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# UNIFIED FACILITIES CRITERIA (UFC)

# SURFACE AND SUBSURFACE DRAINAGE

U.S. Department of Transportation
**Federal Aviation Administration**

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# UNIFIED FACILITIES CRITERIA (UFC)

# SURFACE AND SUBSURFACE DRAINAGE[1]



U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by **1**\ ... /**1**/)

| **Change No.** | **Date** | **Location** |
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[1]This UFC supersedes TM 5-820-1/AFM 88-5, Chap 1, dated August 1987; TM 5-820-2/AFM 88-5, Chap 2, dated March 1979; TM 5-820-3/AFM 88-5, Chap 3, dated June 1991; TM 5-820-4/AFM 88-5, Chap 4, dated October 1983; TM 5-852-7/AFM 88-19, Chap 7, dated April 1981; NAVFAC DM 21.06, dated April 1986; EI 02C202, dated October 1995.

# FOREWORD

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- Commander, Atlantic Division, Naval Facilities Engineering Command, 1510 Gilbert Street (ATTN: NAVFAC Engineering Innovation and Criteria Office), Norfolk, Virginia 23511-2699, or ufc@efdlant.navfac.navy.mil, by commercial telephone (757) 322-4200 or DSN 262-4200, or by facsimile machine to (757) 322-4416
- Air Force Civil Engineer Support Agency, 139 Barnes Drive, Tyndall Air Force Base, Florida 32403-5319, or larry.spangler@Tyndall.af.mil.

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# CHAPTER 1

# INTRODUCTION

1-1 **PURPOSE**. This document establishes general concepts and procedures for the hydrologic design of surface and subsurface structures for the U.S. Army, Navy, Air Force, Marine Corps, and Federal Aviation Administration (FAA).

1-2 **SCOPE**. This UFC applies to all service elements and contractors preparing UFC.

1-3 **REFERENCES**. Appendix A contains a list of references used in this UFC.

1-4 **UNITS OF MEASUREMENT**. The unit of measurement system in this document is the International System of Units (SI). In some cases inch-pound (IP) measurements may be the governing critical values because of applicable codes, accepted standards, industry practices, or other considerations. Where the IP measurements govern, the IP values may be shown in parenthesis following a comparative SI value or the IP values may be shown without a corresponding SI value.

1-5 **APPLICABILITY**. This document covers a wide range of topics in the areas of surface and subsurface drainage and serves as the standard for several agencies responsible for hydrologic design for airfields and areas other than airfields. The intended use of the facility under design may differ between agencies and in some cases dictates the need for separate standards. In special cases where more than one standard is presented, or the standard does not apply to all agencies, special care has been given to clearly identify the relevant audience. Any user of this manual should pay close attention to the relevance of each topic to the intended agency.

1-6 **GENERAL INVESTIGATIONS.** An on-site investigation of the system site and tributary area is a prerequisite for study of drainage requirements. Information regarding capacity, elevations, and condition of existing drains will be obtained. Topography, size and shape of drainage area, and extent and type of development; profiles, cross sections, and roughness data on pertinent existing streams and watercourses; and location of possible ponding areas will be determined. Thorough knowledge of climatic conditions and precipitation characteristics is essential. Adequate information regarding soil conditions, including types, permeability on perviousness, vegetative cover, depth to and movement of subsurface water, and depth of frost will be secured. Outfall and downstream flow conditions, including high-water occurrences and frequencies, also must be determined. Effect of base drainage construction on local interests' facilities and local requirements that will affect the design of the drainage works will be evaluated. Where diversion of runoff is proposed, particular effort will be made to avoid resultant downstream conditions leading to unfavorable public relations, costly litigations, or damage claims. Any agreements needed to obtain drainage easements and/or avoid interference with water rights will be determined at the time of design and consummated prior to initiation of construction. Possible adverse effects on water quality due to disposal of drainage in waterways involved in water-supply systems will be evaluated.

## 1-7 ENVIRONMENTAL CONSIDERATIONS

1-7.1 **National Environmental Policy**. The National Environmental Policy Act of 1969 (NEPA), approved 1 January 1970, sets forth the policy of the Federal Government, in cooperation with State and local governments and other concerned public and private organizations, to protect and restore environmental quality. The Act (Public Law 91-190) states, in part, that Federal agencies have a continuing responsibility to use all practicable means, consistent with other essential considerations of national policy, to create and maintain conditions under which man and nature can exist in productive harmony. Federal plans, functions and programs are to be improved and coordinated to (1) preserve the environment for future generations, (2) assure safe, healthful, productive, and aesthetically pleasing surroundings for all, (3) attain the widest beneficial uses of the environment without degradation, risk to health or safety or other undesirable consequences, …and (4) enhance the quality of renewable resources and approach the maximum attainable recycling of depletable resources. All Federal agencies, in response to NEPA, must be concerned not just with the impact of their activities on technical and economic considerations but also on the environment.

1-7.2 **Executive Orders**. Executive Order 11514 of 5 March 1970 states that, “The Federal Government shall provide leadership in protecting and enhancing the quality of the Nation’s environment to sustain and enrich human life. Federal agencies shall initiate measures to direct their policies, plans, and programs so as to meet national environmental goals.” Executive Order 11752 of 17 December 1973 enunciates its purpose “to assure that the Federal Government in the design, construction, management, operation, and maintenance of its facilities shall provide leadership in the nationwide effort to protect and enhance the quality of our air, water, and land resources….”

1-7.3 **Environmental Considerations in DOD Actions**. DOD Directive 6050.1, 19 March 1964, establishes policy of the Department of Defense, as trustee of the environment, to demonstrate leadership and carry out its national security mission in a manner consistent with national environmental policies and host country environmental standards, laws, and policies. The directive requires that DOD components will:

“1. Assess at the earliest practical stage in the planning process and in all instances prior to the first significant point of decision, the environmental consequences of proposed actions.

“2. Review those continuing actions initiated prior to enactment of P.L. 91-190 for which the environmental consequences have not been assessed and ensure that any of the remaining actions are consistent with the provisions of the directive.

"3. Utilize a systematic interdisciplinary approach in planning and decision making.

"4. Prepare and process under the criteria contained in the directive a detailed environmental impact statement on every recommendation or report on proposals for legislation and other major defense actions which are expected to be environmentally controversial or could cause a significant effect on the quality of the human environment.

"5. Study, develop and describe appropriate alternatives to the recommended courses of action in any proposal which involves unresolved conflicts concerning alternative uses of available resources."

1-7.4 **U.S. Army Environmental Quality Program.** AR 200-1, outlines the Army's fundamental environmental policies, management of its programs, and its various types of activities, one of which, water resources management, includes minimizing soil erosion and attendant pollution caused by rapid runoff into streams and rivers. The overall goal is to "plan, initiate, and carry out all actions and programs in a manner that will minimize or avoid adverse effects on the quality of the human environment without impairment of the Army mission." A primary objective is to eliminate the discharge of pollutants produced by Army activities. Provision of suitable surface drainage facilities is necessary in meeting this objective. Among the types of actions listed as requiring close environmental scrutiny because they may either affect the quality of the environment or may create environmental controversy are the following which pertain to surface drainage in the Arctic and Subarctic.

1-7.4.1 Real estate acquisition, disposal, and outleasing.

1-7.4.2 Proposed construction of utilities including drainage systems.

1-7.4.3 Constructing or installing open channels, ditches, culverts, or other barriers that might obstruct migration, passage or free movement of fish and wildlife.

1-7.4.4 Closing or limiting areas, such as roads or recreational areas, that were previously open to public use.

1-7.4.5 Proposed construction on flood plains or construction that may cause increased flooding, erosion or sedimentation activities.

1-7.4.6 Channelization of streams, diversions, or impoundment of water.

1-7.4.7 Proposed construction of pipelines and other drainage structures.

1-7.5 **U.S. Air Force Environmental Quality Program**. AFR 19-1 enunciates Air Force policy in compliance with above-stated NEPA executive orders and DOD directives. Procedures outlined are similar to those described for Army installations. AFR 19-2 establishes policies, assigns responsibilities, and provides guidance for preparation of environmental assessments and statements for Air Force facilities. Sources and types of pollutants, pollution effects, and control measures are discussed.

1-7.6 **U.S. Navy Environmental Quality Program**. The Navy's Environmental Quality Initiative (EQI) is a comprehensive initiative focused on maximizing the use of pollution prevention to achieve and maintain compliance with environmental regulations. The EQI is a fundamental part of the Navy environmental strategy called AIMM to SCORE - Assess, Implement, Manage and Measure to achieve Sustained Compliance and Operational Readiness through Environmental Excellence.

1-7.7 **FAA Environmental Quality Program**. FAA Order 5050.4A, Airport Environmental Handbook, provides instructions and guidance for preparing and processing the environmental assessments, findings of no significant impact (FONSI), and environmental impact statements (EIS) for airport development proposals and other airport actions as required by various laws and regulations.

1-7.8 **Environmental Impact Analysis**. A comprehensive reference, “Handbook for Environmental Impact Analysis,” was issued in September 1974. This document, prepared by the Corps of Engineers Construction Engineering Research Laboratory (CERL), presents recommended procedures for use by Army personnel in preparing and processing environmental impact assessments (EIA) and environmental impact statements (EIS). The procedures list step-by-step actions considered necessary to comply with requirements of NEPA and subsequent guidelines. These require that all Federal agencies use a systematic and disciplinary approach to incorporate environmental considerations into their decision making process. Eight major points to be covered by environmental impact statements are listed as follows:

1. A description of the proposed action, a statement of its purpose, and a description of the environmental setting of the project.

2. The relationship of the proposed action to land-use plans, policies, and controls for the affected area.

3. The probable impact of the proposed action on the environment

4. Alternatives to the proposed action, *including* those not within the existing authority of the responsible agency.

5. Any probable adverse environmental affects that cannot be avoided (summarizing the unavoidable parts Point 3 and, separately, how avoidable parts Point 3 will be mitigated).

6. The relationship between local short-term uses of man’s environment and the maintenance and enhancement of long-term productivity.

7. Any irreversible and irretrievable commitments of resources (including natural and cultural as well as labor and materials).

8. An indication of what other interests and considerations of Federal policy are thought to offset the adverse environmental effects identified.

1-7.9 **Environmental Effects of Surface Drainage Systems.** Such facilities in the Arctic or Subarctic could have either beneficial or adverse environmental impacts affecting water, land, ecology, and socioeconomic (human and economic) considerations. Despite low population density and minimal development, the fragile nature of the ecology in the Arctic and Subarctic has attracted the attention of environmental groups interested in protecting these unique assets. Effects on surrounding land and vegetation may cause changes in various conditions in the existing environment, such as surface water quantity and quality, groundwater levels and quality, drainage areas, animal and aquatic life, and land use. Proposed systems may also have social impacts on the community, requiring relocation of military and public activities, open space, recreational activities, community activities, and quality of life. Environmental attributes related to water could include such items as erosion, aquifer yield, flood potential, flow or temperature variations (the latter affecting permafrost levels and ice jams), biochemical oxygen demand, and content of dissolved oxygen, dissolved solids, nutrients and coliform organisms. These are among many possible attributes to be considered in evaluating environmental impacts, both beneficial and adverse, including effects on surface water and groundwater. Various methods are discussed for presenting and summing up the impact of these effects on the environment.

1-7.10 **Discharge Permits**. The Federal pollution abatement program requires regulatory permits for all discharges of pollutants from point sources (such as pipelines, channels or ditches) into navigable waters or their tributaries. This requirement does *not* extend to discharges from separate storm sewers except where the storm sewers receive industrial, municipal and agricultural wastes or runoff, or where the storm water discharge has been identified by the EPA Regional Administrator, the State water pollution control agency or an interstate agency as a significant contributor of pollution. Federal installations, while cooperating with and furnishing information to State agencies, do not apply for or secure State permits for discharges into navigable waters.

1-7.11 **Effects of Drainage Facilities on Fish.** Natural drainage channels in many locations are environmentally important to preservation of fish resources. Culverts, ditches, and other drainage structures constructed along or tributary to these fish streams must be designed to minimize adverse environmental effects. Culvert hazards to fish include high inverts, excessive velocities, undersized culverts, stream degradation, failed or damaged culverts creating obstructions, erosion and siltation at outlets, blockage by icing, and seasonal timing and methods of drainage construction. Consultation with Federal and State fish and wildlife agencies will provide guidance on probable effects and possible expedients to mitigate them. Special concern will be given to anticipated conditions during fish migration season. Certain conditions are discussed below.

1-7.11.1 **High inverts**. Fish passage is impossible when the culvert outlet is set too high, exceeding jumping ability of the fish and creating a spill velocity exceeding the swimming capability of the fish. Causes can be survey or design error, easier installation, or unexpected degradation of the downstream channel after culvert installation.

1-7.11.2 **High velocities in culverts**. These prevent fish from swimming upstream. Factors affecting velocity include the culvert's area, shape, slope, and internal roughness, and inlet and outlet conditions. Some increases in velocity result from the culvert alignment being straight in lieu of the natural stream's meander. Tailwater elevation, the water level in the downstream channel at the culvert outlet, should be about D/8 where D is the pipe diameter or pipe arch rise, but not less than 2.5 in. This minimum should be set with due consideration to recommendations of local fishery biologists.

1-7.11.3 **Undersized or failed culverts**. These can cause overtopping and washout of an embankment and destroy a fish resource by release of large amounts of sediment and debris.

1-7.11.4 **Erosion along drainageways or at outlets**. Additional sediment from uncontrolled erosion can adversely affect fish. Causes can be high velocities, high inverts, undersized culverts, inadequate bank protection, and lack of suitable culvert endwalls.

1-7.11.5 **Channel filling**. Covering an extensive reach of stream bottom decreases the area most suitable for spawning, depleting renewal of stocks. Proper biological input in siting and designing drainageways will avoid this problem.

1-7.11.6 **Culvert installation**. Scheduling culvert excavation, channel diversion, and channel crossings by equipment should avoid times of the year which are critical to the fish cycle.

1-7.11.7 **Control of icing**. Thawing devices such as electrical cables or steam lines, essential to any design where there is ice buildup, should be in operation to assure freedom from ice blockages during the spring migration period.

## 1-8 DESIGN COMPUTER PROGRAMS

1-8.1 **Hydraulic Design Programs.** "CORPS" is a time-sharing system developed for the Corps of Engineers computer at the Waterways Experiment Station in Vicksburg, Mississippi, with a library or computer programs, principally in the field of hydraulics. Corps offices nationwide have telephone remote terminal access to "CORPS". Use of this computer system is fully explained in step-by-step procedures suited to engineering personnel communicating in discipline-oriented language. Among available hydraulic programs useful to drainage layers are the following.

H6001 GEOMETRICAL ELEMENTS OF TRAP., TRIA., OR RECT. CHANNEL

H6002 GEOMETRIC ELEMENTS OF CIRCULAR CONDUIT

H6005 GEOMETRIC ELEMENTS OF A NATURAL CHANNEL

H6110 NORMAL DEPTH-TRAP., TRIA., OR RECT. SECTION – MANNING FORMULA

H6111 NORMAL DEPTH AND VELOCITY-CIRCULAR CONDUIT – MANNING FORMULA

H6112 NORMAL DISCHARGE – MANNING FORMULA

H6140 CRITICAL DEPTH AND VELOCITY FOR TRAP., TRIA., AND RECT. SECTION

H6141 CRITICAL DEPTH AND VELOCITY FOR CIRCULAR CONDUIT

H6201 FRICTION SLOPE – ANY FLOW SECT – MANNING, CHEZY OR COLEBROOK-WHITE

H6208 FLOW PROFILE – CIRC. COND – MANNING, CHEZY, OR COLEBROOK-WHITE FORM

H7220 EROSION AT CULVERT OUTLETS AND RIPRAP REQUIREMENTS

Details on these and other hydraulic design programs and their use are available from Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180.

1-8.2 **Storm Water Management Programs**. In developed areas, planners, designers and operators of storm water drainage systems are often required to determine quantities of storm water runoff and evaluate its quality as an important component in overall condition of an area or watershed. Two computer models, designed principally for urban areas, are available. These are “STORM”, developed by the Hydrologic Engineering Center of the Corps of Engineers, and “SWMM” (Storm Water Management Model), developed for the Environmental Protection Agency.

1-8.3 **DRIP (Drainage Requirement in Pavements)**. DRIP is a Windows® computer program developed by the FHWA for pavement subsurface drainage design. A design example using this program is detailed in Appendix A.

1-8.4 **CANDE-89 (Culvert Analysis and Design)**. CANDE-89 is a software program used for the structural analysis and design of buried culverts and other soil-structure systems. A variety of buried structures are considered, including corrugated steel and aluminum pipes, long span metal structures, reinforced concrete pipe, concrete box culverts and structural plastic pipes. The CANDE methodology incorporates the soil mass with the structure into an incremental static, plane-strain boundary value problem. The program is available from the following website. http://www-mctrans.ce.ufl.edu

1-8.5 **MODBERG.** ModBerg calculates the maximum depth of frost penetration for a given location. This program is available at the following address. http://www.pcase.com/

1-8.6 **DDSOFT (Drainage Design Software)**. Based on the Rational Formula and Manning Equation, DDSoft determines the size and bed slope of drainage channel or storm sewer. The program works with channels of four different shapes (i.e., vertical curb, triangular, rectangular, and trapezoidal), and one sewer shape (i.e., circular). The program is available from the following website.
http://www.ntu.edu.sg/home/cswwong/software.htm

1-8.7 **NDSOFT (Normal Depth Software)**. Based on the Manning Equation, NDSoft determines the normal depth in drainage channel. It works with channels of five different shapes (i.e., vertical curb, triangular, rectangular, trapezoidal, and circular). Further, the program can also determine the size of a circular sewer based on the normal depth under the full-flow condition. The program is available from the following website.
http://www.ntu.edu.sg/home/cswwong/software.htm

1-8.8 **PIPECAR**. PIPECAR is a program for structural analysis and design of circular and horizontal reinforced concrete pipe. Load analysis includes pipe weight, soil weight, internal fluid load, live loads, and internal pressures up to 50 ft of head. The program is available for download from the following website.
http://www.fhwa.dot.gov/bridge/hyddescr.htm

1-8.9 **Visual Urban (HY-22) Urban Drainage Design Programs**. These programs perform tasks in drainage of highway pavements, open channel flow characteristics, critical depth calculations, development of stage-storage relationships, and reservoir routing. The software is available for download from the following website.
http://www.fhwa.dot.gov/bridge/hyddescr.htm

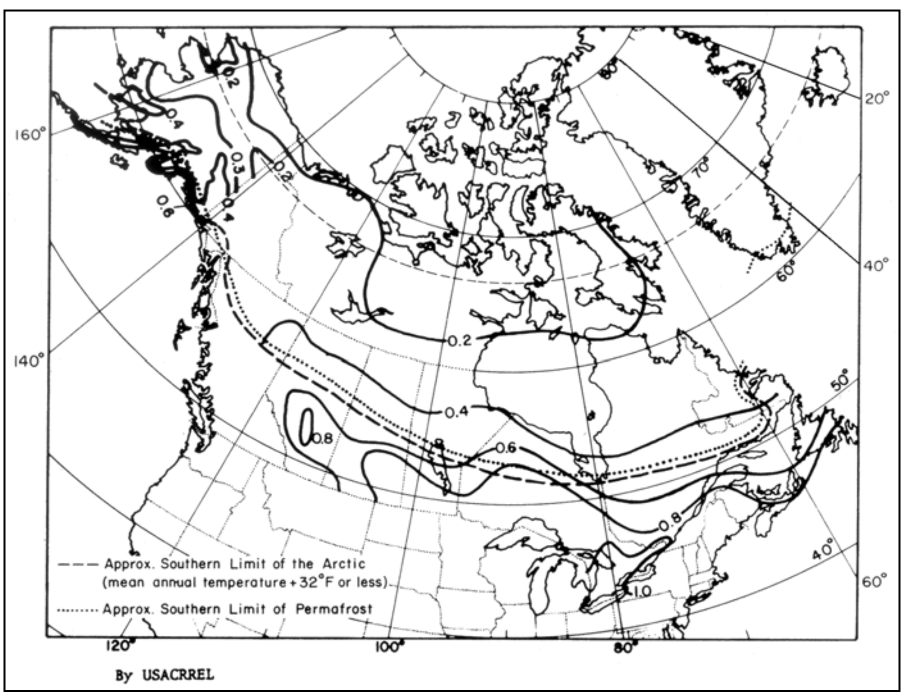
# CHAPTER 2

# SURFACE HYDROLOGY

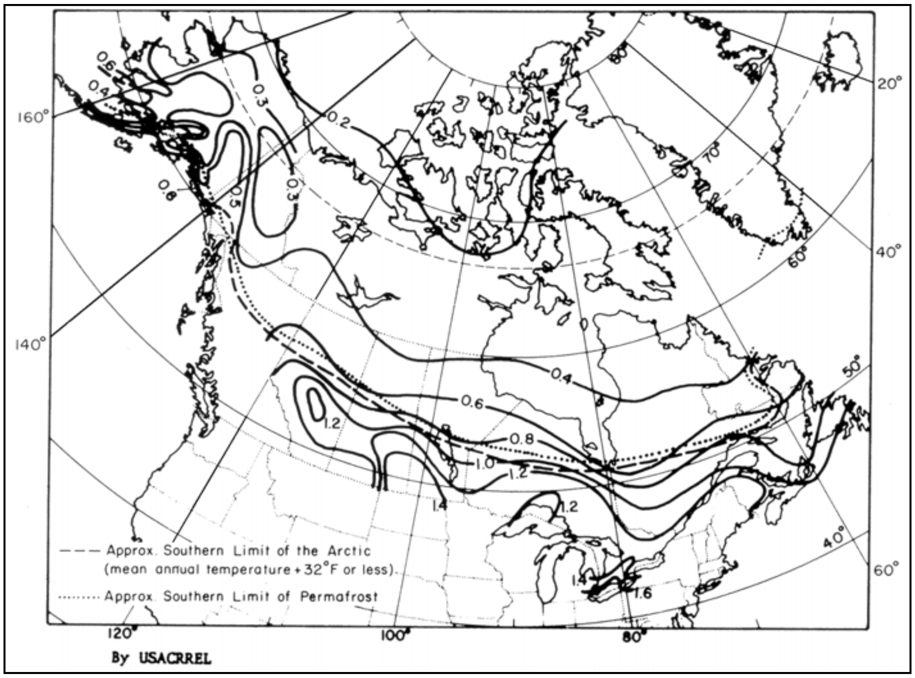
2-1 **PURPOSE AND SCOPE.** This chapter presents discussions and examples to give a better understanding of problems in the design of drainage facilities, and outlines convenient methods of estimating design capacities for airfield and heliport drainage facilities in arctic and subarctic regions. Although the design data herein have been developed primarily for drainage conditions in North America, the data are also generally applicable to other arctic and subarctic regions. For roads and built-over areas, different methods and design rates of rainfall are used in computing required runoff amounts and in determining the size of storm drains, culverts and other drainage facilities. However, the general information in this chapter on icings and special design considerations for arctic and subarctic conditions are applicable. Criteria in Sections 4-4.10 through 4-4.14, together with design storm indexes as determined from Figure 2-1, will be used for design of drainage facilities for other than airfields and heliports.

**Figure 2-1. Design Storm Index for Alaska and Canada: Isolines of maximum 1-hour rainfall (inches) occurring once in 2, 5, 10 and 25 years. Lines correspond to the intensity-duration curves in Figure 2-3. Data from US National Weather Service, the Canadian Department of Transportation, Meteorological Branch, and Quartermaster Research and Development Center**

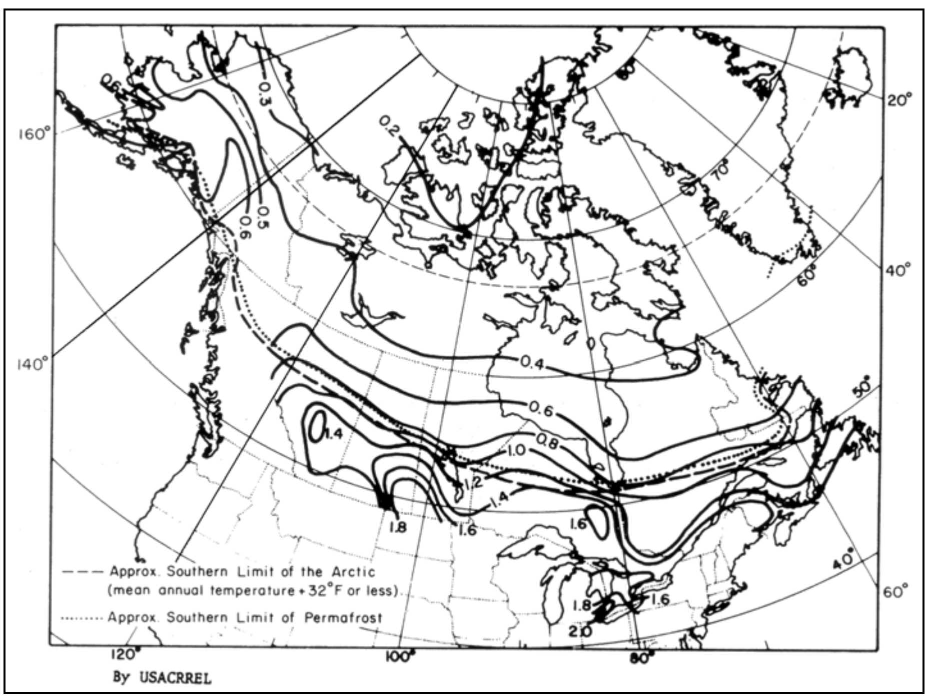
**a. Once in 2 yr**



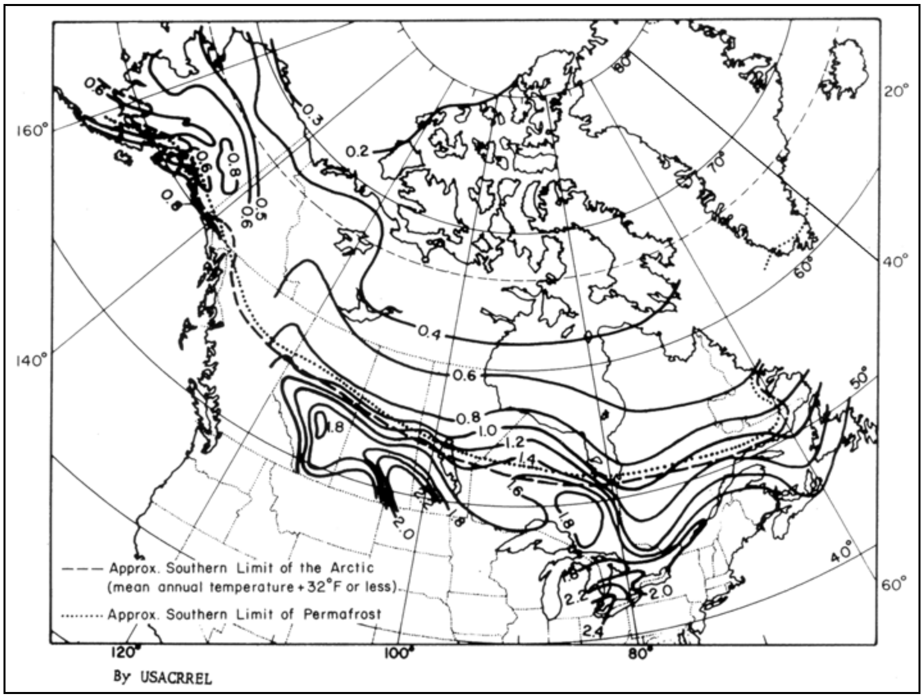
**b. Once in 5 yr**




**c. Once in 10 yr**



**d. Once in 25 yr**




2-2 **HYDROLOGIC CRITERIA.** The Rational Method, developed over 100 yr ago, is widely used for estimating design runoff from urban areas. The Rational Formula, popular because of its simplicity in application, is described in Chapter 4. It is suited mainly to sizing culverts, storm drains or channels to accommodate drainage from small areas, general less than 50 acres. Selection of appropriate values of runoff coefficients in the formula depends on the experience of the designers and the designers' knowledge of local rainfall-runoff relationships. Use of the Rational Method in the design of military airfield drainage systems, with their large, generally level contributory drainage areas, is not recommended. The development of hydrologic criteria in this manual closely follows the procedure outlined in Chapter 3. "Investigation of Airfield Drainage, Arctic and Subarctic Regions, Part I, Field Reconnaissance Report," by L. G. Straub and L. A. Johnson, is one of several confirming that this procedure accurately determines required hydraulic capacity of airfield drainage facilities with lessened dependence on arbitrary assumptions of design factors. Although judgment is important in any engineering design, guesswork is minimized in use of this procedure which is based on theoretical concepts which have been verified in carefully controlled natural and simulated rainfall and runoff tests under widely varying hydrologic and topographic conditions. In the design of drainage facilities for the Arctic and Subarctic, additional capacity must in many cases be provided to compensate for that lost due to icings. This is discussed in Section 2-8.

2-2.1 **Definitions.** The following specialized terms are used in this chapter.

2-2.1.1 **Arctic.** The northern region in which the mean temperature for the warmest month is less than 50 degrees F and the mean annual temperature is below 32 degrees F. In general, the Arctic coincides with the tundra region north of the limit of trees.

2-2.1.2 **Subarctic.** The region adjacent to the Arctic in which the mean temperature for the coldest month is below 32 degrees F, the mean temperature for the warmest month is above 50 degrees F, and in which there are less than 4 months having a mean temperature above 50 degrees F. In general, the subarctic land areas coincide with the circumpolar belt of dominant coniferous forests.

2-2.2 **Design Objectives.** The design capacity of the airfield or heliport surface drainage system should be adequate to accomplish the following objectives as satisfactorily as is economically feasible and with due consideration of the mission and importance of the particular airfield or heliport, effects of icings, and environmental impact.

2-2.2.1 **Surface runoff from design storm.** Surface runoff from the selected design storm will be disposed of without damage to facilities, undue saturation of the subsoil, or significant interruption of normal traffic.

2-2.2.2 **Surface runoff from storms exceeding design storm.** Surface runoff from storms greater than the design storm will be disposed of with the minimum damage to the airfield for heliport. The center 50 percent of runways; the center 50 percent of taxiways serving these runways; and helipad surfaces shall be free from ponding resulting from storms of one hour duration, 25-yr frequency and intensity determined by the graphic location.

2-2.2.3 **Reliability of operation.** The drainage system will have the maximum reliability of operation practicable under all conditions, with due consideration given to abnormal requirements during annual periods of snowmelt and ice jam breakup.

2-2.2.4 **Maintenance.** The drainage system will require minimum maintenance which will be accomplished quickly and economically. Particular reliance will be placed on maintenance of drainage components serving operational facilities.

2-2.2.5 **Future expansion.** Future expansion of drainage facilities will be feasible with the minimum of expense and interruption to normal traffic.

2-2.3 **Degree of Drainage Required.** The degree of protection to be provided by the drainage system depends largely on the importance of the facility as determined by the type and volume of traffic to be accommodated, the necessity for uninterrupted service, and similar factors. Although the degree of protection should increase with the importance of the airfield or heliport, minimum requirements must be adequate to avoid hazards to operation. One severe accident chargeable to inadequate drainage can offset any difference between the cost of reasonably adequate and inadequate drainage facilities. Drainage for military airfields or heliports will be based on a 2-yr design storm frequency, unless exceptional circumstances require greater protection. For design purposes, a minimum supply rate of 0.2 in./hr of rainfall plus snowmelt is to be used, even where intensity frequency studies for the Arctic indicate somewhat lower values. In mountainous areas subject to orographic precipitation, maps showing local variations of the design storm index will prove useful for drainage designs provided that adequate long-term precipitation records are available to warrant such refinements. In some cases one can justify use of design storm frequencies appreciably higher than the 2-yr rate to protect important facilities. In some U.S. designs, portions of the drainage system have been based on as high as a 50-yr design frequency to reduce likelihood of flooding a facility essential to operations and to prevent loss of life. Many designers find that using the 2-yr design with this Corps of Engineers method will usually yield results comparable with use of a 10-yr design based on the Rational Method.

2-3 **RAINFALL.** A study of rainfall intensity-frequency data recorded at arctic and subarctic stations indicates significant variance between the average intensity of rainfall for a period of 1 hr and the average precipitation rates of comparable frequency for shorter intervals. This is also evident when compared with similar rainfall data in the continental United States. Even within the area of Alaska, there is noticeable difference between the orographic rains of Juneau and the convergent and convective precipitation at Fairbanks. The higher values for rainfall intensity were used to develop design intensity-duration (supply) curves. Similar curves for the continental United States are shown in Figure 2-2.

**Figure 2.2 Design Storm Index, 1-hour Rainfall Intensity-Frequency Data for Continental United States Excluding Alaska**

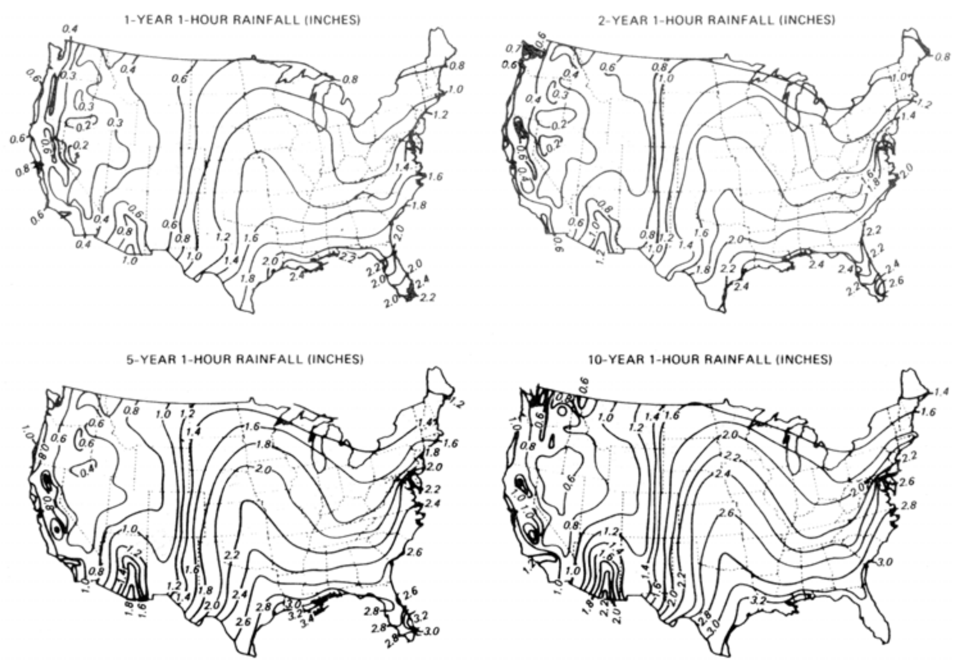



Chart reproduced from US Weather Bureau, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, Washington, DC, May 1961.

2-3.1 **Design Storm Frequency.** Design storm frequencies are normally stated in engineering instructions for the specific project. For airfields and heliports, the 2-yr design storm frequency is most often used. It should be noted that after this design storm frequency is specified, computations must be made to determine the critical duration of rainfall required to produce the maximum rate of runoff for each area. This will depend primarily on the slope and length of overland flow.

2-3.2 **Storms of Greater Severity Than Design Storm.** The design storm frequency alone is not a reliable criterion of the adequacy of storm drain facilities. Under some circumstances, storms much more severe than the design storm may cause very little damage or inconvenience, whereas under other circumstances flooding of important areas may result. It is advisable to investigate the probable consequences of storms more severe and less frequent than the design storm before making final decisions regarding the adequacy of proposed drain-inlet capacities. Additional requirements necessitated by the effects of icings on drainage facilities in arctic and subarctic regions are discussed in Section 2-8.

2-3.3 **Design Storm Index.** One-hour rainfall intensities having various average frequencies of occurrence in the artic and subarctic regions of Alaska and Canada are shown in Figure 2-1. This figure, on which rainfall depth curves are superimposed, is known as a design storm index and is based on reports by the U.S. National Weather Service and the Canadian Department of Transport, Meteorological Branch. The curves are labeled according to the 1-hour amounts of rainfall and are coordinated with the supply curves of Figure 2-3. Figures 2-1 and 2-3 used in combination provide a sufficiently accurate means for determining rainfall intensities for runoff computations for any duration and geographic location. Where data are incomplete for a specific foreign area under study, a generalized method for estimating the 2-yr 1-hr value has been developed using usually available climatic data. This method uses a diagram (Figure 2-4) which relates the 2-yr 1-hr rainfall to the following more commonly known climatic data: mean annual precipitation, mean annual number of days of precipitation, mean annual thunderstorm days, and mean of the annual maximum observational-day rainfall amounts. The diagram gives maximum 60-min, not clock-hour, rainfall for the 2-yr frequency.

**Figure 2-3. Supply Curves for Arctic and Subarctic Regions**

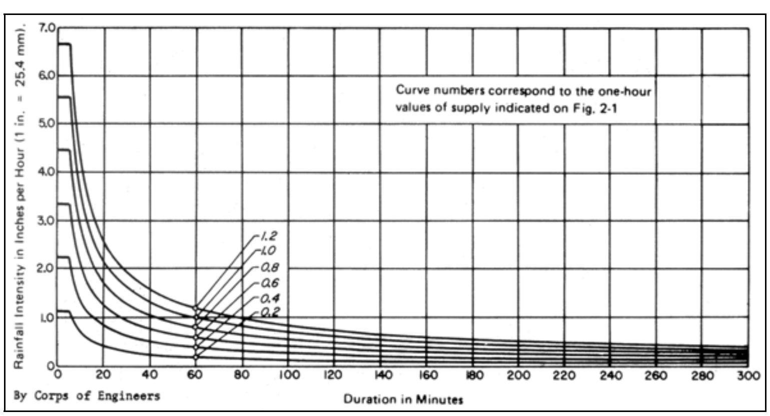
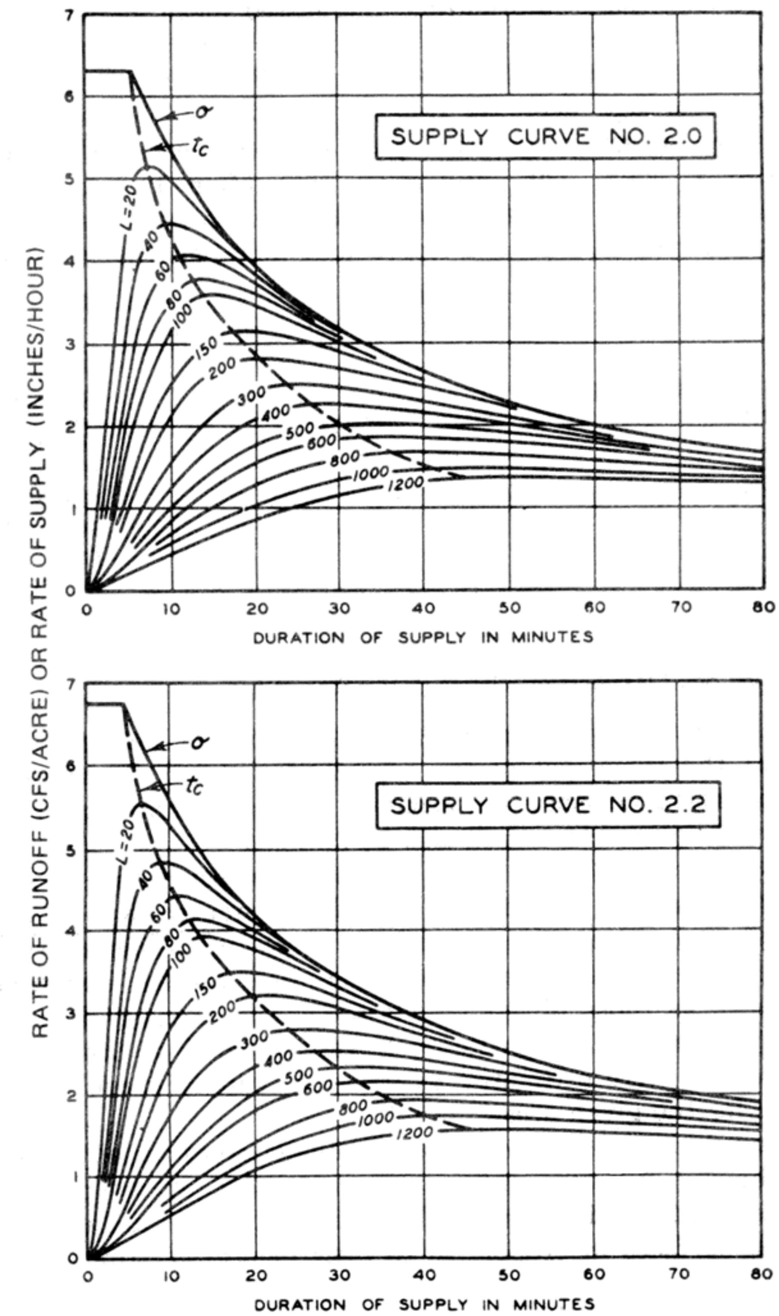


**Figure 2-4. Rates of runoff and Rates of Supply Corresponding to Standard Supply Curves No. 2.0 and 2.2; n = 0.40 and S = 1 percent**




NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

$\sigma$ = RATE OF SUPPLY, IN INCHES PER HOUR.

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## 2-4 INFILTRATION

2-4.1 **Definition.** As used herein, the term "infiltration" refers to the absorption of rainfall by the ground during a design storm. The infiltration capacity, or ability of a soil to absorb precipitation, normally decreases as the duration of rainfall increases, until a fairly definite minimum rate is reached. Variations in the degree of compaction, soil moisture deficiencies at the beginning of rainfall, and the depth to the groundwater table may greatly influence the infiltration capacity of a particular soil.

