = \frac{Q_b}{Q_D} \qquad \frac{20218}{26310} = .77 \qquad J = \frac{A_p}{A_b} = .063$$

k_b = .32 Figure 7-5, page 7-26

ΔK_p = .25 Figure 7-6, page 7-27 = .93 Figure 7-6

ΔK_p = $\Delta K \sigma$ = .25 (.93) = .23

ΔK_e = 0

ΔK_s = 0

$$K^*_{total} = K_b + \Delta K_p + \Delta K_e + \Delta K_s = .32 + .23 = .55$$

a_2 = 1.55 Figure 7-4, page 7-22

$$\text{Backwater} = h_1^* = K^* a_2 \frac{V_2^{\,2}}{2g} + a_1 \left[\left(\frac{A_b}{A_4}\right)^2 - \left(\frac{A_b}{A_1}\right)^2\right] \frac{V_2^{\,2}}{2g}$$

$$h_1^* = .55\ (1.55)\ \frac{11.74^2}{64.4} + 1.7 \left[\left(\frac{2240}{3532.5}\right)^2 - \left(\frac{2240}{4260}\right)^2\right] \frac{11.74^2}{64.4}$$

$$h_1^* = 1.82 + .46 = 2.28$$

If $M > 0.7$, $V_2 < 7$ fps, $\frac{K\ V_2^{\ 2}}{2g} < .5$

Second part of equation can be ignored.

Backwater = h_1 = 2.28

Distance to point to maximum backwater = 185 L*, Figure 7-14, Page 7-49.

S = 0.00189   local slope at bridge

Backwater elevation = Elevation + h_1 + S (L*) =   feet
= 37 + 2.28 + 185 (.00189) = 39.62

BACKWATER COMPUTATION FORM SHEET 2

Sub-Section	n	$\frac{1.49}{n}$	a	p	$r = \frac{a}{p}$	$r^{2/3}$	$k = \frac{1.49}{n} ar^{2/3}$	$q = Q \frac{k}{K_1}$	$v = \frac{q}{a}$	qv^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
0-120	.04	37.25	678	112	6.05	3.32	83786	3642	5.37	105244
120-165	.08	18.63	325	45	7.22	3.74	22645	984	3.03	9020
165-200	.08	18.63	235	35	6.71	3.56	15586	677	2.88	5618
200-315	.035	42.57	1737	119	14.59	5.97	441446	19191	11.04	2342533
315-335	.08	18.63	126	20	6.32	3.42	8053	350	2.77	2683
335-375	.08	18.63	252	40	6.29	3.41	15984	691	2.74	5212
375-416	.04	37.25	180	42	4.28	2.64	17701	770	4.27	14090
		A_n = 3532.5					K_1 = 605201	Q = 26305		Σqv^2 = 2484400

$Q = K_1 \, S^{1/2}$

$Q = (605201) \; 0.00189^{1/2} = 26310$

APPENDIX B

This appendix explains the method of solution of the HOW computer program, and an explanation of the input and output. Also included is the computer solution to the example problem in Appendix A, on pages 7-64 to 7-68.

SCOPE

a. The purpose of this program is to perform the hydraulic analysis of bridge waterways and determine the backwater produced by a bridge.

b. The program consists of two parts. The first part analyzes the nature stream section and produces typical stage-discharge information and cross section and rating curve plots.

c. The second part, which is optional, analyzes the constricted waterway opening and determines velocities and backwater heights for given stage elevations and bridge lengths.

d. Bridge backwater is computed using the method presented in the Federal Highway Administration publication "Hydraulics of Bridge Waterways," HDS No. 1, revised 1970.

SECTION 2. METHOD OF SOLUTION

a. Section properties are computed for a number of stages, beginning at the minimum stage submitted and increasing by the input stage increment until a discharge greater than or equal to 1.15 times the design discharge is attained.

b. If a skewed cross section is submitted, the normal X values will be calculated and used in determing the section properties.

c. If backwater computations are desired, they are obtained by the procedure given in the publication "Hydraulics of Bridge Waterways," HDS No. 1, published by the Federal Highway Administration.

d. The backwater procedure is limited to bridge constrictions with subcritical flow. The program determines the value of the Froude number at the constricted section by the following equation :

$$Fr = \frac{V}{\sqrt{g\ y}}$$

where

V = Average velocity in constriction

g = Acceleration due to gravity

y = Mean depth of flow

If the value of the Froude number is greater than 0.9, indicating the flow is close to supercritical, the method cannot be used and the message "DESIGN METHOD INVALID" is printed out. If several values of backwater are to be obtained, the program will continue to increment the stage elevation and/or bridge length and recalculate the Froude number for each condition. This may result in the above message for each stage elevation and bridge length. However, a bridge length or stage may be obtained where the Froude number is less than 0.9, in which case the program will calculate and output the backwater value.

e. If the flow profile is desired for the upstream section it is calculated by the following equation:

$$Z_1 + \frac{\alpha_1 V_1^2}{2g} = Z_2 + \frac{\alpha_2 V_2^2}{2g} + HF + HE$$

Z_1 = watersurface elevation at new station

α_1 = velocity distribution coefficient at new station

V_1 = mean velocity at new station

Z_2 = watersurface elevation at previous station

α_2 = velocitv distribution coefficient at previous station

V_2 = mean velocity at previous station

HF = friction loss between stations

$$= \frac{S_1 + S_2}{2} \cdot \Delta X$$

$$S_1 = \left[\frac{Q}{\text{CONVEYANCE AT NEW STATION}}\right]^2$$

$$S_2 = \left[\frac{Q}{\text{CONVEYANCE AT PREVIOUS STATION}}\right]^2$$

ΔX = DISTANCE BETWEEN STATIONS

HE = EDDY LOSS

$$= \text{CE x ABS. VALUE} \left[\frac{\alpha_1 V_1^2}{2g} - \frac{\alpha_2 V_2^2}{2g}\right]$$

CE = eddy loss coefficient

$= 0.3 \quad (V_1 > V_2)$

$= 0.1 \quad (V_1 < V_2)$

It is necessary for the user to input a profile increment. The program proceeds to calculate the actual water surface elevations, normal water surface elevations, and backwater, moving upstream from the point of maximum backwater by the input station increment. Elevations are calculated for fifty stations.

The cross-section of the stream may be varied by entering additional cross-sections. A maximum of 50 additional sections may be entered. Input for each section consists of the distance upstream, number of points, skew, and the X, Y, and Mannings N for each point.

If the station being investigated does not coincide with any of the input cross-sections, the previous section downstream is used with the Y values adjusted.

SECTION 3. INPUT AND OUTPUT

Input

a. Project Identification

IDENT	- The five characters which follow the equal sign (BKWTR) uniquely identify the data being submitted. The program initially checks the characters being input and will terminate the execution if the proper combination of characters is not present. The message "IDENT CARD ERROR - EXECUTION TERMINATED" will appear.
COunty	- County number for the project site: from 1 to 67.
LR SEC	- Legislative route and section numbers for the project.
LANE	- Alphabetic designation of the roadway, such as SB for Southbound.
LOG NUMBER	- Enter a number of your own choosing.
ORG CODE	- Your three-digit organization code, i.e. 040 for District 4-0.
PROJECT NUMBER	- The 14-digit project number.
STATION	- Reference station of the structure.
STREAM IDENT	- Stream identification.

b. Stream Data

DESIGN DESCHARGE	- Normal discharge of the channel - c.f.s.
SLOPE	- Slope of the channel - ft./ft.
MINIMUM STAGE ELEVATION	- The beginning stage elevation for the computation of section properties - feet.
STAGE INCR.	- The stage increment used while computing section properties and backwater values - feet.
PROFILE INCR.	- Stationing increment for the calculation of the flow profile - feet. Leave blank if the flow profile is not desired.

NUMBER X-SECS - The number of upstream cross sections which will be entered. Enter only if the flow profile is desired and the stream cross section varies.

BKWT - Enter 'Y' if backwater computations are desired and bridge data is to be entered. Otherwise leave blank.

The following three items are entered only if backwater computations are desired.

ADD. BW - If 'Y' is entered, backwater will be computed from the design stage to the stage where discharge is greater than or equal to 1.15 times the design discharge. If left blank, backwater will be calculated for the design stage only.