2-4.2 **Variability.** Because of the several variables that affect the infiltration capacity of a given soil, it is impracticable to determine accurately the infiltration capacities assumed to apply during storms. The rate of infiltration changes not only during the course of a storm but also during a season. The infiltration rate also varies with the type of soil structure, the soil cover, the temperature of air, soil, and water, the moisture content of soil, turbidity of the water, and the amount of organic matter in the soil. The total porosity of a soil determines to a considerable extent the total amount of water that may filter into it. Available data indicate that the rate of infiltration increases with a rise in the temperature of the air, soil and water, and conversely, the rate of infiltration lessens with an increase in the moisture content of the soil. Soils with a high organic matter content also have high infiltration rates. Vegetative cover serves as a protection from the impact of rain, retards the rate of runoff, and thereby reduces the velocity of overland flow and turbidity, and permits greater infiltration of water into the soil. Rates of infiltration on bare soil can be expected to be considerably less than those for turfed areas. For use in the design of storm drains for a particular airfield or heliport, the infiltration capacity that is estimated to be characteristic of the given soil, *following a rainfall of 1 hr*, serves as the most convenient index to the probable volume of loss through infiltration during the design storm. Antecedent rainfall conditions such as those ordinarily occurring during seasons in which the adopted design storm is likely to occur will be assumed in estimating the 1-hr infiltration rate referred to above.

2-4.3 **Rate.** In permafrost regions, groundwater percolation rates are much lower than in thawed soils and the rate of infiltration for design purposes should be considered zero. In other areas, a good guide can be obtained when test borings are made. Rates would normally not exceed about 0.5 in./hr for clayey soils with low permeability.

2-5 **SNOWMELT.** Airfields, heliports, and other pavement areas in the Arctic and Subarctic are subjected to their most critical drainage requirements during spring thaw and other periods of snow and ice melting. Initial periods of higher temperatures and longer days result in densification or "ripening" of snow, subsequently converted to snowmelt runoff. With banked water-laden snow on or adjacent to pavements, inlets and drainage ditches, a maximum rate of runoff from snowmelt, exclusive of rainfall, is about 0.1 in./hr. In regions of lesser snowfall accumulation, snowmelt runoff at half this rate, 0.05 in./hr, would be expected. Accordingly, an amount of 0.05 to 0.1 in/hr for snowmelt will be added to the design rainfall intensity rate for drainage facilities in the Arctic and Subarctic.

2-6 **SUPPLY.** The term "rate of supply" refers to the rainfall intensity plus snowmelt minus the infiltration capacity at the same instant of a particular storm. To simplify computation procedures, the rainfall intensity, rate of snowmelt and infiltration capacity are assumed to be constant during any specific storm. On this premise, the rate of supply during a particular storm would also be uniform.

2-6.1 **Average Rates of Supply.** Average rates of supply corresponding to storms of different durations and the same average frequency of occurrence can be computed by subtracting estimated infiltration capacities from rainfall plus snowmelt intensities represented by the proper standard rainfall intensity-duration curve in Figure 2-3. For convenience, standard supply curves are assumed to have same shape as the rainfall intensity-duration curves. For example, if curve 0.8 in Figure 2-3 was indicated by Figure 2-1 as the design rainfall plus snowmelt, and infiltration loss at the rate of 0.2 in./hr was estimated to be applicable, curve 0.6 would be adopted as the supply curve for that area.

2-6.2 **Weighted Standard Supply Curves.** In most cases, drainage areas consist of combinations of paved and unpaved areas having different infiltration capacities. To simplify computations, weighted standard supply curves should be estimated for composite tributary drainage areas by weighting the standard supply curve numbers adopted for paved and unpaved surfaces in proportion to their respective areas.

## 2-7 RUNOFF

2-7.1 **Notation.** Symbols used in equations and discussions contained in the following paragraphs are defined below:

- $L$ = effective length of overland flow, ft (See discussion of effective length in 2-7.3 and 2-7.5 below.)
- $n$ = retardance coefficient
- $Q$ = discharge capacity, ft$^3$/sec, at a designated point
- $Q_d$ = drain-inlet capacity, ft$^3$/sec
- $q$ = rate of overland flow at the lower end of an elemental strip of turfed, bare, frozen or paved surface, in./hr or in ft$^3$/sec per acre of drainage area
- $q_d$ = drain-inlet capacity, or maximum rate of outflow from a ponding area, ft$^3$/sec per acre of tributary drainage area
- $q_p$ = peak runoff rate, in./hr or ft$^3$/sec per acre of drainage area
- $S$ = slope of surface, or hydraulic gradient
- $t$ = time, or duration, min; time from beginning of supply
- $t_c$ = critical duration or supply, min; that is, the duration of rainfall plus snowmelt excess (rate of supply) for a given standard supply curve that would produce the maximum rate of outflow from a given drainage area, taking into account surface detention and surface runoff characteristics

$t_d$ = time required for water to travel from a specified inlet to a given point in the drainage system, min
$t_r$ = duration of supply, min
$\sigma$ = rate of supply or rainfall plus snowmelt in excess of the rate of infiltration, in./hr
tanh = hyperbolic tangent (defined as the quotient of the hyperbolic sine divided by the hyperbolic cosine, i.e., tanh

$$x = \frac{\sinh x}{\cosh x},$$

the hyperbolic functions having the same relationship to the equilateral hyperbola as the trigonometric functions do to the circle).

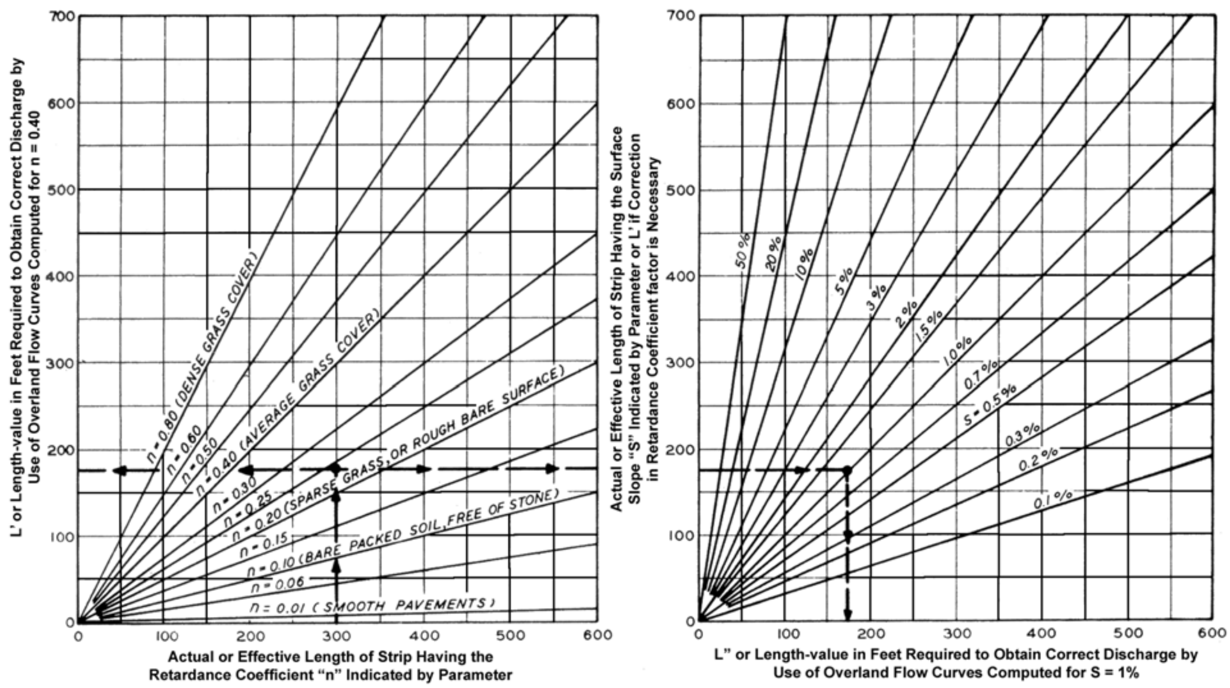
2-7.2 **Overland Flow Equation.** The term “overland flow” as used herein relates to surface runoff, resembling sheet flow, before it has reached a defined channel or ponding basin. Horton developed an equation for the rate of overland flow to be expected from a uniform rate of rainfall excess, or rate of supply, which in a form modified for this manual is as follows:

$$q = \sigma \tanh^2 \left[ 0.922t \ \ (\sigma / nL)^{0.50} S^{0.25} \right]$$

2-7.3 **Effective Length.** In the basic derivation of the above equation, the term L, effective length, represents the length of overland sheet flow measured in a direction parallel to the maximum slope, before the runoff has reached a defined channel. In actuality, particularly in large drainage areas and under many conditions of grading, considerable channelized flow will occur during the design storm conditions. Investigation of many runoff records for watersheds similar to typical airfield and heliport areas in the continental United States indicates that by modifying the determination of effective length, satisfactory reproduction of runoff by hydrographs can be obtained regardless of channelization of flow. The effective length L is the sum of the channelized flow length and the overland flow length, each converted to an equivalent length for n = 0.40 and S = 1.0 percent by means of Figure 2-5. The length of channel flow is measured along the proposed collecting channel or swale for that section in which appreciable depth of flow may occur during the design storm. Length of overland flow is the average distance from the end of the effective channel, if any, or the drain to the outer periphery of the drainage area. Even with excellent grading, overland flow lengths seldom exceed a few hundred feet before channelization occurs. Typical values of the retardance coefficient n for use in determining equivalent length of overland flow are shown in Table 2-1. A guide to selection of n values in the case of channelized flow is shown in Figure 2-6. A more detailed description of the procedure for selecting “n” value is contained in Chapter 3 and Section 4-2.1.3.

2-7.4 **Ponding.** Although provision of ponding areas is advantageous in temperate zone drainage designs, ponding on or alongside paved areas should be avoided in permafrost regions. There, water accumulated alongside airfield or roadway pavement embankments can cause thermal as well as mechanical erosion. Saturation of fine-grained soil and subsoil shortly before freezeup in the fall may greatly increase subsequent destructive frost heaving damage.

**Figure 2-5. Airfield Drainage-Overland Flow Relations. Modification in L Required to Compensate for Differences in n and S**




A. Correction for Differences in Retardance Coefficient (n)

B. Correction for Differences in Slope (S)

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**Table 2-1. Retardance Coefficients for Overland Flow**

| Surface | Value of n |
|---|---|
| Pavements and frozen ground | 0.01 |
| Bare packed soil free of stone | 0.10 |
| Sparse grass cover, tundra, or moderately rough bare surface | 0.20 |
| Average grass cover | 0.40 |
| Dense grass cover | 0.80 |

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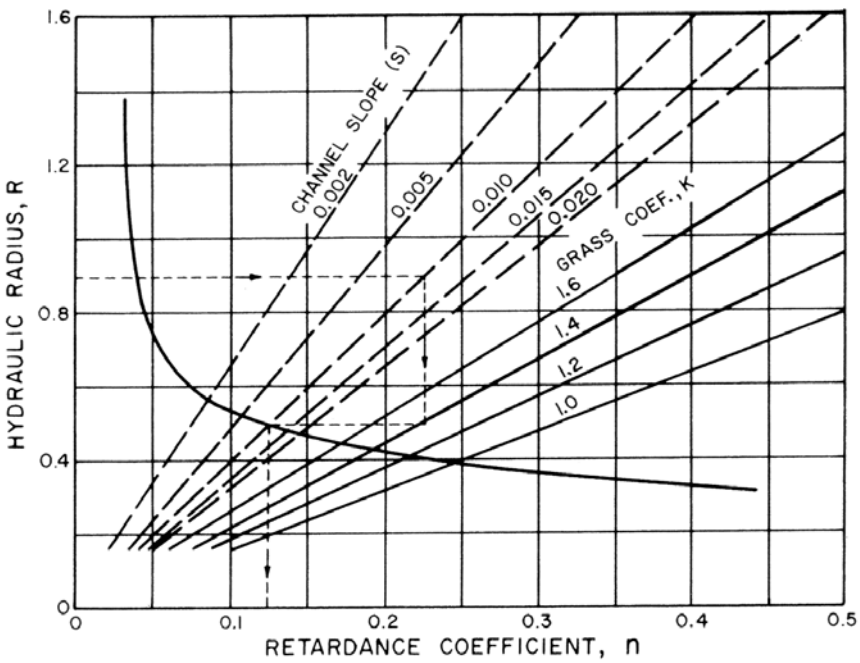
2-7.5 **Effect of Paved Area on Determination of Effective Length.** The time required for water to run off the average paved or ice-covered area is normally very short. Consequently, the length of the paved areas need be given little weight in estimating the effective length L for a composite area. As q is inversely proportional to L, it is helpful to grade the slopes so that the drain inlet is located as far as practicable from the watershed center. In a rectangular area, a drain inlet located near a corner would require less discharge capacity than one located in or near the center of the plot.

**Figure 2-6. Retardance Coefficients for Flow in Turfed Channels**




AVERAGE GRASS COEFFICIENT (K) FOR DENSE AIRFIELD TURF

| AVERAGE LENGTH OF GRASS IN INCHES | Under 6 in. (0-15 cm) | 6 to 12 in. (15-30 cm) | Over 12 in. (30 cm) |
|---|---|---|---|
| GRASS COEF. (K) | 1.4 | 1.3 | 1.2 |

EXAMPLE:

Determine "n" for a channel with 4 inches of dense grass, R = 0.9, and S = 0.010.

From table, K = 1.4; from graph, by following dotted line, "n" is equal to 0.12.

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2-7.6 **Relation of Overland Flow is Standard Supply Curves.** The curves shown in Figures 2-7 through 2-12 were obtained by computing the rates of discharge, at appropriate time intervals that would result from various rates of supply, corresponding to the respective standard supply curves of Figure 2-3. The procedure is illustrated by the sample computations in Table 2-2. The curves shown are not hydrographs for any specific design storm but represent the peak rates of runoff from individual storm events of various durations, all of which have the same average frequency of occurrence. The duration of supply corresponding to the greatest discharge for a particular standard supply curve and value of L in Figures 2-7 through 2-12 is defined as the critical duration of supply $t_c$ for runoff from an area.

**Figure 2-7. Supply Curve No. 0.2**

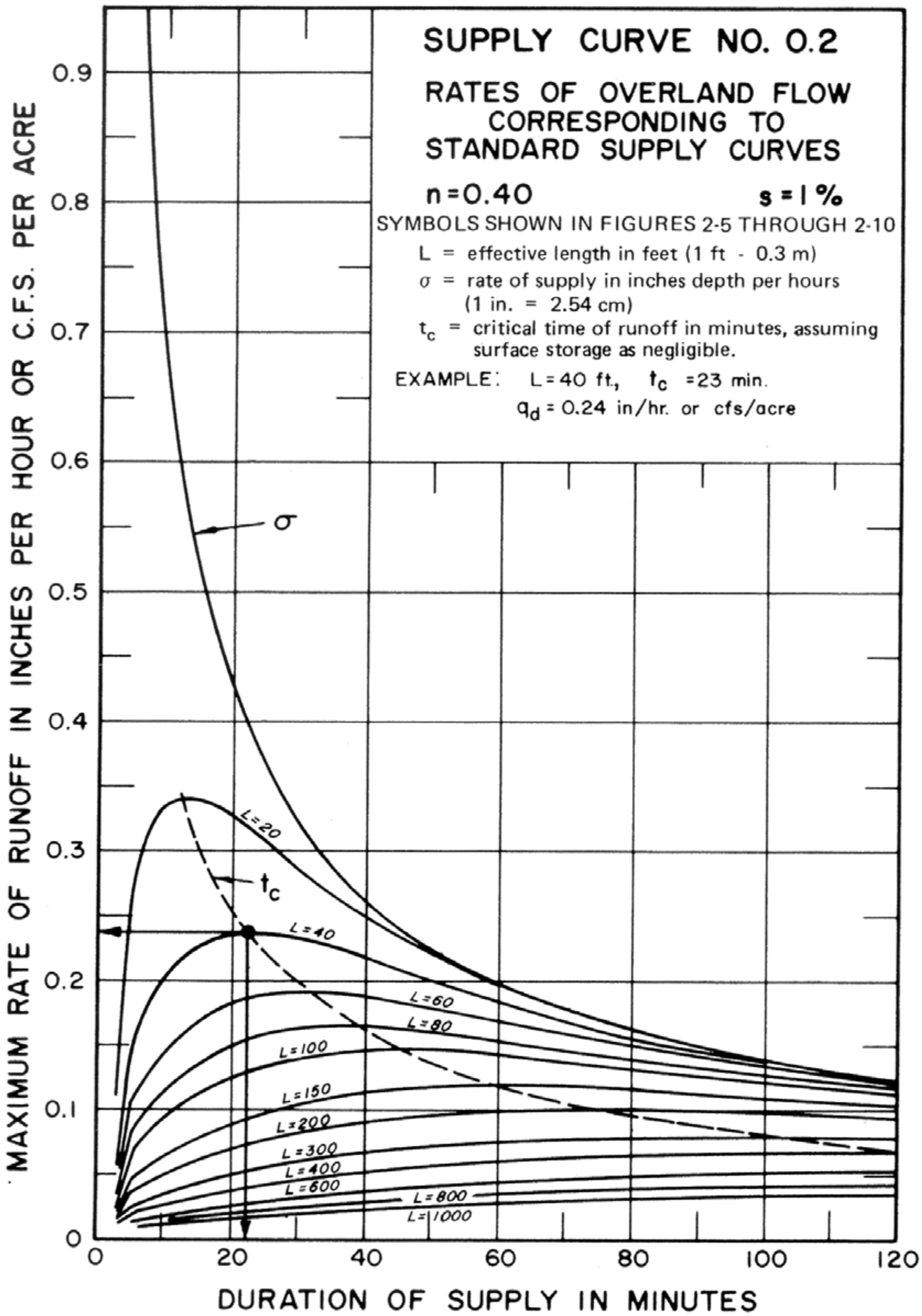


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**Figure 2-8. Supply Curve No. 0.4**

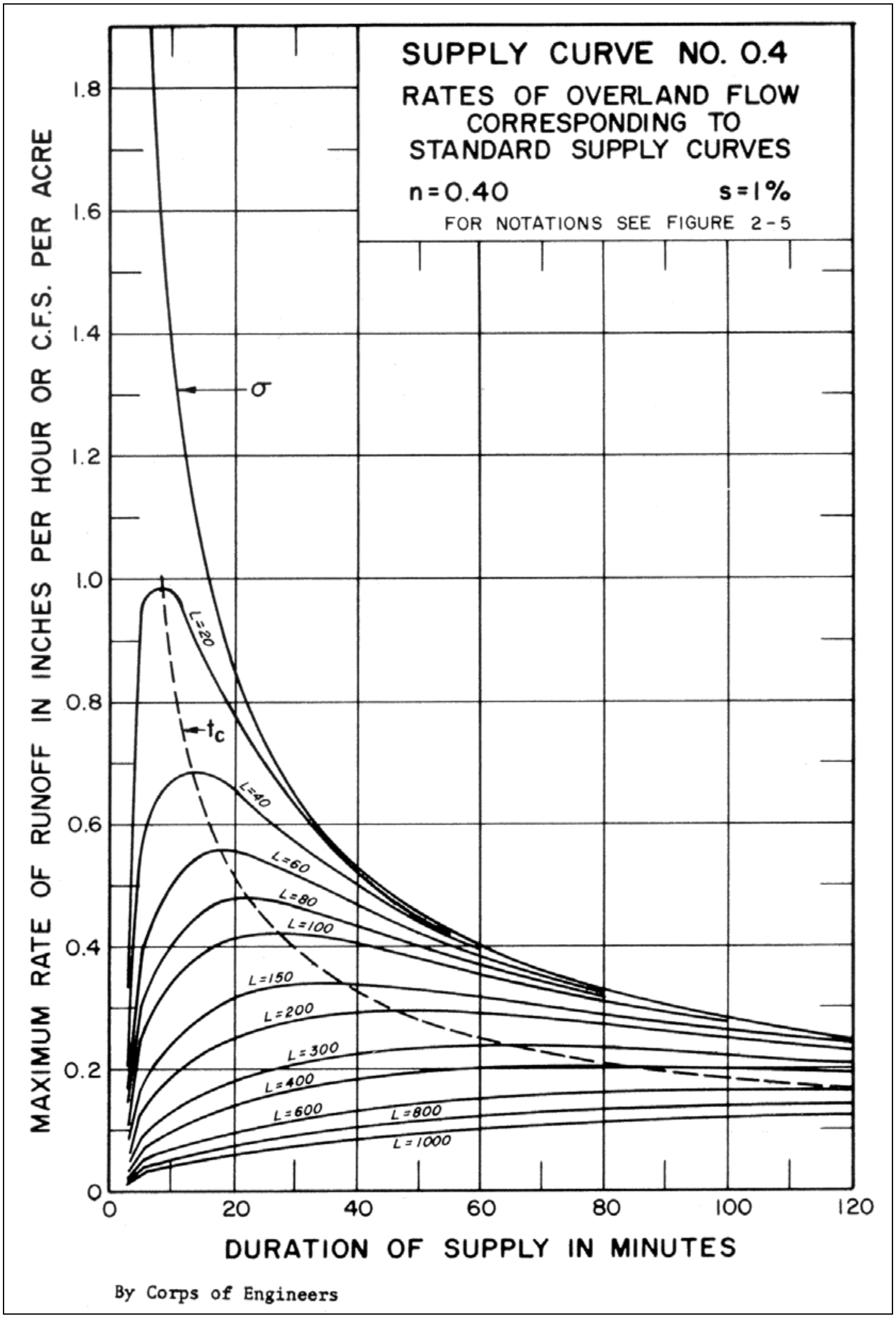

**Figure 2-9. Supply Curve No. 0.6**

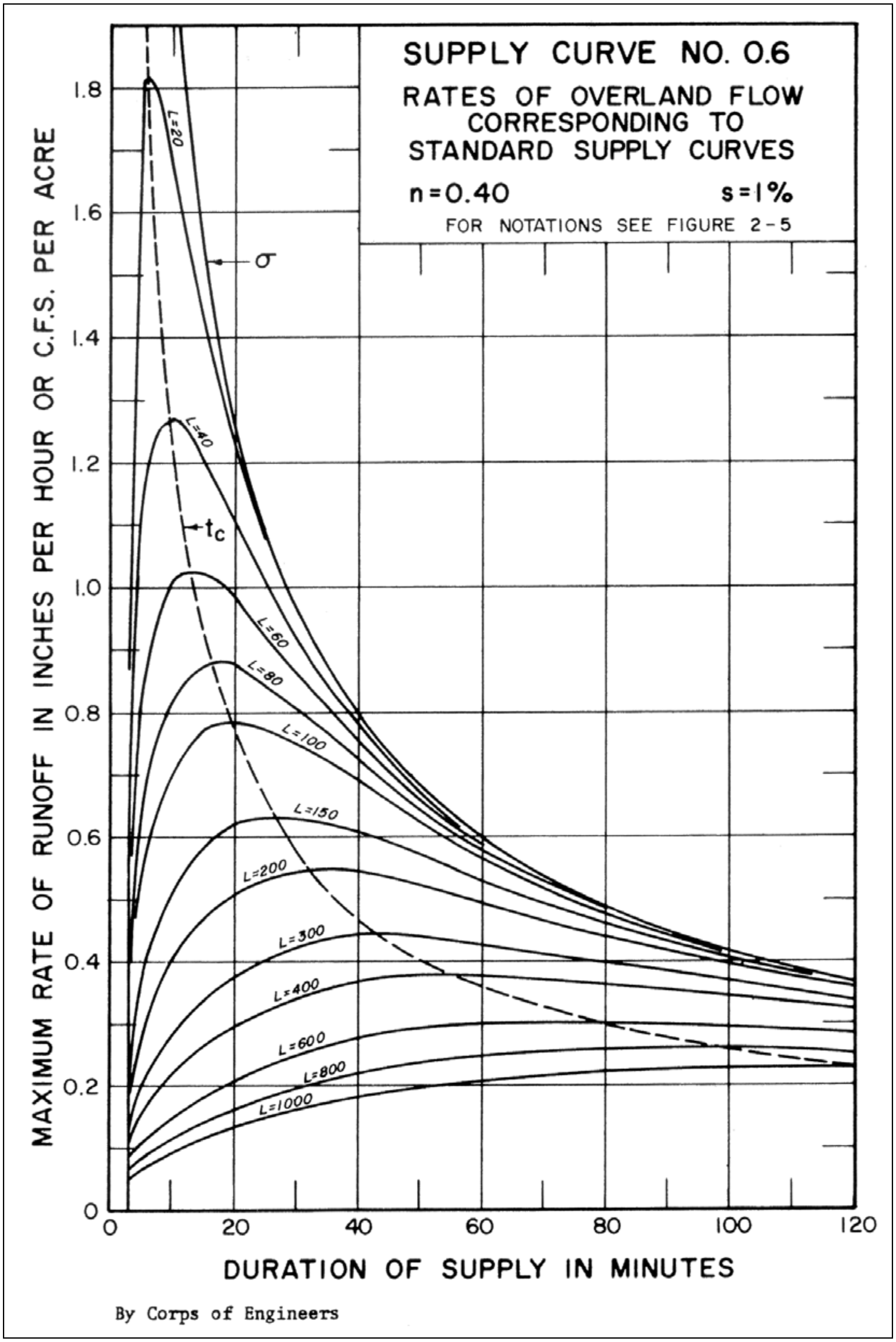

**Figure 2-10. Supply Curve No. 0.8**

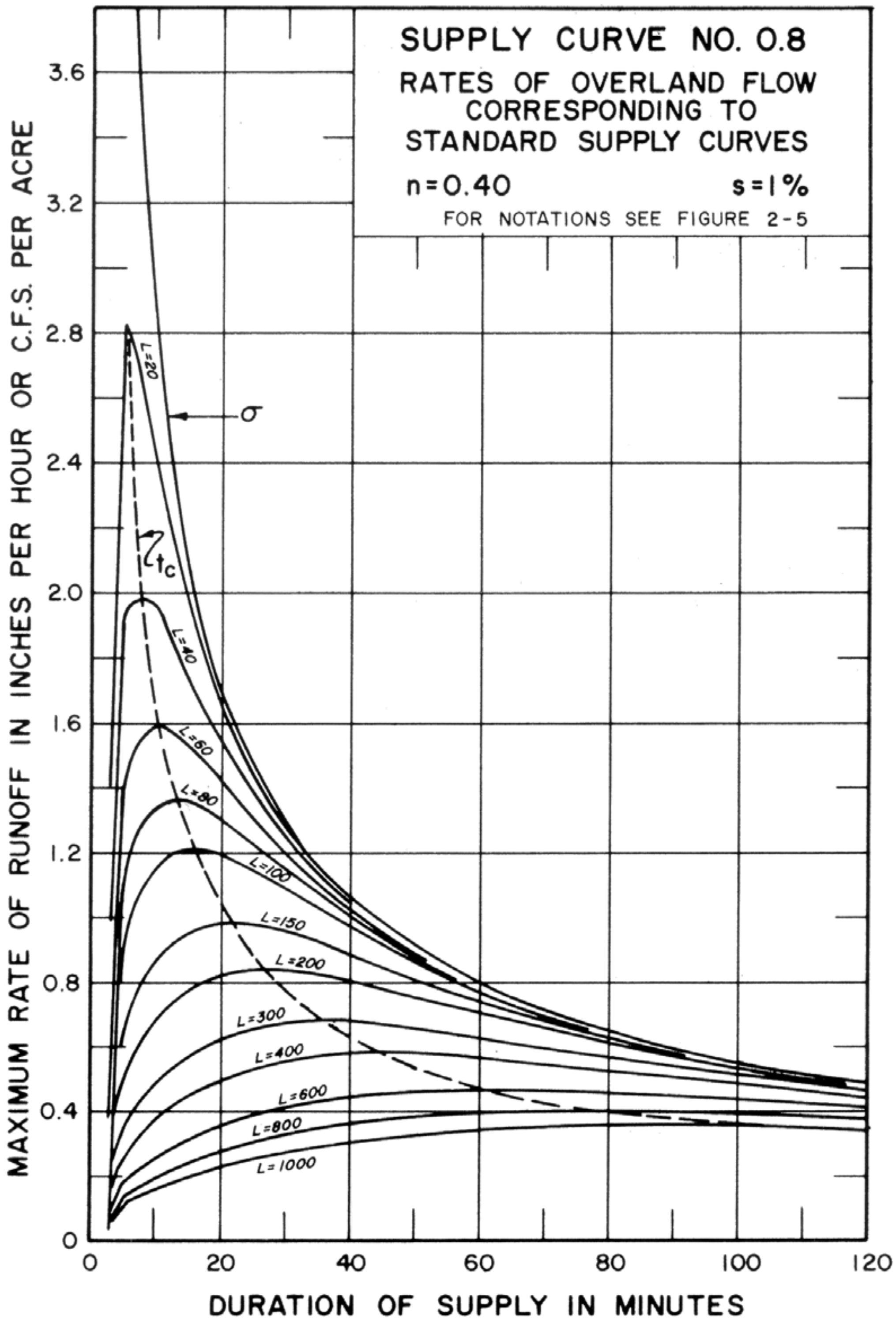


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**Figure 2-11. Supply Curve No. 1.0**

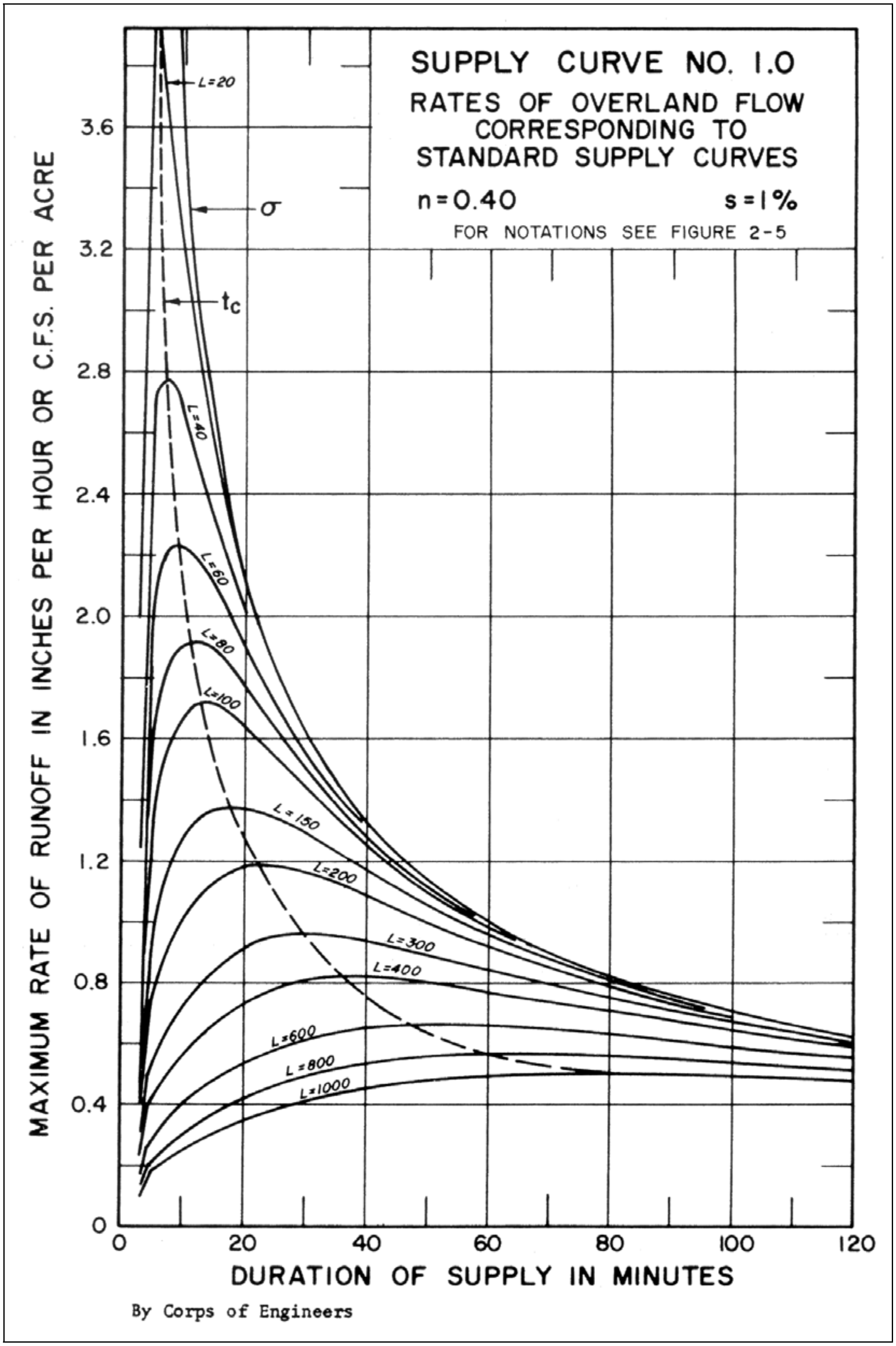

**Figure 2-12. Supply Curve No. 1.2**

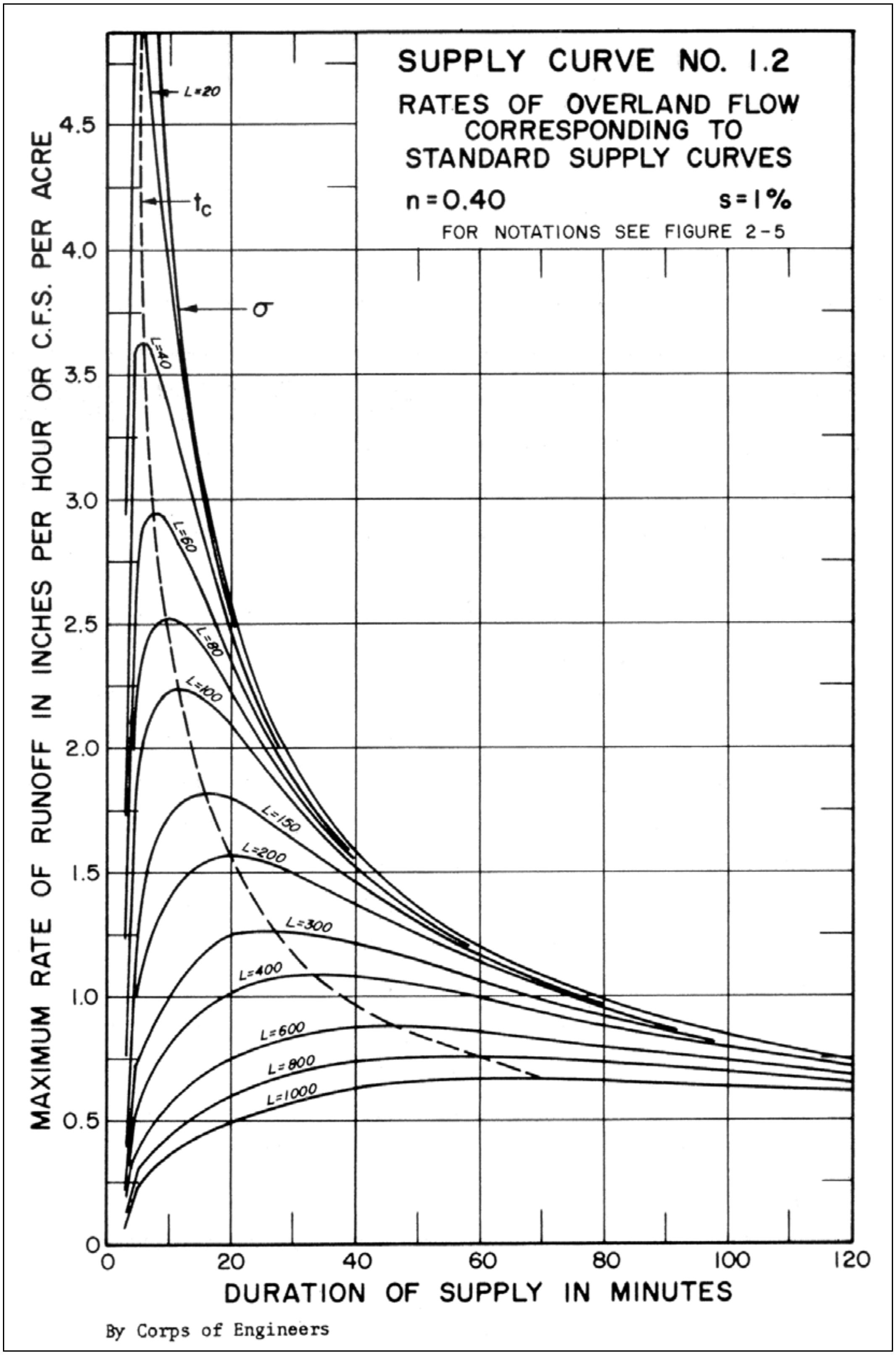

**Table 2-2. Rates of Overland Flow Corresponding to Intensities Shown on Supply Curve 0.2 in Figure 2-3**

| Duration of Supply, min. | Rate of Supply, in./hr | Rate of overland flow in c.f.s. for various durations and rates of supply where L equals | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 ft | 40 ft | 60 ft | 80 ft | 100 ft | 150 ft | 200 ft | 300 ft | 400 ft | 600 ft | 800 ft |
| 3 | 1.113 | 0.111 | 0.058 | 0.039 | 0.031 | 0.024 | 0.017 | 0.013 | 0.009 | 0.008 | 0.006 | 0.003 |
| 5 | 1.113 | 0.273 | 0.149 | 0.104 | 0.080 | 0.065 | 0.043 | 0.035 | 0.023 | 0.018 | 0.011 | 0.009 |
| 7 | 0.883 | 0.306 | 0.175 | 0.122 | 0.093 | 0.077 | 0.053 | 0.041 | 0.027 | 0.022 | 0.015 | 0.011 |
| 9 | 0.743 | 0.328 | 0.194 | 0.137 | 0.107 | 0.087 | 0.060 | 0.046 | 0.031 | 0.025 | 0.016 | 0.013 |
| 12 | 0.608 | 0.340 | 0.213 | 0.154 | 0.122 | 0.100 | 0.069 | 0.053 | 0.036 | 0.028 | 0.019 | 0.015 |
| 15 | 0.522 | 0.339 | 0.227 | 0.167 | 0.133 | 0.110 | 0.078 | 0.060 | 0.041 | 0.031 | 0.022 | 0.017 |
| 20 | 0.430 | 0.329 | 0.237 | 0.184 | 0.148 | 0.125 | 0.090 | 0.069 | 0.048 | 0.037 | 0.030 | 0.020 |
| 25 | 0.367 | 0.308 | 0.236 | 0.190 | 0.157 | 0.132 | 0.097 | 0.076 | 0.054 | 0.041 | 0.029 | 0.023 |
| 30 | 0.323 | 0.287 | 0.232 | 0.191 | 0.162 | 0.139 | 0.103 | 0.081 | 0.058 | 0.045 | 0.031 | 0.024 |
| 35 | 0.292 | 0.269 | 0.226 | 0.192 | 0.164 | 0.145 | 0.109 | 0.088 | 0.063 | 0.049 | 0.034 | 0.026 |
| 40 | 0.265 | 0.250 | 0.217 | 0.188 | 0.164 | 0.145 | 0.112 | 0.091 | 0.065 | 0.052 | 0.036 | 0.028 |
| 45 | 0.245 | 0.235 | 0.210 | 0.184 | 0.164 | 0.147 | 0.115 | 0.094 | 0.069 | 0.054 | 0.038 | 0.030 |
| 50 | 0.227 | 0.220 | 0.201 | 0.179 | 0.161 | 0.145 | 0.116 | 0.096 | 0.071 | 0.056 | 0.040 | 0.031 |
| 60 | 0.200 | 0.197 | 0.184 | 0.170 | 0.155 | 0.143 | 0.117 | 0.100 | 0.075 | 0.060 | 0.043 | 0.034 |
| 80 | 0.163 | 0.162 | 0.157 | 0.149 | 0.141 | 0.133 | 0.115 | 0.100 | 0.079 | 0.065 | 0.048 | 0.038 |
| 100 | 0.140 | -- | 0.138 | 0.134 | 0.129 | 0.123 | 0.110 | 0.099 | 0.081 | 0.068 | 0.051 | 0.041 |
| 120 | 0.123 | -- | -- | 0.120 | 0.117 | 0.113 | 0.104 | 0.095 | 0.080 | 0.069 | 0.054 | 0.043 |

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## 2-8 ICING

2-8.1 **Description**. The term icing (sometimes misnamed “glaciering”) applies to a surface ice mass formed by the freezing of successive sheets of water, the source of which may be a river or stream, a spring, or seepage from the ground. When icing occurs at or near airfields, heliports, roadways or railroads, the drainage structures and channels gradually fill with ice, which may spread over pavements or structures, endangering and disrupting traffic and operations. Ice must be removed from pavements or structures, and drainage facilities must be cleared to avoid or limit the re-forming of icing. Obstruction of flow through drainage facilities—culverts, bridges, pipelines or channels—can lead to washout of pavement embankments or undermining of structures. The spring thaw period is most critical in this regard. Prevention or control of icing at or near drainage structures and the related effects on pavements and other facilities is a key objective of drainage design and maintenance in the Arctic and Subarctic. As icing can occur throughout both seasonal frost and permafrost areas, they are a widespread cause of recurring operational and maintenance problems. Drainage designs based only on conventional criteria will not fulfill the abnormal hydraulic conveyance requirements of icing-prone regions and will be subject to troublesome maintenance problems. Special design and maintenance concepts, based mainly on field experience under similar situations, are required.

2-8.2 **Types**. Icing is classed conveniently as river or stream icing, ground icing, or spring icing, although sometimes it is difficult to assign a specific type to a particular situation. The three general types of icing are discussed below.

2-8.2.1 **River or stream icing.** These occur more commonly on shallow streams with large width/depth ratios. Braided or meandering channels are more prone to icing formation than well defined single channels. River or stream icing normally begins to develop soon after normal ice cover forms on a stream surface, generally during October to December. The icing begins with the appearance of unfrozen water on the surface of the normal ice cover. This water may originate from cracks in the ice cover, from seepage through unfrozen portions of soil forming the channel banks, from adjacent springs which normally discharge into the channel, or other sources. This water, flowing in sheets of an inch or less in thickness to a foot or more, freezes in a layer. Each overflow even is followed by another, with new flow atop the previously frozen sheet, the icing growing higher layer upon layer with its boundaries extending laterally according to the topography. River icing may grow for only part of the winter or throughout the period of below-freezing temperatures. Icing behavior usually varies but little year by year, depending on availability of the feeding water. An icing surface is generally flat but can be gently terraced with each step marking the frozen edge of a thin overflow layer. Occasionally ice mounds form with cracks developing therein providing outlets for the confined water forming the mounds. The water flows out, continuing the growth of the icing for a limited period. Smaller icing is generally confined to the stream or drainage channel; larger ones may spread over floodplains or pavements. With onset of the spring thawing season, runoff cuts channels through the icing to the streambed. Channels are widened by thawing, collapse of the ice forming the sides, and erosion. Depending on the size of the icing and its geographic location, its remnants may last only until May or June, or in colder regions it may last all summer. In extreme locations, they never completely melt and are known as perennial icing. River or stream icing occurring at culverts is also objectionable in that fish migration is obstructed.

2-8.1.2 **Ground icing.** Unlike river or stream icing; ground icing, while developing on certain topographic features, does not have clearly defined areas of activity. These icings are commonly referred to as seepage icings, due to the way their feed waters appear on the ground surface. Seepage icings may develop on nearly level ground or at points of contact of two different types of relief (such as at the base of a slope) or as encrustations on slopes. Ground icing begins to form at different times of the year depending on the sources and modes of discharge of the feeding waters. Where water seeps from the ground often or continuously, icing may begin to form in September or October, in which case it might also be termed a spring icing. Those forming where water does not usually issue from the ground generally begin to form in November or December or even later in the winter. A characteristic of ground icing is that its development begins with unfrozen water appearing on the ground surface or with the saturation and subsequent freezing of snow on the ground. This water may seep from the soil or fractures in the bedrock, or it may travel along the roots of vegetation, or it may issue from frost-induced cracks in the ground. As the seepage flows are exposed to the cold atmosphere, they freeze; to be followed repeatedly by additional seepages onto the icing surface that also freeze, building up successive thin ice layers, seldom over an inch thick. Ground icings may grow during the winter, being extremely sensitive to weather and local hydrologic conditions of the winter and its preceding seasons. Normally ground icings are limited in size as compared with stream spring icing since their source of supply is limited. Some rapid growth may occur with advent of thawing weather. When general thawing occurs, the ground icing will slowly waste away. This disintegration is unlike that of stream icings where sizable runoff streams can rapidly erode icing.

2-8.1.3 **Spring icing.** Springs found in a variety of topographic situations sustain continuous discharge, leading to early winter formation of icing, generally prior to ground icing. Spring outlets generally remain fixed in location and continue to grow throughout the winter, ultimately reaching a larger size than ground icing. A flow of 1 cu ft/min can create a 1-ft-deep icing covering an acre in one month. Spring icing melts away slowly on all sides and these icings are also eroded by spring water channel flow.

2-8.3 **Natural Factors Conducive to Icing Formation**. These can be summarized as follows:

2-8.3.1 A rainy season prior to freeze-up producing an abundance of groundwater in the annual frost zone of the soil or in the ground above the permafrost.

2-8.3.2 Low air temperatures and little snow during the first half of the winter, that is through January. Early heavy snow minimizes occurrence of icing.

2-8.3.3 Nearness of an impervious horizon such as the permafrost table to the ground surface.

2-8.3.4 Heavy snow depth accumulations during the latter part of winter.

2-8.4 **Effects of Man's Activities on Icing.** Airfields and heliports, in altering the natural physical environment, have profound effects on icing. The widespread clearing of vegetative cover, cutting and filling of soil, excavation of rock, and provisions for drainage, for example, greatly affect the natural thermal regime of the ground and the hydrologic regimes of both groundwater and surface water. Some of these effects are discussed below.

2-8.4.1 Removal of vegetation and organic soil with their generally higher insulation values than those of the construction materials replacing them results in increased seasonal frost penetration. This may create or aggravate nearby damming of groundwater flow and cause icing. Airfield and heliport pavement areas, kept clear of snow, lack its insulating value and are subject to deeper seasonal frost penetration, causing icing.

2-8.4.2 Cut faces may intersect the water table, and fill sections may block natural drainage channels. Construction compaction operations can reduce permeability of natural soils, blocking natural discharge openings.

2-8.4.3 In cut sections, water comes into contact with the cold atmosphere, forming ground icing where none occurred prior to the construction. Icing grows on the cut face, fills the adjacent drainage ditches with ice, and eventually reaches the pavement surface. In these conditions, deep snow on the slope and ditch insulates seepage from the cut face. Seepage water passes under the snow without freezing and reaches the snow-free pavement where it is sufficiently exposed to freeze. This type of man-made icing is the most common and troublesome type along pavements.

2-8.4.4 Snowplowing and storage of snow greatly affect the location and extent of icing by changing insulation values and damming seepage waters.

2-8.4.5 Channel realignment and grading into wider, more shallow sections, commonly done in airfield and heliport construction, renders the stream more susceptible to high heat losses and extensive freezing and formation of icing.

2-8.4.6 Drainage designers customarily size hydraulic structures to accommodate runoff from a specified design storm. In the Arctic and Subarctic, the size of hydraulic structures based solely on these well-founded hydrologic principles will usually result in inadequate capacity which will contribute or intensify icing formation. Culverts, small bridges, storm drains and inlets designed to accommodate peak design discharges are generally much too small to accommodate icing volumes before becoming completely blocked by ice. Once the drainage openings become blocked, icing upstream from the affected structures will grow markedly. The inadequacy of drainage facilities, both in capacity and number, because of failure to accommodate icing, leads to more serious effects of icing on engineering works.

2-8.5 **Methods of Counteracting Icing.** Several techniques are available for avoiding, controlling, or preventing icing. Although sound in principle, the methods are often applied without adequate understanding of the icing problems encountered, leading to unsuccessful or poor results. Selection of a particular method from the many that might be applied for the given set of conditions is based principally on economics. One must use a systems approach considering costs of installation plus costs of operation and maintenance, energy conservation, and environmental impact. Where feasible, methods requiring no fuel or electrical energy output or little or no service by maintenance personnel are preferred. The techniques for dealing with icings fall into two categories: *avoidance and control* and *prevention*. These are discussed below.

2-8.5.1 **Methods of icing avoidance and control**. These deal with the effects of the icing at the location being protected, so that the type of icing (river or stream, ground, or spring) is of little significance. Methods are as follows:

2-8.5.1.1 **Change of location.** Site facilities where icings do not occur. This is an economic consideration difficult to resolve in siting an airfield with its extensive area, grading and lateral clearance requirements.

2-8.5.1.2 **Raising grade.** This will deter or postpone icing formation but is costly and depends on availability of ample fill. There is also threat of embankment washouts resulting from ice-blocked facilities, and possibility of objectionable seepage effects.

2-8.5.1.3 **More and larger drainage structures.** Susceptibility to icing problems can be reduced by providing more and larger drainage facilities. Openings as much as 2 or 3 times as large as those required by conventional hydraulic design criteria will accommodate sizable icing volumes without encroaching on design flows. Culverts with large vertical dimensions, or small bridges in lieu of culverts, are advantageous. Provision for adequate drainage channels and conduits will facilitate diversion of meltwater runoff from icings, protecting the installation from washouts.

2-8.5.1.4 **Storage space**. This can be provided as a ponding basin or by shifting a cut face further back from the airfield or heliport. There, an icing can grow in an area where it will not encroach on operational facilities.

2-8.5.1.5 **Dams, dikes or barriers.** Known also as ice fences, these are often used to limit the horizontal extent of icings. Permanent barriers of earth, logs or lumber may be built between the source of the icing and the area to be protected. Temporary barriers may be erected of snow embankments, movable wooden fencing, corrugated metal, burlap, plastic sheeting, or expedient lumber construction. In some situations, a second or even third fence is required above the first as the icing grows higher.

2-8.5.1.6 **Culvert closures.** To prevent a culvert being filled with snow and ice, which requires a laborious spring clearing operation, closures are sometimes placed over the culvert ends in the fall. These can be of rocks to permit minor flows prior to freeze-up.

2-8.5.1.7 **Staggered (or stacked) culverts.** This involves placement of two (or more) culverts, one at the usual location at the base of the fill, the other(s) higher in the fill. When the lower culvert becomes blocked by an icing accumulation, the higher ones carry initial spring runoff over the icing. As the spring thaw progresses, the lower one becomes cleared, eventually carrying the entire flow. In cases where there is limited height, the second culvert is placed to the side with its invert at a slightly higher elevation. The ponding area available for icing accumulations must be large enough to store an entire winter's ice without having the icing reach the upper culverts or the elevation of the area being protected.

2-8.5.1.8 **Heat.** Icing is commonly controlled by the application of heat in any of several ways, the objective being not to prevent icing but to establish and maintain thawed channels through it to minimize their growth and to pass spring runoff.

2-8.5.1.9 **Steam.** This method, common in North America, is used to thaw culvert openings and to thaw channels into icing for collecting icing feed water or early spring runoff. Steam, generated in truck-mounted boilers, is conducted through hoses to portable steam lances, or through hoses temporarily attached to permanently installed thaw pipes supported inside the tops of the culverts. Thaw pipes of 3/8- to 2-inch diameter have been used. The thaw pipe is terminated by a vertical riser at each end of the culvert, extending high enough to permit access above accumulated ice and snow. The pipe is filled with antifreeze, with the risers capped when not in use.

2-8.5.1.10 **Fuel oil heaters.** These heaters, known as firepots, are in common use. They consist of a 55-gallon oil drum, equipped with an oil burner unit (railroads often use coal or charcoal as fuel). The drum fed from a nearby fuel supply, is usually suspended from a tripod at the upstream end of the culvert. A continuous fire maintains a thaw pit in the icing. Fuel consumption varies, averaging about 30 gallons per day. Water, flowing over the icing, enters the pit where it receives heat, passes through the culvert, hopefully without refreezing before it flows beyond the area to be protected. While firepots are simple devices, they are inefficient energy sources due to loss of most heat to the atmosphere rather than to the water or icing. Firepots are in decreasing favor due to high maintenance requirements and difficulty in preventing theft of fuel in remote locations.

2-8.5.1.11 **Electrical heating.** Use of insulated heating cables to heat culverts is a recent adaptation successfully used where electrical power is available or, in important locations, where small generating stations would prove feasible. Heating cables have been used, not to prevent icing, but to create and maintain a thawed tunnel-like opening in an icing to minimize its growth and to provide for spring runoff. Cable can be strung in the fall within the culvert and, in some cases, along its upstream drainageway and removed in the spring. Cable can also be permanently installed in a small diameter metal pipe inside the culvert or buried at shallow depth under a drainage ditch or channel. Common heat output is 40 to 50 watts/lineal ft with minimum heat lost to the atmosphere. A tunnel about 2-3 ft wide and 4-5 ft high is achieved by later winter. Electrical heating requires much less attention by maintenance personnel than steam thawing.

2-8.5.1.12 **Breaking and removing accumulated ice.** This common technique, whether by manual or mechanical equipment, should be practiced only as an expedient or emergency measure. Timing of such operations, as for the following two methods, critically limits their effectiveness.

2-8.5.1.13 **Blasting.** This has a twofold objective—physical removal of ice and fracturing ice to provide paths for water flow deep in the icing. This flow can enlarge openings and still remain protected from the atmosphere and refreezing.

2-8.5.1.14 **Deicing chemicals.** Chemicals such as sodium or calcium chloride are sometimes used to prevent refreezing of a drainage facility, once it has been freed of ice by other means. A common practice is to place a burlap bag containing the salt at a culvert inlet, allowing the compound to be slowly dissolved by flow, the solution lowering the freezing point of the water. Objections are the detrimental effects on fish and wildlife, vegetation, and other downstream water uses and corrosive effects on metal pipe.

2-8.5.2 **Methods of icing prevention.** These preventive techniques are best classified according to the general type of icing (Section 2-8.2), as follows:

2-8.5.2.1 **River or stream icing**

a. **Channel modification.** Straightening and deepening a channel can prevent icing, although frequent maintenance is usually required to counteract the stream's tendency to resume natural configuration by erosion and deposition. Rock-fill gabions have been used to create a deep, narrow channel for low winter discharges. Such deepened channels permit formation of ice cover to normal thickness while providing adequate space beneath for flow. Deepening at riffles, rapids, or drop structures is especially important as icing is more likely to form in these shallow areas.

b. **Insulation of critical sections.** This icing may be prevented by insulating critical sections of the stream where high heat losses cause excessive thickening of the normal ice cover, to constrict or completely block flow and result in icing formation. These sections may be located under a bridge or taxiway or at riffles or rapids. The insulation which may be placed on the initial ice cover may consist of soil, snow, brush, peat, sawdust or other material, typically 1 to 2 ft thick. Another way is to cover the stream before ice forms, using logs, timber, or corrugated metal as a support for insulating material, later augmented by snowfall. Insulating covers, while beneficial in lessening heat losses from the stream, must be removed each spring before annual freshets. They may also be washed downstream to become obstructions if high water occurs prior to cover removal.

c. **Frost belts.** Known also as "permafrost belts," these are further discussed below under Ground Icing. A frost belt is essentially a ditch or cleared strip of land upstream or upslope from the icing problem area. If organic soil and vegetative cover are removed and the area is kept clear of snow during the first half of the winter, deep seasonal frost will act as a dam to water seeping through the ground, forcing it to the surface where it will form an icing upstream or upslope from the belt. In applying this technique to a drainage channel, a belt is formed by periodically cutting transversely into the ice to cause the bottom of the ice cover to lower and merge with the bed. In this way, the icing is induced to form away from the bridge or culvert entrance being protected.

2-8.5.2.2 **Ground icing.** The most successful methods of preventing ground icing involve drainage. Other procedures depend on preventing formation in one location by inducing formation elsewhere. Principal methods are cited below.

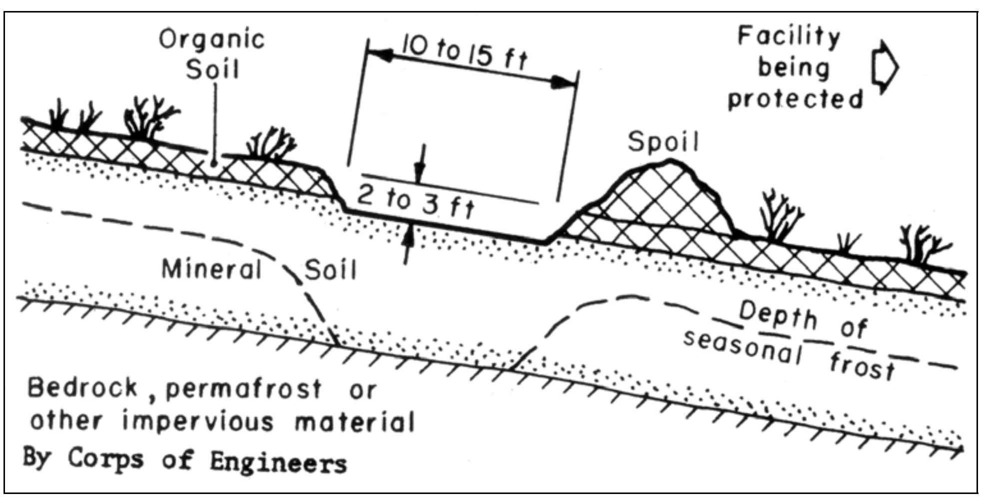
a. **Surface drainage.** This may be accomplished by a network of ditches located so as to drain the soil surface in the region of icing development. Ideally these ditches will be sited in compliance with airfield/heliport lateral safety clearance criteria and be narrow and deep so as to drain the soil to an appreciable depth and to expose only a small surface area to heat loss to the atmosphere. In some cases, these drainage ditches are covered and insulated to maintain flow in winter. Open ditches can be as narrow as 1 ft or, if insulated, about 3 ft wide by 3 ft deep.

b. **Subsurface drainage.** In seasonal frost areas, subsurface drainage systems are more suitable than surface drains because of their better resistance to freezing and ability to intercept more groundwater. They are not suitable for use in permafrost areas due to freezing. Subsurface drainage systems can use any of numerous types of perforated, slotted or open-jointed pipe materials most commonly in 6-in.-diameter size. Improved resistance to freezing can be obtained by placing an insulation layer above the usual granular backfill surrounding the subdrain but beneath the final native soil backfill. In any case, water collected must be conveyed to an outlet away from the area being protected even if it forms an icing at that point.

c. **Insulation of ground.** In some cases ground icings can be prevented by insulating the ground in areas where deep seasonal frost penetration forms a dam, blocking groundwater flow. Insulating material may be snow, soil, brush, or peat. This technique may merely shift the location where an impervious frost dam occurs. It is essential that the insulation of the ground extend under the pavement being protected to assure that ground water flow is maintained past it. Otherwise, seasonal frost penetration under a snow-free airfield pavement would act as a frost dam and cause an icing to form upslope from the area. Suitable insulation materials for pavements are available and have been used.

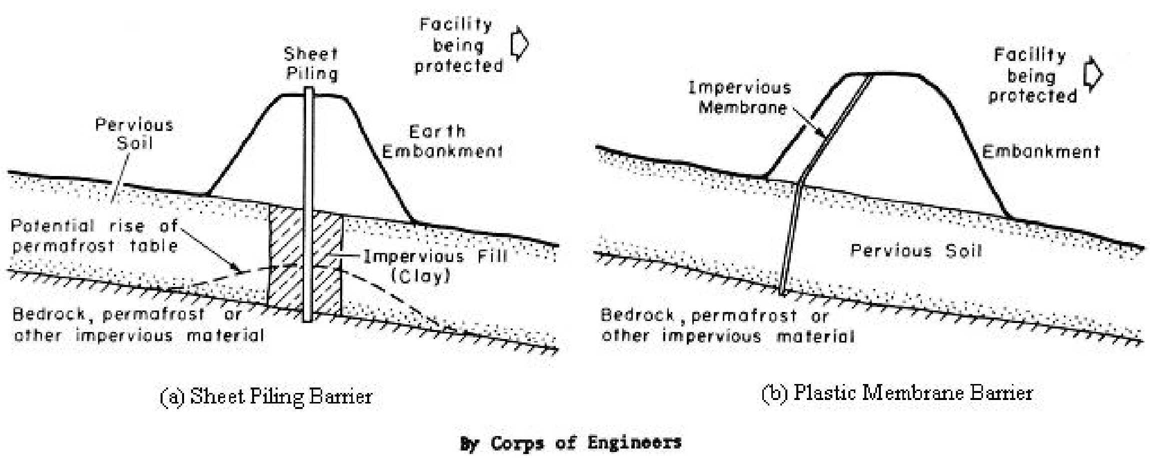
d. **Frost belts.** Successful use of frost belts requires careful siting, planning and maintenance. They may be either permanent or seasonal. The permanent type belt, as mentioned above for control of river or stream icing, is a strip of land cleared of organic soil and vegetation, extending across a slope normal to the direction of seepage flow. Seasonal frost beneath this belt, merging with or approaching some impervious base, causes an icing to form upslope from the belt location. The belt must be long enough to prevent the icing from extending around the ends of the belt and approaching the airfield or other areas being protected. Such a belt is usually about 2 to 3 ft deep and 10 to 15 ft wide. Spoil from the excavation is placed as a low ridge on the downslope side of the belt (Figure 2-13). The shape of the frost belt depends on the topography; often it is slightly convex downslope, or made of two straight segments meeting at an angle of 160-170 degrees on the upslope side of the belt. Sometimes more than one belt is needed, the belts being arranged parallel to each other with their spacing depending on the channel slope. Permanent frost belts require attention to avoid degradation of the permafrost table underneath as the insulation of the ground has been reduced by removing the organic soil and vegetative cover. After a few years, the permafrost table may lower so much that the seasonal frost penetration in the winter will not reach it. In such a case, seepage flow in the soil would not be stopped at the belt; an icing does not develop at the belt but occurs instead downslope at the airfield or other facility intended to be protected. This can be avoided by covering the belt area in the spring with an insulating material and removing it in the fall before the onset of winter frosts. The belt must be kept clear of snow through the first half of the winter to permit rapid and deep seasonal frost penetration. Seasonal type frost belts are free from most maintenance requirements associated with the permanent type and are much simpler and more economical to construct. Instead of preparing a ditch in the ground, one merely clears a strip of snow at the desired belt location and keeps it free of snow during the first half of the winter. The cleared snow is piled downslope of the belt, forming a ridge. The chief advantage of the seasonal belt is that it is less likely to degrade the underlying permafrost; this objective can be further assured by relocating the belt up- or downslope in successive winters. A disadvantage of the seasonal belt is that seasonal frost penetrates below it more slowly, owing to the high specific heat of the wet organic soil and the insulation afforded by the vegetation left in place. It therefore takes longer for a frost dam to form and stop the flow of seepage water. This may permit formation of some icing at the downslope protected area early in the winter before the seasonal frost belt attains full effectiveness. Frost belts have not been widely accepted because of neglect in placement of summer insulation and priority attention to snow removal from pavements rather than from frost belt areas in the winter. Frost belts are much easier to maintain in locations where the impervious base which restrict groundwater flow is other than permafrost, and thus is not subject to degradation.

**Figure 2-13. Typical Cross Section of a Frost Belt Installation**



e. **Earth embankments and impervious barriers.** Ground icing formation can also be prevented by use of earth embankments combined with impervious barriers to groundwater flow. These are placed well away from the area to be protected and function similarly to frost belts in that they dam seepage flow through the soil, causing it to rise to the ground surface where it freezes to form an icing. In southern permafrost zones where permafrost is close to freezing temperatures, embankments may cause the permafrost to melt, leading to subsidence. Methods of developing the impervious barrier include trenching across the slope down to the impervious stratum, filling the trench with clay and then driving a row of sheet piling through it extending several feet above the surface to aid in ponding (Figure 2-14a). Other expedients include use of plastic membrane instead of piling (Figure 2-14b) or burial or horizontal air duct pipe (12 to 18 in.), located usually 4 to 6 ft below the bottom of the embankment.

**Figure 2-14. Earth Embankments with Impervious Barriers**




Vertical air shafts from the horizontal ducts permit cold winter air to permeate the system, removing heat from the ground and freezing the soil beneath the embankment to create an impervious barrier. The vertical air shafts are sealed in the summer to prevent excessive thawing in the soil. A problem which has arisen in some duct installations is that if they are not completely watertight, infiltrated water will freeze in the duct, causing an obstruction, generally difficult to clear. As this type installation would obstruct seepage flow year-round, rather than just in winter, gated openings must be provided to allow accumulated water to flow downslope during the summer. The openings are closed all winter to assure that the icing will form upslope from the embankment. An innovation is use of a steel mesh grid with apertures 8 to 32 in. square. These permit passage of water when the air is warm, but gradually freeze until a blockage forms in subfreezing weather. Grids must be removed in the summer to avoid debris accumulation.

## 2-9. AREAS OTHER THAN AIRFIELDS

### 2-9.1 Design Storm

2-9.1.1 For such developed portions of military installations as administrative, industrial, and housing areas, the design storm will normally be based on rainfall of 10-yr frequency. Potential damage or operational requirements may warrant a more severe criterion; in certain storage and recreational areas, a lesser criterion may be appropriate. (With concurrence of the using Service, a lesser criterion may also be employed in regions where storms of an appreciable magnitude are infrequent and either damages or operational capabilities are such that large expenditures for drainage are not justified.)

2-9.1.2 The design of roadway culverts will normally be based on 10-yr rainfall. Examples of conditions where greater than 10-yr rainfall may be used are areas of steep slope in which overflows would cause severe erosion damage; high road fills that impound large quantities of water; and primary diversion structures, important bridges, and critical facilities where uninterrupted operation is imperative.

2-9.1.3 Protection of military installations against floodflows originating from areas exterior to the installation will normally be based on 25-yr or greater rainfall, again depending on $\frac{HW}{D}$ operational requirements, cost-benefit considerations, and nature and consequences of flood damage resulting from the failure of protective works. Justification for the selected design storm will be presented, and, if appropriate, comparative costs and damages for alternative designs should be included.

2-9.1.4 Rainfall intensity will be determined from the best available intensity-duration-frequency data. Basic information of this type will be taken from such publications as (see Appendix A for referenced publications):

*Rainfall Frequency Atlas of the United States.* Technical Paper No. 40.

*Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands.* Technical Paper No. 42.

*Rainfall-Frequency Atlas of the Hawaiian Islands. Technical Paper No. 43.*

*Probable Maximum Precipitation and Rainfall Frequency Data for Alaska. Technical Paper No. 47.*

*TM 5-785/AFM 88-29/NAVFAC P-89.*

These publications may be supplemented as appropriate by more detailed publications of the Environmental Data and Information Center and by studies of local rainfall records. For large areas and in studies involving unit hydrography and flow-routing procedures, appropriate design storms must be synthesized from areal and time-distribution characteristics of typical regional rainfalls.

2-9.1.5 For some areas, it might reasonably be assumed that the ground would be covered with snow when the design rainfall occurs. If so, snowmelt would add to the runoff. Detailed procedures for estimating snowmelt runoff are given in Section 2-5. It should be noted, however, that the rate of snowmelt under the range of hydro-meteorological conditions normally encountered in military drainage design would seldom exceed 0.2 in. per hour and could be substantially less than that rate.

2-9.1.6 In selecting the design storm and making other design decisions, particular attention must be given to the hazard to life and other disastrous consequences resulting from the failure of protective works during a great flood. Potentially hazardous situations must be brought to the attention of the using service and others concerned so that appropriate steps can be taken.

2-9.2 **Infiltration and Other Losses**

2-9.2.1 Principal factors affecting the computation of runoff from rainfall for the design of military drainage systems comprise initial losses, infiltration, transitory storage, and, in some areas, percolation into natural streambeds. If necessary data are available, an excellent indication of the magnitudes of these factors can be derived from thorough analysis of past storms and recorded flows by the unit-hydrograph approach. At the onset of a storm, some rainfall is effectively retained in “wetting down” vegetation and other surfaces, in satisfying soil moisture deficiencies, and in filling surface depressions. Retention capacities vary considerably according to surface, soil type, cover, and antecedent moisture conditions. For high intensity design storms of the convective, thunderstorm type, a maximum initial loss of up to 1 in. may be assumed for the first hour of storm precipitation, but the usual values are in the range of 0.25 to 0.50 in./hr. If the design rainfall intensity is expected to occur during a storm of long duration, after substantial amounts of immediately prior rain, the retention capacity would have been satisfied by the prior rain and no further assumption of loss should be made.

2-9.2.2 Infiltration rates depend on type of soils, vegetal cover, and the use to which the areas are subjected. Also, the rates decrease as the duration of rainfall increases. Typical values of infiltration for generalized soil classifications are shown in Table 2-3. The soil group symbols are those given in MIL-STD-619, Unified Soil Classification System for Roads, Airfields, Embankments, and Foundations. These infiltration rates are for uncompacted soils. Studies indicate that compacted soils decrease infiltration values from 25 to 75 percent, the difference depending on the degree of compaction and the soil type. Vegetation generally decreases the infiltration capacity of coarse soils and increases that of clayey soils.

2-9.2.3 Peak rates of runoff are reduced by the effect of transitory storage in watercourses and minor ponds along the drainage route. The effects are reflected in the C factor of the Rational Formula (given below) or in the shape of the unit hydrography. Flow-routing techniques must be used to predict major storage effects caused by natural topography or man-made developments in the area.

**Table 2-3. Typical Values of Infiltration Rates**

| Description | Soil Group Symbol | Infiltration, in./hr |
|---|---|---|
| Sand and gravel mixture | GW, GP<br>SW, SP | 0.8-1.0 |
| Silty gravels and silty sands to inorganic silt, and well-developed loams | GM, SM<br>ML, MH<br>OL | 0.3-0.6 |
| Silty clay sand to sandy clay | SC, CL | 0.2-0.3 |
| Clays, inorganic and organic | CH, OH | 0.1-0.2 |
| Bare rock, not highly fractured | … | 0.0-0.1 |
| U.S. Army Corps of Engineers | | |

2-9.2.4 Streambed percolation losses to direct runoff need to be considered only for sandy, alluvial watercourses, such as those found in arid and semiarid regions. Rates of streambed percolation commonly range from 0.15 to 0.5 cfs/acre of wetted area.

2-9.3 **Runoff Computations**

2-9.3.1 Design procedures for drainage facilities involve computations to convert rainfall intensities expected during the design storm into runoff rates which can be used to size the various elements of the storm drainage system. There are two basic approaches: first, direct estimates of the proportion of average rainfall intensity that will appear as the peak runoff rate; and, second, hydrography methods that depict the time-distribution of runoff events after accounting for losses and attenuation of the flow over the surface to the point of design. The first approach is exemplified by the Rational Method which is used in the large majority of engineering offices in the United States. It can be employed successfully and consistently by experienced designers for drainage areas up to 1 square mile in size. *Design and Construction of Sanitary and Stem Sewers*, ASCE Manual No. 37, and *Airport Drainage*, FAA AC 150/5320-5B, explain and illustrate use of the method. A modified method is outlined below. The second approach encompasses the analysis of unit-hydrograph techniques to synthesize complete runoff hydrography.

2-9.3.2 To compute peak runoff the following empirical formula can be used

$$Q = C\,(I-F)\,A$$

where

Q = discharge or peak rate of runoff, cfs
C = coefficient
I = rainfall intensity, in./hr

A = drainage area, acres, total area of clear opening, or cross-sectional are of flow, $ft^2$

This equation is known as the modified rational method.

2-9.3.2.1 C is a coefficient expressing the percentage to which the peak runoff is reduced by losses (other than infiltration) and by attenuation owing to transitory storage. Its value depends primarily on surface slopes and irregularities of the tributary area, although accurate values of C cannot readily be determined. For most developed areas, the apparent values range from 0.6 to 1.0. However, values as low as 0.20 for C may be assumed in areas with low intensity design rainfall and high infiltration rates on flat terrain. A value of 0.6 may be assumed for areas left ungraded where meandering-flow and appreciable natural-ponding exists, slopes are 1 percent or less, and vegetal cover is relatively dense. A value of 1.0 may be assumed applicable to paved areas and to smooth areas of substantial slope with virtually no potential for surface storage and little or no vegetal cover.

2-9.3.2.2 The design intensity is selected from the appropriate intensity-duration-frequency relationship for the critical time of concentration and for the design storm frequency. Time of concentration is usually defined as the time required, under design storm conditions, for runoff to travel from the most remote point of the drainage area to the point in question. In computing time of concentration, it should be kept in mind that, even for uniformly graded bare or turfed ground, overland flow in "sheet" form will rarely travel more than 300 or 400 ft before becoming channelized and thence move relatively faster; a method which may be used for determining travel-time for sheet flow is given in Chapter 3. Also, for design, the practical minimum time of concentration for roofs or paved areas and for relatively small unpaved areas upstream of the uppermost inlet of a drainage system is 10 min; smaller values are rarely justifiable; values up to 20 min may be used if resulting runoff excesses will not cause appreciable damage. A minimum time of 20 min is generally applicable for turfed areas. Further, the configuration of the most remote portion of the drainage area may be such that the time of concentration would be lengthened markedly and thus design intensity and peak runoff would be decreased substantially. In such cases, the upper portion of the drainage areas should be ignored and the peak flow computation should be based only on the more efficient, downstream portion.

2-9.3.2.3 For all durations, the infiltration rate is assumed to be the constant amount that is established following a rainfall of 1 hour duration. Where F varies considerably within a given drainage area, a weighted rate may be used; it must be remembered, however, that previous portions may require individual consideration, because a weighted overall value for F is proper only if rainfall intensities are equal to or greater than the highest infiltration rate within the drainage area. In design of military construction drainage systems, factors such as initial rainfall losses and channel percolation rarely enter into runoff computations involving the Rational Method. Such losses are accounted for in the selection of the C coefficient.

2-9.3.3 Where basic hydrologic data on concurrent rainfall and runoff are adequate to determine unit hydrography for a drainage area, the uncertainties inherent in application of the Rational Method can largely be eliminated. Apparent loss rates determined from unit-hydrograph analyses of recorded floods provide a good basis for estimating loss rates for storms of design magnitude. Also, flow times and storage effects are accounted for in the shape of the unit-hydrograph. Where basic data are inadequate for direct determination of unit-hydrographs, use may be made of empirical methods for synthesis. Use of the unit-hydrograph method is particularly desirable where designs are being developed for ponds, detention reservoirs, and pump stations; where peak runoff from large tributary areas is involved in design; and where large-scale protective works are under consideration. Here, the volume and duration of storm runoff, as opposed to peak flow, may be the principal design criteria for determining the dimensions of hydraulic structures.

2-9.3.4 Procedures for routing storm runoff through reservoir-type storage and through stream channels can be found in publications listed in Appendix B and in the available publications on these subjects.

# CHAPTER 3

## SURFACE DRAINAGE FACILITIES FOR AIRFIELDS AND HELIPORTS

3-1 **PURPOSE AND SCOPE.** This chapter prescribes standards of design of surface drainage of airfields and heliports. Problems involved in the design of drainage facilities are discussed, and convenient methods of estimating design capacities are outlined. These standards can be altered when necessary to meet special problems or unusual conditions on the basis of good engineering practice. Design of drainage facilities for arctic or subarctic regions is discussed in Chapter 8 (see Appendix A for referenced publications).

3-2 **DESIGN OBJECTIVES FOR AIRFIELD AND HELIPORT SURFACE DRAINAGE.** Surface drainage facilities will be designed to suit the mission and the importance of airfields or heliports; the design capacity will be adequate to accomplish the following objectives:

3-2.1 **Surface Runoff from the Design Storm**. Surface runoff from the selected design storm will be disposed of without damage to the airfield facilities or significant interruption of normal traffic.

3-2.2 **Surface Runoff from Storms Exceeding the Design Storm**. Surface runoff from storm exceeding the design storm will be disposed of with minimum damage to the airfield facilities and with the shortest practicable interruption of normal traffic. The primary runway will remain operational under all conditions.

3-2.3 **Reliability of Operation**. The drainage system will provide maximum practicable reliability of operation under all climatic conditions.

3-2.4 **Maintenance**. The drainage system in the immediate vicinity of operational facilities will require minimum maintenance.

3-2.5 **Coordination**. Basic data obtained during preliminary field investigations will be coordinated with the facility master plan and with other agencies having jurisdiction over conservation, flood control, drainage, and irrigation.

3-2.6 **Safety Requirement**. Separate drainage and containment should be provided in areas with a high potential for fuel spills. This provision will allow spilled fuel to be promptly separated, collected, and removed from the rest of the drainage system.

3-2.7 **Future Expansion**. Drainage design should allow for future expansion with a minimum of expense and traffic interruption.

3-2.8 **Environmental Impact**. Drainage facilities will be constructed with minimal impact on the environment.

## 3-3 DRAINAGE PROTECTION REQUIRED

3-3.1 **Degree of Drainage Protection**. The degree of drainage protection depends largely on the importance of the airfield or heliport, the mission and volume of traffic to be accommodated, and the necessity for uninterrupted service. Within certain limits the degree of drainage protection should be sufficient so that hazards can be avoided during operation.

3-3.2 **Frequency of the Design Storm**. Drainage for military airfields and heliports will be based on a 2-yr design frequency, unless exceptional circumstances require greater protection. Temporary ponding will be permitted on graded areas adjacent to runway and taxiway aprons, or airfield or heliport pavements other than primary runways. Ponding will not be permitted on primary runways under any condition. To determine the extent of ponding permissible on areas where ponding is allowed, possible damage of pavement subgrades and base courses as a result of occasional flooding must be considered. In addition, ponding basins must conform to grading standards.

## 3-4 HYDROLOGIC CONSIDERATIONS

3-4.1 **Definitions**. The following definitions are used in the development of hydrologic concepts.

3-4.1.1 **Design frequency**. The average frequency with which the design event, rainfall or runoff, is equaled or exceeded. The reciprocal of frequency is the annual probability of occurrence. Design frequency is selected to afford the degree of protection deemed necessary. Except in special circumstances, the 2-yr frequency, that is, an annual probability of occurrence of 0.5, is considered satisfactory for most airfields.

3-4.1.2 **Design storm**. The standard rainfall intensity-frequency relation, lasting for various durations of supply. The design storm is used to compute the runoff to be carried in drainage facilities.

3-4.1.3 **Rainfall-excess**. The amount of rainfall which appears as surface runoff. Rainfall-excess is rainfall less losses to infiltration or other abstractions.

3-4.1.4 **Standard supply**. The standard intensity-frequency-duration relationship of the selected design storm less losses for infiltration. Standard supply is usually designated by the average rainfall intensity in inches per hour at the 1-hr duration.

3-4.2 **Design Methods**. The design procedures for drainage facilities involve computations to convert the rainfall intensities expected from the design storm into runoff rates which can be used to size the various elements of the storm drainage system. There are two basic approaches: direct estimates of the proportion of the average rainfall intensity which will appear as the peak rate of runoff and hydrographic methods which account for losses such as infiltration and for the effects of flow over the surface to the point of design. The first approach is exemplified by the "Rational Method," which is used in most engineering offices in the United States. This approach can be used successfully by experienced designers for drainage areas up to 1 square mile in size. ASCE Manual of Practice No. 37 and FAA AC 150/5320-5B explain and illustrate the use of the Rational Method. Chapter 4 presents a modified Rational Method. The second approach includes techniques to synthesize hydrography of runoff. Where studies of large drainage areas or complex conditions of storage require hydrography, the designer should refer to the sources listed in the Bibliography and other publications on these subjects. The method described in Sections 3-5 through 3-9 and developed and illustrated in Section 3-11 and Design Example C-3 combines features from both basic approaches to determine runoff.

## 3-5 RAINFALL

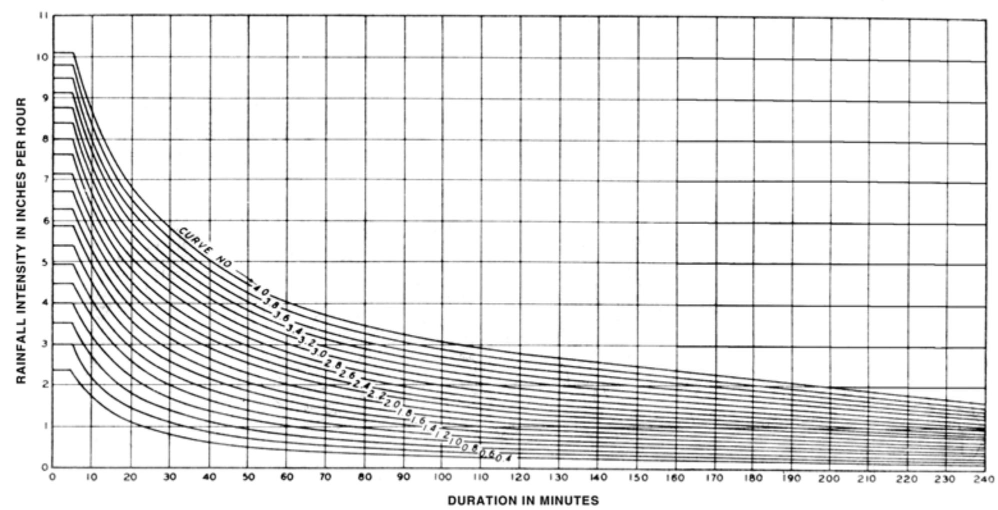
3-5.1 **Intensity-Frequency Data.** Studies of rainfall intensity-frequency data indicate a fairly consistent relation between the average intensities of rainfall for a period of 1 hr and the average intensities at the same frequency for periods less than 1 hr, regardless of the geographical location of the stations. The average rainfall for a 1-hr period at various frequencies for the continental United States, excluding Alaska, may be determined from Figure 2-2. Data for other locations are available from the Office, Chief of Engineers, and the National Oceanic and Atmospheric Administration, National Weather Service (formerly the U.S. Weather Bureau). For Alaska, data may be obtained from Figure 2-1 and U.S. Weather Bureau Technical Paper No. 47. Data for Puerto Rico and the Virgin Islands and for Hawaii may be obtained from U.S. Weather Bureau Technical Papers No. 42 and 43, respectively. For any frequency, the 1-hr rainfall intensity is considered a design-storm index for all average intensities and duration of storms with the same frequency.

3-5.2 **Standard Rainfall Intensity-Duration Curves.** Figure 3-1 shows the standard curves that have been compiled to express the rainfall intensity-duration relationships and the standard supply (infiltration subtracted) which are satisfactory for the design of airfield drainage systems in the continental United States. The curves may be used for all locations until standard curves are developed for any region under consideration. As an example, assume the average rainfall intensity is required for a 40-min design storm based on a 2-yr frequency in central Kentucky. From Figure 2-2 the 2-yr 1-hr rainfall is found to be 1.4 in./hr. In Figure 3-1, supply curve No. 1.4 is used with the 40-min duration of storm to determine a rainfall intensity of 1.9 in./hr.

3-5.3 **Incomplete Data.** In areas where rainfall data are incomplete or unavailable, the methods described in Section 3-11 can be used to develop design rainfall information.

3-5.4 **Design Frequency.** Drainage systems are normally designed for the maximum runoff from rainfall with a certain frequency of occurrence. The design frequency indicates the average frequency at which some portions or all of the drainage system will be taxed to capacity. After the design frequency is selected, computations must be made to determine the critical duration of rainfall necessary to produce the maximum rate of runoff for the specific areas involved. Ordinarily, the maximum rate of runoff occurs when all tributary areas are contributing to the system. However, in cases of odd-shaped areas and areas containing both paved and turfed areas, peak runoff rates may occur before all areas are contributing. Factors affecting the critical duration of rainfall are primarily the length of overland flow, extent of surface detention, ponding, and characteristics of the runoff surfaces.

**Figure 3-1. Standard Rainfall Intensity-Duration Curves or Standard Supply Curves**




NOTE: CURVE NUMBERS CORRESPOND TO 1-HOUR VALUES OF RAINFALL OR SUPPLY INDICATED BY RESPECTIVE CURVES. ALL POINTS ON THE SAME CURVE ARE ASSUMED TO HAVE THE SAME AVERAGE FREQUENCY OF OCCURRENCE.