BW - If ADD. BW is entered as 'Y' and backwater computations are also desired for stages below the design stage, enter the number desired.

SCOUR AREA - Area of scour measured on downstream side of bridge - square feet.

c. Bridge Data

Enter only if BKWT = 'Y'

SKEW - Skew angle of the bridge measured from the centerline of the stream to a line perpendicular to the centerline of the bridge - degrees.

BR. DIST. - Enter 'N' if abutment distances are measured along a normal to the centerline of stream. Enter 'S' if the bridge is skewed and abutment distances are measured along the centerline of the bridge.

MOVES - Enter the number of times the bridge length is to be increased or decreased for backwater computations.

DISTANCE PER MOVE - The increase (+) or decrease (-) in bridge length for each move.

DIRECT - If 'L' or 'R' is entered the left or right abutment will be moved by DISTANCE PER MOVE. If 'B' is entered the left and right abutment will each be moved by this distance.

ABUT - Enter 'W' for a wingwall abutment, 'S' for a spill-through abutment.

CURVE - Designates the specific base backwater curve to be used. Refer to Figure 7-5, page 7-26.

Enter '1' for:
All spillthrough abutments.
All 45^{o} and 60^{o} wingwall abutments.
Bridges longer than 200 feet with 30^{o} and 90^{o} wingwall abutments.

Enter '2' for bridges shorter than 200 feet with 30^{o} wingwall abutments.

Enter '3' for bridges shorter than 200 feet with 90^{o} wingwall abutments.

The bridge length is computed within the program. If '2' or '3' is entered and the bridge is longer than 200 feet, curve 1 will be used.

NUMBER PIERS - The number of piers in the cross section. There may be a maximum of 20.

PIER - The number of the curve used to obtain the incremental backwater coefficient due to the piers. Refer to Figure 7-6, page 7-27.

BENT - The number of the curve used to obtain SIGMA, the multiplication factor for the influence of the opening ratio on the incremental backwater coefficient for piers. Refer to Figure 7-17, page 7-77.

Ld - If dual bridges are present, enter the distance between the upstream face of the upstream bridge and the downstream face of the downstream bridge - feet.

l - The overall width of the roadway or bridge - feet.

LEFT ABUTMENT, RIGHT ABUTMENT
Refer to Figure 7-18, page 7-78.

DISTANCE - The horizontal distance from the datum to the top of the abutment - feet.

ELEVATION - The elevation of the top of the abutment - feet.

ANGLE - The angle between the face of the abutment wall and the horizontal - degrees.

d. Pier Data

Data must be entered for the number of piers specified on the previous line of input. Do not enter any data if backwater computations are not desired or NUMBER PIERS = "0".

BASE ELEVATION - The elevation of each pier at the point where it intersects the ground line.

WIDTH - The average width of each pier normal to the flow - ft.

e. Stream Cross Sections

One set of data must be entered for the cross-section at the bridge. If the flow profile is desired for a waterway with a varying cross section , also enter the number of upstream cross sections entered under NUMBER X-SECS.

DISTANCE UPSTREAM - The distance along the centerline of the waterway from the bridge to the cross section - ft. This value must be '0' for the first cross section entered.

NO. PTS. - The number of points on the cross section.

SKEW - The skew angle of the cross section, measured from the cross section to a line normal to the flow of the stream.

f. Cross Section Data

Two datum planes are chosen. One a horizontal datum, must be below the lowest point on the channel. The other, a vertical datum, must be outside of and to the left of the cross section so that all horizontal measurements (i.e. X distances) will be positive.

Data is entered from left to right along the cross section, looking upstream.

DISTANCE - The distance from the datum to the point on the cross section along the X - axis - ft.

GROUND ELEVATION - The ground elevation of the point on the cross section - ft.

N - Manning's N for the segment to the left of the point.

Output

g. Cross section input data.

h. A line printer plot of the Cross Section. The vertical scale of the plot is set at 5 or 10 feet per inch, depending on the maximum Y difference. The horizontal scale is set at 60 feet per inch.

Storage allocation restricts plotting to sections under 3000 feet in length. Sections not beginning at X equals zero, even if less than 3000 feet, will not be plotted if the maximum X exceeds 3000 feet. This limitation can be overcome by adjusting the X values to begin at zero. If the section exceeds the 3000 feet limitation, a message will be printed out without program termination.

If the input cross section is skewed, the program will plot the skewed cross section, calculate the normal values for the X distances and plot the normal cross section.

i. Section Properties

The total area, total conveyance and total discharge are output for each stage.

The area, wetted perimeter, hydraulic radius, conveyance, discharge and velocity are output for each sub-section.

j. Rating Curve

A line printer plot of discharge versus stage elevation.

k. Backwater computations

The section properties listed under i. are output for each stage at which backwater is computed.

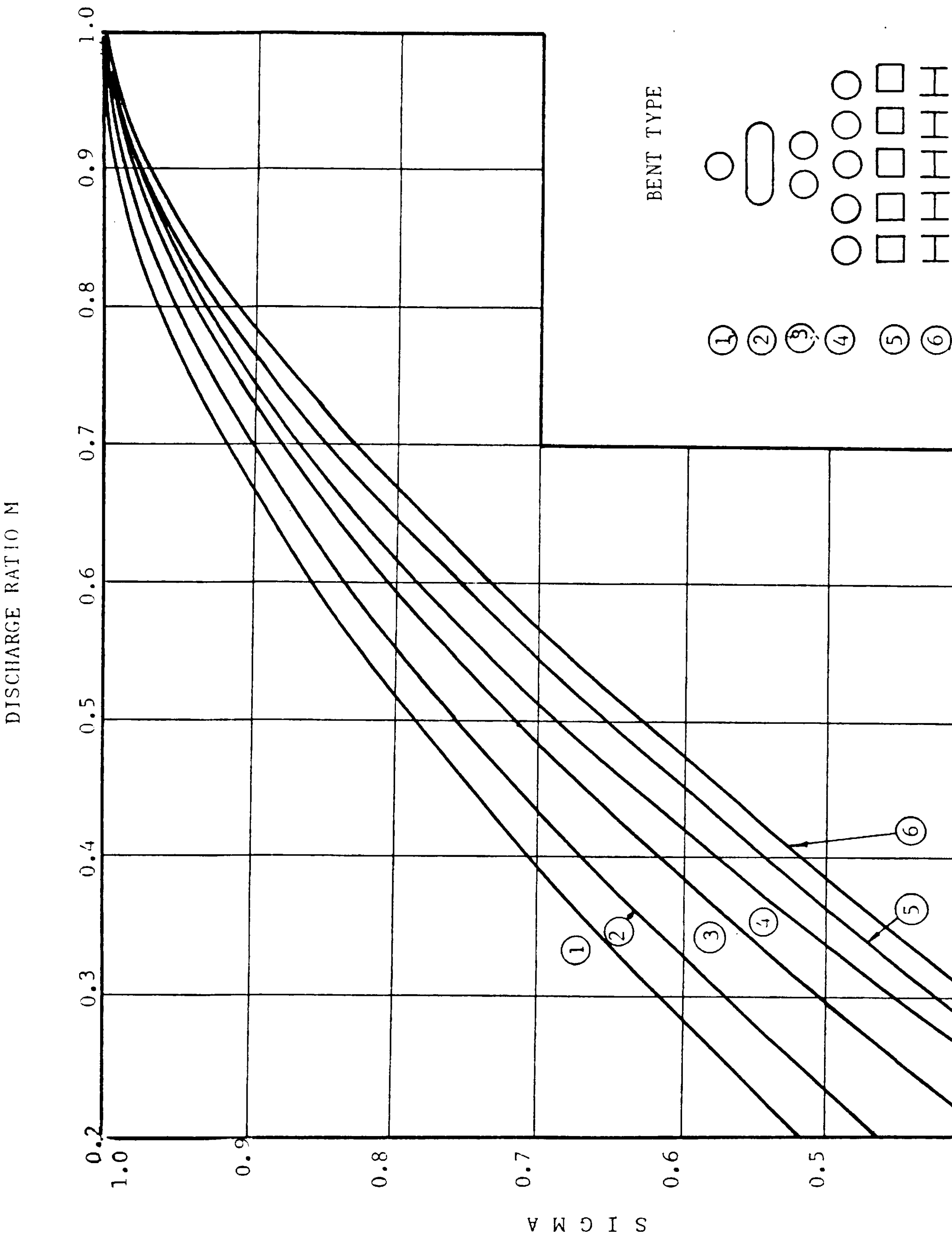
Bridge length, skew and abutment positions.

Major variables used in the computation of backwater.

Final backwater approximation.

The distance to the point of maximum backwater.

l. Flow profile.



DISCHARGE RATIO M
SIGMA
BENT TYPE
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
1
2
3
4
5
6

FIGURE 7-17

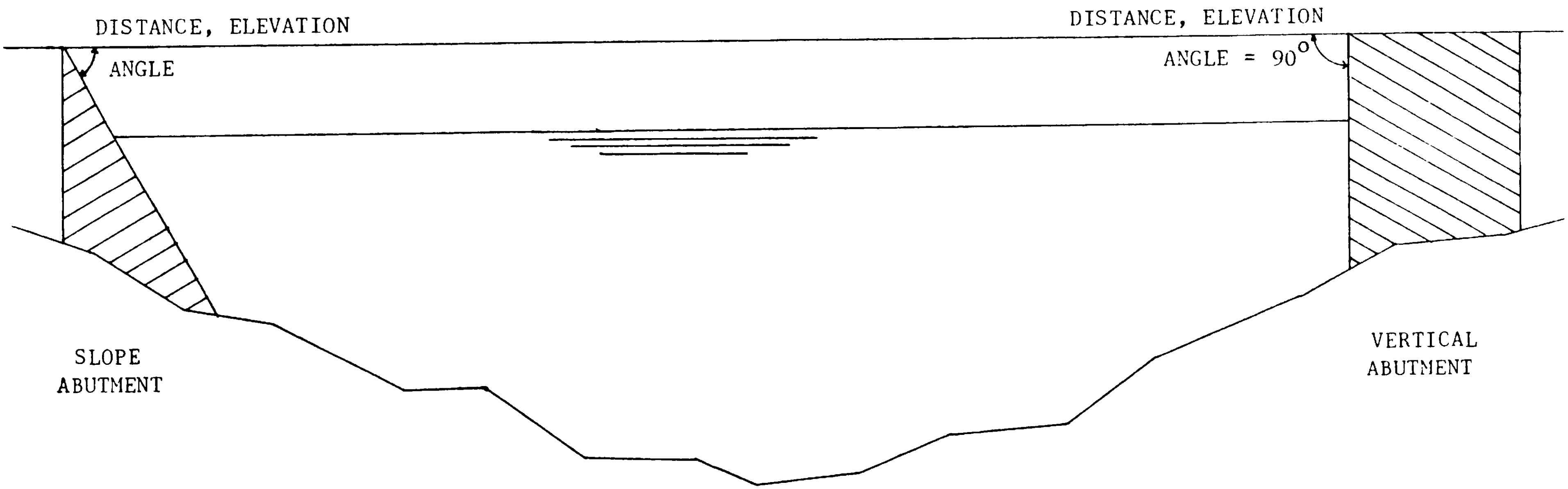

DISTANCE, ELEVATION
ANGLE
SLOPE
ABUTMENT
DISTANCE, ELEVATION
ANGLE = 90°
VERTICAL
ABUTMENT

Figure 7-18

HYDRAULICS OF BRIDGE WATERWAYS

FORM 1 OF 2

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

PROGRAM DEVELOPED BY PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PROJECT IDENTIFICATION

IDENT	REMARKS, PROJECT NUMBER, NAME, ETC.
1	7
=BKWTR	

STREAM DATA

DESIGN DISCHARGE	SLOPE	MIN. STAGE ELEV.	STAGE INCR.	PROFILE INCR.	NUMBER X-SECS	BKWT	ADD.BW	= BW	SCOUR AREA	XYPL	SECT	RATI
1	9	14	19	22	25	27	29	29	31	35	37	39

ENHANCEMENT SEP. 81

XYPL - ENTER A "N" IN COLUMN 36 TO DELETE PLOT OF VALLEY SECTION

SECP - ENTER A "N" IN COLUMN 37 TO DELETE LIST OF SECTION PROPERTIES

RATI - ENTER A "N" IN COLUMN 38 TO DELETE PLOT OF STAGE DISCHARGE CURVE

BRIDGE DATA

SKEW	BR DIST	= MOVES	DIST. PER MOVE	DIRECT	ABUT	CURVE	NUMBER PIERS	PIER	BENT	LD	I	LEFT ABUTMENT			RIGHT ABUTMENT		
												DIST.	ELEV.	ANG.	DIST.	ELEV.	ANG.
1	4	5	6	10	11	12	13	15	15	17	20	22	27	32	35	40	45

PIER DATA

BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.	BASE ELEV.	WTH.
1	6	9	14	17	22	25	30	33	39	41	45	43	54	57	62	55	70

PREPARED BY________________ DATE____/____/____

HYDRAULICS OF BRIDGE WATERWAYS

FORM 2 OF 2

ARKANSAS STATE HIGHWAY AND TRANSPORTATIONS DEPARTMENT

PROGRAM DEVELOPED BY PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

STREAM CROSS SECTION

DIST. UP.	NO. PTS.	SKEW
1	5	7

DIST.	GRD. ELEV.	N	DIST.	GRD. ELEV.	N	DIST.	GRD. ELEV.	N	DIST.	GRD. ELEV.	N	DIST.	GRD. ELEV.	N
1	6	11	15	20	25	29	34	39	43	48	53	57	62	67

PREPARED BY——————————— DATE——/——/——

FORM 1 OF 2

HYDRAULICS OF BRIDGE WATERWAYS

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT
PROGRAM DEVELOPED BY PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

PROJECT IDENTIFICATION

IDENT (1)	REMARKS, PROJECT NUMBER, NAME, ETC. (7)
=BKWTR	EXAMPLE PROBLEM HY/TS813.CEL

STREAM DATA

DESIGN DISCHARGE (1)	SLOPE (9)	MIN. STAGE ELEV. (14)	STAGE INCR. (19)	PROFILE INCR. (22)	NUMBER X-SECS (25)	BKWT (27)	ADD.5W (28)	=BW (29)	SCOUR AREA (31)	XYPL (35)	SECT (37)	RATI (39)
	001.89	25.0	.05			Y	Y	1				

ENHANCEMENT SEP. 81
XYPL - ENTER A "N" IN COLUMN 36 TO DELETE PLOT OF VALLEY SECTION
SECP - ENTER A "N" IN COLUMN 37 TO DELETE LIST OF SECTION PROPERTIES
RATI - ENTER A "N" IN COLUMN 38 TO DELETE PLOT OF STAGE DISCHARGE CURVE

BRIDGE DATA

SKEW (1)	BR DIST (4)	= MOVES (5)	DIST. PER MOVE (6)	DIRECT (10)	ABUT (11)	CURVE (12)	NUMBER PIERS (13)	PIER (15)	BENT (16)	LD (17)	I (20)	LEFT ABUTMENT DIST. (22)	LEFT ABUTMENT ELEV. (27)	LEFT ABUTMENT ANG. (32)	RIGHT ABUTMENT DIST. (35)	RIGHT ABUTMENT ELEV. (40)	RIGHT ABUTMENT ANG. (45)
0	W	0	0		S	1	4	3	4		4.0	150.0	38.0	26.6	350.0	38.0	26.0

PIER DATA

BASE ELEV. (1)	WTH. (6)	BASE ELEV. (9)	WTH. (14)	BASE ELEV. (17)	WTH. (22)	BASE ELEV. (25)	WTH. (30)	BASE ELEV. (33)	WTH. (38)	BASE ELEV. (41)	WTH. (46)	BASE ELEV. (49)	WTH. (54)	BASE ELEV. (57)	WTH. (62)	BASE ELEV. (65)	WTH. (70)
30.0	3.0	20.5	3.0	20.5	3.0	30.2	3.0										

PREPARED BY——————————DATE——/——/——

HYDRAULICS OF BRIDGE WATERWAYS

FORM 2 OF 2

ARKANSAS STATE HIGHWAY AND TRANSPORTATIONS DEPARTMENT
PROGRAM DEVELOPED BY PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