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3-5.5 **Storms of Greater Severity than the Design Storm.** The design storm alone is not a completely reliable criterion for the adequacy of drainage facilities. Often storms more severe than the design storm can cause excessive damage and affect operations. Therefore, the probable consequences of storms greater than the design storm should be considered before deciding on the adequacy of facilities designed to handle only the design storm.

3-6 **INFILTRATION.** Infiltration refers to the rate of absorption of rainfall into the ground during a design storm, which is assumed to occur after a 1-hr period of antecedent rainfall. Wherever possible, determine average infiltration rates from a study of runoff records near the airfield from infiltrometer studies or from similar acceptable information. Suggested mean values of infiltration for classifications are shown in Table 2-3. The soil group symbols are those given in generalized soil MIL-STD-619. Infiltration values are for uncompacted soils. Studies indicate that where soils are compacted, infiltration values decrease; the percentage decrease ranges from 25 to 75 percent, depending on the degree of compaction and the types of soil. Vegetation generally decreases infiltration capacity of coarse soils and increases that of clayey soils. The infiltration rate after 1 hr of rainfall for turfed areas is approximately 0.5 in./hr and seldom exceeds 1.0 in./hr. The infiltration rate for paved or roofed areas, blast protective surfaces, and impervious dust-palliative-treated areas is zero.

3-7 **RATE OF SUPPLY.** Rate of supply refers to the difference between the rainfall intensity and the infiltration capacity at the same instant for a particular storm. To simplify computations, the rainfall intensity and the infiltration capacity are assumed to be uniform during any specific storm. Thus the rate of supply during the design storm will also be uniform.

3-7.1 **Average Rate of Supply.** Average rates of supply corresponding to storms of different lengths and the same average frequency of occurrence may be computed by subtracting estimated infiltration capacities from rainfall intensities represented by the selected standard rainfall intensity-duration curve in Figure 3-1. For convenience and since no appreciable error results, standard supply curves are assumed to have the same shapes as those of the standard rainfall intensity-duration curves shown in Figure 3-1. For example, if supply curve No. 2.2 in Figure 3-1 were selected as the design storm and the infiltration loss during a 1-hr storm were estimated as 0.6 in., supply curve No. 1.6 would be adopted as the standard supply curve for the given areas.

3-7.2 **Weighted Standard Rate of Supply Curves.** Drainage areas usually consist of combinations of paved and unpaved areas having different infiltration capacities. A weighted standard supply should be established for the composite drainage areas by weighting the standard supply curve numbers adopted for paved and unpaved surfaces in proportion to their respective tributary area.

3-8 **RUNOFF.** The method of runoff determination described herein is based on an overland flow model. Details are given in Section 3-11.3.

3-8.1 **Overland Flow.** The surface runoff resulting from a uniform rate of supply is termed “overland flow.” If the rate of supply were to continue indefinitely, the runoff would rise to a peak rate and remain constant. Ordinarily, the peak rate is established after all parts of the drainage surface are contributing to runoff. However, in cases of odd-shaped areas and areas containing both paved and turfed areas, peak runoff rates may occur before all areas are contributing. The elapsed time for runoff to build to a peak is termed the “time of concentration,” which depends primarily on the coefficient of roughness, the slope, and the effective length of the surface. When the supply terminates, the runoff rate diminishes, but continues until the excess stored on the surface drains away.

3-8.2 **Effective Length.** The effective length to the point under consideration must account for the effects of overland and channel flow and for the differences in roughness and slope of the drainage surface. Methods for determining effective length are presented in Section 3-11.2.

3-8.3 **Maximum Rate of Runoff.** Figure 2-4 shows the results of overland flow computations using standard supply curves No. 2.0 and 2.2. Curves for other supply rates are given in Figures 3-2 through 3-9). Figure 2-4 depicts the relationships between the rate of supply, σ, in inches per hour; critical duration of supply or time of concentration, $t_c$; the effective length of overland flow, L; and the resulting maximum rate of runoff. The curves are not complete hydrography for any specific design storm, but are peak rates of runoff from individual storm events of various durations, all having the same frequency of occurrence. Use of the curves can be illustrated by using supply curve No. 2.0, as follows:

3-8.3.1 Assume the effective length of overland flow is 300 ft:

3-8.3.1.1 The critical duration of supply, that is, the time of concentration, to provide maximum runoff is obtained by reading vertically downward from the point where $t_c$ and L = 300 ft curves intersect. This value is found to be 24 min.

3-8.3.1.2 The maximum rate of runoff from overland flow is obtained by reading horizontally across from the point where $t_c$ and L = 300 ft curves intersect. This value is found to be 2.5 in./hr or 2.5 cubic feet per second per acre (cfs/acre).

3-8.3.1.3 The average rate of supply over the area is obtained by reading vertically upward from the point where the $t_c$ and L = 300 ft curve intersect to the σ curve and then reading horizontally across from this point. This value is found to be 3.6 in./hr or 3.6 cfs/acre.

3-8.3.2 Assume the critical duration of supply is 30 min:

3-8.3.2.1 The average rate of supply is obtained by reading horizontally across from the point where the duration of supply = 30 min and σ intersect. This value is found to be 3.2 in./hr or 3.2 cfs/acre.

3-8.3.2.2 The effective length is obtained by reading the point where $t_c$ and the duration of supply = 30 min intersect. This is found to be 500 ft.

3-8.3.2.3 The maximum rate of runoff is obtained by reading horizontally across from this point. This is found to be 2.0 in./hr or 2.0 cfs/acre.

3-9 **STORAGE.** The supply curves in Figure 2-4 assume no surface storage. Where surface storage or ponding is permitted, the overland flow will be stored temporarily and released as the pond drains. The discharge rate from the pond will depend on the volume of storage provided and the extent to which the surface area of the pond reduces the effective length of overland flow. Methods for designing with temporary storage or ponding are given in Section 3-11.4.

**Figure 3-2. Rates of Runoff Corresponding to Supply Curves No. 0.4 and 0.6; n = 0.40 and S = 1 percent**

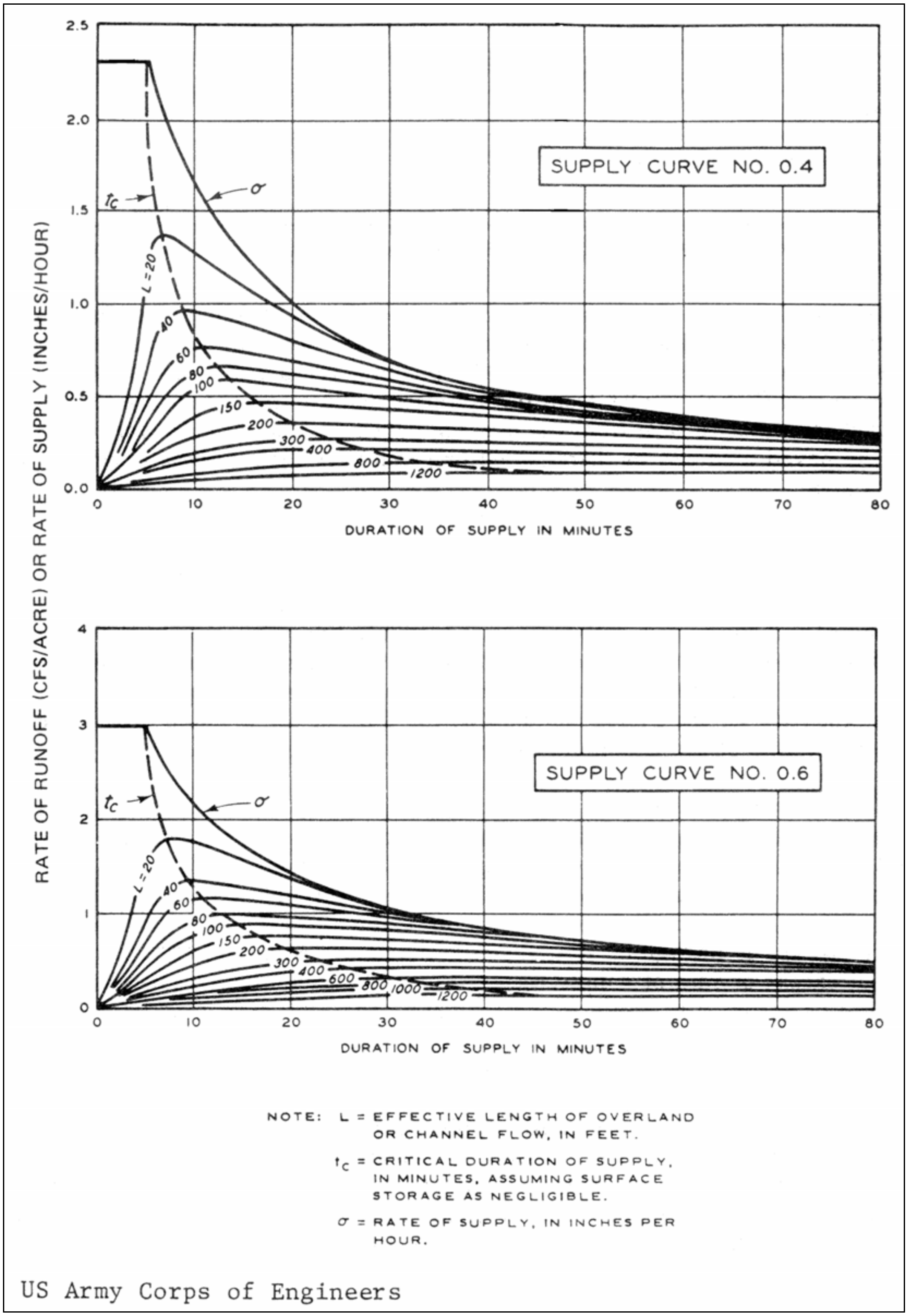
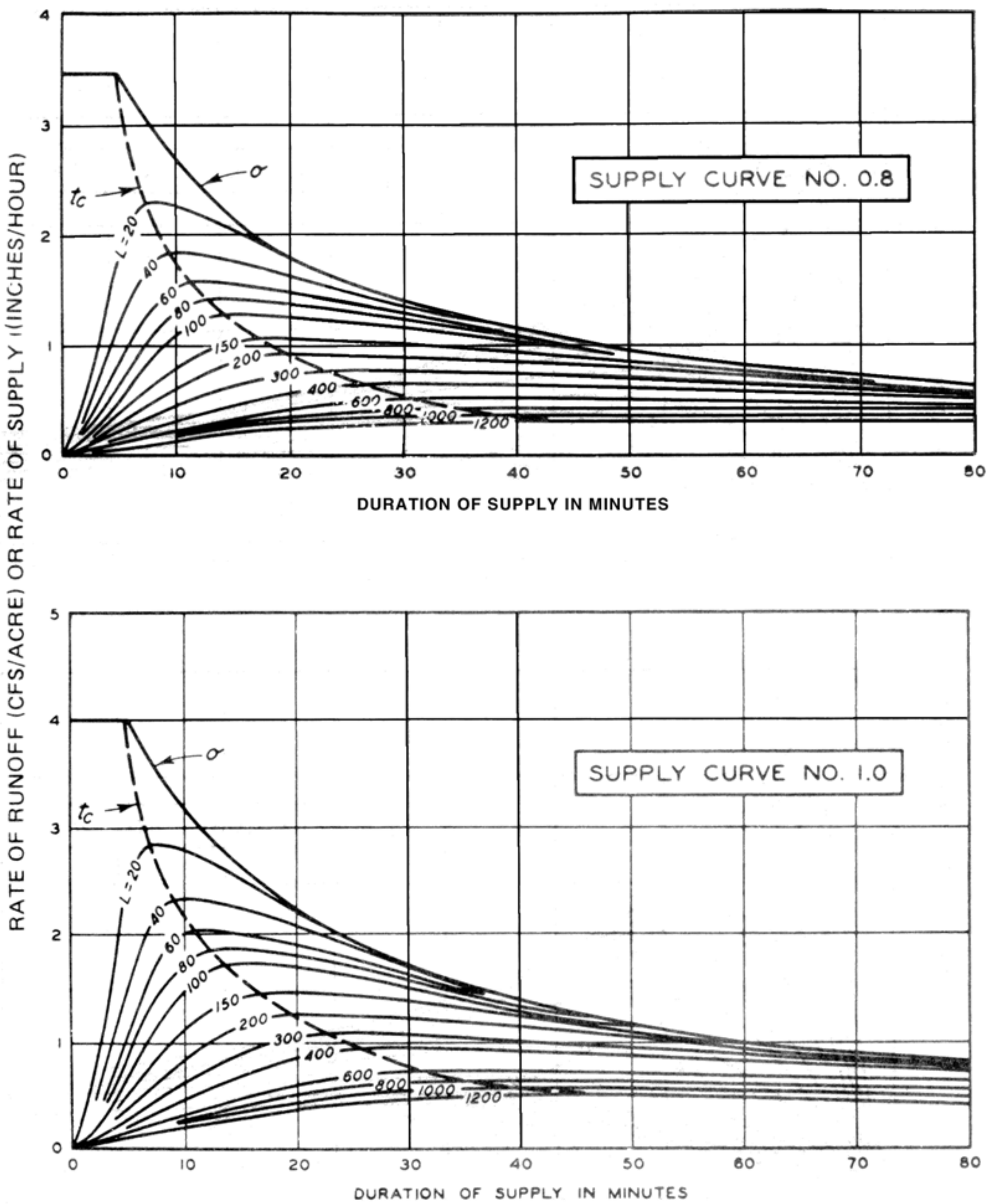


**Figure 3-3. Rates of Runoff Corresponding to Supply Curves No. 0.8 and 1.0; n = 0.40 and S = 1 percent**




NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

σ = RATE OF SUPPLY, IN INCHES PER HOUR.

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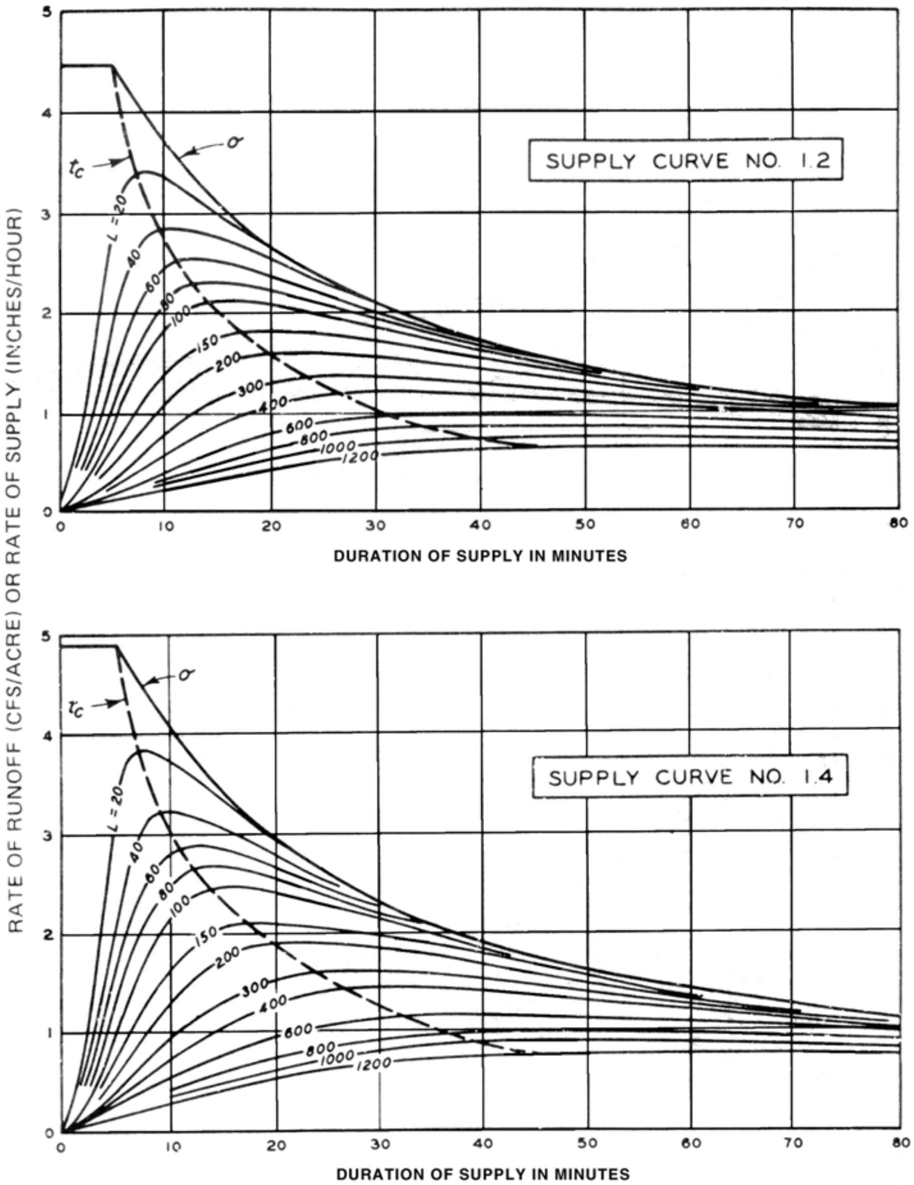
**Figure 3-4. Rates of Runoff Corresponding to Supply Curves No. 1.2 and 1.4; n = 0.40 and S = 1 percent**



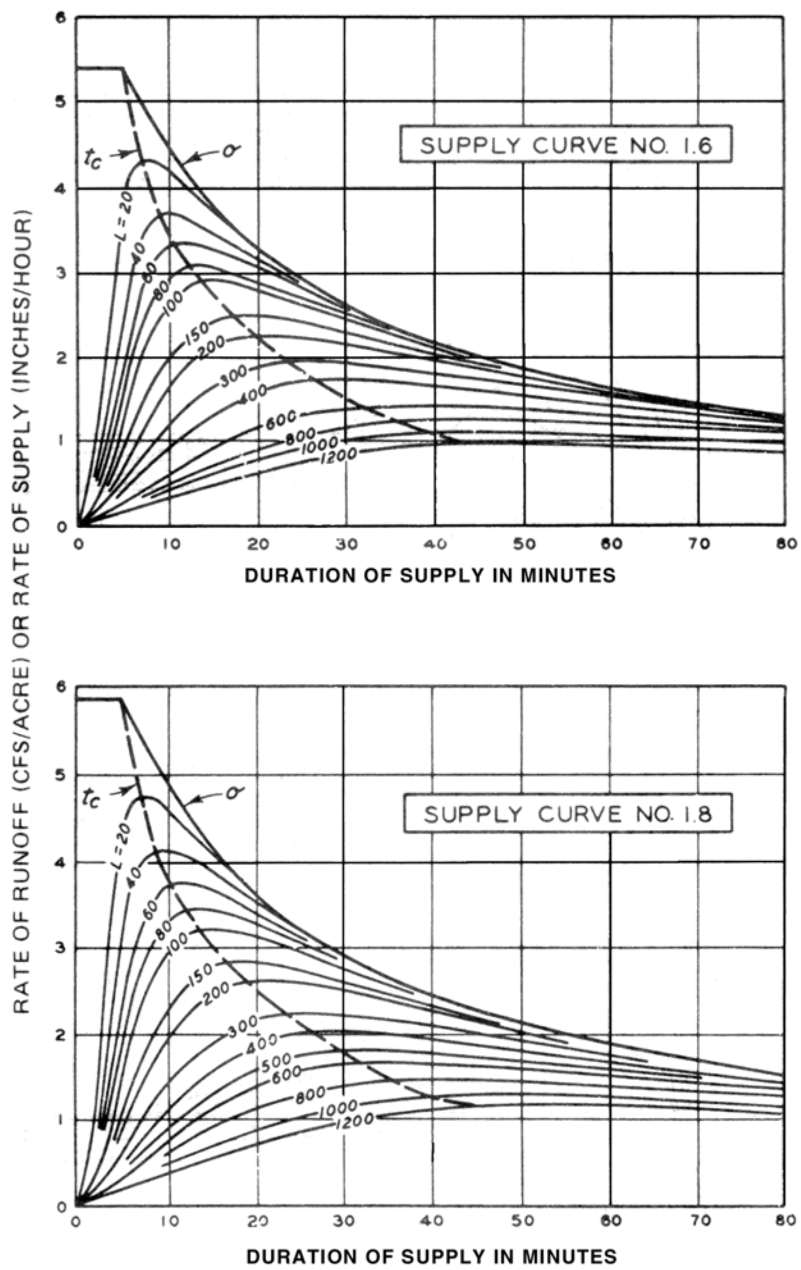

NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

$\sigma$ = RATE OF SUPPLY, IN INCHES PER HOUR.

US Army Corps of Engineers

**Figure 3-5. Rates of Runoff Corresponding to Supply Curves No. 1.6 and 1.8; n = 0.40 and S = 1 percent**




NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

$\sigma$ = RATE OF SUPPLY, IN INCHES PER HOUR.

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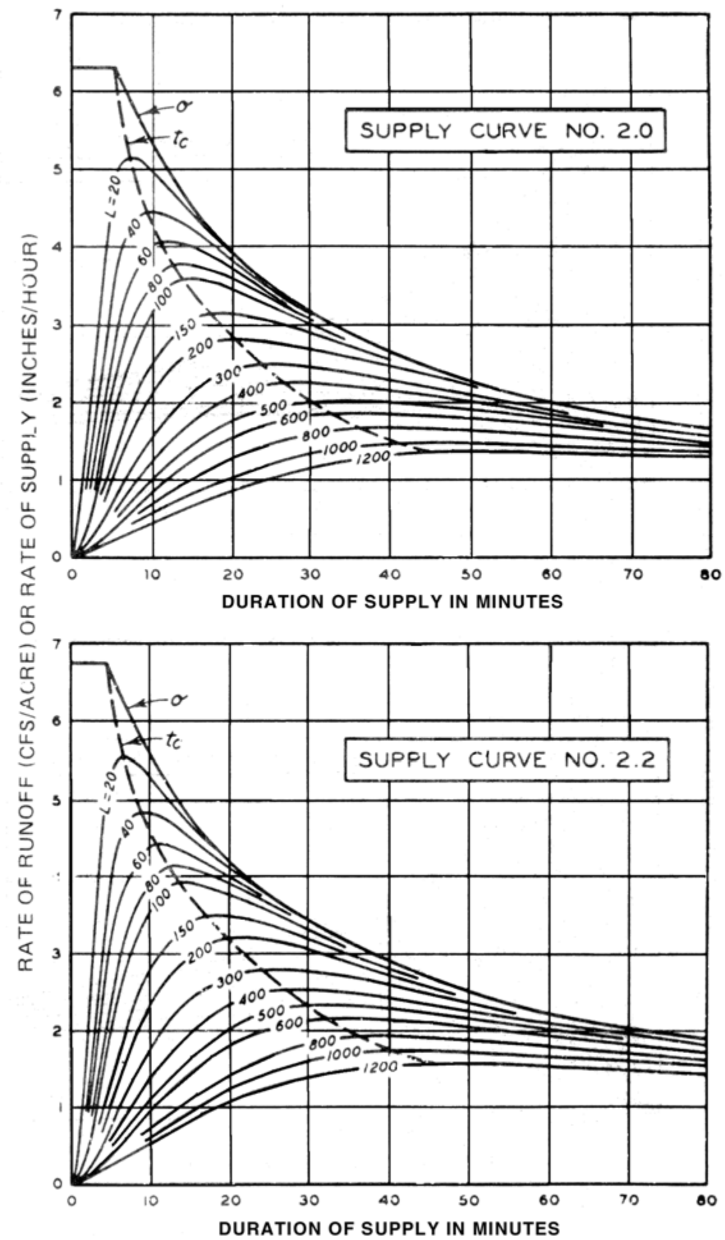
**Figure 3-6. Rates of Runoff Corresponding to Supply Curves No. 2.0 and 2.2; n = 0.40 and S = 1 percent**




NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET.

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

$\sigma$ = RATE OF SUPPLY, IN INCHES PER HOUR.

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**Figure 3-7. Rates of Runoff Corresponding to Supply Curves No. 2.4 and 2.6; n = 0.40 and S = 1 percent**

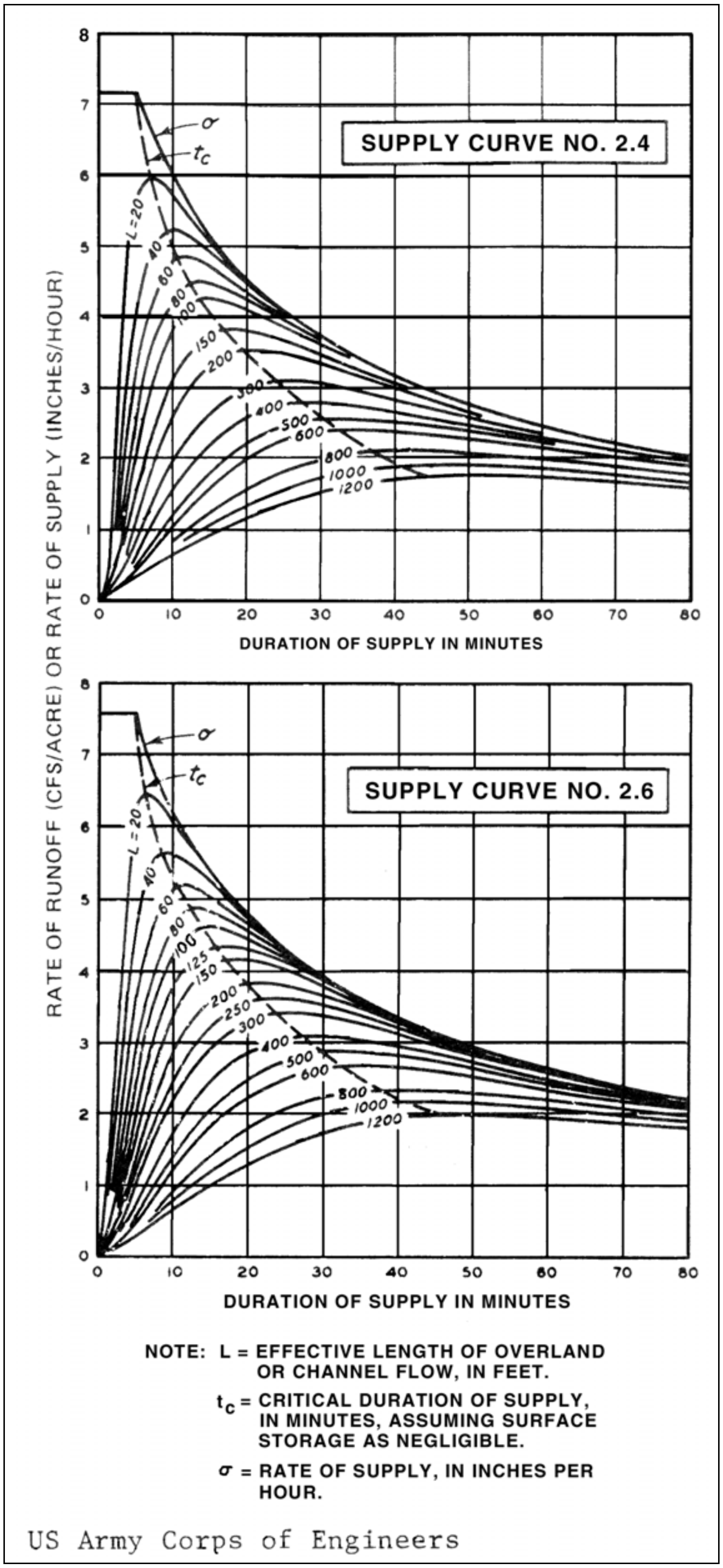
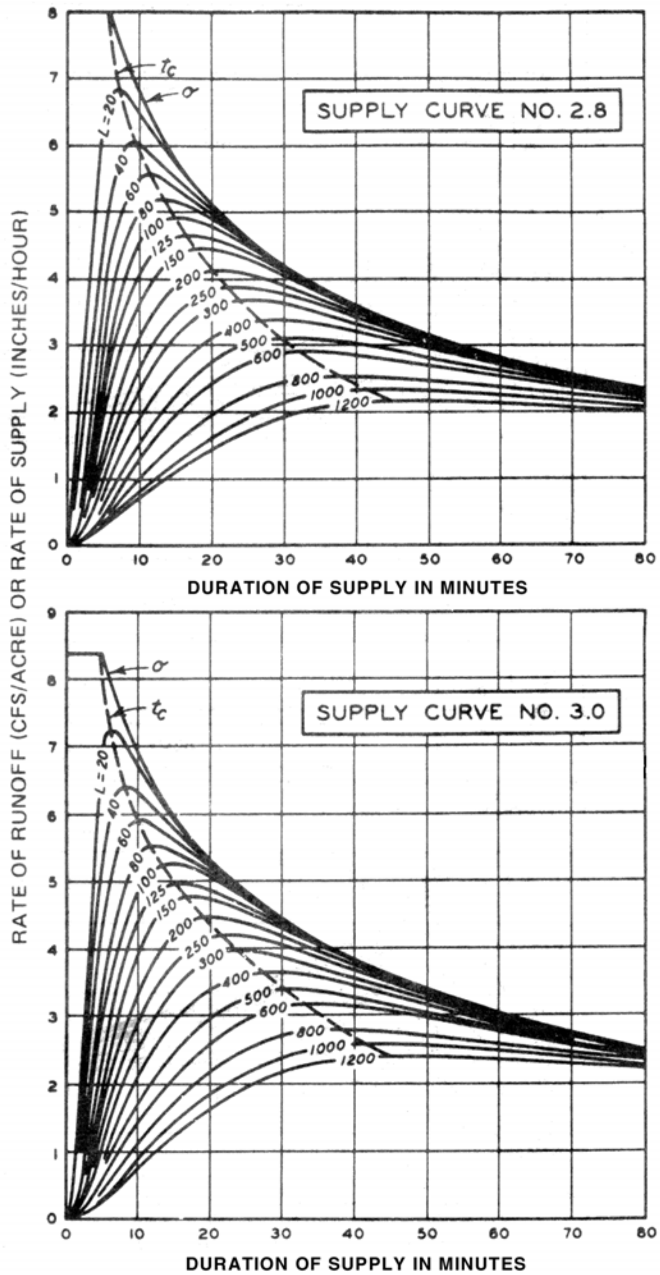


**Figure 3-8. Rates of Runoff Corresponding to Supply Curves No. 2.8 and 3.0; n = 0.40 and S = 1 percent**




NOTE: L = EFFECTIVE LENGTH OF OVERLAND OR CHANNEL FLOW, IN FEET

$t_c$ = CRITICAL DURATION OF SUPPLY, IN MINUTES, ASSUMING SURFACE STORAGE AS NEGLIGIBLE.

$\sigma$ = RATE OF SUPPLY, IN INCHES PER HOUR.

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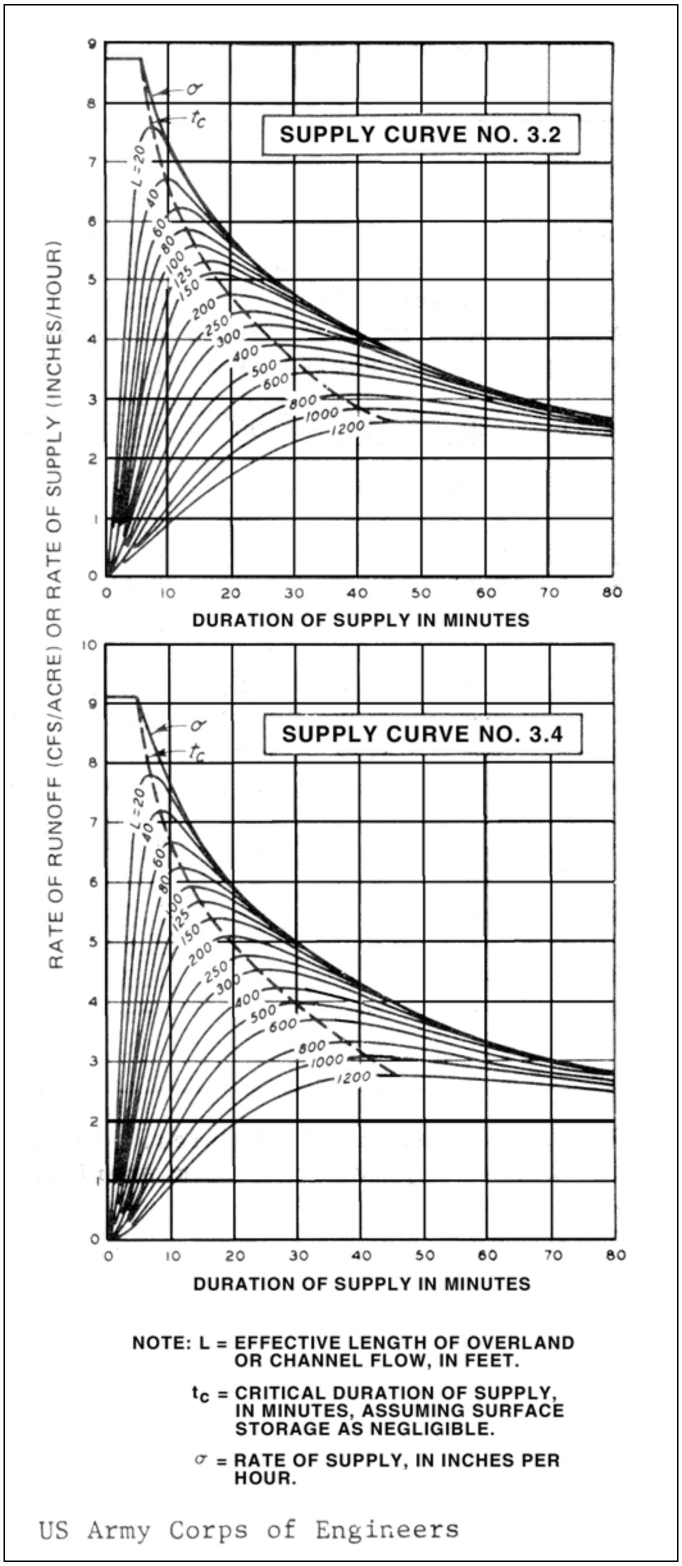
**Figure 3-9. Rates of Runoff Corresponding to Supply Curves No. 3.2 and 3.4; n = 0.40 and S = 1 percent**



3-10 **DESIGN PROCEDURES FOR THE DRAINAGE SYSTEM.** Design-storm runoff must be efficiently removed from airfields and heliports to avoid interruption of operations during or following storms and to prevent temporary or permanent damage to pavement subgrades. Removal is accomplished by a drainage system unique to each airfield and heliport site. Drainage systems will vary in design and extent depending upon local soil conditions and topography; size of the physical facility; vegetation cover or its absence; the anticipated presence or absence of ponding; and most importantly, upon local storm intensity and frequency patterns. The drainage system should function with a minimum of maintenance difficulties and expense and should be adaptable to future expansion. Open channels or natural water courses are permitted only at the periphery of the airfield or heliport facility and must be well removed from the landing strips and traffic areas. Provisions for subsurface drainage, the requirements for which are provided in Chapter 6, may necessitate careful consideration. Subdrains are used to drain the base material, lower the water table, or drain perched water tables. Fluctuations of the water table must be considered in the initial design of the airfield or heliport facility.

3-10.1 **Information Required.** Before proceeding with the design calculations, as illustrated in Section 3-11 and Example C-3, certain additional information and data must be developed. These include:

3-10.1.1 A topographic map.

3-10.1.2 A layout of the helipad, runways, taxiways, aprons, and other hardstands with tentative finished grading contours at 1-ft intervals.

3-10.1.3 Profiles of runways, taxiways, apron areas, and other hardstands.

3-10.1.4 Soil profiles based on soil tests to include, whenever possible, infiltration properties of local soils to be encountered.

3-10.1.5 Groundwater elevation and fluctuation if known or obtainable.

3-10.1.6 A summary of climatic conditions including temperature ranges, freezing and thawing patterns and depth of frost penetration.

3-10.1.7 Snowfall records, snow cover depths, and convertibility factors to inches of rainfall.

3-10.1.8 Runoff records for drainage areas in the same locality having similar characteristics and soil conditions.

3-10.2 **Grading.** Proper grading is the most important single factor contributing to the success of the drainage system. Development of grading and drainage plans must be fully coordinated. Grading criteria in AFR 86-14 for Air Force facilities and TM 5-803–4 for Army airfields and heliports provide adequate grading standards to insure effective drainage.

3-10.2.1 **Minimum slopes.** For satisfactory drainage of airfield pavements, a minimum gradient of 1.5 percent in the direction of drainage is recommended except for rigid pavements where 1.0 percent is adequate. In some cases, gradients less than 1.5 percent are adequate because of existing grades; arid or semiarid climatic conditions; presence of noncohesive, free-draining subgrades; and locations of existing drainage structures. Such factors may allow a lesser transverse slope; thus, construction economies are effected and preferred operational grades are obtained.

3-10.2.2 **Shoulder slopes.** In Attachment 5 of AFR 86-14, transverse grades of shoulders are specified for runways, taxiways, and aprons. In areas of moderate or heavy rainfall or excessive turf encroachment, use of a steeper transition shoulder section immediately adjacent to the airfield pavement is permitted. In designing shoulders, the first 10-ft strip of shoulder adjacent to the pavement edges of runways, taxiways, or aprons should have a 5 percent slope. The elevation of the pavement edge and the shoulder will coincide. The shoulder gradient beyond the 10-ft strip will conform to the minimum 2 percent and maximum 3 percent specified in AFR 86-14. Waivers will not be required for the 5 percent slope discussed above. Paved shoulders will normally have the same transverse slope as that of the contiguous runways and taxiways.

3-10.2.3 **Determination of drainage area.** Use the completed grading plan as a guide and sketch the boundaries of specific drainage areas tributary to their respective drain inlets. Compute the area of paved and unpaved areas tributary to the respective inlets by planimetering.

3-10.2.4 **Drainage patterns.** Drainage patterns consisting of closely spaced interior inlets in pavements with intervening ridges are to be avoided. Such grading may cause taxiing problems including bumping or scraping of wing tanks. Crowned sections are the standard cross sections for runways, taxiways, and safety areas. Crowned sections generally slope each way from the center line of the runway on a transverse grade to the pavement. Although crowned grading patterns result in most economical drainage, adjacent pavements, topographic considerations, or other matters may necessitate other pavement grading.

3-10.3 **Classification of Storm Drains.** Storm drains for airfields and heliports may be classified in two groups, primary and auxiliary.

3-10.3.1 **Primary drains.** Primary drains consist of main drains and laterals that have sufficient capacity to accommodate the project design storm, either with or without supplementary storage in ponding basins above the drain inlets. To lessen construction requirements for drainage facilities, maximum use of ponding consistent with operational and grading requirements will be considered. The location and elevation of the drain inlets are determined in the development of the grading plans.

3-10.3.2 **Auxiliary drains.** Auxiliary drains normally consist of any type or size drains provided to facilitate the removal of storm runoff, but lacking sufficient capacity to remove the project design storm without excessive flooding or overflow. Auxiliary storm drains may be used in certain airfields to provide positive drainage of long flat swales located adjacent to runways or in unpaved adjacent areas. During less frequent storms of high intensity, excess runoff should flow overland to the primary drain system or other suitable outlet with a minimum of erosion. An auxiliary drain may also be installed to convey runoff from pavement gutters wherever a gutter capacity of less than design discharge is provided.

3-10.4 **Storm-Drain Layout.** The principal procedures in the determination of the storm-drain layout follow:

3-10.4.1 **Preliminary layout.** Prepare a preliminary map (scale 1 in. = 200 ft or larger) showing the outlines of runways, taxiways, and parking aprons. Contours should represent approximately the finished grade for the airfield or heliport. Details of grading, including ponding basins around primary drain inlets, need not be shown more accurately than with 1-ft contour intervals.

3-10.4.2 **Profiles.** Plot profiles of all runways, taxiways, and aprons so that elevations controlling the grading of intermediate areas may be determined readily at any point.

3-10.4.3 **Drain outlets.** Consider the limiting grade elevations and feasible channels for the collection and disposition of the storm runoff. Select the most suitable locations for outlets of drains serving various portions of the field. Then select a tentative layout for primary storm drains. The most economical and most efficient design is generally obtained by maintaining the steepest hydraulic gradient attainable in the main drain and maintaining approximately equal lateral length on each side of the main drain.

3-10.4.4 **Cross-sectional profiles of intermediate areas.** Assume the location of cross-sectional profiles of intermediate areas. Plot data showing controlling elevations and indicate the tentatively selected locations for inlets by means of vertical lines. Projections of the runways, taxiways, or aprons for limited distances should be shown on the profiles, to facilitate a comparison of the elevations of intermediate areas with those of the paved areas. Generally, one cross-sectional profile should follow each line of the underground storm-drain system. Other profiles should pass through each of the inlets at approximately right angles to paved runways, taxiways, or aprons.

3-10.4.5 **Correlation of the controlling elevations and limiting grades.** Begin at points corresponding to the controlling elevations, such as the edges of runways, and sketch the ground profile from the given points to the respective drain inlets. Make the grades conform to the limiting slopes. Review the tentative grading and inlet elevations and make such adjustments in the locations of drain inlets and in grading details as necessary to obtain the most satisfactory general plan.

3-10.4.6 **Trial drainage layouts.** Several trial drainage layouts will be necessary before the most economical system can be selected. The first consideration will be the tentative layout serving all of the depressed areas in which overland flow will accumulate. The inlet structures will be located, during the initial step, at the lowest points within the field areas. The pipelines will be shown next. Each of the inlet structures will be connected to the field pipelines, which in turn will be connected to the major outfalls.

3-10.4.7 **Rechecking of finished contours.** Before proceeding further, recheck the finished contours to determine whether the surface flow is away from the paved areas, that the flow is not directed across them, that no field structures fall within the paved areas (except in aprons), that possible ponding areas are not adjacent to pavement edges, and that surface water will not have to travel excessively long distances to flow into the inlets. If there is a long, gradually sloping swale between a runway and its parallel taxiway (in which the longitudinal grade, for instance, is all in one direction), additional inlets should be placed at regular intervals down this swale. Should this be required, ridges may be provided to protect the area around the inlet, prevent bypassing, and facilitate the entry of the water into the structure. If the ridge area is within the runway safety area, the grades and grade changes will need to conform to the limitations established for runway safety areas in other pertinent publications.