STREAM CROSS SECTION

DIST. UP. (1)	NO. PTS. (5)	SKEW (7)
0	13	0.0

DIST. (1)	GRD. ELEV. (6)	N (11)	DIST. (15)	GRD. ELEV. (20)	N (25)	DIST. (29)	GRD. ELEV. (34)	N (39)	DIST. (43)	GRD. ELEV. (48)	N (53)	DIST. (57)	GRD. ELEV. (62)	N (67)
0.0	40.0	04	25.0	32.0	04	50.0	30.0	04	120.0	30.0	04	200.0	30.0	08
215.0	25.0	035	230.0	19.0	035	250.0	16.0	035	300.0	25.0	035	315.0	30.0	035
375.0	31.0	08	400.0	32.0	04	425.0	40.0	040						

PREPARED BY__________ DATE___/___/___

* * * * * HYDRAULICS OF BRIDGE WATERWAYS * * * * *

* * * INPUT DATA * * *

* * * PROJECT IDENTIFICATION * * *

EXAMPLE PROBLEM (HY/T5813.CEL)

* * * STREAM DATA * * *

DESIGN DISCHARGE	CHANNEL SLOPE	MIN. STAGE ELEV.	STAGE INCREMENT	PROFILE INCREMENT	NO. OF X-SECT	BKWT	ADD BKWT	NO. BKWT	SCOUR AREA
26109.0	.001890	25.0	0.5			Y	Y	1	

XYPL SECP RATI

* * * BRIDGE DATA * * *

SKEW	BRIDGE DISTANCE N OR S	NO. MOVES	DIST PER MOVE	DIRECT MOVED (L/R/B)	ABUT TYPE (W/S)	BKWT CURVE NO.	NO. PIER	PIER CURVE NO.	BENT CURVE NO.	"LD" DUAL BR.	"L" BR. WIDTH
0.0	N	0	0.0		S	1	4	3	4		40

LEFT ABUTMENT			RIGHT ABUTMENT		
DISTANCE	ELEVATION	ANGLE	DISTANCE	ELEVATION	ANGLE
150.0	38.0	26.6	350.0	38.0	26.6

* * * PIER DATA * * *

BASE ELEV	WIDTH	BASE ELEV	WIDTH	BASE ELEV	WIDTH	BASE ELEV	WIDTH	BASE ELEV	WIDTH	BASE ELEV	WIDTH
30.0	3.0	20.5	3.0	20.5	3.0	30.2	3.0				

* * * STREAM CROSS SECTION * * *

DISTANCE UPSTREAM	NO. PTS.	SKEW
0	13	0.0

DIST	ELEV	N	DIST	ELEV	N	DIST	ELEV	N	DIST	ELEV	N	DIST	ELEV	N
00	400	04	250	320	04	500	300	04	1200	300	04	2000	300	08
2150	250	035	2300	190	035	2500	160	035	3000	250	035	3150	300	035
3750	314	080	4000	320	04	4250	400	04						

* * * * * HYDRAULICS OF BRIDGE WATERWAYS * * * * *

EXAMPLE PROBLEM (HY/T5813.CEL)

INPUT DATA

STAGE ELEVATION	=	37.00	FEET
SLOPE OF RIVER	=	0.001890	FEET PER FOOT
DESIGN DISCHARGE	=	26109.00	CFS
SKEW ANGLE	=	0.0	DEGREES

RESULANT DATA

X BEG	X END	MANNINGS N	AREA	WETTED PER	HYD. RADIUS	CONVEYANCE	DISCHARGE	VELOCITY
9.4	25.0	0.0400	39.1	16.4				
25.0	50.0	0.0400	150.0	25.1				
50.0	120.0	0.0400	490.0	70.0				
SUB-SECTION TOTAL * * * *			679.1	111.5	6.1	84416.1	3669.9	5.40
120.0	150.0	0.0800	210.0	30.0				
SUB-SECTION TOTAL * * * *			210.0	30.0	7.0	14321.7	622.6	2.96
150.0	166.0	0.0800	111.8	16.0				
SUB-SECTION TOTAL * * * *			111.8	16.0	7.0	7625.7	331.5	2.96
166.0	200.0	0.0800	238.2	34.0				
SUB-SECTION TOTAL * * * *			238.2	34.0	7.0	16242.9	706.1	2.96
200.0	215.0	0.0350	142.5	15.8				
215.0	230.0	0.0350	225.0	16.2				
230.0	250.0	0.0350	390.0	20.2				
250.0	300.0	0.0350	825.0	50.8				
300.0	315.0	0.0350	142.5	15.8				
SUB-SECTION TOTAL * * * *			1725.0	118.8	14.5	437451.0	19017.8	11.02
315.0	335.0	0.0800	135.0	20.0				
SUB-SECTION TOTAL * * * *			135.0	20.0	6.8	9000.9	391.3	2.90
335.0	350.0	0.0800	95.7	15.0				
350.0	375.0	0.0800	147.3	25.0				
SUB-SECTION TOTAL * * * *			243.0	40.1	6.1	15059.7	654.7	2.69
375.0	400.0	0.0400	132.5	25.0				
400.0	415.6	0.0400	39.1	16.4				
SUB-SECTION TOTAL * * * *			171.6	41.4	4.1	16492.3	717.0	4.18

TOTAL AREA	=	3513.62	SQUARE FEET
TOTAL CONVEYANCE	=	600610.45	CFS
TOTAL DISCHARGE	=	26111.02	CFS

BRIDGE INFORMATION

BRIDGE SKEW = 0.0 DEGREES

BRIDGE LENGTH = 200.00

		DISTANCE	ELEVATION
LEFT ABUTMENT	LEFT FACE	150.00	30.00
	RIGHT FACE	165.98	30.00
RIGHT ABUTMENT	LEFT FACE	334.95	30.47
	RIGHT FACE	350.00	30.82

BRIDGE OPENING AT WATER SURFACE = 196.01

BASE BACKWATER CURVE USED = 1

CALCULATED INFORMATION

PORTION OF DISCHARGE LEFT OF OPENING	(QA)	=	4539.28	CFS
PORTION OF DISCHARGE THRU OPENING	(QB)	=	20270.60	CFS
PORTION OF DISCHARGE RIGHT OF OPENING	(QC)	=	1301.13	CFS
AREA OF PIERS BELOW WATER SURFACE		=	140.40	SQ FEET
ALPHA1		=	1.70	
ALPHA2		=	1.54	
TOTAL BACKWATER COEFFICIENT		=	0.53	
BRIDGE WATERWAY OPENING BELOW NORMAL DEPTH	(AN2)	=	2189.76	SQ FEET
MEAN VELOCITY THRU BRIDGE OPENING	(VN2)	=	11.92	FPS
V2		=	14.51	FPS
DISCHARGE RATIO	(M)	=	0.776	
BACKWATER APPROXIMATION NO.1		=	1.81	FEET
FINAL BACKWATER APPROXIMATION		=	2.37	FEET
NUMBER OF ITERATIONS TO OBTAIN FINAL BACKWATER		=	3	
DISTANCE TO POINT OF MAXIMUM BACKWATER		=	196.	FEET

ERRATA July 1982

Chapter Number 8, Storm Water Management, is not included in this printing of the manual.

It will be furnished to all holders of the manual upon completion.

TABLE OF CONTENTS

Chapter 9

ARKANSAS STORM WATER LAW

9-100 INTRODUCTION

This chapter should not in any way be treated as a manual upon which to base legal advice or make legal decisions. Various drainage laws and rules applicable to highway facilities are discussed. It is not a summary of all existing drainage laws. Most emphatically, this chapter is not intended as a substitute for legal counsel. It is to provide information and guidance on the designer's role in the legal aspects of highway drainage.

The following generalizations can be made in reaching the proper conclusion regarding liability:

1. A goal in highway drainage design should be to perpetuate natural drainage, insofar as practicable.
2. The courts look with disfavor upon infliction of damage that could reasonably have been avoided, even where some alteration in flow is legally permissible.
3. The basic laws relating to the liability of government entities are undergoing radical change, with a trend toward increased government liability.

There are numerous publications on the legal aspects of drainage and water laws. Most of the information included in this chapter was obtained from two principle sources:

1. Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, Washington, D. C., 1979.
2. Arkansas Water Law, Arkansas Soil and Water Conservation Commission, Little Rock, Arkansas, 1981.

9-200 FEDERAL LAWS

Federal law consists of the Constitution of the United States, Acts of Congress, regulations which government agencies issue to implement these acts, Executive Orders issued by the President, and case law. Acts of Congress are published immediately upon issuance in slip law form and are cumulated for each session of Congress and published in the UNITED STATES STATUTES AT LARGE. Compilations of Federal Statutory Law, revised annually, are available in the United States Code (USC) and United States Code Service (USCS).

The Federal Register, which is published daily, provides a uniform system for making regulations and legal notices available to the public. Presidential Proclamations and Executive Orders, Federal agency regulations and documents having general applicability and legal effect, documents required to be published by an act of Congress and other Federal agency documents of public interest are published in the Federal Register. Compilations of Federal regulatory material revised annually, are available in the Code of Federal Regulations (CFR).

Federal law does not deal with drainage per se, but many laws have implications which affect drainage design. These include laws concerning flood insurance and construction in flood hazard areas, navigation and construction in navigable waters, water pollution control, environmental protection, protection of fish and wildlife, and coastal zone management. Federal agencies formulate and promulgate rules and regulations to implement these laws, and highway hydraulics engineers should attempt to keep informed regarding proposed and final regulations.

Some of the more significant Federal laws affecting highway drainage are listed below with a brief description of their subject area.

1. Department of Transportation Act (80 Stat. 941, 49 U.S.C. 1651 et seq.). This Act established the Department of Transportation and sets forth its powers, duties, and responsibilities to establish, coordinate, and maintain an effective administration of the transportation programs of the Federal Government.

2. Federal-aid Highway Acts (23 U.S.C. 101 et seq.). The Federal-Aid Highway Acts provide for the administration of the Federal-aid Highway Program. Proposed Federal-aid projects must be adequate to meet the existing and probable future traffic needs and conditions in a manner conducive to safety, durability, and economy of maintenance, and must be designed and constructed according to standards best suited to accomplish these objectives and to conform to the needs of each locality.

The Federal-aid Highway Act of 1970 (84 Stat. 1713, 23 U.S.C. 109(h)) provides for the establishment of general guidelines to insure that possible adverse economic, social and environmental effects relating to any proposed Federal-aid project have been fully considered in developing the project. In compliance with the Act, the Federal Highway Administration issued process guidelines for the development of environmental action plans. These guidelines are contained in the Federal-aid Highway Program Manual Volume 7, Chapter 7, Section 1 (FHPM 7-7-1), and in 23 CFR 795 et seq.

The Federal-aid Highway Act of 1966 (80 Stat. 766), amended by the Act of 1970 (84 Stat. 1713), 23 U.S.C. 109(g), required the issuance of guidelines for minimizing possible soil erosion from highway construction. In compliance with these requirements, the Federal Highway Administration issued guidelines which are applicable to all Federal-aid highway projects. These guidelines are included in FHPM 6-7-3-1. Regulatory material is found in 23 CFR 650.201.

The following sections contain brief discussions of other Federal laws and regulations current on the date of publication of this guideline which significantly affect highway drainage design.

9-201 FLOOD DISASTER PROTECTION

The Flood Disaster Protection Act of 1973 (PL 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. Formula grants to States are excluded from the definition of financial assistance, and the definition of construction in the Act does not include highway construction; therefore, Federal aid for highways is not affected by the Act. The Act does require communities to adopt certain land use controls in order to qualify for flood insurance. These land use requirements could impose restrictions on the construction of highways in floodplains and floodways in communities, which have qualified for flood insurance. A floodway, as used here and as used in connection with the National Flood Insurance Program, is that portion of the floodplain required to pass a flood that has a 1-percent chance of occurring in any 1-year period with no significant increase in profile due to marginal confinement.

Regulations pertaining to Federal flood insurance are contained in 24 CFR, Housing and Urban Development, Chapter X. This subject is discussed further under Section 9-501, pages 9-12, Flood Insurance.

9-202 NAVIGABLE WATERS

The Congress of the United States is granted constitutional power to regulate "commerce among the several states". A part of that power is the right to legislate on matters concerning the instrumentalities of interstate commerce such as navigable waters. The definition of navigable waters expands and contracts depending upon the breadth required to adequately carry out the Federal purpose. The result is the Congress can properly assert regulatory authority over at least some aspects of waterways that are not in themselves subject to navigation.

Basically four Federal agencies carry out existing Federal regulations.

The Coast Guard has authority under Section 9 of the Rivers and Harbors Act of 1899, 33 U.S.C. 401 (delegated through the Secretary of Transportation in accordance with 49 U.S.C. 1655(g) to approve plans and issue permits for bridges and causeways across navigable rivers. For this purpose, navigable rivers are defined as "those waters that are subject to the ebb and flow of the tide and/or presently being used, or have been used in the past, or may be susceptible to use to transport interstate or foreign commerce" (33 CFR 329).

The Corps of Engineers has regulatory authority over the construction of dams, dikes or other obstructions (which are not bridges and causeways) under Section 9 (33 U.S.C. 401). The Corps also has authority to regulate Section 10 of the River and Harbor Act of 1899 (33 U.S.C. 403) which prohibits the alteration or obstruction of any navigable waterway with the excavation or deposition of fill material in such waterway. Section 11 of the River and Harbor Act of 1899 (33 U.S.C. 404) authorizes the Secretary of the Army to establish harbor lines. Work channelward of those lines requires separate approval of the Secretary of the Army and work shoreward requires Section 10 Permits.

Section 404 of Clean Water Act (33 U.S.C. 1344), prohibits the unauthorized discharge of dredged or fill material into waters of the United States, including navigable waters. Such discharges require a Permit. The term "discharges of fill material" means the addition of rock, sand, dirt, concrete or other material into the waters of the United States incidental to construction of any structure. Under the provisions of 33 CFR 330.5(a)(15), fill associated with construction of bridges across navigable waters of the

United States, including cofferdams, abutments, foundation seals, piers, temporary construction and access fills are authorized under the Nationwide Section 404 Permit providing such fill has been permitted by the U.S. Coast Guard under Section 9 of the River and Harbor Act of 1899 as part of the bridge permit. Therefore, formal application to the Corps of Engineers for a Section 404 Permit is not required unless bridge approach embankment is located in a wetland area contiguous to said navigable stream. The Corps of Engineers has Section 404 regulatory authority over streams the Coast Guard has placed in the "advance approval" category. This category of navigable streams is defined as navigable in law but not actually navigated other than by logs, log rafts, rowboats, canoes and motorboats.

The Federal Highway Administration has the authority to implement the Section 404 Permit Program (Clean Water Act of 1977) for Federal-aid highway projects processed under 23 CFR 771.115(b) categorical exclusions. This authority was delegated to the Federal Highway Administration by the Corps of Engineers to reduce unneccessary Federal regulatory controls over activities adequately regulated by another agency. This permit is granted for projects where the activity, work or discharge is categorically excluded from environmental documentation because such activity does not have individual or cumulative significant effect on the human environment.

Section 404 of the Clean Water Act (33 U.S.C. 1344) requires any applicant for a Federal Permit for any activity that may affect the quality of waters of the United States to obtain water quality certification from the Arkansas Department of Pollution Control and Ecology.

The Environmental Protection Agency (EPA) is authorized to prohibit the use of any area as a disposal site when it is determined that the discharge of materials at the site will have an unacceptable adverse effect on municipal water supplies, shellfish beds and fishery areas, wildlife, or recreational areas (Section 404 (c), Clean Water Act (33 U.S.C. 1344).

Federally recognized navigable streams in Arkansas requiring a Coast Guard Permit are listed in "Applications for Coast Guard Bridge Permits", published by the U.S. Coast Guard, Second Coast Guard District, 1430 Olive Street, St. Louis, Missouri, 63103. A copy of this publication is in the Hydraulics Section's Library.

9-203 FISH AND WILDLIFE

The Fish and Wildlife Act of 1956 (16 U.S.C. 742 et seq.), the Migratory Game-Fish Act (16 U.S.C. 760c-760g), and the Fish and Wildlife Coordination Act (16 U.S.C. 611-666c) express the concern of Congress with the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. The Fish and Wildlife Coordination Act requires that "whenever the waters of any stream or body of water are proposed or authorized to be impounded, diverted, the channel deepened, or the stream or other body of water otherwise controlled or modified for any purpose whatever, including navigation and drainage, by any department or agency of the United States, or by any public or private agency under Federal permit or license, such department or agency shall first consult with the United States Fish and Wildlife Service, Department of the Interior, and with the head of the agency exercising administration over the wildlife resources of the particular state with a view to the conservation of wildlife resources by preventing loss of and damage to such resources as well as providing for the development and improvement thereof."