3.10.4.8 **Maximum ponding area and volume.** Estimate the maximum elevation of storage permissible in the various ponding areas and indicate the elevations on the profiles referred to in (4) and (5) above. Scale the distances from the respective drain inlets to the point where the elevation of maximum permissible ponding intersects the ground line, transfer the scaled distances to the map prepared in (1) above, and sketch a line through the plotted points to represent the boundary of the maximum ponding area during the design storm. Determine the area within the various ponding areas and compute the volume of permissible storage at the respective drain inlets. All ponding area edges will be kept at least 75 ft from the edges of the pavement to prevent saturation of the base or subbase and of the ground adjacent to the pavement during periods of ponding.

3-10.4.9 **Ditches.** A system of extensive peripheral ditches may become an integral part of the drainage system. Ditch size and function are variable. Some ditches carry the outfall away from the pipe system and drainage areas into the natural drainage channels or into existing water courses. Others receive outfall flow from the airport site or adjacent terrain. Open ditches are subject to erosion if their gradients are steep and if the volume of flow is large. When necessary, the ditches may be turfed, sodded, stabilized, or lined to control erosion.

3-10.4.10 **Study of the contiguous areas.** After the storm drain system has been tentatively laid out and before the actual computations have been started, the areas contiguous to the graded portion of the airport which may contribute surface flow upon it should again be studied. A system of open channels, intercepting ditches, or storm drains should be designed where necessary to intercept this storm flow and conduct it away from the airport to convenient outfalls. A study of the soil profiles will assist in locating porous strata which may be conducting subsurface water into the airport. If this condition exists, the subsurface water should be intercepted and diverted.

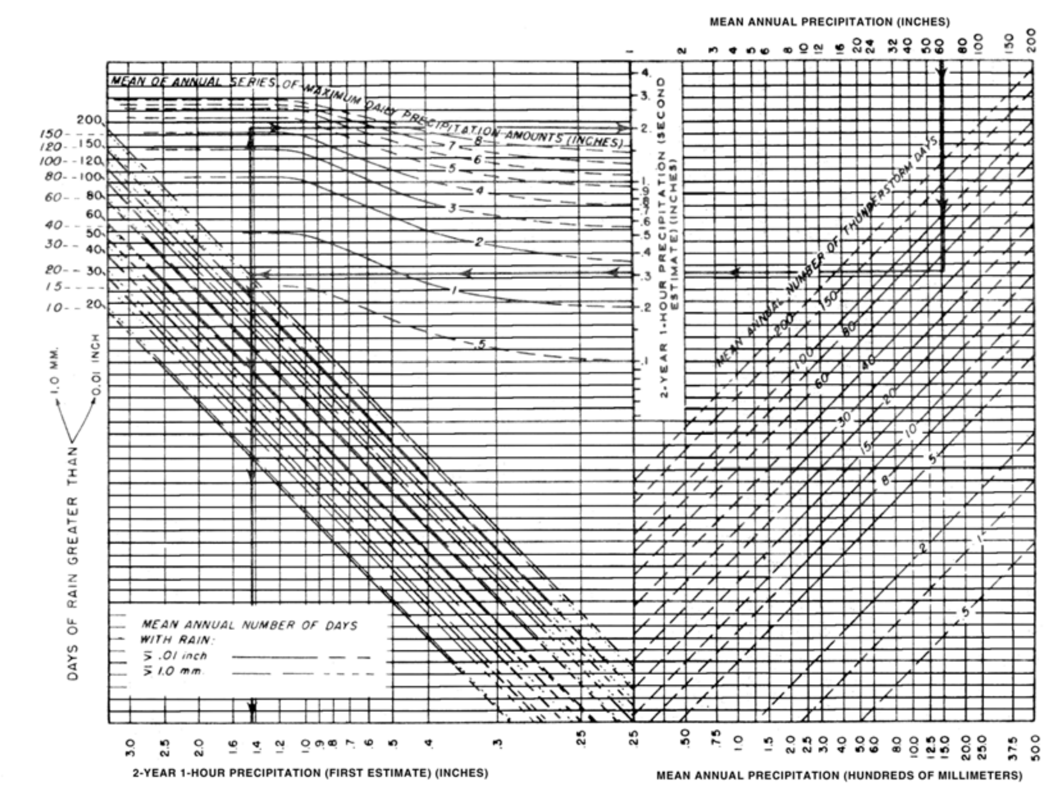
3-10.5 **Typical Design Procedures.** The procedures in Sections 3-2 through 3-10 are illustrated and annotated in the design computations contained in Example C-3. Comparative designs with and without provisions for temporary ponding have been prepared for the airfield shown.

## 3-11 DESIGN PROCEDURE

### 3-11.1 Rainfall

3-11.1.1 **Intensity-frequency data.** In areas where intensity-frequency data are incomplete or unavailable, the 2-yr 1-hr rainfall can be estimated from the following parameters: mean annual precipitation—the average of total yearly rainfall for a specified number of years; mean annual number of days of precipitation–the average number of days for a specified number of years in which greater than 0.01 in. of rain occurred; mean annual thunderstorm days—the average number of days for a specified number of years in which thunder was heard; and the mean of the annual maximum observational-day rainfall amounts—the average of the maximum rainfall on any calendar day within the year for a specified number of years. Correlation of the 2-yr 1-hr rainfall with these four climatic parameters appears in Figure 3-10.

**Figure 3-10. Diagram for Estimating 2-yr 1-hr Rainfall**




NOTE: DASHED LINES ARE UNSUPPORTED BY DATA.

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Engineers and US Weather Bureau

3-11.1.1.1 When daily rainfall data are not available, the 2-yr 1-hr value can be estimated using the other three parameters, namely, mean annual precipitation, mean annual number of precipitation days, and mean annual number of thunderstorm days. Three parameters are not as accurate as four, and the diagram should be supplemented wherever possible by correlation with other data.

3-11.1.1.2 As an example of the use of Figure 3-10, assume the mean annual precipitation is 60 in., the mean annual number of thunderstorm days is 50, and the mean annual number of precipitation days is 200. Enter the diagram at the upper right with the mean annual precipitation; proceed vertically down to the mean annual number of thunderstorm days; move horizontally to the left to the number of days of precipitation, and then vertically downward to the 2-yr 1-hr precipitation value (first estimate). In this example, the first estimate for the 2-yr 1-hr precipitation is approximately 1.4 in. Now assume the fourth parameter, the mean of annual series of maximum daily precipitation, is 4.3 in. The same procedure is followed to the mean annual days of precipitation; from there, move vertically upward to the mean of annual series of maximum daily precipitation value and then horizontally to the right to the 2-yr 1-hr precipitation value (second estimate). In this example, the second estimate would be 2.0 in. The second estimate is preferable, if four parameters are available.

3-11.1.1.3 For frequencies other than 2 years, the factors in Table 3-1 can be used to approximate intensity-frequency values, using the 2-yr 1-hr value as a base.

3-11.1.2 **Standard rate of supply curves.** Standard supply curves for areas with zero infiltration loss will be the same as the standard rainfall intensity curves in Figure 3-1. Where infiltration losses occur, the standard supply curve number corresponding to a given standard rainfall curve number is computed by subtracting the estimated 1-hr infiltration value from the 1-hr rainfall quantity.

**Table 3-1. Approximate Intensity-Frequency Values**

| Factor | Intensity-Frequency Values |
|---|---|
| 0.80 | 1-year 1-hour |
| 1.00 | 2-year 1-hour |
| 1.35 | 5-year 1-hour |
| 1.60 | 10-year 1-hour |
| 1.90 | 25-year 1-hour |
| 2.10 | 50-year 1-hour |
| 2.30 | 100-year 1-hour |

3-11.1.3 **Weighted standard rates of supply.** For composite areas, the rate of supply should be the average weighted supply. Mathematically, the weighted supply curve, $SC_W$, can be expressed by the equation:

$$SC_W = \frac{\left[\left(SC_1 \times A_1\right) + \left(SC_2 \times A_2\right) + \ldots \left(SC_n \times A_n\right)\right]}{A_1 + A_2 + \ldots + A_n} \qquad \text{(eq. 3-1)}$$

where the SC's are standard supply rates for the various areas, A. For example, if the drainage area under consideration has a 1-hr rainfall intensity of 2.5 in.; estimated infiltration values of 0.0 for paved area $A_1$, 0.6 for turfed area $A_2$, and 0.2 for bare clay area $A_3$; and drainage area $A_l$ is 1.5 acres, $A_2$ is 5.0 acres, and $A_3$ is 6.5 acres; then the weighted standard supply curve for the composite drainage area would be:

$$SC_W = \frac{(2.5-0.0)(1.5)+(2.5-0.6)(5.0)+(2.5-0.2)(6.5)}{1.5+5.0+6.5}$$

$$SC_W = 2.2$$

3-11.1.4 **Overland flow.** The rate of overland flow to be expected from a continuous and uniform rate of rainfall excess, or rate of supply, can be determined from Equation 3-2 as interpreted by G. A. Hathaway (American Society of Civil Engineers, Transactions, Vol. 110):

$$q = \sigma \tanh^2 \left[0.922t\ (\sigma / nL)^{0.50}\ S^{0.25}\right] \qquad \text{(eq. 3-2)}$$

where

| | | |
|---|---|---|
| q | = | rate of overland flow at the lower end of an elemental strip, in./hr or cfs/acre |
| $\Phi$ | = | rate of supply or intensity of rainfall excess, in./hr |
| t | = | time, or duration, from beginning of supply, min |
| n | = | coefficient of roughness of the surface |
| L | = | effective length of overland, or channel flow, ft |
| S | = | slope of the surface (absolute, that is, 1 percent = 0.01) |
| tanh | = | hyperbolic tangent |

3-11.1.4.1 The curves shown in Figures 3-11 through 3-13 were computed using Equation 3-2, assuming n = 0.40 and S = 0.01. The overland flow curves are the hydrography that would result from continuous and uniform rates of rainfall-excess or rates of supply. From the curves, hydrography can be developed for any selected duration and rate of rainfall-excess by the procedure shown in Figure 3-14. Hydrography 1 and 1-A in Figure 3-14 represent rates of runoff under given conditions assuming supply continues indefinitely. However, by lagging the hydrography for a selected period of rainfall-excess, $t_r$ (20 min in this example), and subtracting runoff in hydrography 1-A from hydrography 1, a hydrography can be obtained that represents the runoff pattern for the selected period of rainfall-excess (hydrography 2 in the example).

**Figure 3-11. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 20, 40, 60, 80, 100, and 125 ft**

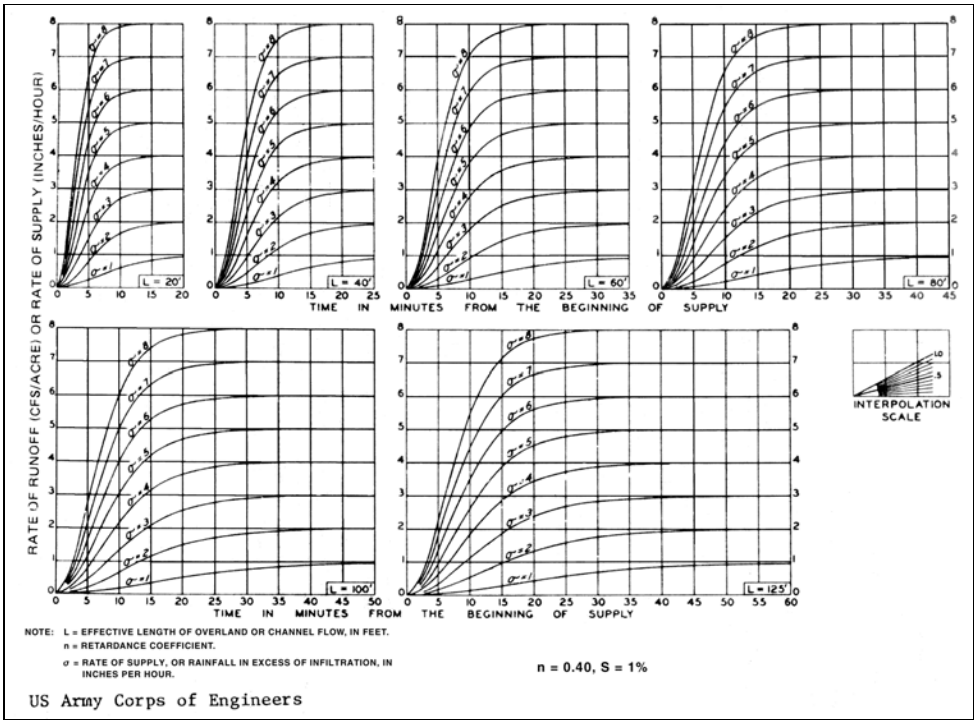


**Figure 3-12. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 150, 200, 250, 300, 400, 500, and 600 ft**

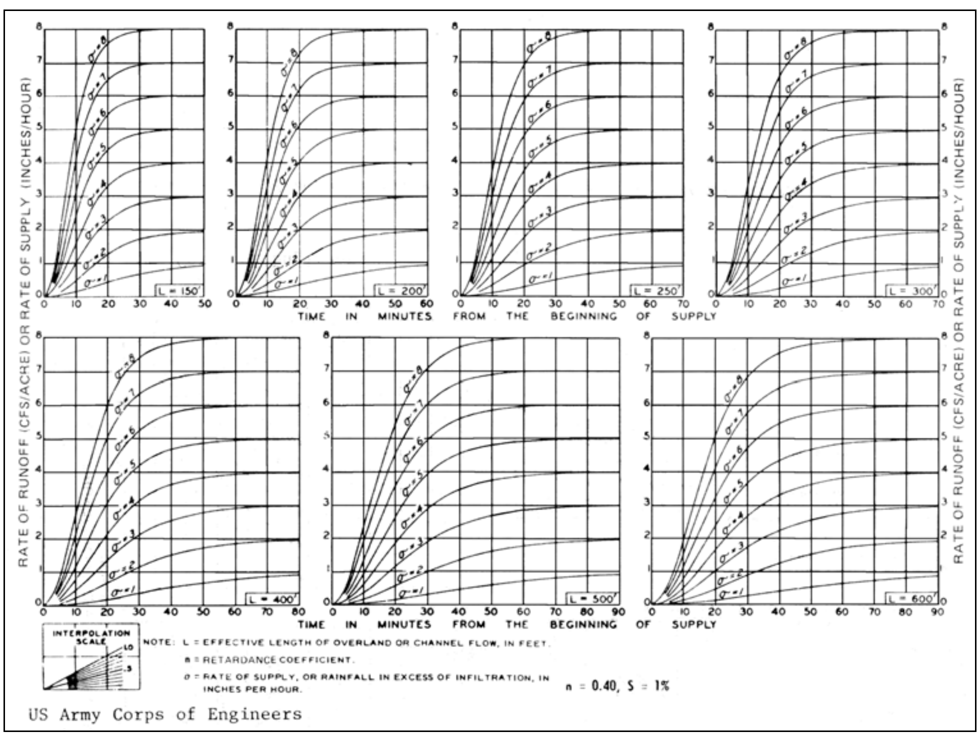

**Figure 3-13. Rates of Runoff Versus Duration of Supply for Turfed Areas; L = 800, 1,000, and 1,200 ft**

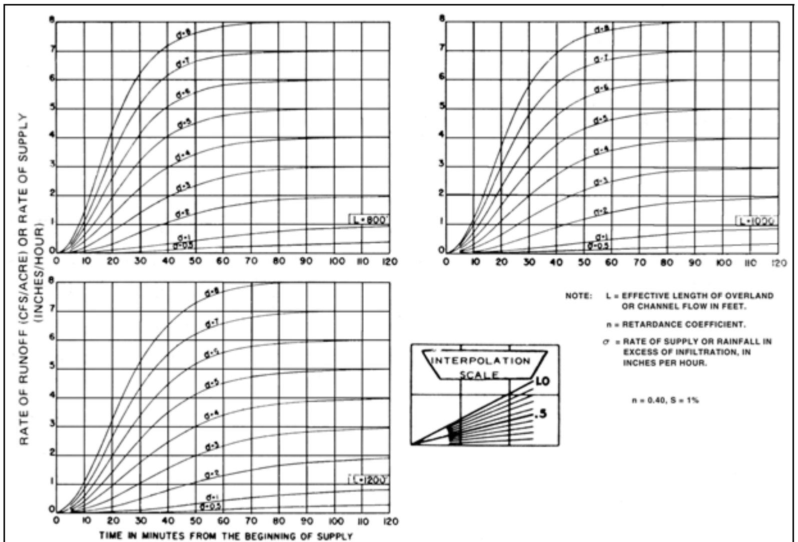


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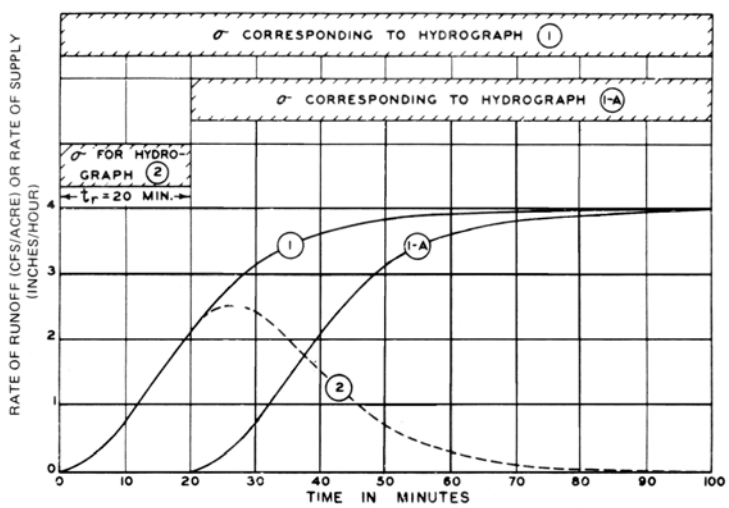
**Figure 3-14. Computation of Hydrograph to Represent Runoff from a Supply of Specified Duration**




EXPLANATION

EXAMPLE: L = 400 FT.; S = 1%; n = 0.40; σ = 4 IN. PER HR.; $t_r$ = 20 MIN.

HYDROGRAPH 1 REPRESENTS RATE OF RUNOFF UNDER GIVEN CONDITIONS ASSUMING SUPPLY BEGINS AT TIME ZERO AND CONTINUES INDEFINITELY (SEE FIG. 3-12).

HYDROGRAPH 1-A IS IDENTICAL WITH HYDROGRAPH 1 EXCEPT THAT SUPPLY AND RUNOFF ARE ASSUMED TO BEGIN 20 MIN. LATER THAN HYDROGRAPH 1.

HYDROGRAPH 2, OBTAINED BY SUBTRACTING ORDINATES OF HYDROGRAPH 1-A FROM HYDROGRAPH 1, REPRESENTS APPROXIMATELY THE RUNOFF TO BE EXPECTED FROM A SUPPLY RATE OF 4 IN. PER HR. AND A DURATION OF 20 MIN.

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3-11.1.4.2 Overland flow curves may be used for surfaces having other coefficients of roughness or slopes by using, instead of actual length of the flow involved, a hypothetical length that is greater or less than the actual by a sufficient amount to compensate for the difference between the correct values of n and S and those used in preparing Figures 3-11 through 3-13. The necessary conversions to get an effective length may be accomplished by substituting the quantity nL $\sqrt{4}\,\sqrt{S}$ for L or by using Figure 3-4 as explained below (Effective Length).

## 3-11.2 Effective Length

3-11.2.1 **General.** In Equation 3-2, the effective length, L, represents the length of overland flow, measured in a direction parallel to the maximum slope, from the edge of the drainage area to a point where runoff has reached a defined channel or pOnding basin. In large drainage areas, considerable channelized flow will occur under design-storm conditions. Investigation of many runoff records for watersheds has indicated that by modifying the actual length, satisfactory reproduction of runoff hydrography may be obtained regardless of channelization of flow. The values for L are determined by summing the length of channel flow and the length of overland flow after each has been reduced to an effective length for n = 0.40 and S = 1.0 percent by means of Figure 2-5. The length of channel flow is measured along the proposed collecting channel for that section in which appreciable depth of flow may reasonably be expected to occur during the design storm. Length of overland flow is the average distance from the end of the effective channel or from the drain inlet to the edge of the drainage area, measured in the direction of flow as indicated on the proposed grading plans. Airfield and heliport grading is such that overland flow will normally channelize in distances of 600 feet or less, although this distance may be exceeded. Whenever the distance is exceeded, the actual length may be divided by a number so that the quotient conveniently falls on the horizontal axis of graph A on Figure 2-5. The length derived from graph B on the figure would then be multiplied by this same number to determine the final effective length. Typical values of the coefficient of roughness, n, for use in determining effective length of overland flow are given in Table 2-1. Chapter 4 gives additional n values for turfed channels. For example, to find the effective length of overland flow for an actual length of 900 ft on a sparse grass ground cover where n = 0.20, and the overall slope is 0.7 percent, use the following procedure. Divide the 900-ft actual length by 2 and enter graph A of Figure 2-5 with 450 ft on the horizontal axis. Project a line vertically upward until it intersects the coefficient of roughness line; proceed horizontally to the intersection of the slope line equal to 0.7 percent on graph B, and proceed vertically down to obtain a length of 275 ft, which must be multiplied by 2, resulting in a total effective length of overland flow of 550 ft.

3-11.2.2 **Effect of paved area on determination of effective length.** Ponding areas are frequently located in intermediate turfed areas bordered by paved runways, taxiways, or aprons. Runoff from paved areas ultimately passes over turfed slopes to reach the ponding areas and drain inlets, and is retarded in a manner similar to runoff that results from precipitation falling directly on the turfed area. Inasmuch as the time required for water to flow from the average paved area is normally very short (5 to

10 min), the length of the paved area can be disregarded or given very little weight in estimating the value of L for a composite area.

3-11.2.3 **Determination of effective length for ponding conditions.** The true value of L applicable to a particular area varies as the size of the storage pond fluctuates during storm runoff. As water accumulates in the relatively flat storage area during storm runoff, the size of the pond increases rapidly and progressively reduces the distance from the edge of the pond to the outer limits of the drainage area. In the majority of cases, it is satisfactory to estimate the value of L as the distance from the outer limits of the drainage area to the average limits of the ponding area during the period of design-storm runoff. If the drain inlet is not located near the centroid of the drainage area, the value of L can be estimated approximately as the average distance to the limit of the ponding area, which corresponds to a depth equal to two-thirds of the maximum depth caused by the design storm.

3-11.3 **Runoff**

3-11.3.1 **General.** The curves shown in Figures 2-5 and 3-2 through 3-9 describe the relationship between rate of supply, $\Phi$; critical duration of supply, $t_c$; effective length of overload flow, L; and maximum rate of runoff for the various supply curves presented in Figure 3-1. The curves portray the data presented in the flow curves shown in Figure 3-11 through 3-13 in another format. Table 3-2 illustrates the computational procedure. The runoff values obtained are assumed to be the maximum because surface storage is negligible. Actually, the maximum runoff would normally occur a short time after the rainfall excess or rate of supply ceases. For practical purposes, however, the maximum rate of overland flow can be assumed to occur at approximately the same time that the rate of supply ends.

3-11.3.2 **Peak runoff rates.** Figures 2-5 and 3-2 through 3-9 are not hydrography for any specified design storm, but represent the peak rates of runoff from individual storm events of various durations, all of which have the same average frequency of occurrence. The duration of supply corresponding to the greatest discharge for a particular standard supply curve and value of L in these figures is defined as the critical duration of supply, $t_c$, for runoff from an area not affected by surface ponding. However, experience indicates that adopting minimum values for $t_c$ of 10 min for paved areas and 20 min for turfed areas in the actual design of storm drains is feasible and practical. For combined turfed and paved areas, minimum values of $t_c$ are to be used even though the calculated effective length of overland flow indicates a shorter critical duration of supply. For combined turfed and paved areas, where only the minimum values of $t_c$ are of concern, the following equation should be used in selecting $t_c$:

$$t_c = (10A_p + 20A_t)/(A_p + A_t) \qquad \text{(eq 3-3)}$$

where

$A_p$ = area paved, acres

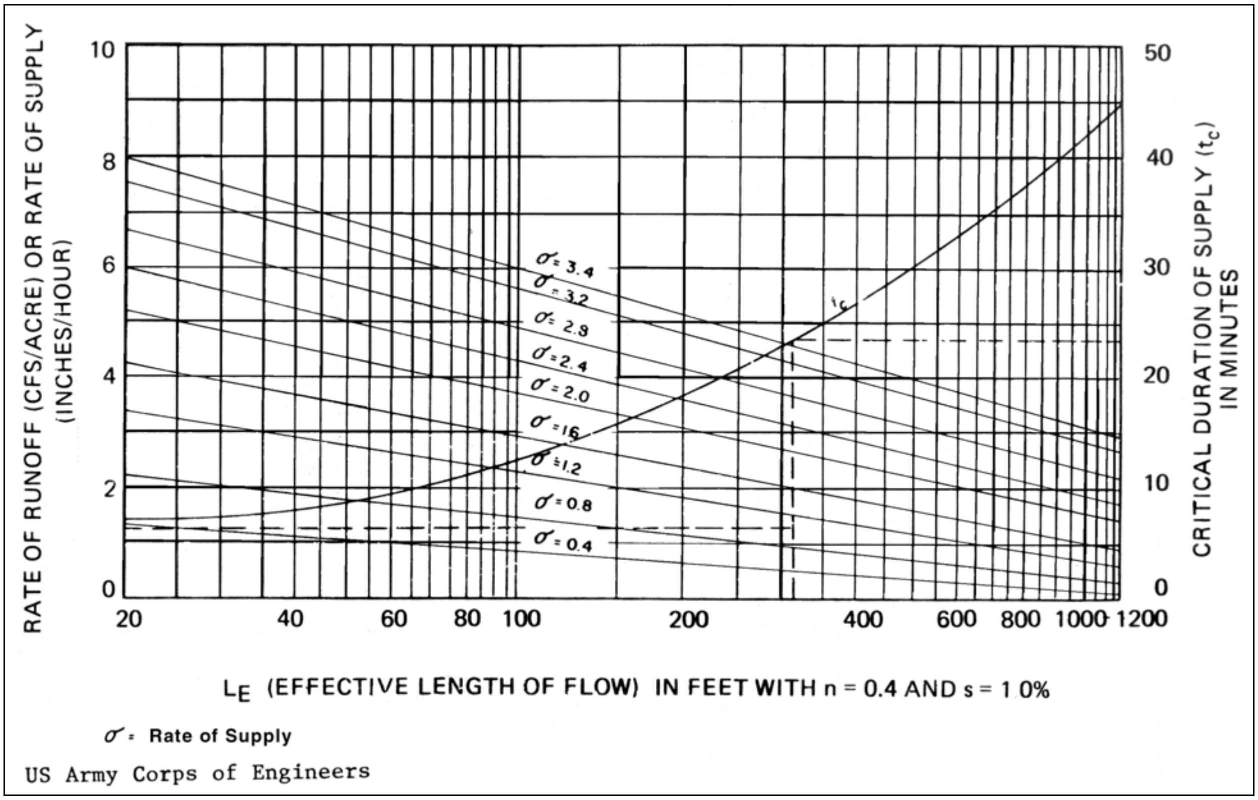
$A_t$ = area turfed, acres

**Table 3-2. Rates of Runoff Corresponding to Intensities and Durations of Supply Represented by Standard Supply Curve No. 2 in Figure 3-11 (n = 0.40, S = 1 percent)**

| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
|---|---|---|---|---|---|---|---|---|
| | | Rate of runoff (in in./hr or cfs/acre) corresponding to durations shown in column 1 and rates of supply given in column 2 as scaled from Figures B-2 and B-3 L, feet | | | | | | |
| Duration of supply minutes | Rate of supply in./hr (scaled from curve No. 2.0, Figure 3-1) | 20 | 60 | 100 | 200 | 300 | 400 | 600 |
| 3 | 6.30 | 2.68 | 1.12 | 0.75 | 0.39 | 0.25 | 0.22 | 0.13 |
| 5 | 6.30 | 4.74 | 2.59 | 1.76 | 0.96 | 0.64 | 0.52 | 0.33 |
| 7 | 5.81 | 5.16 | 3.41 | 2.55 | 1.54 | 1.12 | 0.83 | 0.58 |
| 9 | 5.35 | 5.06 | 3.84 | 3.02 | 1.94 | 1.42 | 1.10 | 0.76 |
| 12 | 4.83 | 4.75 | 4.07 | 3.43 | 2.41 | 1.80 | 1.49 | 1.02 |
| 15 | 4.41 | 4.39 | 4.02 | 3.59 | 2.70 | 2.12 | 1.76 | 1.26 |
| 20 | 3.85 | 3.85 | 3.70 | 3.46 | 2.86 | 2.39 | 2.05 | 1.55 |
| 25 | 3.44 | -- | 3.38 | 3.27 | 2.85 | 2.49 | 2.20 | 1.73 |
| 30 | 3.12 | -- | 3.12 | 3.02 | 2.77 | 2.49 | 2.25 | 1.85 |
| 35 | 2.84 | -- | -- | 2.81 | 2.60 | 2.39 | 2.20 | 1.86 |
| 40 | 2.62 | -- | -- | 2.62 | 2.48 | 2.32 | 2.15 | 1.86 |
| 45 | 2.43 | -- | -- | -- | 2.32 | 2.21 | 2.09 | 1.86 |
| 50 | 2.27 | -- | -- | -- | 2.20 | 2.11 | 2.00 | 1.82 |
| 60 | 2.00 | -- | -- | -- | 1.96 | 1.92 | 1.86 | 1.72 |
| 80 | 1.62 | -- | -- | -- | 1.60 | 1.59 | 1.56 | 1.50 |
| 100 | 1.38 | -- | -- | -- | 1.38 | 1.35 | 1.33 | 1.28 |
| 120 | 1.16 | -- | -- | -- | -- | 1.16 | 1.16 | 1.12 |
| Modified from "Design of Drainage Facilities," by G. A. Hathaway, *Transactions*, Vol. 110, with permission from American Society of Civil Engineers. | | | | | | | | |

3-11.3.3 **Consolidated design curve.** The data presented in Figures 2-5 and 3-2 through 3-9 with respect to peak runoff rates and critical durations of supply have been consolidated into one diagram, Figure 3-15. Use of Figure 3-15 is not as precise as using Figures 2-5 and 3-2 through 3-9, but Figure 3-158 may be applied to most drainage problems. The following example is provided to illustrate the use of Figure 3-15. Assume an effective length of overland flow of 315 ft and a rate of supply of 1.0 in./hr. To determine the critical duration of supply, project a line vertically upward from the effective length to the intersection of the $t_c$ curve and proceed horizontally to the right to the critical duration of supply which, in this example, is 23 min. To determine the maximum rate of runoff, proceed vertically upward from the effective length to the intersection of the rate of supply line and proceed horizontally to the left to the maximum rate of runoff, which is 1.2 cfs/acre of drainage area.

**Figure 3-15. Consolidated Design Curve Composite of Peak Runoff Rates and Critical Durations of Supply Shown in Figures 3-2 through 3-9**




3-11.4 **Storage**

3-11.4.1 **Temporary storage or ponding.** If the rate of outflow from a drainage area is limited by the capacity of the drain serving the area, runoff rates exceeding the drain capacity must be stored temporarily. As soon as the rate of inflow into the ponding basin becomes less than the drain capacity, the accumulated storage may be drawn off at a rate equal to the difference between the drain capacity and the rate of inflow into the basin. The general relation between inflow, storage, and outflow is expressed as: outflow = inflow ∀ storage.

3-11.4.1.1 The rate of outflow from a ponding basin is affected by the elevation of the water surface at the drain inlet serving the area. The rate of outflow increases as the head on the inlet increases. However, because of the flat slopes of airfield areas, the surfaces of the storage ponds surrounding drain inlets are usually very large in comparison to the depth of water at the inlets. The rate of outflow through a particular drain inlet would be approximately constant as long as the rate of runoff and accumulated storage are sufficient to maintain the full discharge capacity of the drain inlet. The rate of outflow equals the rate of inflow into the pond until the full discharge capacity of the drain inlet is attained.

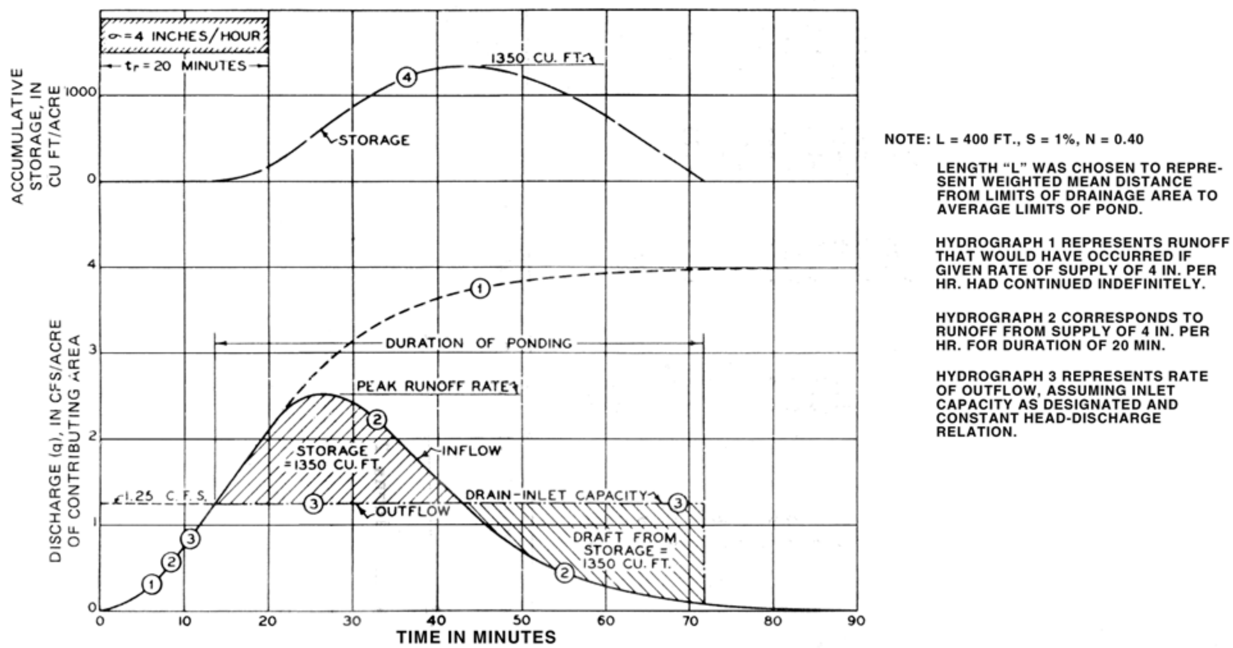
3-11.4.1.2 To illustrate these assumptions, reference is made to the curves shown in Figure 3-16 and the computations in Table 3-3. Hydrography 1 and 2 are developed as for Figure 3-14. Hydrograph 3 of Figure 3-16 represents the constant rate of outflow corresponding to inflow hydrography 2, when the drain-inlet capacity is assumed to be 1.25 cfs/acre of drainage area. Storage volume can be calculated from the area between curves 2 and 3. The volume of storage above outflow hydrography 3 and below hydrography 2 that would be accumulated at successive intervals of time under these conditions is indicated by curve 4 of Figure 3-16. The maximum storage that would accumulate under these particular conditions is 1,350 cu ft/acre of drainage area. The end of the accumulation period occurs approximately 43 min after the beginning of runoff.

**Table 3-3. Design Example**

| Duration of supply min (1) | Rate of runoff cfs/acre (2) | Rate of runoff + 20 min cfs/acre (3) | Rate of runoff to inlet[b] cfs/acre (4) | Drain inlet capacity cfs (5) | Storage increment[c] cu ft (6) | Total storage cu ft (7) |
|---|---|---|---|---|---|---|
| 0 | 0.0 | | 0.0 | 0.0 | 0 | 0 |
| 5 | 0.2 | | 0.2 | 1.25 | 0 | 0 |
| 10 | 0.8 | | 0.8 | 1.25 | 0 | 0 |
| 13 | | | | 1.25 | 0 | 0 |
| 15 | 1.5 | | 1.5 | 1.25 | +15 | 15 |
| 20 | 2.2 | 0.0 | 2.2 | 1.25 | +180 | 195 |
| 25 | 2.7 | 0.2 | 2.5 | 1.25 | +330 | 525 |
| 30 | 3.1 | 0.8 | 2.3 | 1.25 | +345 | 870 |
| 35 | 3.5 | 1.5 | 2.0 | 1.25 | +270 | 1,140 |
| 40 | 3.6 | 2.2 | 1.4 | 1.25 | +165 | 1,305 |
| 43 | | | | 1.25 | +32 | 1,337 |
| 45 | 3.7 | 2.7 | 1.0 | 1.25 | -15 | 1,322 |
| 50 | 3.8 | 3.1 | 0.7 | 1.25 | -120 | 1,202 |
| 55 | 3.85 | 3.5 | 0.35 | 1.25 | -218 | 984 |
| 60 | 3.9 | 3.6 | 0.3 | 1.25 | -277 | 707 |
| 65 | 3.95 | 3.7 | 0.25 | 1.25 | -292 | 415 |
| 70 | 4.0 | 3.8 | 0.2 | 1.25 | -308 | 107 |
| 72 | | | | 1.25 | -125 | 0 |
| 75 | 4.0 | 3.85 | 0.15 | 1.25 | | |
| 80 | 4.0 | 3.9 | 0.1 | 1.25 | | |
| 85 | 4.0 | 3.95 | 0.05 | 1.25 | | |
| 90 | 4.0 | 4.0 | 0.0 | | | |

Note: L = 400 feet; S = 1.0 percent; n = 0.40; σ = inches per hour; $t_c$ = 20 minutes.
[a] From Figure 3-12.
[b] Difference between columns 2 and 3.
[c] Example for 20- to 25-minute increment.
V = [(2.2 – 1.25) + (2.5 – 1.25)]/2 × (5 × 60) = 330 cubic feet.

**Figure 3-16. Sample Computations of Storage Required with Selected Drain-Inlet Capacity to Provide for Runoff from an Acre of Turf Under Assumptions Designated**

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3-11.4.2 **Drain-inlet capacity-storage diagrams.** The concepts presented by G. A. Hathaway (American Society of Civil Engineers, Transactions, Vol 110) and discussed in Section 3-11.4 have been included in the preparation of Figures 3-17 through 3-21. These graphs are presented to facilitate the determination of the drain-inlet capacity (Diagram A) and the critical duration of supply (Diagram B) for drainage areas where temporary ponding can be permitted. Where temporary ponding is permitted, $t_c$ reflects the time associated with both the overland flow and the time to obtain maximum temporary storage. The diagrams presented in Figures 3-20 through 3-24 have been prepared for use with effective lengths reduced to n = 0.40 and S = 1.0 percent. As an example of the use of these figures, assume:

- Effective length of overland flow = 300 ft.
- Maximum storage allowable = 1,000 cu ft/acre of drainage area.
- Rate of supply = 3.0 in./hr.