The Fish and Wildlife Service's role in the permit review process is to review and comment on the effects of a proposal on fish and wildlife resources. It is the function of the regulatory agency (e.g. Corps of Engineers, U. S. Coast Guard) to consider and balance all factors, including anticipated benefits and costs in accordance with NEPA, in deciding whether to issue the permit (40 FR 55810), December 1, 1975).

9-204 EXECUTIVE ORDERS

Presidential Executive Orders (E.O.) have the effect of law in the administration of programs by Federal agencies. Executive Order 11296, issued in August 1966 because of ever increasing flood losses, directed Federal agencies to avoid uneconomic, hazardous and unnecessary use of flood plains. In May 1972, the Water Resources Council (WRC) published "Guidelines for Federal Executive Agencies for Flood Hazard Evaluations" containing guidance for the implementation of provisions of the Executive Order. Federal-aid highway drainage designs, to qualify for Federal-aid participation, must meet minimum requirements established to comply with the provisions of the Executive Order and the Water Resources Council Guidelines. These requirements are contained in the Federal-aid Highway Program Manual, Volume 6, Chapter 7, Section 3, Subsection 2, and were published in the Federal Register, April 26, 1979 (44 FR 24678), and in 23 CFR 650.

E. O. 11988, May 24, 1977, requires each Federal agency, in carrying out its activities, to take action (1) to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains; (2) to evaluate the potential effect of any actions it may take in a floodplains, to ensure its planning programs reflect consideration of flood hazards and floodplain management; and (3) submit a report to the CEQ and the WRC on the status of procedures and the impact of the Order on the agency's operations. E. O. 11988 revoked E. O. 11296 and all actions, procedures and issuances taken under E. O. 11296 remain in effect until modified to implement E. O. 11988.

E. O. 11990, May 24, 1977, orders each Federal agency (1) to take action to minimize the destruction, loss or degradation of wetlands, and to preserve and enhance the natural and beneficial values of wetlands; (2) to avoid undertaking or providing assistance for new construction in wetlands unless the head of the agency finds that there is no practicable alternative and all practicable measures are taken to minimize harm which may result from the action; (3) to consider factors relevant to the proposal's effects on the survival and quality of the wetlands; and (4) to amend existing or issue new procedures to comply with the Order.

9-300 ARKANSAS LAW

9-301 CONSTITUTION

The Constitution of Arkansas has no provision regarding the use and/or diversion of either surface water or waterways.

9-302 COMMON LAW

The basic statement of Arkansas common law (case law) is that every owner of land through which a stream of water flows (riparian owner) is entitled to the use and enjoyment of the water and to have it flow in its natural and accustomed course without obstruction, diversion, or corruption. This law has developed around three principal issues: allocation, pollution, maintenance of the natural flow. Case law is controlling unless altered by specific statutory enactment.

9-302.1 ALLOCATION

Like most states in the Eastern United States where water is plentiful, Arkansas has adopted the "reasonable use" doctrine for allocating surface waters. All riparian owners share an equal right to use the stream. There is no special priority for prior appropriation of upstream use. Users are held to a standard of "reasonableness", the focus of which is no undue interference with other users. Arkansas, early in its history, recognized a priority for domestic use over irrigation and industrial uses in time of shortage. Also the Water Conservation Commission is authorized by statute, Ark. Stat. 21-1301 et. seq. to allocate water in time of drought.

9-302.2 POLLUTION

Riparian owners share a right to enjoy their surface waters without "corruption", i.e. pollution. A downstream owner who is injured as a result of a change in water quality caused by an upstream use can maintain a suit for damages. Problems are primarily those of proof: identifying the source of the pollution; identifying the value of the injury. It is not clear whether a downstream owner in Arkansas can legally enjoin any alteration in water quality or whether an owner's interest in limited to situations in which the pollution damages the owner's ability to enjoy a prior existing use. In the last ten or fifteen years source pollution has become intensively regulated under the Federal Water Pollution Control Act, as amended (PL 92-500).

9-302.3 MAINTENANCE OF THE NATURAL FLOW

Arkansas recognizes an almost "pure" doctrine of natural flow. Not surprisingly the focus in the case law is not on the alteration of the flow of surface waters but on damage caused to others by any change in the flow. Generally a property owner is entitled to both fend off surface waters and otherwise alter the natural flow of water courses on his own land so long as his actions do not damage fellow riparian owners. If he constructs artificial ditches, culverts, etc., he undertakes an obligation to keep them open. Moreover, in designing such artificial structures he is under an obligation to design for all reasonably foreseeable contingencies. No one is obliged, however, to construct any artificial improvements to handle the flow from his neighbor's land. It is unclear if a riparian owner is required to prevent natural obstructions of a natural drain through his property. In time of flood a property owner is allowed to take whatever means are necessary to fend off this extraordinary amount of water so long as he has no intent to harm other owners. Within drainage districts the mutual obligations of riparian owners are controlled by statue (Ark. Stat. 21-501 et seq.).

9-400 EMINENT DOMAIN

The Arkansas Constitution grants the state the right of eminent domain which allows that taking of property for public purposes, including the development of watercourse and watershed areas. It is important to remember, however, that whenever any property is taken under eminent domain, the private landowner must be compensated for his loss.

There are numerous statutory provisions delegating the right of eminent domain. The Arkansas Soil and Water Conservation Commission has the right to condemn land, rights-of-way and easements for its use expect within the boundaries of any levee or drainage district now existing or hereafter organized.

County governments have the right of eminent domain to construct, operate, repair or maintain any floodway, reservoir spillway, levee or diversion, or other flood control improvements.

Similarly, any levee or drainage district, through its Board of Directors, has eminent domain powers as long as it is declared necessary by the Chief of Engineers, United States Army, for the location, construction, operation or maintenance of any levee, channel rectrification, drainage canal, floodway, reservoir, spillway or diversion to be constructed by the United States Government. Several other entities have the power of eminent domain if the construction of public water supplies is involved.

9-500 LOCAL LAWS

Local governments (cities, counties, improvement districts) have ordinances and codes which require consideration during design. For example, zoning ordinances can have a substantial effect on the design of a highway and future drainage from an area.

On occasion, a question may arise as to whether the State must comply with local ordinances. Generally, the State is not legally required to comply with local ordinances except where compliance is required by specific State statute. Quite often, however, the State conforms with local ordinances as a matter of courtesy especially when it can be done without imposing a burden on the State.

9-501 FLOOD INSURANCE

The National Flood Insurance Act of 1968, as amended, (42 U.S. C. 4001-4127) requires that communities adopt adequate land use and control measures to qualify for insurance. Federal criteria promulgated to implement this provision (24 CFR 1909) contain the following requirements which can affect certain highways:

1. In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require

that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it is demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood more than 1 foot at any point within the community.

2) After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 foot at any point and prohibit, within the designated floodway, fill, encroachments, and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.

3) The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistant with the intent, standards and criteria set by the National Flood Insurance Program.

9-501.1 COMPLIANCE BY THE ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

There is no requirement that the State qualify for flood insurance. However, if the State is to qualify it must comply with 44 CFR, 60.12. According to that regulation a State shall either comply with the floodplain management requirements of all local communities participating in the program in which State owned properties are located or establish and enforce its own floodplain management regulations, which, at a minimum, satisfy the criteria set forth in 44 CFR, 60.3, 60.4, and 60.5.

Pursuant to State Executive Order signed by Governor David Pryor and effective May 20, 1977, the Arkansas State Building Services Council promulgates rules and regulations for floodplain management. All state agencies are required to follow the procedures set by the Arkansas State Building Services Council except the Arkansas State Game and Fish Commission and the Arkansas State Highway and Transportation Department. Since the Highway Department, under Arkansas law, is not required to follow the state floodplain management regulations, the Department is required to adhere to the restrictions of the local communities in order for the state to qualify for flood insurance.

Adhering to the local communities' requirements is the only prudent course for the State. This will minimize, if not eliminate any potential liability and will relieve the State from any embarrassment for failure to not conform to regulations, causing a local community to be disqualified from the flood insurance program (Reference 9-5).