**Figure 3-17. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 0 and 40 ft**

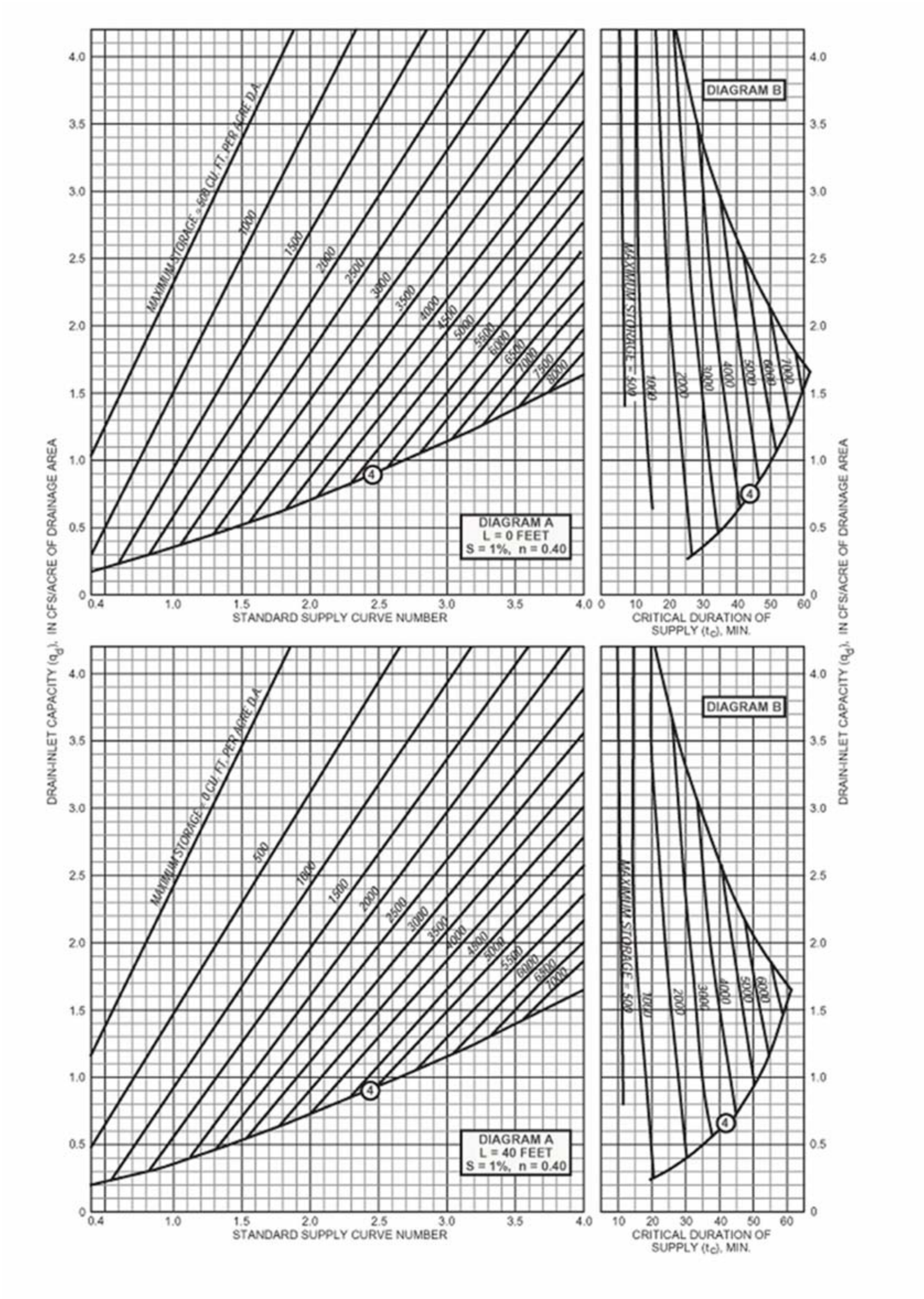


**Figure 3-18. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 100 and 200 ft**

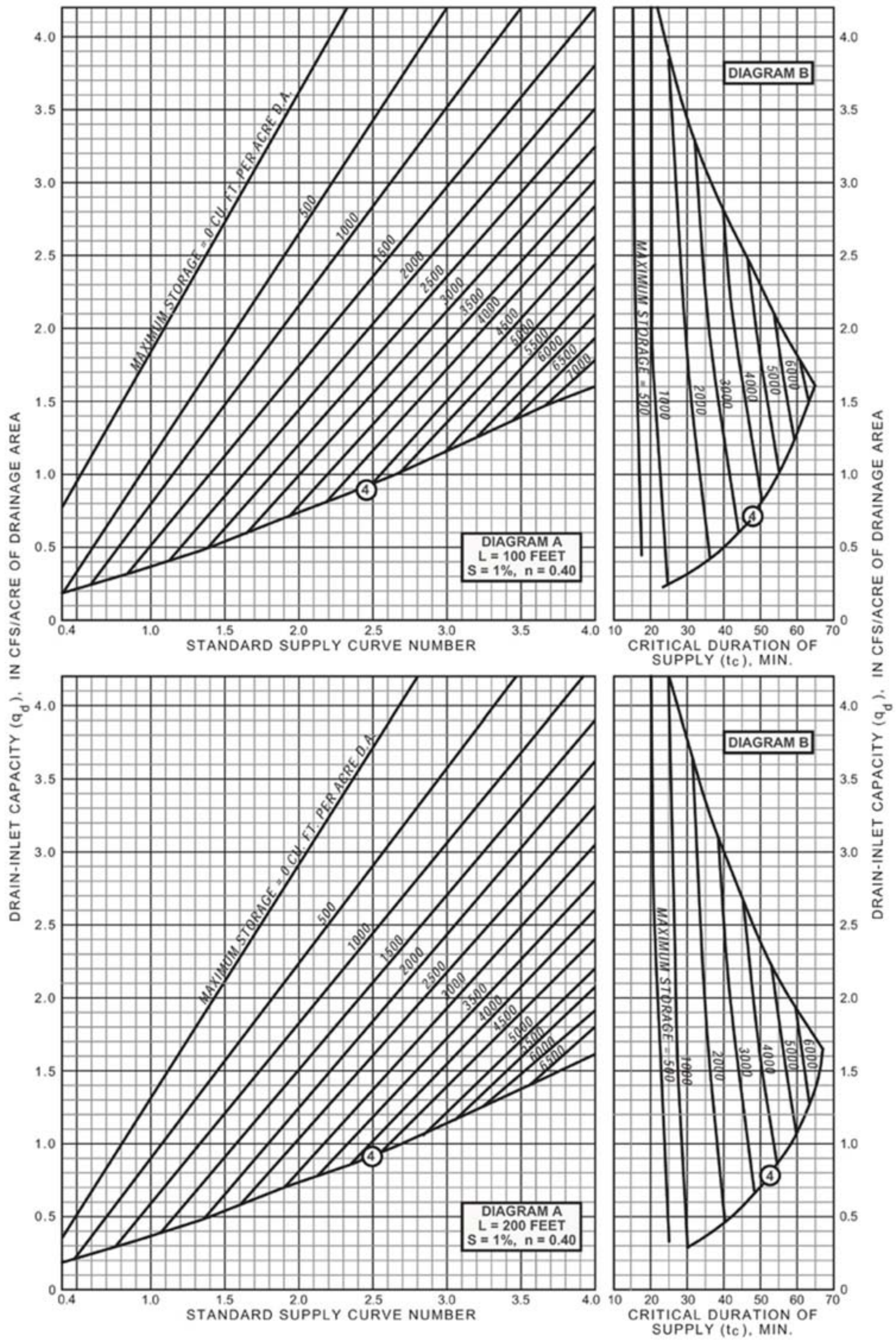


**Figure 3-19. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 300 and 400 ft**

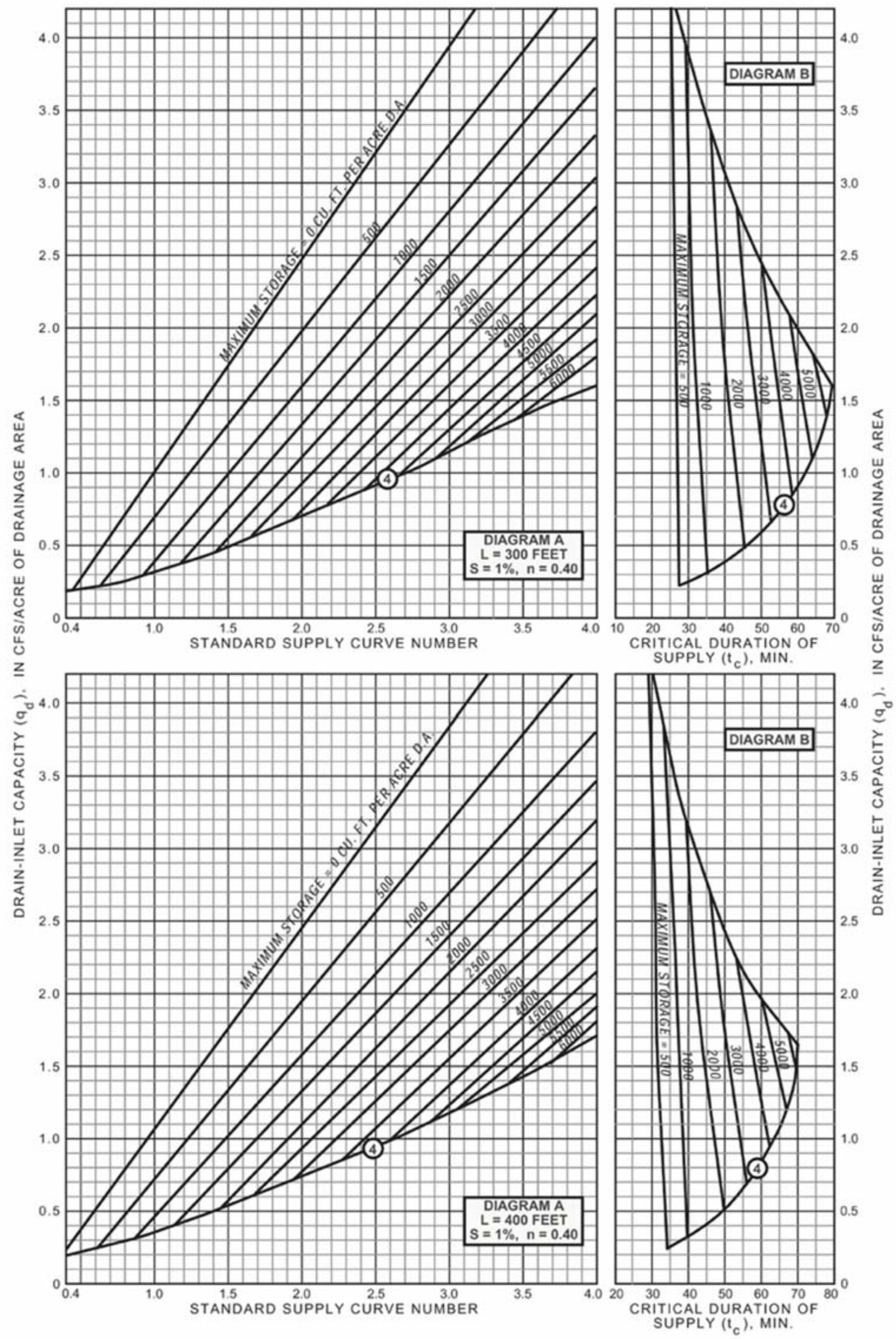


**Figure 3-20. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 600 and 800 ft**

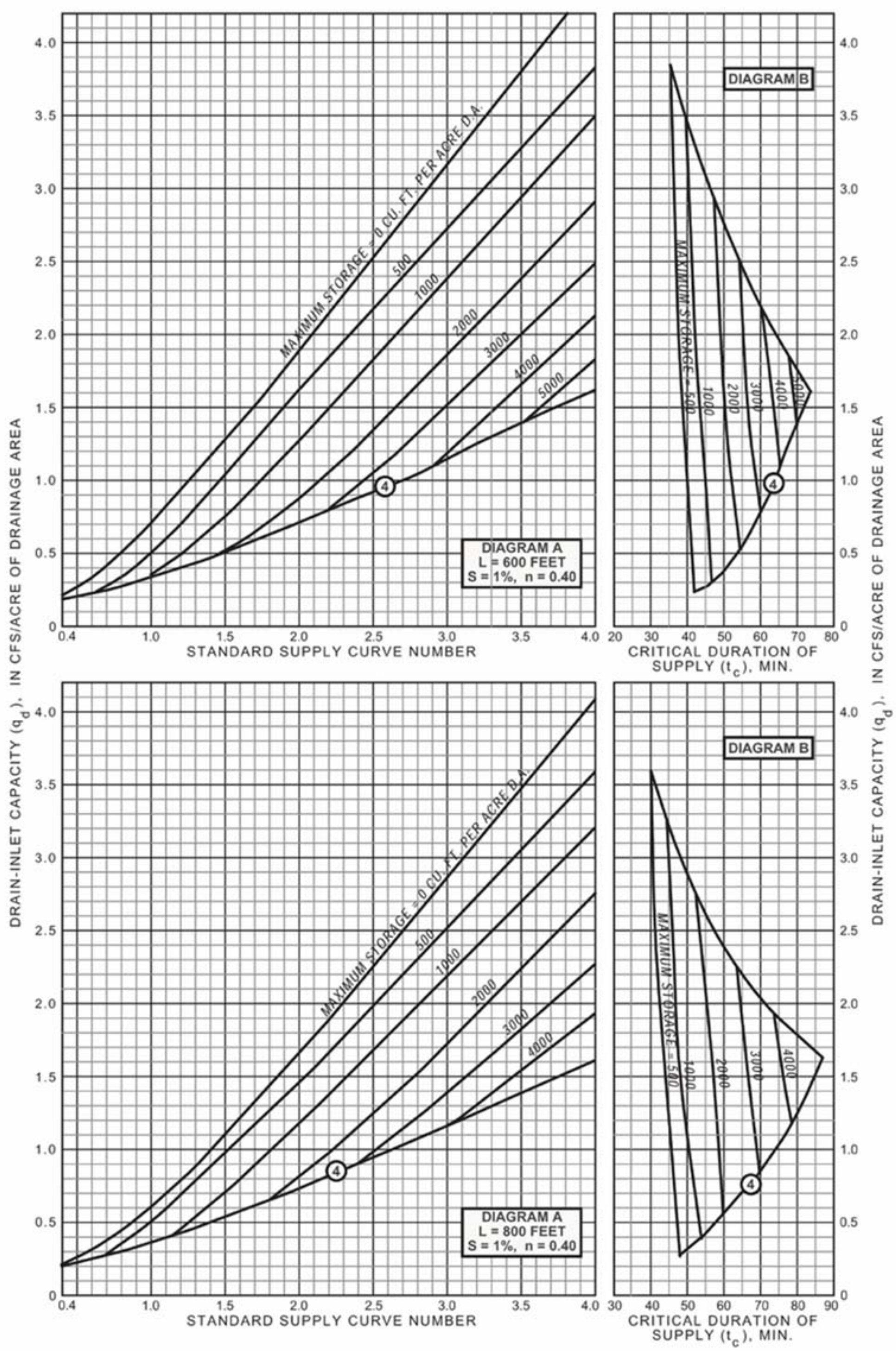
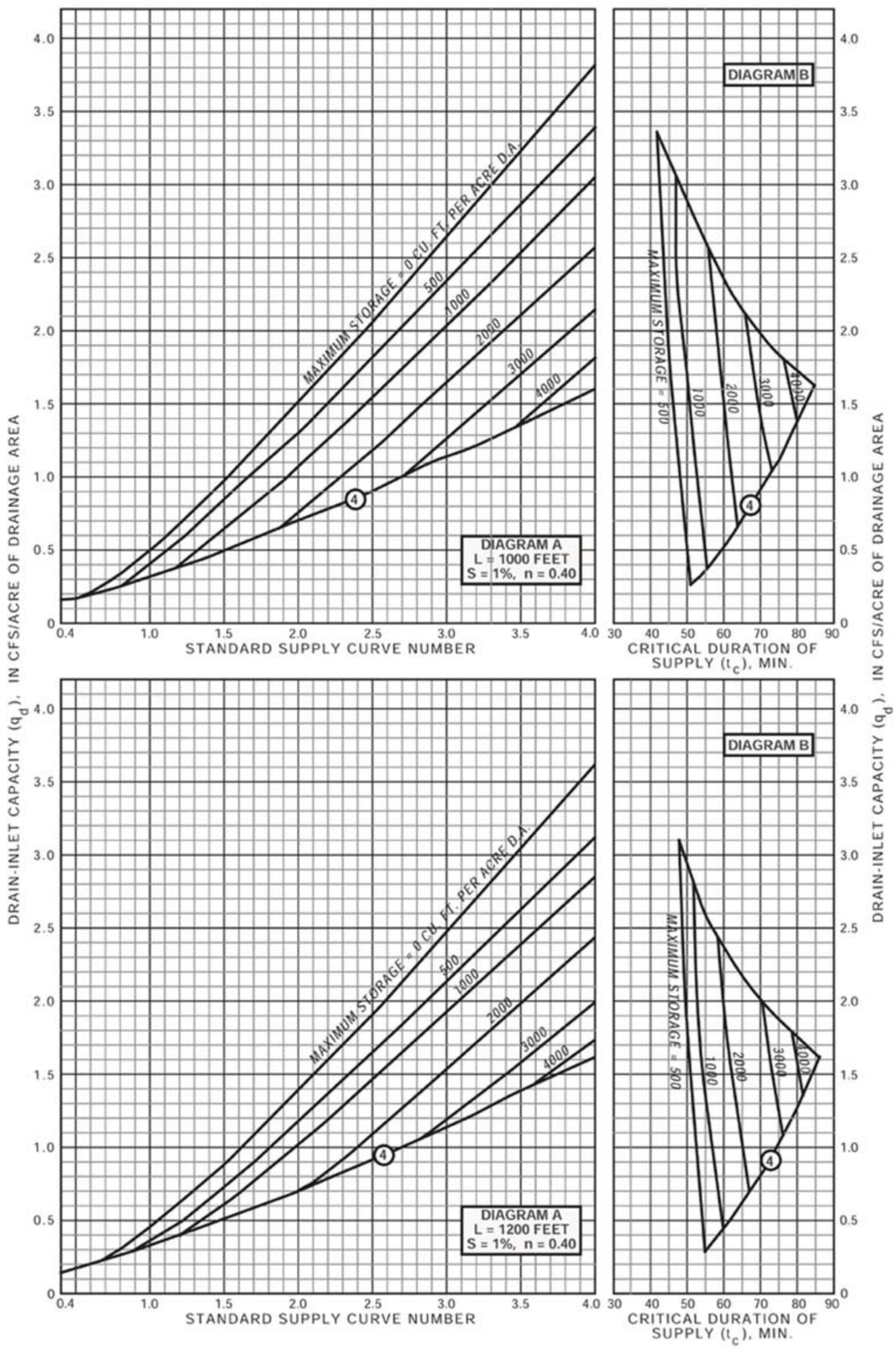


**Figure 3-21. Drain-Inlet Capacity Versus Maximum Surface Storage; L = 1,000 and 1,200 ft**



3-11.4.2.1 From the 3.0 in./hr line on the top portion of Figure 3-19, proceed vertically upward to the intersection of the 1,000 cu ft/acre of drainage area maximum storage capacity and then horizontally to the left to the intersection of the minimum design drain-inlet capacity of 2.8 cfs/acre of drainage area. To determine the critical duration of supply, $t_c$, proceed as before to the intersection of the maximum storage capacity on Diagram A; then move horizontally to the right to the intersection of the maximum, storage capacity on Diagram B, and then vertically downward to the intersection of $t_c$ at 30 min.

3-11.4.2.2 If the drain-inlet capacity of an outlet has been previously established and the temporary ponding capacity is known, Diagram B can be entered directly to find $t_c$. Diagram B of Figure 3-19, for an effective length of 400 ft, offers a quick check on the example presented in Table 3-3 and Figure 3-16.

3-11.4.3 **Minimum drain-inlet capacity.** Curve 4 in Diagram A (Figures 3-17 through 3-21) represents the minimum drain-inlet capacities that are considered desirable, regardless of the volume of storage that may be permitted. The drain-inlet capacities represented by Curve 4 of Diagram A are equal to the rates of supply corresponding to durations of 4 hr on the standard supply curves given in Figure 3-1. If the drain-inlet capacity indicated by Curve 4 is adopted in a particular case, some storage may result in the ponding basin during all storms less than 4 hr in duration that produce rates corresponding to the given standard supply curve.

3-11.5. **Drain-Inlet and Drain Capacities**

3-11.5.1 **Determination of drain-inlet capacities without ponding.** From Figures 3-5 through 3-9 and 3-15 through 3-17, select the supply curve number corresponding to the weighted standard supply curve determined previously. The critical duration of supply, $t_c$, and the maximum rate of runoff, $q_d$, in cubic feet per second per acre, for the individual inlet drainage area can be read directly from the graph for the given value of effective length. If Figure 3-15 is used, the same data can be obtained by following the procedure described in Section 3-11.3.3.

3-11.5.1.1 To obtain the maximum rate of runoff at a given point in a drainage system, during a supply of uniform intensity, the storm must continue long enough to produce the maximum rate of runoff into each upstream inlet and to permit the inflow to travel through the drain from the “critical inlet” to the point of design. “Critical inlet” is defined as the upstream inlet from which the critical duration of supply causes the maximum runoff to the point of design. The critical duration of supply necessary for these purposes is referred to as $t'_c$ and is expressed as

$$t'_c - t_c + t_d \qquad \text{(eq 3-4)}$$

where $t_c$ is the duration of supply that would provide the maximum design-storm runoff from the area tributary to the critical drain inlet, and $t_d$ is the time required for water to flow from the critical drain inlet to the point of design. The critical drain inlet normally may be assumed to be the inlet located the greatest distance upstream from the given point. Care should be taken to check whether $t_c$ to an inlet along a drainage line exceeds the time required for water falling on a more distant area to reach this same inlet. Problems which arise in this regard must be investigated individually to determine under what conditions of time and flow the maximum volume of water can be expected at the point of design.

3-11.5.1.2 In order to simplify the determination of drain-inlet capacities, the computed value of $t'_c$ may be rounded off to the nearest 5 min. Inspection of Figures 3-2 through 3-9 will disclose that for large values of effective length and low values of supply curves the maximum rate of runoff is approximately constant after $t_c$ duration of supply. In order to facilitate design computations, the drain-inlet capacity values, $q_d$, obtained from the 0 storage capacity line of Diagram A of Figures 3-20 and 3-21 should be used as a replacement for the maximum rate of runoff when the duration of supply is greater than $t_c$, when the values of effective length are large, and when low values of the supply curve are in effect.

3-11.5.2 **Determination of drain-inlet capacities with temporary ponding.** From Figures 3-17 through 3-21, select the graph corresponding to the effective length and determine the drain-inlet capacity from the given standard supply curve value and maximum permissible ponding. In a drainage system where ponding is used, the maximum rate of flow at any given point in the drainage system may be determined, in most cases, by the simple addition of the peak discharges for the upstream inlets based on drain-inlet capacities. This procedure is justified in view of the prolonged period where temporary ponding takes place as shown in Figure 3-16. Curve 4 in Figures 3-17 through 3-21 represents the minimum drain-inlet capacities that are considered desirable, regardless of the volume of flooding exceeding allowable limits. The drain-inlet capacities represented by curve 4, in cubic feet per second per acre of drainage area, are equal to the rates of supply corresponding to durations of 4 hr on the respective standard supply curve given in Figure 3-1. If the drain-inlet capacity indicated by curve 4 is adopted in a particular case, some storage may result in the ponding basin during all storms less than 4 hr in duration that produce supply rates corresponding to the given standard supply curve. The proper criteria to be followed in estimating minimum drain-inlet capacities depend largely on the extent of drainage desired and the characteristics of the soil involved.

3-11.5.3 **Computation of pipe sizes.** The size and gradient of storm drain required to discharge design-storm runoff may be determined by use of Manning's formula presented in nomograph form in Figures 3-22 through 3-25. Storm drains will have a minimum diameter of 12 in. to lessen possibilities of clogging. Design of drain-inlet facilities is discussed in Chapter 4.

3-11.5.3.1 For conditions of instantaneous runoff the hydraulic gradient will be kept at the top of the pipe. Where temporary ponding is proposed, considerable saving in pipe sizes may be accomplished by designing the pipeline under pressure, provided undesirable backflow does not result in some critical areas.

3-11.5.3.2 Where flooding from a temporary ponding area due to rates of supply greater than design will cause a hazard to the adjacent areas, special provisions must be made to assure adequate control. An auxiliary drainage system or a diversionary channel to another inlet or ponding area is a method that has been used successfully. The designer must consider each case individually to arrive at the most economical solution to provide the desired results.

**Figure 3-22. Nomograph for Computing Required Size of Circular Drain Smooth Interior, Flowing Full; n = 0.012**

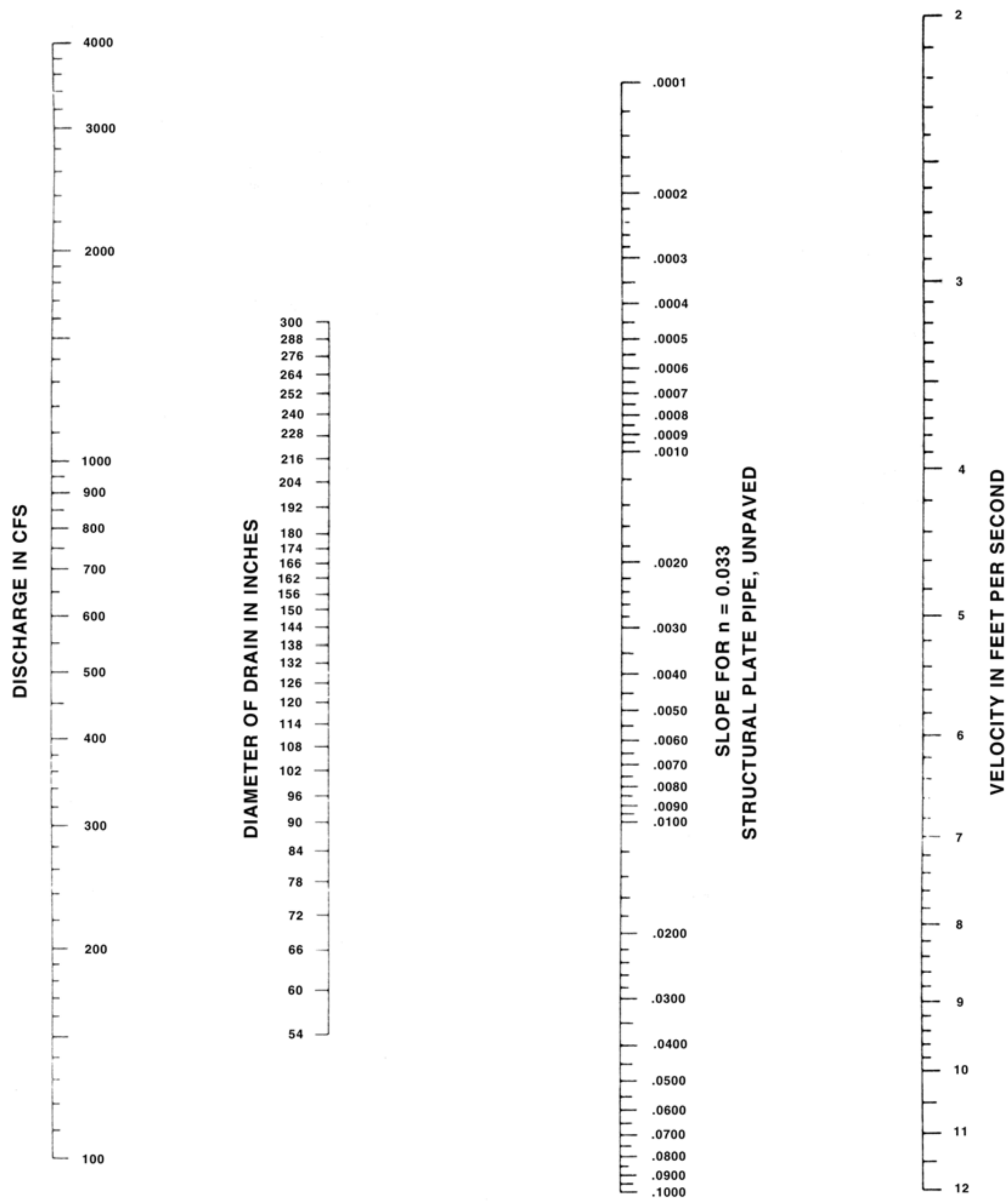



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**Figure 3-23. Nomograph for Computing Required Size of Circular Standard Corrugated Metal Pipe, 25 percent Paved Invert, Flowing Full; n = 0.021**

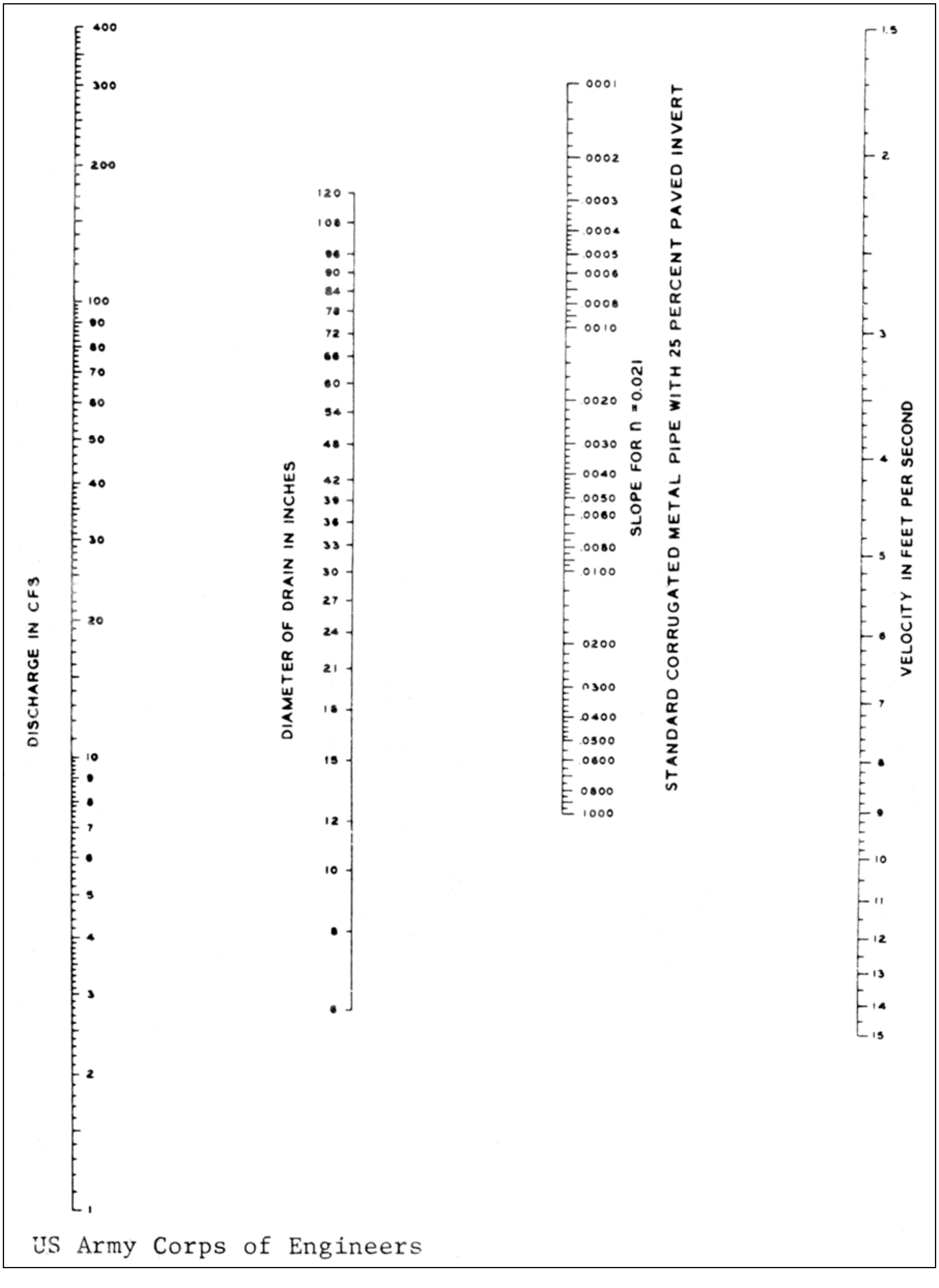

**Figure 3-24. Nomograph for Computing Required Size of Circular Standard Corrugated Metal Pipe, Unpaved, Flowing Full; n = 0.021**

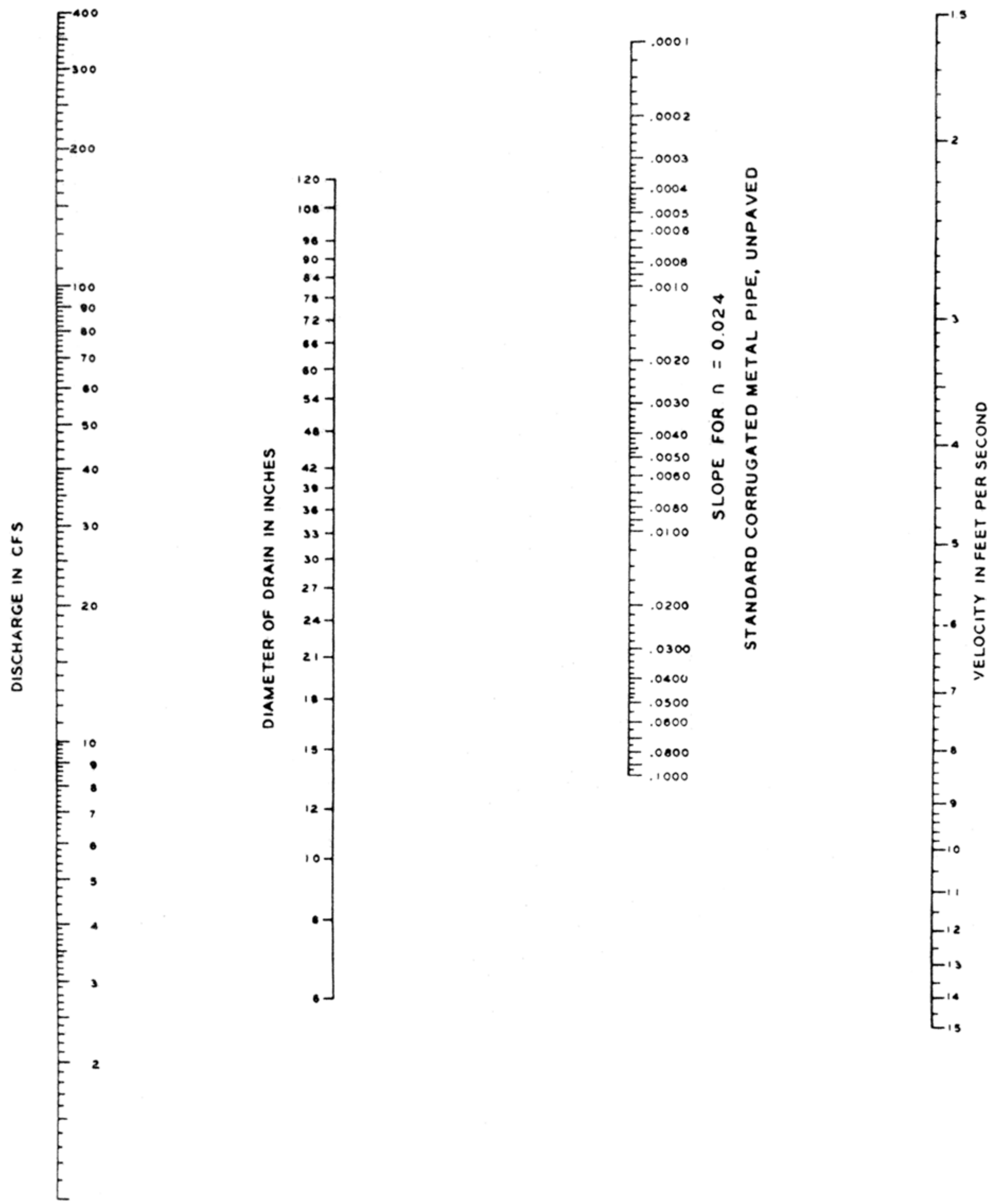


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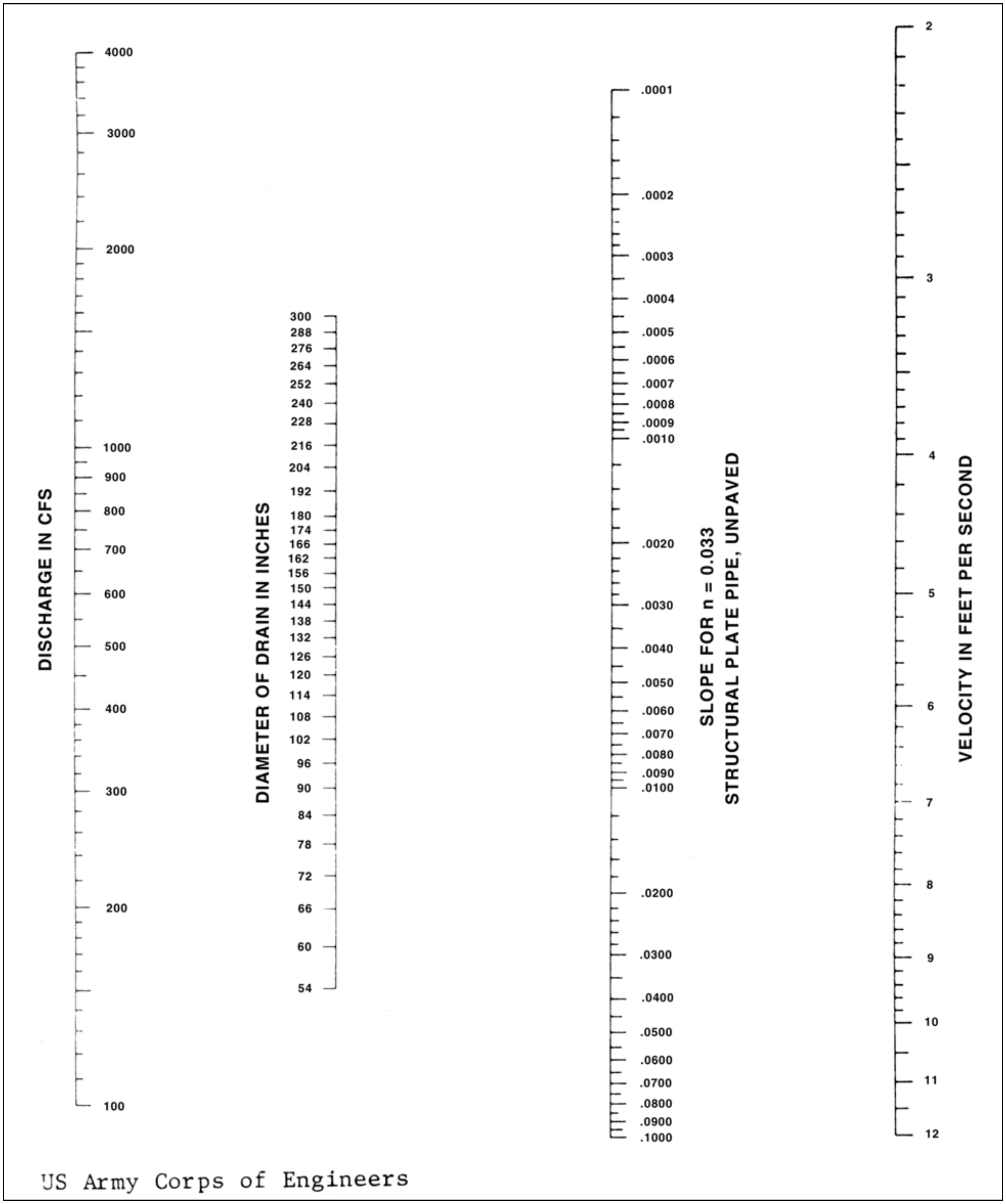
**Figure 3-25. Nomograph for Computing Required Size of Circular Structural Plate Pipe, Unpaved, Flowing Full; n = 0.033**



# CHAPTER 4

# HYDRAULICS AND DRAINAGE STRUCTURES

## 4-1 HYDRAULICS

4-1.1 **Purpose**. This chapter discusses water disposal methods which ensure the safe and efficient operation of airport and heliport facilities, to describe an efficient drainage system, and to detail problems that can be caused by inadequate drainage systems.

4-1.2 **Scope**. This chapter provides design criteria for common drainage and erosion-control structures, cover requirements for several types of pipe for varying wheel loads, and protection of storm drains against freezing conditions in seasonal frost areas.

### 4-1.3 Problem Areas

4-1.3.1 The problem areas include culverts, underground storm drainage systems, scour, riprap requirements at culvert and storm drain outlets, outlet energy dissipators, natural and artificial open channels, and drop structures.

4-1.3.2 Problems in the design of drainage and erosion-control structures for airfields and heliports result from failure to follow a long-range master development plan, inadequate basic data, and limitation in time or funding. Problems in construction and operation result from poor inspection and construction procedures, and lack of periodic inspections and follow-up maintenance. There is also the misconception that drainage is considered to be the least important factor affecting the performance of an installation.

4-1.3.3 Adequate initial drainage facilities provide satisfactory performance with little maintenance and good long run economy, while faulty installations will require extensive repairs, replacements or other remedies.

### 4-1.4 Design

4-1.4.1 Improper design and careless construction of various drainage structures may render airfields and heliports ineffective and dangerous to the safe operations of military aircraft. Consequently, the necessity of applying basic hydraulic principles to the design of all drainage structures must be emphasized. Care should be given to both preliminary field surveys which establish control elevations and to construction of the various hydraulic structures in strict accordance with proper and approved design procedures. A successful drainage system can only be obtained by the coordination of both the field and design engineers.

4-1.4.2 Fuel spillage will not be collected in storm or sanitary sewers. Fuel spillage may be safely disposed of by providing ponded areas for drainage so that any fuel spilled can be removed from the water surface. Bulk-fuel-storage areas will not be considered as built-over areas. Curbs, gutters, and storm drains will not be provided for drainage around tank-car or tank-truck unloading areas, tank-truck loading stands, and tanks in bulk-fuel-storage areas.

4-1.4.3 Waste water from cleaning floors, machines, and airplanes is also prohibited from entering storm or sanitary sewers directly. Treatment facilities, traps, or holding facilities will be provided as appropriate.

4-1.5 **Outfall Considerations**. In some localities the upstream property owner may artificially drain his property onto the downstream properties without liability for damages from the discharge of water, whereas in other areas he may be liable for damage caused by such drainage. Local law and practices should be reviewed prior to the design of a drainage system, and the advice of the Division real estate office should be obtained.

4-1.6 **Drainage Law**

4-1.6.1 There are two basic rules of law applied in drainage problems, Roman civil law and common-enemy rule.

4-1.6.2 A number of states follow Roman civil law which specifies that the owners of high land are entitled to discharge their drainage water onto lower land through natural depressions and channels without obstruction by the lower owner. The elevation of land gives the owners of high land an advantage allowing them to accelerate the flow of surface water by constructing ditches or by improving natural channels on the property or by installing tile drains. The owners of lower land, however, cannot prevent natural drainage from entering their property from above because water may not be carried across a drainage divide and discharged on land which would not have received the water naturally.

4-1.6.3 Other states employ the common-enemy rule which recognizes that water is a common enemy of all and that any landowners have the right to protect themselves from water flowing onto their land from a higher elevation. Under this law, the higher landowners cannot construct drainage works which damage the property of the lower owners without first securing an easement. The lower owners, however, are allowed to construct dikes or other facilities to prevent the flow of surface water onto their property.

4-1.6.4 Both Roman civil and the common-enemy rule place the responsibility for damages on the party altering the natural stream pattern of an area or creating an obstacle which blocks the flow of a natural stream.

## 4-2 AIRFIELDS

### 4-2.1 Drainage Pipe

4-2.1.1 **General**. A drainage pipe is a structure (other than a bridge) used to convey water through or under a runway fill or some other obstruction. Materials for permanent-type installations include plain or nonreinforced concrete, reinforced concrete, corrugated steel, asbestos cement, and day and aluminum corrugated pipe.

#### 4-2.1.2 Selection of type of pipe

4-2.1.2.1 The selection of a suitable construction conduit will be governed by the availability and suitability of pipe materials for local conditions with due consideration of economic factors. It is desirable to permit alternates so that bids can be received with contractor's options for the different types of pipe suitable for a specific installation. Allowing alternates serves as a means of securing bidding competition. When alternate designs are advantageous, each system will be designed economically, taking advantage of full capacity, best slope, least depth, and proper strength and installation provisions for each material involved. Where field conditions dictate the use of one pipe material in preference to others, the reasons will be clearly presented in the design analysis.