9-600 AGENCIES OF ARKANSAS GOVERNMENT

Most Arkansas State institutions and agencies are under the administration of the Governor's office with the exception of the Arkansas State Highway and Transportation Department and the Arkansas Game and Fish Commission. In addition, however, local boards and political entities derive authority from the municipalities and counties in which they are situated. The following is a list of some of these agencies that relate to watercourse and surface water regulations:

a. Arkansas Soil and Water Conservation Commission
b. Arkansas Waterway Commission
c. Arkansas Department of Pollution Control and Ecology
d. Arkansas Game and Fish Commission
e. Arkansas Geological Commission
f. White River Navigation District Commission
g. Department of Arkansas Natural and Cultural Heritage
h. Red River Commission
i. Water Resources Research Center

REFERENCES

9-1 Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, Washington, D. C., 1979.

9-2 Arkansas Water Law, Arkansas Soil and Water Conservation Commission, Little Rock, Arkansas, 1981.

9-3 Federal-Aid Highway Program Manual, Transmittal 315, FHPM 6-7-3-2, Washington, D. C., 1979.

9-4 The Floodway: A Guide for Community Permit Officials, Federal Emergency Management Agency, Federal Insurance Administration, Washington, D. C., Community Assistance Series Number 4.

9-5 LEGAL OPINION Of The Arkansas State Highway Department, Little Rock, Arkansas, December 2, 1980.

G L O S S A R Y

ABRASION - Wear or scour by hydraulic traffic.

ABSORPTION - The assimilation or taking up of water by soil.

AGGRADATION - General and progressive raising of the stream bed by deposition of sediment.

ALLUVIAL - Referring to deposits of silts, sands, gravels and similar detrital material which have been transported by running water.

ANTECEDENT MOISTURE - The degree of wetness of the soil at the beginning of a runoff period; frequently expressed as an index determined by summation of weighted daily rainfalls for a period preceding the runoff in question.

BACKWATER - The rise in water surface measured at a specified location upstream from the constriction causing the increased height.

BASE FLOOD - The flood or tide having a 1-percent chance of being exceeded in any given year (commonly known as a 100-year flood).

BASE FLOODPLAIN - The area subject to flooding by the base flood.

BED LOAD - Sediment that moves by rolling, sliding, or skipping along the bed and is essentially in contact with the stream bed.

BRAIDED STREAM - A stream in which flow is divided at normal stage by small islands. This type of stream has the aspect of a single large channel within which there are subordinate channels.

BUOYANCY - The power of supporting a floating body, including the tendency to float an empty pipe (by exterior hydraulic pressure).

CAPILLARY RISE - The height above a free water elevation to which water will rise by capillary action.

CAPILLARY SUCTION - Capillary force that pulls or draws water against the force of gravity in dry soils.

CAISSON - Watertight box or cylinder used in excavating for foundations or tunnel pits - to hold out water so concreating or other construction can be carried on.

COFFERDAM - A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the areas within.

COMPOSITE HYDROGRAPH - A plot of mean daily discharges for a number of years of record on a single year time base for the purpose of showing the occurrence of high and low flows.

CONDUIT - A pipe of other opening, buried or above ground for conveying hydraulic traffic, pipelines, cables or other utilities.

CONFLUENCE - A function of streams.

CONTRACTION - The reduction in cross sectional area of flow.

CONTROL - A section or reach of an open conduit or stream channel which maintains a stable relationship between stage and discharge.

CONVEYANCE - A measure of the water carrying capacity of a stream or channel.

CRITICAL FLOW - That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of one.

CULVERT - A conduit for conveying water through an embankment.

CURRENT METER - An instrument for measuring the velocity of a current. It is usually operated by a wheel equipped with vanes or cups which is rotated by the action of the impinging current. An indicating or recording device is provided to indicate the speed of rotation which is correlated with the velocity of the current.

DEBRIS - Any material including floating woody materials and other trash, suspended sediment, or bed load, moved by a flowing stream.

DEFLECTION - Change in shape or decrease in diameter of a conduit, produced without fracture of the material.

DEGRADATION - General and progressive lowering of the longitudinal profile of a channel by erosion.

DESIGN FLOOD - The peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood.

DETENTION BASIN - An open excavation of depression in the ground surface used for temporary storage of storm water prior to release downstream.

DETENTION TANK - A tank used to temporarily store storm water underground. Inlet and outlet flow controls are usually provided and tank can be perforated to exfiltrate water into soil during the detention time.

DIAPHRAGM - A metal collar at right angles to a drain pipe for the purpose of retarding seepage or the burrowing of rodents.

DIKE - An embankment to confine or control water, especially one built along the banks of a river to prevent overflow of low lands or to deflect water away from a bank. Also called Levee.

DIKE, FINGER - Relatively short embankments constructed normal to a larger embankment, such as an approach fill to a bridge. Their purpose is to impede flow and direct it away from the major embankment.

DIKE, SPUR - Relatively short embankments constructed at the upstream side of a bridge end for the purpose of aligning flow with the waterway opening and to move scour away from the bridge abutment.

DIKE, TOE - Embankments constructed to prevent lateral flow from scouring the corner of the downstream side of an abutment embankment. Sometimes referred to as training dikes.

DIKE, TRAINING - Embankments constructed to provide a transition from the natural stream channel or floodplain, both to and from a constricting bridge crossing.

DRAWDOWN - The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and at weirs or vertical spillways.

DYNAMIC EQUILIBRIUM - That delicate balance of the many factors which must occur in a stream reach so that the channel is neither aggrading or degrading.

ENCROACHMENT - Extending beyond the original, or customary limits, such as by occupancy of the river and/or floodplain by earth fill embankment. An action within the limits of the base floodplain.

ENERGY GRADE LINE - The line which represents the total energy gradient along the channel. It is established by adding together the potential energy expressed as the water surface elevation referenced to a datum and the kinetic energy (usually expressed as velocity head) at points along the flowing water.

ENERGY HEAD - The elevation of the hydraulic gradient at any section, plus the velocity head.

EQUALIZER - A culvert placed where there is no channel but where it is desirable to have standing water at equal elevations on both sides of a fill.

FLOOD FREQUENCY - Also referred to as exceedance interval, recurrence interval or return period; the average time interval between actual occurrences of a hydrological event of a given or greater magnitude; the percent chance of occurrence is the reciprocal of flood frequency, e.g., a 2 percent chance flood is the reciprocal statement of a 50-year flood.

FLOODPLAIN - Normally dry land areas subject to periodic temporary inundation by stream flow or tidal overflow. Land formed by deposition of sediment by water; alluvial land.

FLOODPROOF - To design and construct individual buildings, facilities, and their sites to protect against structural failure, to keep water out or to reduce the effects of water entry.

FLOW, CRITICAL - That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of one.

FLOW, SUBCRITICAL - In this state, gravity forces are dominant, so that the flow has a low velocity and is often described as tranquil and streaming. Also, that flow which has a Froude number less than one.

FLOW, SUPERCRITICAL - In this state, inertia forces are dominant, so that flow has a high velocity and is usually described as rapid, shooting and torrential. Also, that flow which has a Froude number greater than one.

FLOW REGIME. The system or order characteristic of streamflow with respect to velocity, depth and specific energy.

FLUME - An open channel or conduit of metal, concrete or wood, on a prepared grade, trestle or bridge.

FORD - A shallow place where a stream may be crossed by traffic.

FREEBOARD - The vertical clearance of the lowest structural member of the bridge superstructure above the water surface elevation of the overtopping flood; the vertical distance between the level of the water surface usually corresponding to the design flow and a point of interest such as a levee top or specific location on the roadway grade.

FREE OUTLET - (as pertaining to critical flow) - Exists when the backwater does not diminish the discharge of a conduit.

FRENCH DRAIN - An underground passageway for water through interstices among stones placed loosely in a trench.

FROUDE NUMBER - A dimensionless expression of the ratio of inertia forces to gravity forces, used as an index to characterize the type of flow in a hydraulic structure in which gravity is the force producing motion and interia is the resisting force. It is equal to a characteristic flow velocity (mean, surface, or maximum) of the system divided by the square root of the product of a characteristic dimension (as diameter or depth) and the gravity constant (acceleration due to gravity) all expressed in consistent units. $F_r = V/(gy)^{0.5}$

GAGING STATION - A location on a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is uniform, a control downstream from this reach and usually a small building to house the recording instruments.