4-2.1.2.2 Factors which should be considered in selecting the type of pipe include strength under maximum or minimum cover, bedding and backfill conditions, anticipated loadings, length of sections, ease of installation, corrosive action by liquids carried or surrounding soil, jointing methods, expected deflection, and cost of maintenance. Although it is possible to obtain an acceptable pipe installation to meet design requirements by establishing special provisions for several possible materials, ordinarily only one or two alternates will economically meet the individual requirements for a proposed drainage system.

4-2.1.3 **Selection of n values**. Whether the coefficient of roughness, n, should be based on the new and ideal condition of a pipe or on anticipated condition at a later date is a difficult problem. Sedimentation or paving in a pipe will affect the coefficient of roughness. Table 4-1 gives the n values for smooth interior pipe of any size, shape, or type and for annular and helical corrugated metal pipe both unpaved and 25 percent paved. When n values other than those listed are selected, such values will be amply justified in the design analysis.

4-2.1.4 **Restricted use of bituminous-coated pipe**. The installation of corrugated-metal pipe with any percentage of bituminous coating should be restricted where fuel spillage, wash rack waste, and/or solvents can be expected to enter the pipe. Polymeric coated steel pipe is recommended where solvents might be expected.

**Table 4-1. Roughness Coefficients for Various Pipes**

| **n = 0.012 for smooth interior pipes of any size, shape, or type*** | | |
|---|---|---|
| **n value for annular corrugated metal** | | |
| **Corrugation size** | **Unpaved** | **25% Paved** |
| 2 + 2/3 by 1/2 in. | 0.024 | 0.021 |
| 3 by 1 in. | 0.027 | 0.023 |
| 6 by 2 in. | 0.028-0.033 | 0.024-0.028 |
| 9 by 2 + 1/2 in. | 0.033 | 0.028 |
| **n values for helical corrugated metal (2 + 2/3 by 1/2 in. corrugations)** | | |
| **Pipe diameter** | **Unpaved** | **25% Paved** |
| 12-18 in. | 0.011-0.014 | X |
| 24-30 in. | 0.016-0.018 | 0.015-0.016 |
| 36-96 in. | 0.019-0.024 | 0.017-0.021 |
| * Includes asbestos cement, plastic, cast iron, clay, concrete (precast or cast-in-place) or fully paved corrugated metal pipe. | | |

4-2.1.5 **Minimum and maximum cover**

4-2.1.5.1 Heliport and airport layout will typically include underground conduits which pass under runways, taxiways, aprons, helipads, and other hardstands. In the design and construction of the drainage system, it will be necessary to consider both minimum and maximum earth cover allowable in the underground conduits to be placed under both flexible and rigid pavements as well as beneath unsurfaced airfields and medium-duty landing-mat-surfaced fields. Underground conduits are subject to two principal types of loads: dead loads caused by embankment or trench backfill plus superimposed stationary surface loads, uniform or concentrated; and live or moving loads, including impact.

4-2.1.5.2 Drainage systems should be designed to provide the greatest possible capacity to serve the planned pavement configuration. Additions to or replacements of drainage lines following initial construction are both costly and disrupting to aircraft traffic.

4-2.1.5.3 Investigations of in-place drainage and erosion control facilities at military installations were made during the period 1966 to 1972. The facilities observed varied from 1 to more than 30 years of age. The study revealed that buried conduits associated storm drainage facilities installed from the early 1940s until the mid-1960s appeared to be in good to excellent structural condition. However, many failures of buried conduits were reported during construction. Therefore, it should be noted that minimum conduit cover requirements are not always adequate during construction. When construction equipment, which may be heavier than live loads for which the conduit has been designed, operated over or near an already in-place underground conduit, it is the contractor's responsibility to provide any additional cover during construction to avoid damage to the conduit.

4-2.1.5.4 Since 1940 gross aircraft weight has increased twenty-fold, from 35,000 lb to approximately 700,000 lb. The increases in aircraft weight have had a significant effect on design criteria, construction procedures, and material used in the manufacture and construction of buried conduits. Major improvements in the design and construction of buried conduits in the two decades mentioned include among other items increased strength of buried pipes and conduits, increased compaction requirements, and revised minimum and maximum cover tables.

4-2.1.5.5 For minimum and maximum cover design, H-20, 15-K, F-15, C-5A, C-141, C-130, B-1 and B-52 live loads and 120 lb/ft$^3$ backfill have been considered. Cover heights for flexible pipes and reinforced concrete pipes were based on an analysis of output (Juang and Lee 1987) from the CANDE computer program (FHWA-RD-77-5, FHWA-RD-77-6, FHWA-RD-80-172). Wall crushing, seam separation, wall buckling, formation of a plastic hinge, and excessive deflection, as functions of pipe size and stiffness, backfill conditions, fill height, and live load were considered for flexible pipes. Steel yield and concrete crushing, shear failure and tensile cracking, as functions of pipe size, backfill conditions, full height, concrete strength, steel content, and live load were considered for real inforced concrete pipe. Nonreinforced concrete and vitrified clay pipe designs are based on the American Concrete Pipe Association's D-load design procedure based on a 0.01-in. crack.

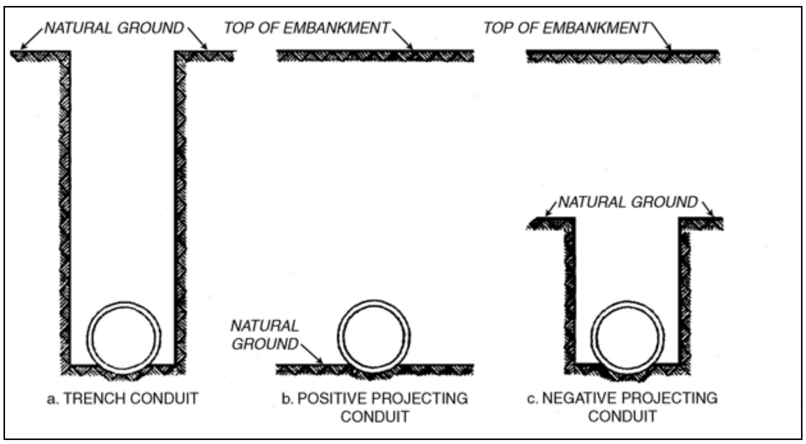
4-2.1.5.6 The tables in Appendix D identify the recommended minimum and maximum cover requirements for storm drains and culverts. These cover depths are valid for the specified loads and conditions, including average bedding and backfill. Deviations from these loads and conditions significantly affect the allowable maximum and minimum cover, requiring a separate design calculation. Most pipe seams develop the full yield strength of the pipe wall. However, there are some exceptions which occur in standard metal pipe manufacture. To maintain a consistent safety factor of 2.0 for these pipes, the maximum ring compression must be one-half of the seam strength rather than one-half of the wall strength for these pipes. Table 4-2 shows cover height reductions for standard riveted and bolted seams which do not develop a strength equivalent to $f_y$ = 33,000 lb/in$^2$. The reduction factors shown are the ratios of seam strength to wall strength. The maximum cover height for pipes with weak seaming as identified in Table 4-2 can be determined by multiplying the maximum cover height for a continuously-welded or lock seam pipe (Appendix D) by the reduction factors shown in Table 4-2.

4-2.1.5.7 Figures 4-1, 4-2, 4-3, and 4-4 indicate the three main types of rigid conduit burial, the free-body conduit diagrams, trench bedding for circular pipe, and beddings for positive projecting conduits, respectively. Figure 4-5 is a schematic representation of the subdivision of classes of conduit installation which influences loads on underground conduits.

**Table 4-2. Maximum Cover Height Reduction Factors for Riveted and Bolted Seams**

| Thickness, in. | Gage | 5/16 in. Rivets 2-2/3 × 1/2 in. | | 3/8 in. Rivets | | | 7/16 in. Rivets 3 × 1 in. | 3/4 in. Bolts 6 × 2 in. |
|---|---|---|---|---|---|---|---|---|
| | | | | 2-2/3 × 1/2 in. | | 3 × 1 in. | | |
| | | Single | Double | Single | Double | Double | Double | 4 bolts/ft |
| 0.064 | 16 | 0.65 | 0.84 | | | 0.98 | | |
| 0.079 | 14 | 0.57 | 0.93 | | | 0.97 | | |
| 0.109 | 12 | | | 0.52 | | | | 0.82 |
| 0.138 | 10 | | | 0.43 | 0.85 | | 0.96 | 0.97 |
| 0.168 | 8 | | | 0.36 | 0.73 | | 0.87 | |

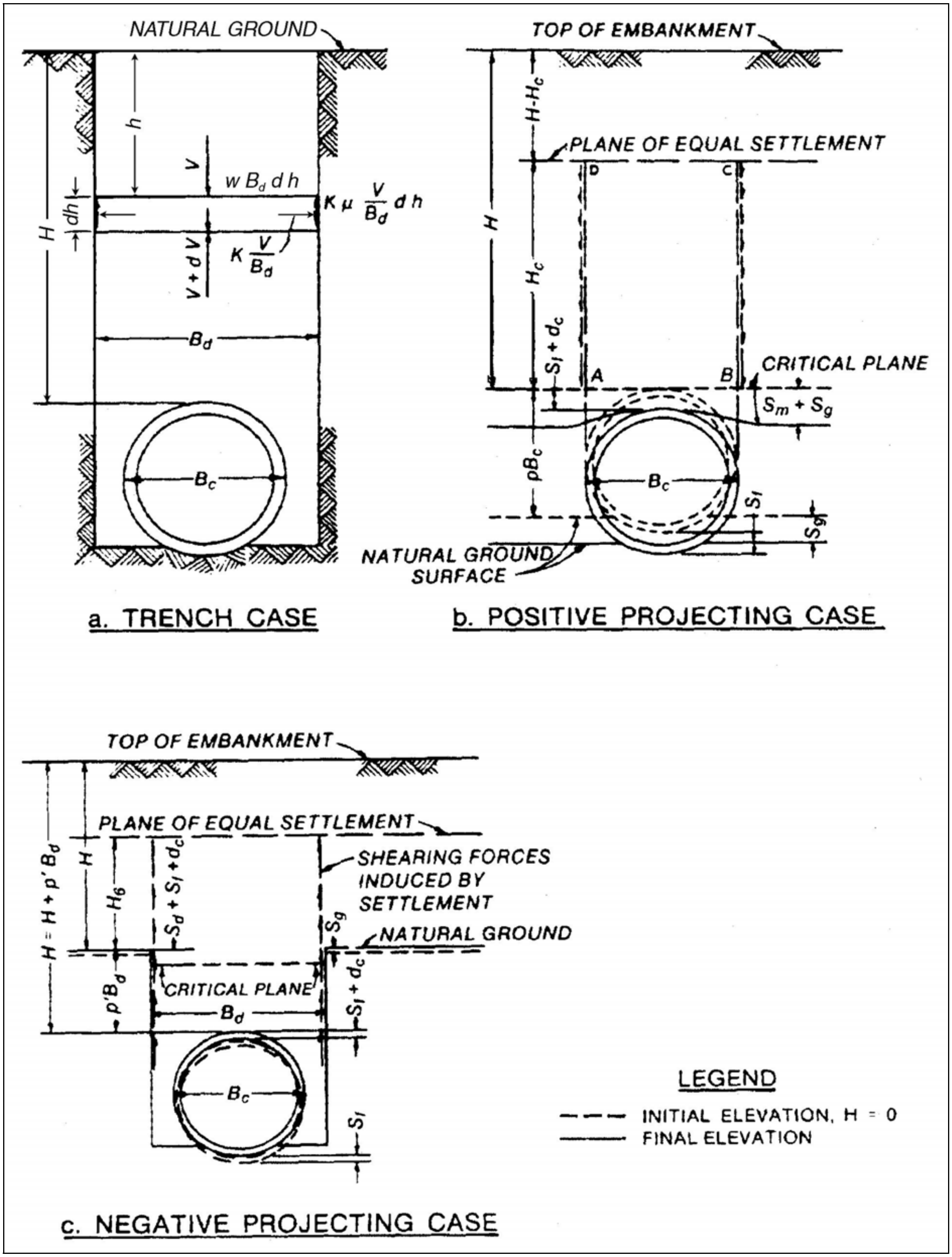
**Figure 4-1. Three Main Classes of Conduits**



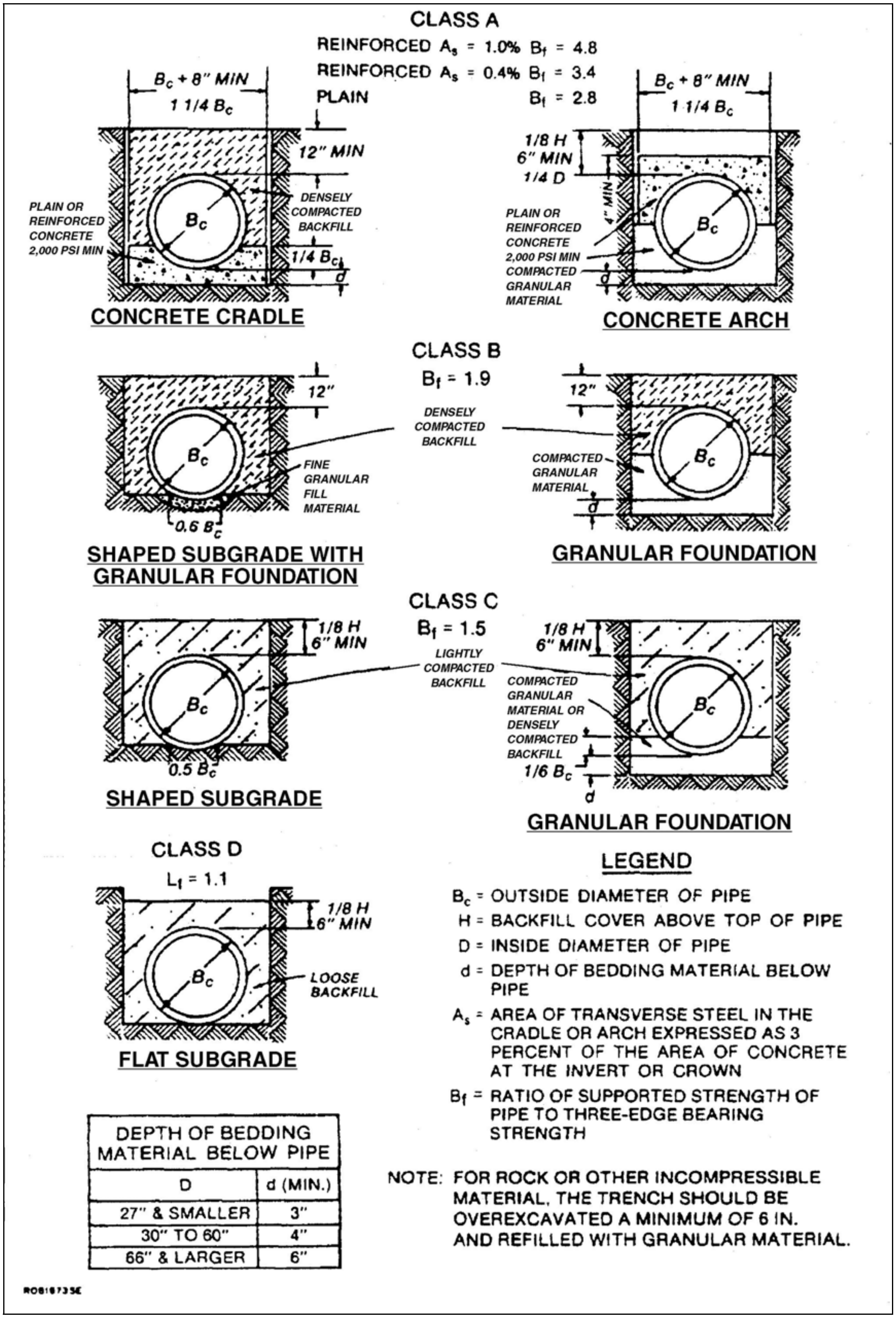

4-2.1.6 **Frost condition considerations**. The detrimental effects of heaving of frost-susceptible soils around and under storm drains and culverts are principal considerations in the design of drainage systems in seasonal frost areas. In such areas, freezing of water within the drainage system, except icing at inlets, is of secondary importance provided the hydraulic design assures minimum velocity flow.

4-2.1.6.1 Drains, culverts, and other utilities under pavements on frost-susceptible subgrades are frequently locations of detrimental differential surface heaving. Heaving causes pavement distress and loss of smoothness because of abrupt differences in the rate and magnitude of heave of the frozen materials. Heaving of frost-susceptible soils under drains and culverts can also result in pipe displacement with consequent loss of alignment, joint failures, and in extreme cases, pipe breakage. Placing drains and culverts beneath pavements should be minimized to the extent possible. When this is unavoidable, the pipes should be installed before the base course is placed in order to obtain maximum uniformity. The practice of excavating through base courses to lay drain pipes and other conduits is unsatisfactory since it is almost impossible to attain uniformity between the compacted trench backfill and the adjacent material.

**Figure 4-2. Free-Body Conduit Diagrams**



## Figure 4-3. Trench Beddings for Circular Pipe




LEGEND

$B_c$ = OUTSIDE DIAMETER OF PIPE
H = BACKFILL COVER ABOVE TOP OF PIPE
D = INSIDE DIAMETER OF PIPE
d = DEPTH OF BEDDING MATERIAL BELOW PIPE
$A_s$ = AREA OF TRANSVERSE STEEL IN THE CRADLE OR ARCH EXPRESSED AS 3 PERCENT OF THE AREA OF CONCRETE AT THE INVERT OR CROWN
$B_f$ = RATIO OF SUPPORTED STRENGTH OF PIPE TO THREE-EDGE BEARING STRENGTH

NOTE: FOR ROCK OR OTHER INCOMPRESSIBLE MATERIAL, THE TRENCH SHOULD BE OVEREXCAVATED A MINIMUM OF 6 IN. AND REFILLED WITH GRANULAR MATERIAL.

DEPTH OF BEDDING MATERIAL BELOW PIPE

| D | d (MIN.) |
|---|---|
| 27" & SMALLER | 3" |
| 30" TO 60" | 4" |
| 66" & LARGER | 6" |

**Figure 4-4. Beddings for Positive Projecting Conduits**

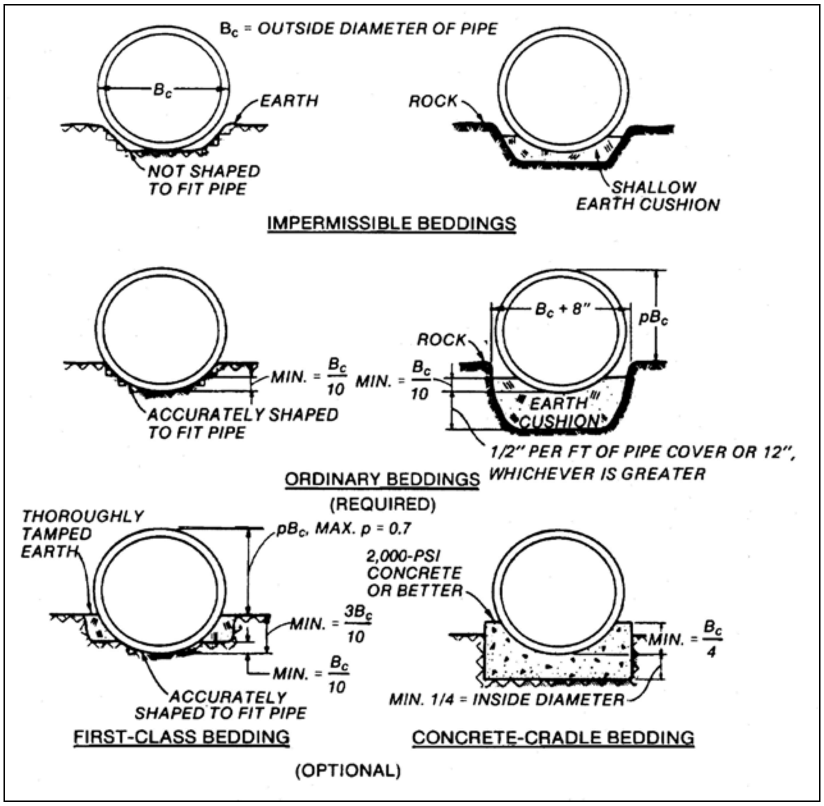
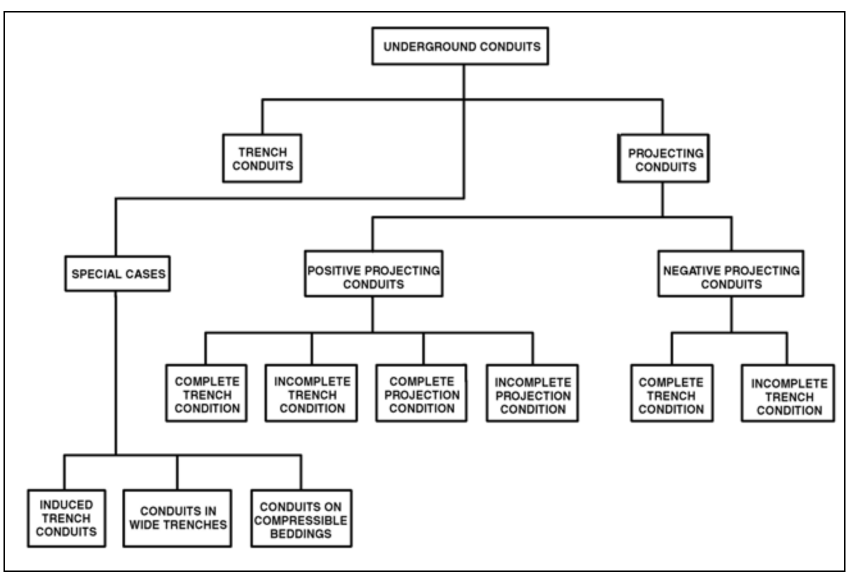



**Figure 4-5. Installation Conditions Which Influence Loads on Underground Conduits**



4-2.1.6.2 No special measures are required to prevent heave in nonfrost-susceptible subgrades. In frost-susceptible subgrades where the highest groundwater table is 5 ft or more below the maximum depth of frost penetration, the centerline of the pipe should be placed at or below the depth of maximum frost penetration. Where the highest ground-water table is less than 5 ft below the depth of maximum frost penetration and the pipe diameter is 18 in. or more, one of the following measures should be taken:

a. Place the centerline of the pipe at or below the depth of maximum frost penetration and backfill around the pipe with a highly free-draining nonfrost-susceptible material.

b. Place the centerline of the pipe one-third diameter below the depth of maximum frost penetration.

4-2.1.6.3 To prevent water from freezing in the pipe, the invert of the pipe should be placed at or below the depth of maximum frost penetration. In arctic and subarctic areas it may be economically infeasible to provide sufficient depth of cover to prevent freezing of water in subdrains; also, in the arctic, no residual thaw layer may exist between the depth of seasonal frost penetration and the surface of permafrost.

Subdrains are of little value in such areas because, unless protected from freezing, they are usually blocked with ice during the spring thawing period. Water freezing in culverts also presents a serious problem in arctic and subarctic regions. The number of such structures should be held to a minimum and should be designed based on twice the normal design capacity. Thawing devices should be provided in all culverts up to 48 in. in diameter. Large diameter culverts are usually cleaned manually immediately prior to the spring thaw. Drainage requirements for arctic and subarctic regions are presented in Chapter 8.

4-2.1.6.4 The following design notes should be considered for installations located in seasonal frost areas.

a. Note 1. Cover requirement for traffic loads will apply when such depth exceeds that necessary for frost protection.

b. Note 2. Sufficient granular backfill will be placed beneath inlets and outlets to restrict frost penetration to nonheaving materials.

c. Note 3. Design of short pipes with exposed ends, such as culverts under roads, will consider local icing experience. If necessary, extra size pipe will be provided to compensate for icing.

d. Note 4. Depth of frost penetration in well-drained, granular, nonfrost-susceptible soil beneath pavements kept free of snow and ice will be determined from data found in Figure 3-5 of TM 5-818-2/AFM 88-6, Chapter 4. For other soils and/or surface conditions, frost penetrations will be determined by using conservative surface condition assumptions and methods outlined in TM 5-852-6/ AFM 88-19, Volume 6. In all cases, estimates of frost penetration will be based on the design freezing index, which is defined as the average air-freezing index of the three coldest winters in a 30-yr period, or the air-freezing index for the coldest winter in the past 10-yr period if 30 years of record are unavailable. Further information regarding the determination of the design freezing index is included in TM 5-818-2/AFM 88-6, Chapter 4 and TM 5-852-6/AFM 88-19, Volume 6.

e. Note 5. Under traffic areas, and particularly where frost condition pavement design is based on reduced subgrade strength, gradual transitions between frost-susceptible subgrade materials and nonfrost-susceptible trench backfill will be provided within the depth of frost penetration to prevent detrimental differential surface heave.

4-2.1.7 **Infiltration of fine soils through drainage pipe joints**

4-2.1.7.1 Infiltration of fine-grained soils into drainage pipelines through joint openings is one of the major causes of ineffective drainage facilities. This is a serious problem along pipes on relatively steep slopes such as those encountered with broken back culverts or stilling wells. Infiltration is not confined to non-cohesive soils. Dispersive soils have a tendency to slake and flow into drainage lines.

4-2.1.7.2 Infiltration, prevalent when the water table is at or above the pipeline, occurs in joints of rigid pipelines and in joints and seams of flexible pipe, unless these are made watertight. Watertight jointing is especially needed in culverts and storm drains placed on steep slopes to prevent infiltration and/or leakage and piping that normally results in the progressive erosion of the embankments and loss of downstream energy dissipators and pipe sections.

4-2.1.7.3 Culverts and storm drains placed on steep slopes should be large enough and properly vented so that full pipe flow can never occur, in order to maintain the hydraulic gradient above the pipe invert but below crown of the pipe, thereby reducing the tendency for infiltration of soil water through joints. Pipes on steep slopes may tend to prime and flow full periodically because of entrance or outlet condition effects until the hydraulic or pressure gradient is lowered enough to cause venting or loss of prime at either the inlet or outlet. The alternating increase and reduction of pressure relative to atmospheric pressure is considered to be a primary cause of severe piping and infiltration. A vertical riser should be provided upstream of or at the change in slope to provide sufficient venting for establishment of partial flow and stabilization of the pressure gradient in the portion of pipe on the steep slope. The riser may also be equipped with an inlet and used simultaneously to collect runoff from a berm or adjacent area.

4-2.1.7.4 Infiltration of backfill and subgrade material can be controlled by watertight flexible joint materials in rigid pipe and with watertight coupling bands in flexible pipe. Successful flexible watertight joints have been obtained in rigid pipelines with rubber gaskets installed in close-tolerance tongue- and-groove joints and factory-installed plastic gaskets installed on bell-and-spigot pipe. Bell-and-spigot joints caulked with oakum or other similar rope-type caulking materials and sealed with hot-poured joint compound have also been successful. Metal pipe seams may require welding, and the rivet heads may have to be ground to lessen interference with gaskets. There are several kinds of connecting bands which are adequate both hydraulically and structurally for joining corrugated metal pipes on steep slopes.

4-2.1.7.5 A conclusive infiltration test will be required for each section of pipeline involving watertight joints, and installation of flexible watertight joints will conform closely to manufacturers' recommendations. Although system layouts presently recommended are considered adequate, particular care should be exercised to provide a layout of subdrains that does not require water to travel appreciable distances through the base course due to impervious subgrade material or barriers. Pervious base courses with a minimum thickness of about 6 in. with provisions for drainage should be provided beneath pavements constructed on fine-grained subgrades and subject to perched water table conditions. Base courses containing more than 10 percent fines cannot be drained and remain saturated continuously.

### 4-2.2 **Inlets and Box Drains**

#### 4-2.2.1 **General**

4-2.2.1.1 Inlet structures to collect storm runoff at airfields and heliports may be built of any suitable construction material. The structures must ensure efficient drainage of design-storm runoff in order to avoid interruption of operations during or following storms and to prevent temporary or permanent damage to pavement subgrades. Most frequently, reinforced concrete is the material used although brick, concrete block, precast concrete, or rubble masonry have also been used. The material, including the slotted drain corrugated metal pipe to handle surface flow if employed, should be strong enough to withstand the loads to which it will be subjected.

4-2.2.1.2 Field inlets are usually those located away from paved areas. Box drains, normally more costly than field inlets, are usually located within paved areas to remove surface drainage.

4-2.2.1.3 Local practices and requirements governing field inlets greatly influence design and construction details. Experience has indicated that the designer should consider the features described in Section 4-2.2.2.

4-2.2.2 **Inlets versus catch basins.** Catch basins are required to prevent solids and debris from entering the drainage system; however, their proper maintenance is difficult. Unless the sediment basin is frequently cleaned, there is no need for catch basins. Since catch basins are not necessary when storm drainage lines are laid on self-cleaning grades, proper selection of storm drain gradients greatly reduce the need for catch basins. Whenever practical ordinary inlets should be used instead of catch basins.

4-2.2.3 **Design features**

4-2.2.3.1 Structures built in connection with airport drainage are similar to those used in conventional construction. Although standard type structures are usually adequate, occasionally special structures will be needed.

4-2.2.3.2 Grating elevations for field inlets must be carefully coordinated with the base or airport grading plan. Each inlet must be located at an elevation which will ensure interception of surface runoff. Increased overland velocities immediately adjacent to field inlet openings may result in erosion unless protective measures are taken. A solid sod annular ring varying from 3 to 10 ft around the inlet reduces erosion if suitable turf is established and maintained on the adjacent drainage area. Prior to the establishment of turf on the adjacent area, silt may deposit in a paved apron around the perimeter or deposit in the sod ring thereby diverting flow from the inlet. In lieu of a sod ring, a paved apron around the perimeter of a grated inlet may be beneficial in preventing erosion and differential settlement of the inlet and the adjacent area as well as facilitating mowing operations.

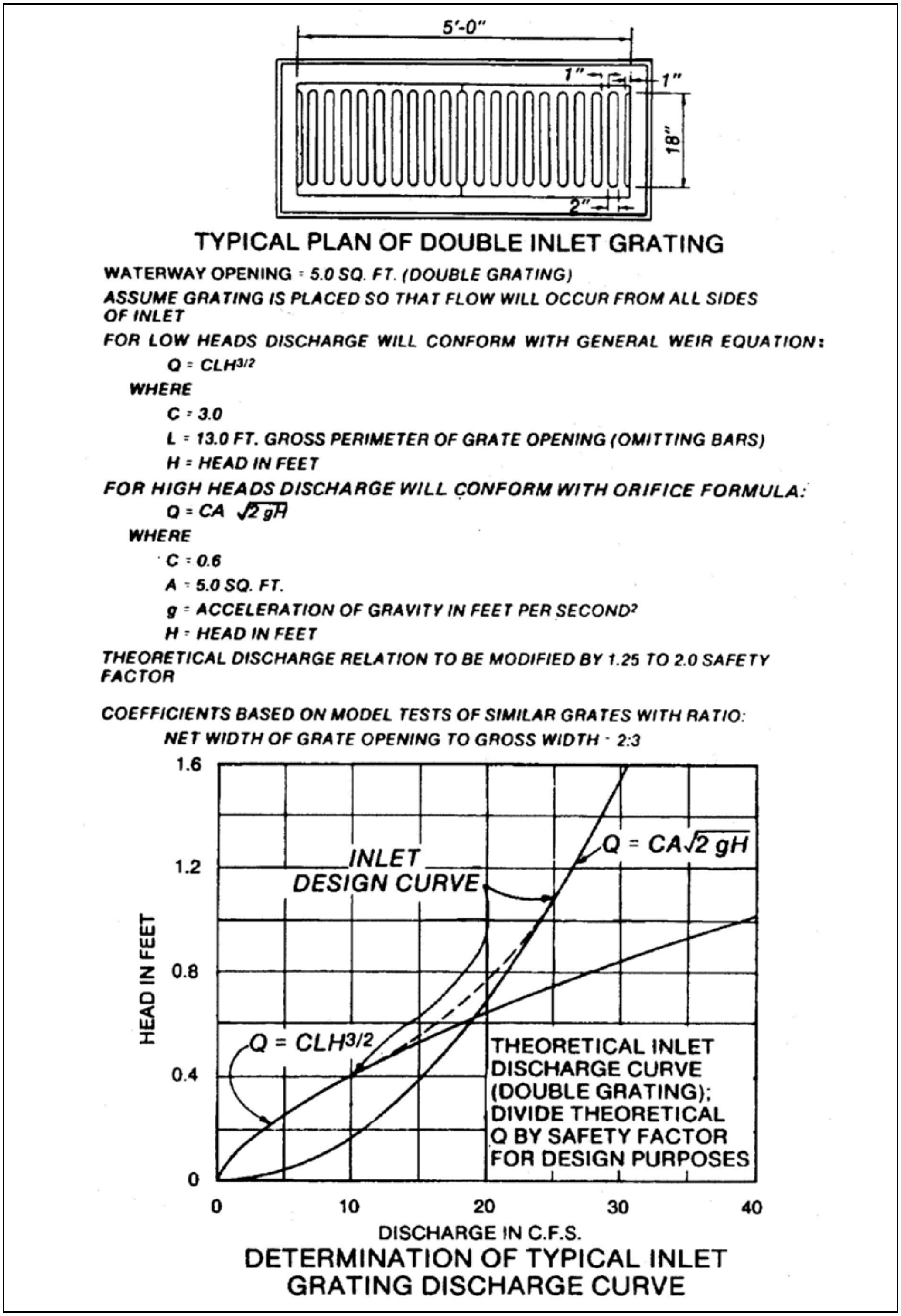
4-2.2.3.3 Drainage structures located in the usable areas on airports should be designed so that the grating does not extend above the ground level. The tops of such structures should be 0.2 of a foot below the ground line (finished grade) to allow for possible settlement around the structure, to permit unobstructed use of the area by equipment, and to facilitate collection of surface runoff.

4-2.2.3.4 A grating in a ponded area operates as a weir under low head situations. At higher heads, however, the grating acts as an orifice. Model tests of a grating shown in the typical plan of a double inlet grating (Figure 4-6) indicate that vortex action influences the discharge characteristics when the head exceeds 0.4 ft. Hydraulically acceptable grates will result if the design criteria in the above figure are applied. For the entire area, the system of grates and their individual capacity will depend on the quantity of runoff to be handled and the allowable head at the grates. Head limitations should not exceed 0.5 ft.

4-2.2.3.5 A grating in a sloping gutter will intercept all approaching the gross width of grate opening if the length of grate is greater than the upper of inflow. Grating bars will be placed parallel to the direction of gutter flow, and spacers between bars will be avoided or located below the surface of the grate. Eighteen inches is the minimum length of opening necessary for grates with a ratio of net to gross width of opening of

2:3. To prevent possible clogging by debris, the safety factors mentioned below will be applied.

**Figure 4-6. Determination of Typical Inlet Grating Discharge Curve**



4-2.2.3.6 Discharge characteristics of gratings are primarily dependent on design and the local rainfall characteristics. A safety factor of 1.5 to 2.0 will be used to compensate for collection of debris on the field gratings in turfed areas. In extensively paved areas a safety factor of 1.25 may be used in design.

4-2.2.3.7 Grates may be made of cast iron, steel, or ductile iron. Reinforced concrete grates, with circular openings, may be designed for box drains. Inlet grating and frame must be designed to withstand aircraft wheel loads of the largest aircraft using or expected to use the facility. As design loads vary, the grates should be carefully checked for load-carrying capacities. Selection of grates and frames will depend upon capacity, strength, anchoring, or the requirement for single or multiple grates. Suggested design of typical metal grates and inlets is shown in Figures 4-7 and 4-8.

4-2.2.3.8 Commercially manufactured grates and frames for airport loadings have been designed specifically for airport loadings from 50 to 250 $lb/in^2$. Hold-down devices have also been designed and are manufactured to prevent grate displacement by aircraft traffic. If manufactured grates are used, the vendor must certify the design load capacity.

4-2.2.3.9 The size and spacing of bars of grated inlets are influenced by the traffic and safety requirements of the local area. Nevertheless, in the interest of hydraulic capacity and maintenance requirements, it is desirable that the openings be made as large as traffic and safety requirements will permit.

4-2.2.3.10 For rigid concrete pavements, grates may be protected by expansion joints around the inlet frames. Construction joints, which match or are equal to the normal spacing of joints, may be required around the drainage structure. The slab around the drainage structure should include steel reinforcements to control cracking outwardly from each corner of the inlet.

4-2.2.4 **Box drains**

4-2.2.4.1 Where box drains are used within paved areas to remove surface drainage, no special inlet structures are required and a continuous-type grating, generally covering the entire drain, is used to permit entrance of water directly into the drain. Box drains are generally more costly than conventional inlets. Accordingly, their use will be restricted to unusual drainage and grade situations where flow over pavement surface must be intercepted such as near hangar doors. The design and construction details of the box drain will depend on local conditions in accordance with hydraulic and structural requirements. However, certain general details to be followed are illustrated by the typical section through a box drain in a paved area shown in Figure 4-9. The walls of the box drain will extend to the surface of the pavement. The will have a free thickened edge at the drain. An approved expansion-joint filler covering the entire surface of the thickened edge of the pavement will be installed at all joints between the pavement and box drain. A 3/4-in.-thick filler is usually sufficient, but thicker fillers may be required.

Grating for box drains can be built of steel, cast iron, or reinforced concrete with adequate strength to withstand anticipated loadings. Where two or more box drains are adjacent, they will be interconnected to provide equalization of flow and optimum hydraulic capacity.