GRADUALLY VARIED FLOW - In this type of flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces are neglected.

GROUNDWATER - Subsurface water occupying the saturation zone, from which wells and springs are fed. In a strict sense the term applies only to water below the water table. Also called phreatic water, plerotic water.

GROUNDWATER RECHARGE - Water descending to the zone of saturation from the atmosphere which gravitates to the zone of saturation under natural conditions or which is added to the zone of saturation by infiltration of storm water using subsurface disposal systems as defined herein.

GROUNDWATER TABLE - (or level) - Upper surface of the zone of saturation in permeable rock or soil. (When the upper surface is confined by impermeable rock, the water table is absent.)

HEAD - The energy, either kinetic or potenial, possessed by each unit weight of a liquid expressed as the vertical height through which a unit weight would have to fall to release the average energy possessed.

HYDRAULIC RADIUS - The cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduits; the ratio of area to wetted perimeter.

HYDROGRAPH - A graph of stage or discharge versus time.

IMPERVIOUS - Not allowing, or allowing only with great difficulty, the movement of water; impermeable. Completely resisting entrance of fluids.

INFILTRATION - The passage of water through the soil surface into the ground. Normally used interchangeably with the word percolation.

INLET TIME - The time required for storm runoff to flow from the most remote point of a drainage area to the point where it enters a drain or culvert.

INUNDATE - To cover or fill as with a flood.

INVERT - That part of a pipe or sewer below the springing line - generally the lowest point of the internal cross section.

MEANDER - In connection with streams, a winding channel usually in an erodible, alluvial valley. A reverse of S-shaped curve or series of curves formed by erosion of the concave bank, especially at the downstream end, characterized by curved flow and alternating shoals and bank erosion. Meandering is a stage in the migratory movement of the channel as a whole down the valley.

NATURAL AND BENEFICIAL FLOODPLAIN VALUES - Include but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

NONUNIFORM FLOW - A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc.

NORMAL WATER SURFACE (NATURAL WATER SURFACE) - The free surface associated with flow in natural streams.

OPEN CHANNEL - Any conveyance in which water flows with a free surface.

OUTFALL (or outlet) - In hydraulics, the discharge end of drains and sewers.

OVERTOPPING FLOOD - The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

PERIPHERY - Circumference or perimeter of a circle, ellipse, pipearch, or other closed curvilinear figure.

PERMEABILITY - The property of a material that permits appreciable movement of water through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.

PERVIOUS SOIL - Soil containing voids through which water will move under ordinary hydrostatic pressure.

PONDING - Refers to water backed up in a channel or ditch as the result of a culvert of inadequate capacity or design to permit the water to flow unrestricted.

PRACTICABLE - Capable of being done within reasonable natural, social, or economic constraints.

PRESERVE - To avoid modification to the functions of the natural floodplain environment or to maintain it as closely as practicable in its natural state.

RAINFALL INTENSITY - Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour at the same rate.

RAPIDLY VARIED FLOW - In this type of flow, changes in depth and velocity take place over short distances, acceleration forces dominate, and energy loss due to friction is minor.

REACH - A length of stream channel.

RECHARGE - Addition of water to the zone of saturation from precipitation or infiltration.

RECHARGE BASIN - A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.

REGIME - The system or order characteristic of a stream; its behavior with respect to velocity and volume, form of and changes in channel, capacity to transport sediment, amount of material supplied for transportation, etc.

REGULATORY FLOODWAY - The floodplain area that is reserved in an open manner by Federal, State of local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Managment Agency (FEMA) for administering the National Flood Insurance Program).

RESTORE - To reestablish a setting or environment in which the functions of the natural and beneficial floodplain values adversely impacted by the highway agency action can again operate.

RISK - The consequences associated with the probability of flooding attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the highway.

RISK ANALYSIS - An economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least total expected cost to the public. It shall include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway-aggravated flood damage to other property, and for additional or interrupted highway travel.

ROUGHNESS COEFFICIENT - A factor in the Kutter, Manning, and other flow formulas representing the effect of channel (or conduit) roughness upon energy losses in the flowing water.

RUNOFF - That part of the precipitation which runs off the surface of a drainage area and reaches a stream or other body of water or a drain or sewer.

SCOUR - The result of erosive action of running water, primarily in streams, excavating and carrying away material from the bed and banks.

SCOUR, GENERAL - The removal of material from the bed and banks across all or most of the width of a channel, as a result of a flow contraction which causes increased velocities and bed shear stress. Also known as CONTRACTION SCOUR.

SCOUR, LOCAL - Removal of material from the channel bed or banks which is restricted to a minor part of the width of a channel. This scour occurs around piers and embankments and is caused by the actions of vortex systems induced by the obstructions to the flow.

SCOUR, NATURAL - Removal of material from the channel bed or banks which occurs in streams with the migration of bed forms, shifting of the thalweg and at bends and natural contractions.

SEDIMENT - Fregmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited by water.

SEDIMENTATION BASIN - A basin or tank in which storm water containing settleable solids is retained to remove by gravity a part of the suspended matter.

SIGNIFICANT ENCROACHMENT - A highway encroachment and any direct support of likely base floodplain development that would involve one or more of the following construction - or flood-related impacts:

(1) a significant potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route,

(2) a significant risk, or

(3) a significant adverse impact on natural and beneficial floodplain values.

SIPHON - (inverted) - A conduit or culvert with a U or V shaped grade line to permit it to pass under an intersecting roadway, stream or other obstruction.

SPECIFIC ENERGY - The energy contained in a stream of water, expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity.

SPILLWAY - A low-level passage serving a dam or reservoir through which surplus water may be discharged; usually an open ditch around the end of a dam, or a gateway or a pipe in a dam. --An outlet pipe, flume or channel serving to discharge water from a ditch, ditch check, gutter or embankment protector.

SPRING BOX - An enclosure constructed to protect a flow of water emerging from the ground.

STAGE - The elevation of a water surface above a datum of reference.

STEADYFLOW - A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant.

STORAGE BASIN - A basin excavated in the earth for detention or retention of water for future flow.

SUBCRITICAL FLOW - Flow with a Froude number less than one. In this state; the role played by gravity forces is more pronounced, so the flow has a low velocity and is often described as tranquil and streaming.

SUPERCRITICAL FLOW - Flow with a Froude number greater than one. In this state, the inertia forces become dominant, so the flow has a high velocity and is usually described as rapid, shooting and torrential.

SUPPORT BASE FLOODPLAIN DEVELOPMENT - To encourage, allow, serve, or otherwise facilitate additional base floodplain development. Direct support results from an encroachment, while indirect support results from an action out of the base floodplain.

SURFACE STORAGE - Storm water that is contained in surface depressions or basins.

SURFACE WATER - Water appearing on the surface in a diffused state, with no permanent source of supply or regular course for a considerable time; as distinguised from water appearing in water courses, lakes, or ponds.

SWALE - A slight depression in the ground surface where water collects.

SYNTHETIC HYDROGRAPH - A graph developed for an ungaged drainage area, based on known physical characteristics of the watershed basin.

TIME OF CONCENTRATION - Time required for storm water runoff to arrive at the point of concentration (usually the inlet to the storm drain) from the most remote point of the drainage area.

TAILWATER - The water surface just downstream from a structure.

UNIFORM FLOW - Flow in which the velocities are the same in both magnitude and direction from point to point along the stream or conduit, all stream lines being parallel.

UNSTEADY FLOW - A flow in which the velocity changes with respect to both space and time.

UNIT HYDROGRAPH - A hydrograph of a direct runoff resulting from 1-inch of effective rainfall generated uniformly over the watershed area during a specified period of time or duration.

WATER COURSE - A natural or artificial channel in which a flow of water occurs, either continuously or intermittently. Natural water courses may be either on the surface or underground.

WATERSHED - Region or area contributing to the supply of a stream or lake; drainage area, drainage basin, catchment area.

WETTED PERIMETER - The length of the wetted contact between the water prism and the containing conduit, (measured along a plane at right angles to the conduit).

ZERO INCREASE IN DISHCHARGE - A storm sewer management concept that suggests no increase in runoff as a result of new development. Any increased flow generated by the development would be taken care of by subsurface disposal (infiltration).