**Figure 4-7. Examples of Typical Inlet Grates**

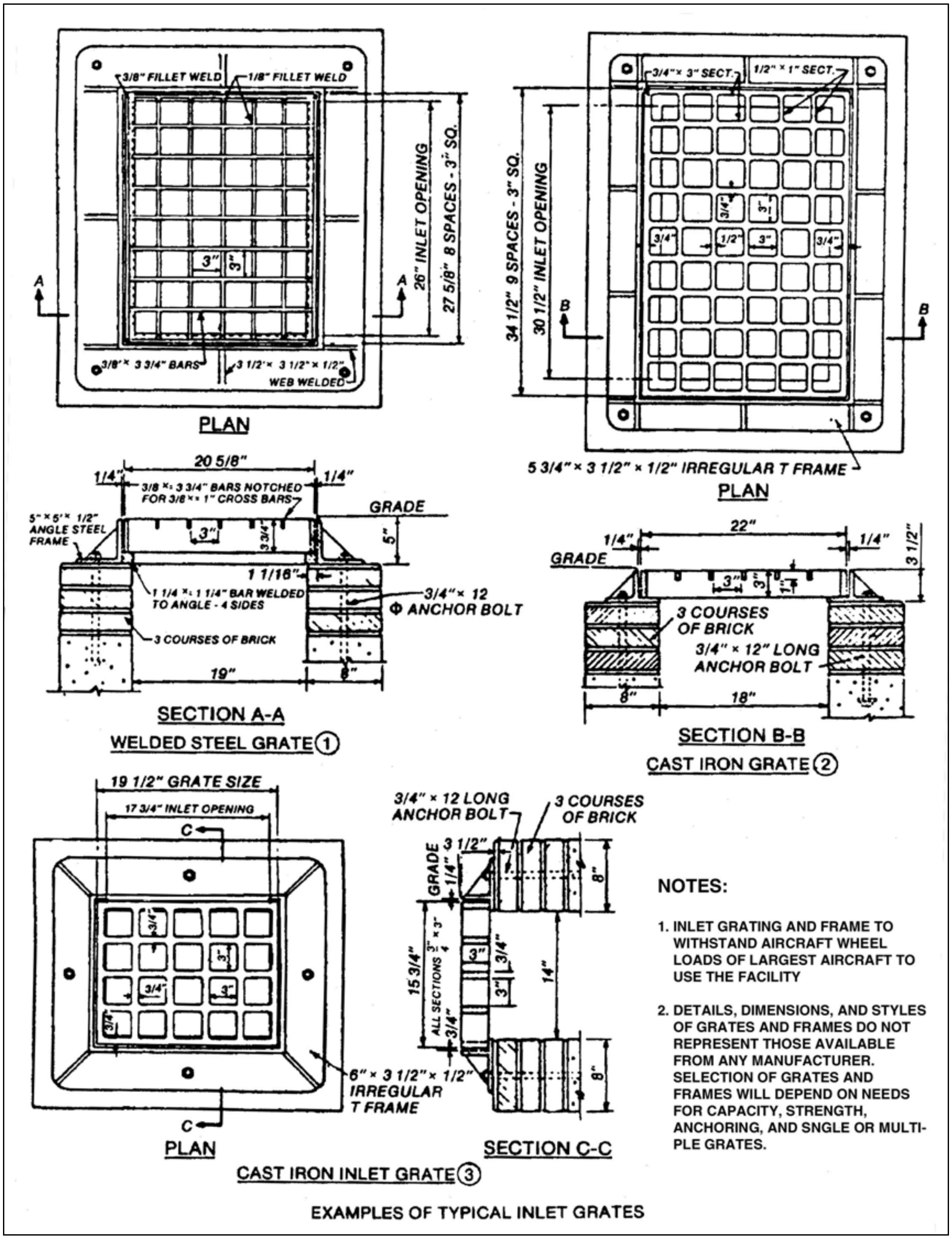
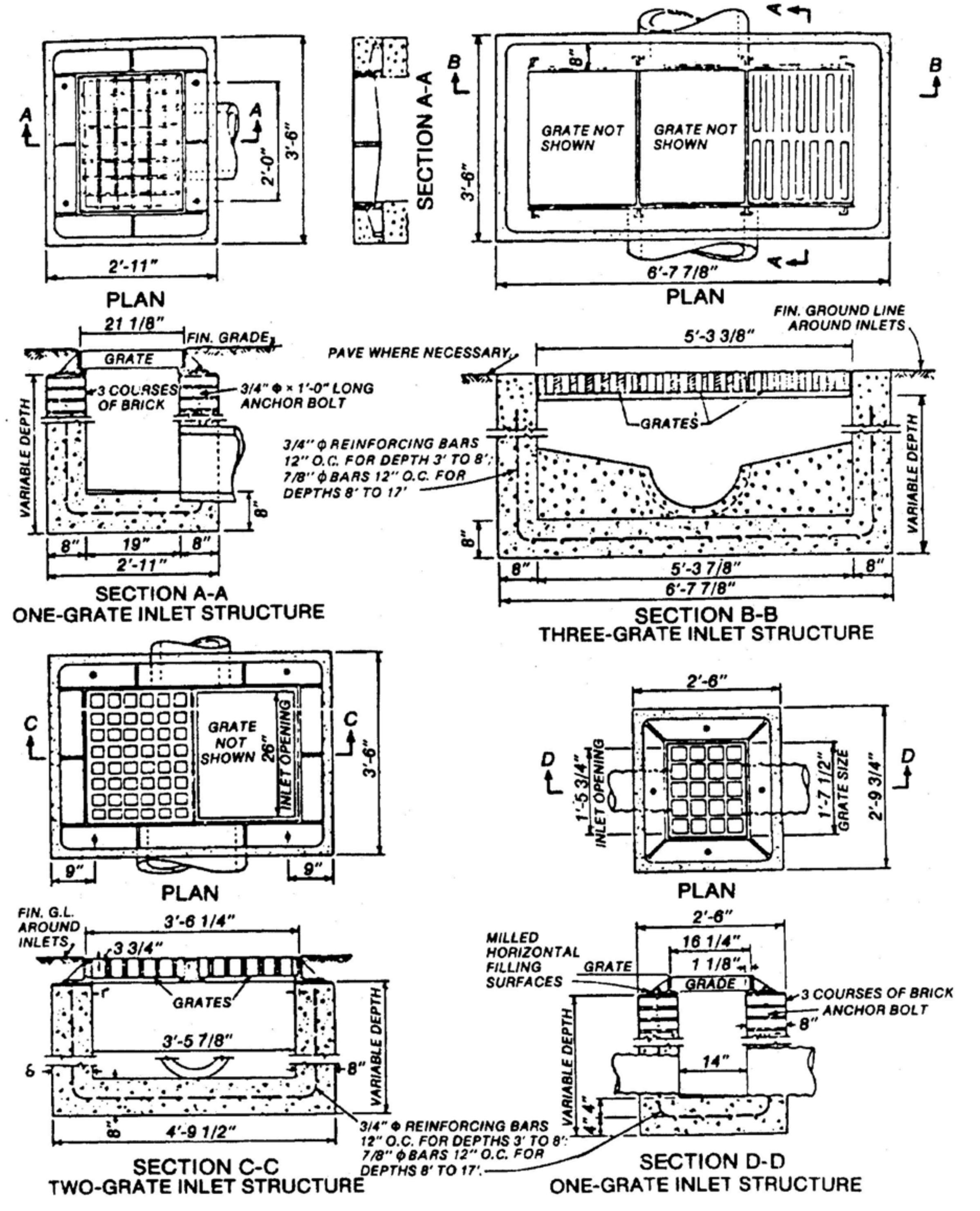
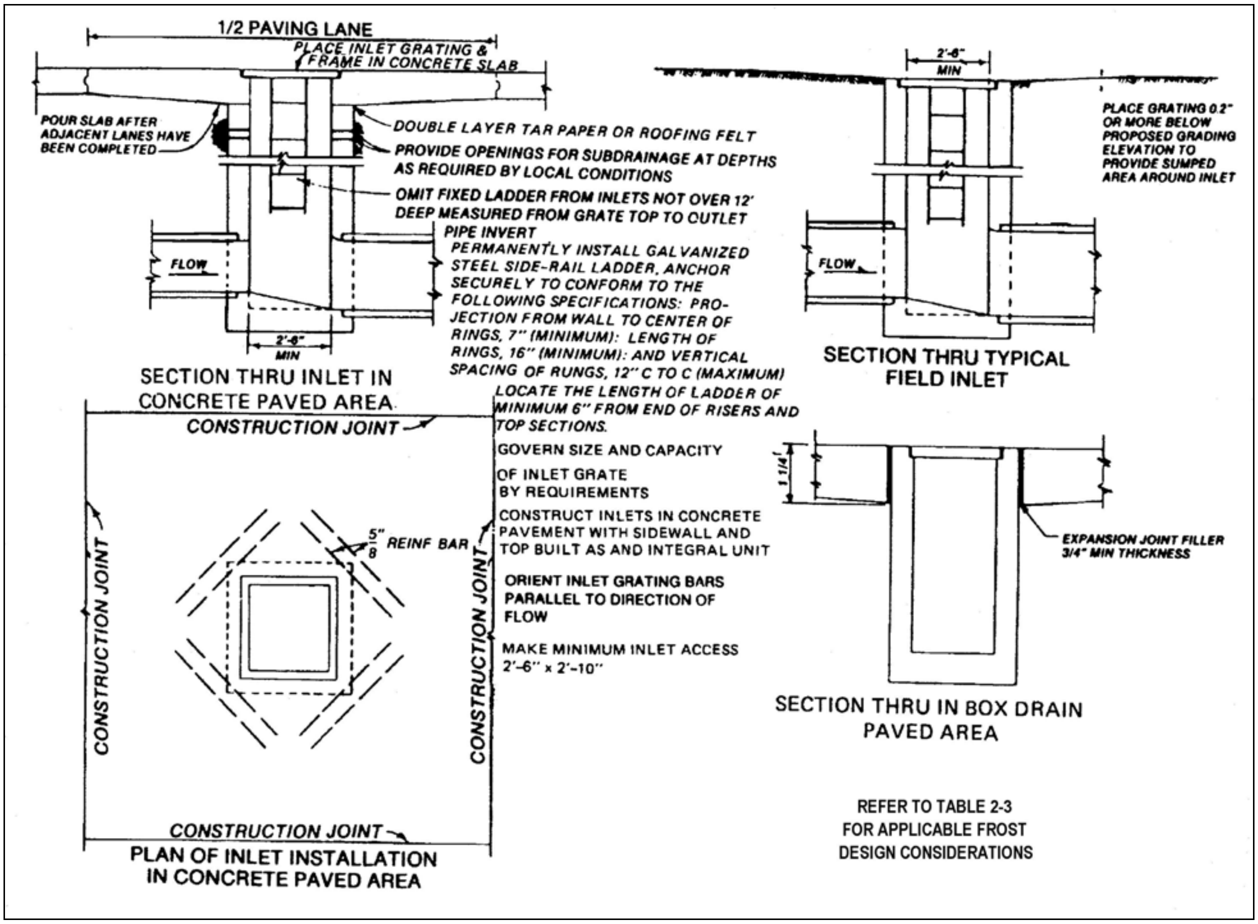


**Figure 4-8. Examples of Inlet Design**




NOTES:
DETAILS DIMENSIONS, AND MATERIALS FOR INLETS AS WELL AS FOR GRATES AND FRAMES ARE ILLUSTRATIVE ONLY.

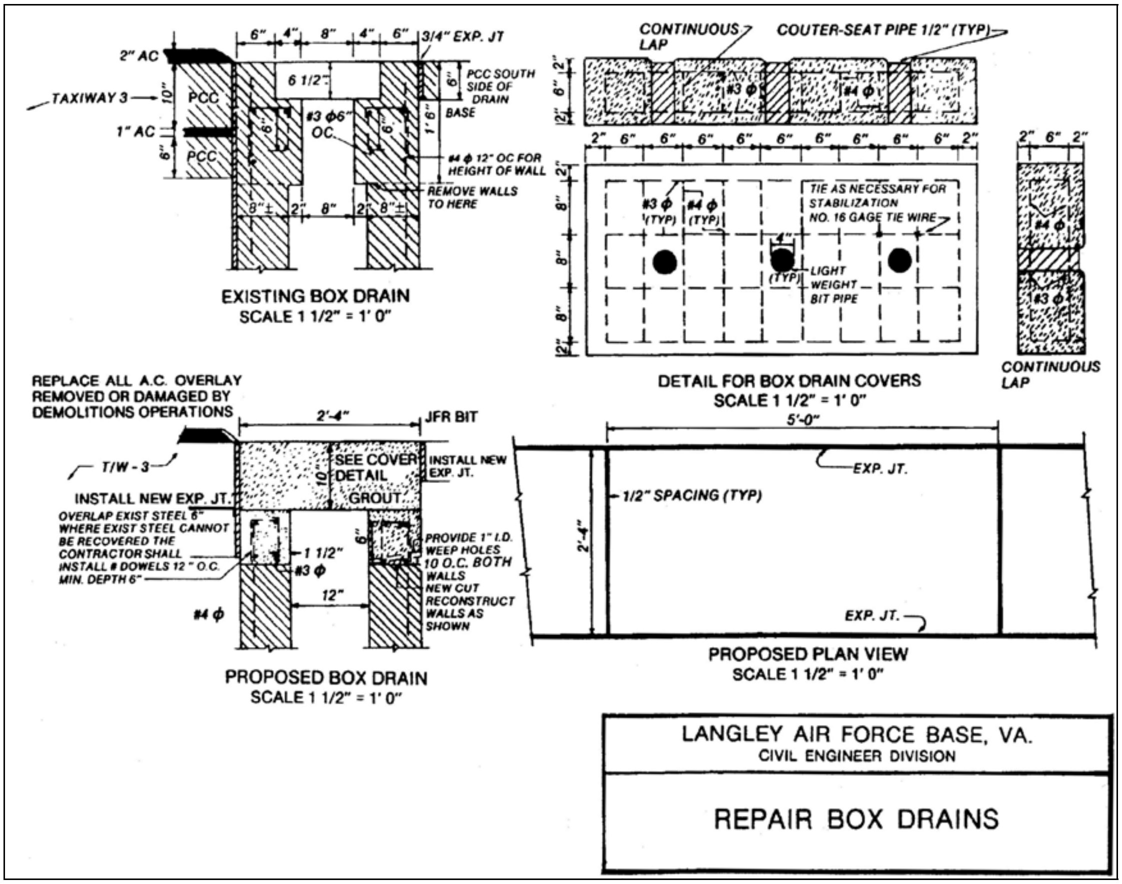
**Figure 4-9. Typical Inlet and Box Drain Designs for Airfield and Heliport Storm Drainage Systems**




4-2.2.4.2 A number of box drains similar to those shown in Figure 4-9 have failed structurally at several installations. Causes of failure are the inability of the drain walls to resist the movement of the abutting pavement under seasonal expansion and contraction, the general tendency of the slope pavement to make an expansion movement toward the drain wall while the thickened edge is restrained from moving away from the drain, and the infiltration of detritus into joints. Figure 4-10 indicates a successful box drain in use at Langley Air Force Base. The design provides for the top of the box drain wall to terminate at the bottom of the abutting pavement. A typical drain cover is a 10-in.-thick reinforced concrete slab with inserted lightweight circular pipes used for the grating openings. While only 4-in.-diameter holes have been indicated in the figure, additional holes may be used to provide egress for the storm runoff. The design may also be used to repair existing box drains which have failed.

4-2.2.4.3 Inlet drainage structures, particularly box drains have been know to settle at rates different from the adjacent pavement causing depressions which permit pavement failure should the subgrade deteriorate. Help construction specifications requiring careful backfilling around inlets will help prevent the differential settling rates.

**Figure 4-10. Repair Box Drains**




4-2.2.5 **Settlement of inlets and drains**. Failure of joints between sections of concrete pipe in the vicinity of large concrete manholes indicates the manhole has settled at a different rate than that of the connecting pipe. Flexible joints should be required for all joints between sections of rigid pipe in the vicinity of large manholes, say 3 to 5 joints along all pipe entering or leaving the manhole.

4-2.2.6 **Gutters**. In general, curb and gutters are not permitted to interrupt surface runoff along a taxiway or runway. The runoff must be allowed unimpeded travel transversely off the runway and thence directly by the shortest route across the turf to the field inlets. Inlets spaced throughout the paved apron construction must be placed at proper intervals and in well-drained depressed locations. Gutters are discussed in Section 4-2.3.

4-2.2.7 **Curb inlets**. The hydraulic efficiency of curb inlets depends upon depression of gutter invert and a relatively high curb; these conditions cannot be tolerated on airfield or heliport pavements and therefore will not be used.

4-2.2.8 **Clogging**. Partial or total restriction of open and grated inlets caused by clogging with debris, sediments, and vegetation is a fairly common problem.

4-2.2.8.1 Major factors responsible for clogging of inlets are inadequate periodic inspection, inadequate maintenance, and improper location of the inlet relative to the hydraulic gradient in the drainage system.

4-2.2.8.2 To prevent clogging of inlets serving drainage basins with characteristics and flows that contribute and transport detritus, debris barriers should be provided upstream of them.

4-2.2.9 **Ladders**. Adequate ladders should be provided to assure that rapid entrance and egress may be made by personnel during inspection of facilities. Ladder rungs should be checked periodically, since they are often lost in the course of regular inspection and maintenance work.

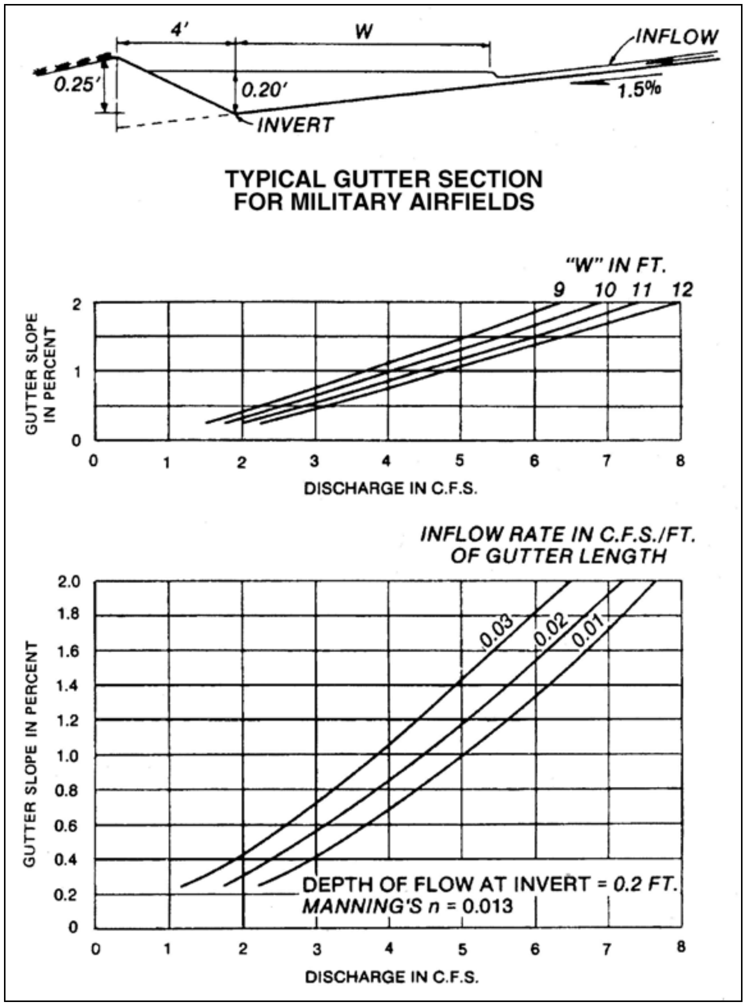
4-2.3 **Gutters**

4-2.3.1 **General**. Shallow, structurally adequate paved gutters adjacent to airfield pavements are frequently required to provide positive removal of runoff from paved areas, to protect easily eroded soils adjacent to the pavement, and to prevent the softening of turf shoulder areas caused by the large volume of runoff from adjoining pavements.

4-2.3.2 **Discharge capacity.** The discharge capacity of gutters depends on their shape, slope, and roughness. Manning's equation may be used for calculating the flow in gutters; however, the roughness coefficient n must be modified somewhat to account for the effect of lateral inflow from the runway. The net result is that the roughness coefficient for the gutter is slightly higher than that for a normal surface of the same type. The assumption of uniform flow in gutters is not strictly correct since runoff enters the gutter more or less uniformly along its length. The depth of flow and the velocity head increase downslope in the gutter, and the slope of the energy gradient is therefore flatter than the slope of the gutter. The error increases rapidly as the gutter slope is flattened, and on very flat slopes, the gutter capacity is much less than that computed using the gutter slope in Manning's equation.

4-2.3.3 **Design charts**. A cross section of a typical runway gutter and the design charts are shown in Figure 4-11. Safety and operational requirements for fast-landing speeds make it desirable to provide a continuous longitudinal grade in the gutter conforming closely to the runway gradient thereby minimizing the use of sumped inlets. A sufficient number of inlets will be provided in the gutter to prevent the depth of flow from exceeding about 2-1/2 in.

**Figure 4-11. Drainage Gutters for Runways and Aprons**




## 4-2.4 Storm Drains and Culverts

4-2.4.1 **General**. The storm-drain system should have sufficient capacity to convey runoff from the design storm within the barrel of the conduit. Hydraulic design of the storm-drain system is discussed later in this chapter. A drainage culvert is a relatively short conduit used to convey flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert hydraulics and diagrams, charts, coefficients, and related information useful in design of culverts are shown later in this chapter.

4-2.4.2 **Headwalls and endwalls**.

4-2.4.2.1 The normal functions of a headwall or wingwall are to recess the inflow or outflow end of the culvert barrel into the fill slope to improve entrance flow conditions, to anchor the pipe and to prevent disjointing caused by excessive pressures, to control erosion and scour resulting from excessive velocities and turbulences, and to prevent adjacent soil from sloughing into the waterway opening.

4-2.4.2.2 Headwalls are particularly desirable as a cutoff to prevent saturation sloughing, piping, and erosion of the embankment. Provisions for drainage should be made over the center of the head-wall to prevent scouring along the sides of the walls.

4-2.4.2.3 Whether or not a headwall is desirable depends on the expected flow conditions and embankment stability. Erosion protection such as riprap or sacked concrete with a sand-cement ratio of 9:1 may be required around the culvert entrance if a headwall is not used.

4-2.4.2.4 In the design of headwalls some degree of entrance improvement should always be considered. The most efficient entrances would incorporate one or more of such geometric features as elliptical arcs, circular arcs, tapers, and parabolic drop-down curves. Elaborate inlet design for a culvert would be justifiable only in unusual circumstances. The rounding or beveling of the entrance in almost any way will increase the culvert capacity for every design condition. These types of improvements provide a reduction in the loss of energy at the entrance for little or no additional cost.

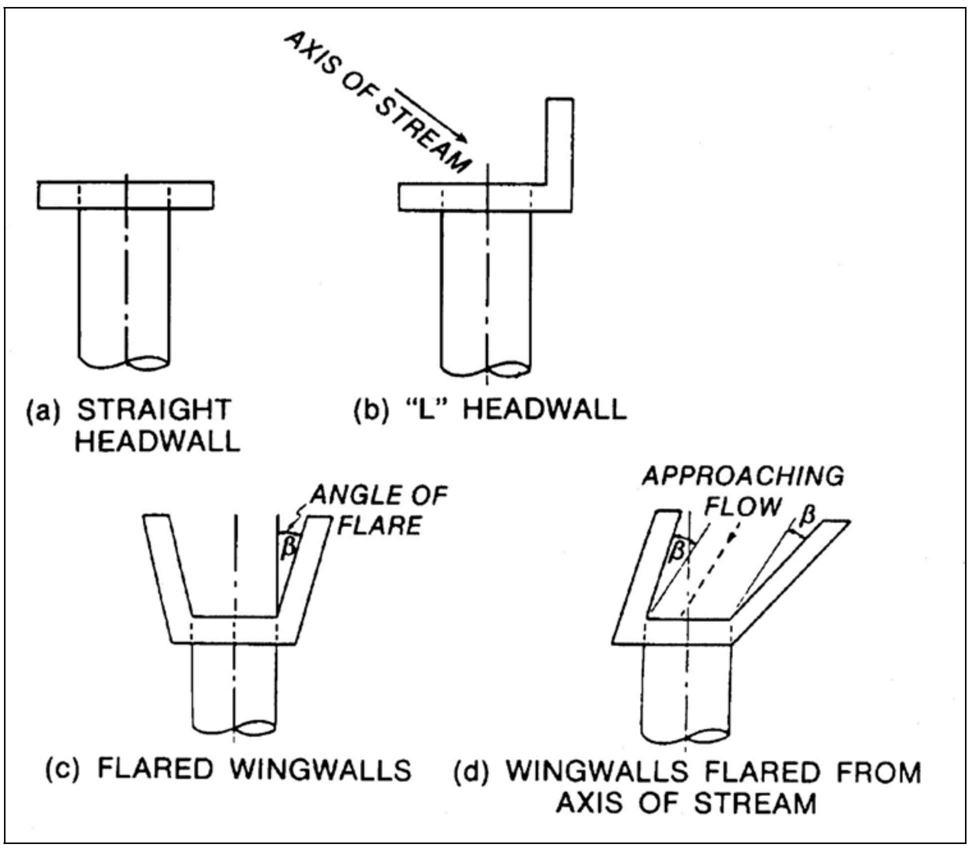
4-2.4.2.5 Entrance structures (headwalls and wingwalls) protect the embankment from erosion and, if properly designed, may improve the hydraulic characteristics of the culvert. The height of these structures should be kept to the minimum that is consistent with hydraulic, geometric, and structural requirements. Several entrance structures are shown in Figure 4-12. Straight headwalls (Figure 4-12a) are used for low to moderate approach velocity, light drift (small floating debris), broad or undefined approach channels, or small defined channels entering culverts with little change in alignment. The "L" headwall (Figure 4-12b) is used if an abrupt change in flow direction is necessary with low to moderate velocities. Winged headwalls (Figure 4-12c) are used for channels with moderate velocity and medium floating debris. Wingwalls are most effective when set flush with the edges of the culvert barrel, aligned with stream axis (Figure 4-12d) and placed at a flare angle of 18 to 45 degrees. Warped wingwalls (not shown) are used for well-defined channels with high-velocity flow and a free water surface. They are used primarily with box culverts. Warped headwalls are hydraulically efficient because they form a gradual transition from a trapezoidal channel to the barrel. The use of a drop-down apron in conjunction with these wingwalls may be particularly advantageous.

4-2.4.2.6 Headwalls are normally constructed of plain or reinforced concrete or of masonry and usually consist of either a straight headwall or a headwall with wingwalls, apron, and cutoff wall, as required by local conditions. Definite design criteria applicable to all conditions cannot be formulated, but the following comments highlight features which require careful consideration to ensure an efficient headwall structure.

a. Most culverts outfall into a waterway of relatively large cross section; only moderate tailwater is present, and except for local acceleration, if the culvert effluent freely drops, the downstream velocities gradually diminish. In such situations, the primary problem is not one of hydraulics but is usually the protection of the outfall against undermining bottom scour, damaging lateral erosion, and perhaps degrading the downstream channel. The presence of tailwater higher than the culvert crown will affect the culvert performance and may possibly require protection of the adjacent embankment against wave or eddy scour. In any event, a determination must be made about downstream control, its relative permanence, and tailwater conditions likely to result. Endwalls (outfall headwalls) and wingwalls will not be used unless justifiable as an integral part of outfall energy dissipators or erosion protection works, or for reasons such as right-of-way restrictions and occasionally aesthetics.

b. The system will fail if there is inadequate endwall protection. Normally the end sections may be damaged first, thus causing flow obstruction and progressive undercutting during high runoff periods which will cause washout of the structure. For corrugated metal (pipe or arch) culvert installations, the use of prefabricated end sections may prove desirable and economically feasible. When a metal culvert outfall projects from an embankment fill at a substantial height above natural ground, either a cantilevered free outfall pipe or a pipe downspout will probably be required. In either case the need for additional erosion protection requires consideration.

**Figure 4-12. Culvert Headwalls and Wingwalls**



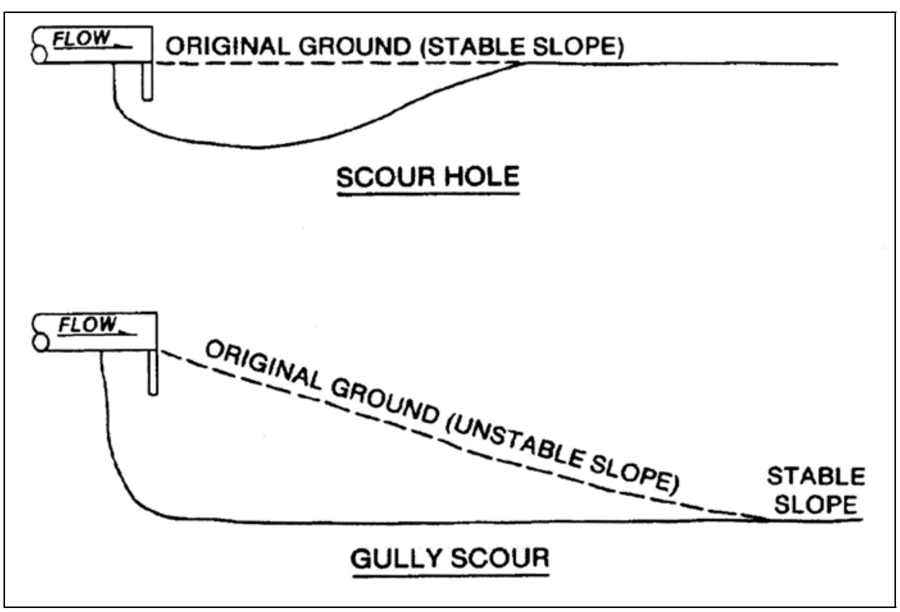
4-2.4.2.7 Headwalls and endwalls incorporating various designs of energy dissipators, flared transitions, and erosion protection for culvert outfalls are discussed in detail in subsequent sections of this chapter.

4-2.4.2.8 Headwalls or endwalls will be adequate to withstand soil and hydrostatic pressures. In areas of seasonal freezing the structure will also be designed to preclude detrimental heave or lateral displacement caused by frost action. The most satisfactory method of preventing such damage is to restrict frost penetration beneath and behind the wall to nonfrost-susceptible materials. Positive drainage behind the wall is also essential. Foundation requirements will be determined in accordance with procedures outlined in Section 4-2.1.6.4. Criteria for determining the depth of backfill behind walls are given in TM 5-818-1.

4-2.4.2.9 The headwalls or endwalls will be large enough to preclude the partial or complete stoppage of the drain by sloughing of the adjacent soil. This can best be accomplished by a straight headwall or by wingwalls. Typical erosion problems result from uncontrolled local inflow around the endwalls. The recommended preventive for this type of failure is the construction of a berm behind the endwall (outfall headwall) to intercept local inflow and direct it properly to protected outlets such as field inlets and paved or sodded chutes that will conduct the water into the outfall channel. The proper use of solid sodding will often provide adequate headwall and channel protection.

4-2.4.3 **Scour at outlets**. In general, two types of channel instability can develop downstream from storm sewer and culvert outlets, i.e., either gully scour or localized erosion termed a scour hole. Distinction between the two conditions can be made by comparing the original or existing slope of the channel or drainage basin downstream of the outlet relative to that required for stability as illustrated in Figure 4-13.

**Figure 4-13. Types of Scour at Storm-Drain and Culvert Outlets**



4-2.4.3.1 Gully scour is to be expected when the Froude number of flow in the channel exceeds that required for stability. It begins at a control point downstream where the channel is stable and progresses upstream. If sufficient differential in elevation exists between the outlet and the section of stable channel, the outlet structure will be completely undermined. The primary cause of gully scour is the practice of siting outlets high, with or without energy dissipators relative to a stable downstream grade in order to reduce quantities of pipe and excavation. Erosion of this type may be extensive, depending upon the location of the stable channel section relative to that of the outlet in both the vertical and downstream directions. To prevent gully erosion, outlets and energy dissipators should be located at sites where the slope of the downstream channel or drainage basin is naturally moderate enough to remain stable under the anticipated conditions or else it should be controlled by ditch checks, drop structures, and/or other means to a point where a naturally stable slope and cross section exist. Design of stable open channels is discussed later in this manual.

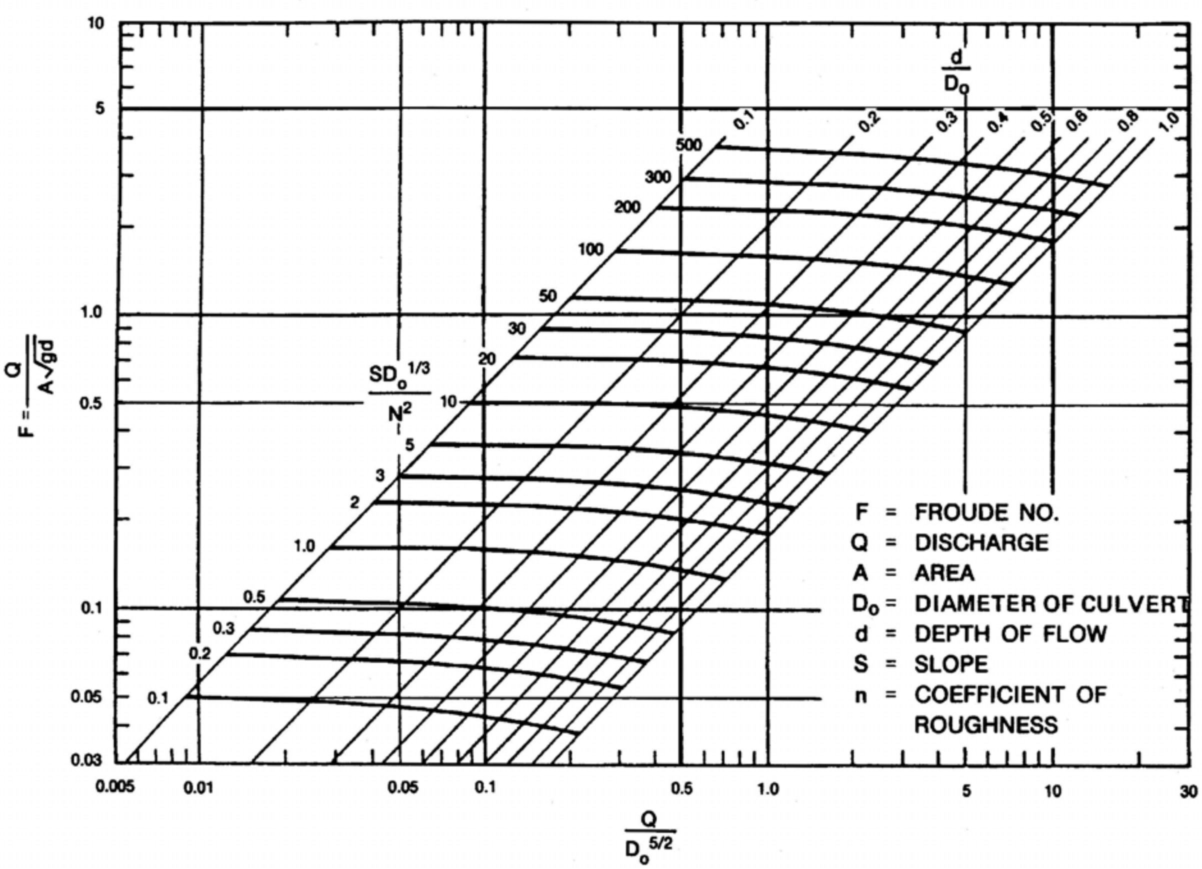
4-2.4.3.2 A scour hole or localized erosion can occur downstream of an outlet even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. In many situations, flow conditions can produce scour resulting in embankment erosion as well as structural damage to the apron, endwall, and culvert.

4-2.4.3.3 Empirical equations have been developed for estimating the extent of the anticipated scour hole in sand, based on knowledge of the design discharge, the culvert diameter, and the duration and Froude number of the design flow at the culvert outlet. However, the relationship between the Froude number of flow at the culvert outlet and a discharge parameter, or $Q/D_o^{5/2}$, can be calculated for any shape of outlet, and this discharge parameter is just as representative of flow conditions as is the Froude number. The relationship between the two parameters, for partial and full pipe flow in square culverts, is shown in Figure 4-14. Terms are defined in Section 4-2.8. Since the discharge parameter is easier to calculate and is suitable for application purposes, the original data were reanalyzed in terms of discharge parameter for estimating the extent of localized scour to be anticipated downstream of culvert and storm drain outlets. The equations for the maximum depth, width, length, and volume of scour and comparisons of predicted and observed values are shown in Figures 4-15 through 4-18. Minimum and maximum tailwater depths are defined as those less than $0.5D_o$ and equal to or greater than $0.5D_o$, respectively. Dimensionless profiles along the center lines of the scour holes to be anticipated with minimum and maximum tailwaters are presented in Figures 4-19 and 4-20. Dimensionless cross sections of the scour hole at a distance of 0.4 of the maximum length of scour downstream of the culvert outlet for all tailwater conditions are also shown in Figures 4-19 and 4-20.

4-2.4.4 **Erosion control at outlet**. There are various methods of preventing scour and erosion at outlets and protecting the structure from undermining. Some of these methods will be discussed in subsequent paragraphs.

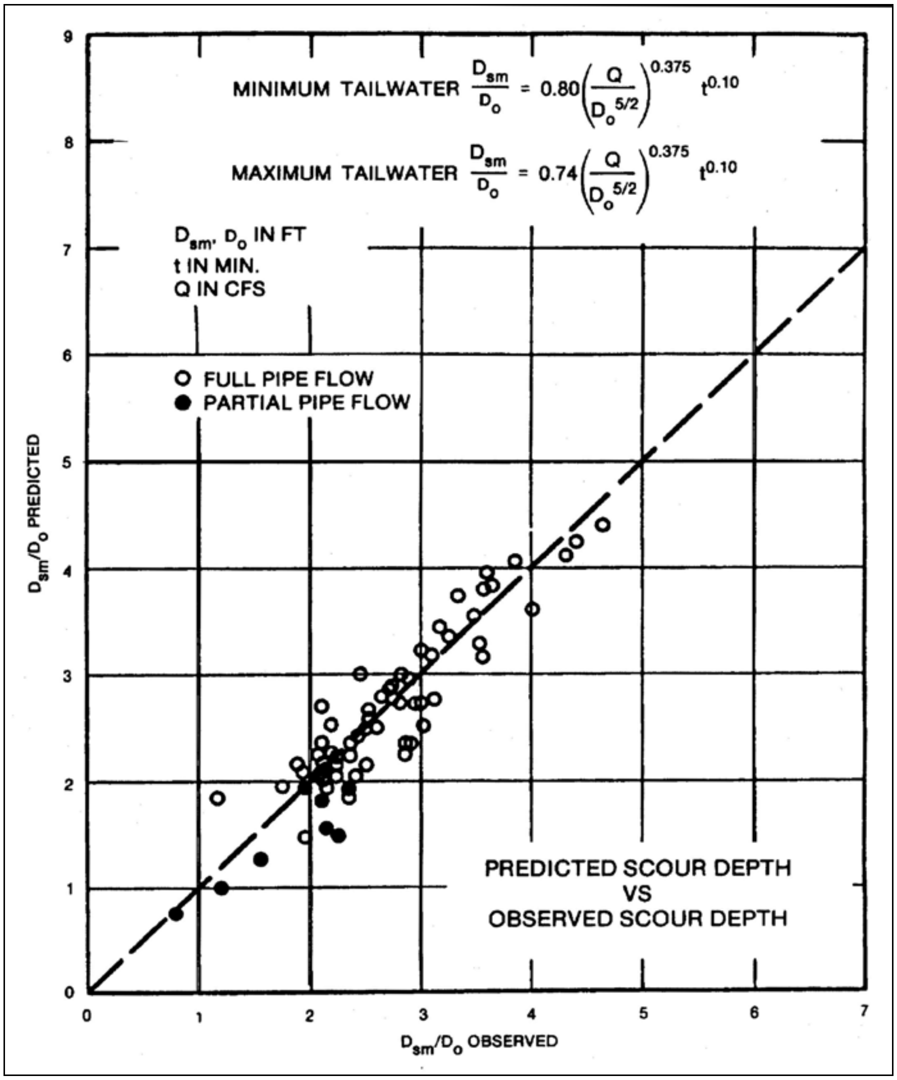
4-2.4.4.1 In some situations placement of riprap at the end of the outlet may be sufficient to protect the structure. The average size of stone ($d_{50}$) and configuration of a horizontal blanket of riprap at outlet invert elevation required to control or prevent localized scour downstream of an outlet can be estimated using the information in Figures 4-21 to 4-23. For a given design discharge, culvert dimensions, and tailwater depth relative to the outlet invert, the minimum average size of stone ($d_{50}$) for a horizontal blanket of protection can be determined using data in Figure 4-21. The length of stone protection (LSP) can be determined by the relations shown in Figure 4-22. The variables are defined in Section 4-2.8 of this chapter and the recommended configuration of the blanket is shown in Figure 4-23.

**Figure 4-14. Square Culvert-Froude Number**




4-2.4.4.2 The relative advantage of providing both vertical and lateral expansion downstream of an outlet to permit dissipation of excess kinetic energy in turbulence, rather than direct attack of the boundaries, is shown in Figure 4-21. Figure 4-21 indicates that the required size of stone may be reduced considerably if a riprap-lined, preformed scour hole is provided, instead of a horizontal blanket at an elevation essentially the same as the outlet invert. Details of a scheme of riprap protection termed "performed scour hole lined with riprap" are shown in Figure 4-24.

**Figure 4-15. Predicted Scour Depth Versus Observed Scour Depth**




4-2.4.4.3 Three ways in which riprap can fail are movement of the individual stones by a combination of velocity and turbulence, movement of the natural bed material through the riprap resulting in slumping of the blanket, and undercutting and raveling of the riprap by scour at the end of the blanket. Therefore, in design, consideration must be given to selection of an adequate size stone, use of an adequately graded riprap or provision of a filter blanket, and proper treatment of the end of the blanket.

4-2.4.4.4 Expanding and lining the channel downstream from a square or rectangular outlet for erosion control can be with either sack revetment or cellular blocks as well as rock riprap, as placed shown in Figure 4-25. The conditions of discharge and tailwater required to displace sack revetment with length, width, and thickness of 2, 1.5, and 0.33 ft, respectively (weight 120 lb); cellular blocks, 0.66 by 0.66 ft and 0.33 ft thick (weight 14 lb); or riprap with a given thickness are shown in Figure 4-26. The effectiveness of the lined channel expansion relative to the other schemes of riprap protection described previously is shown in Figure 4-21.

**Figure 4-16. Predicted Scour Width Versus Observed Scour Width**

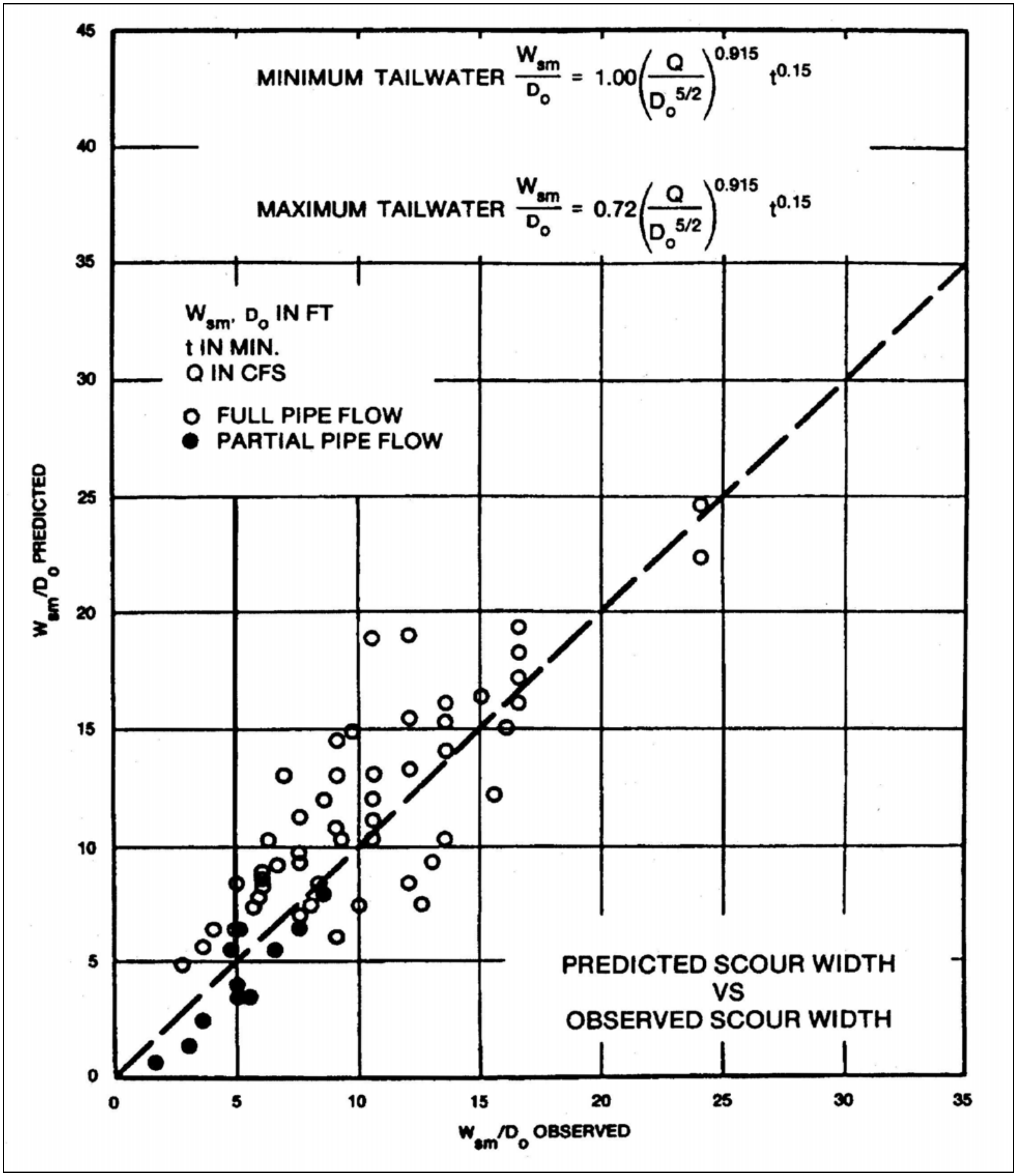


**Figure 4-17. Predicted Scour Length Versus Observed Scour Length**

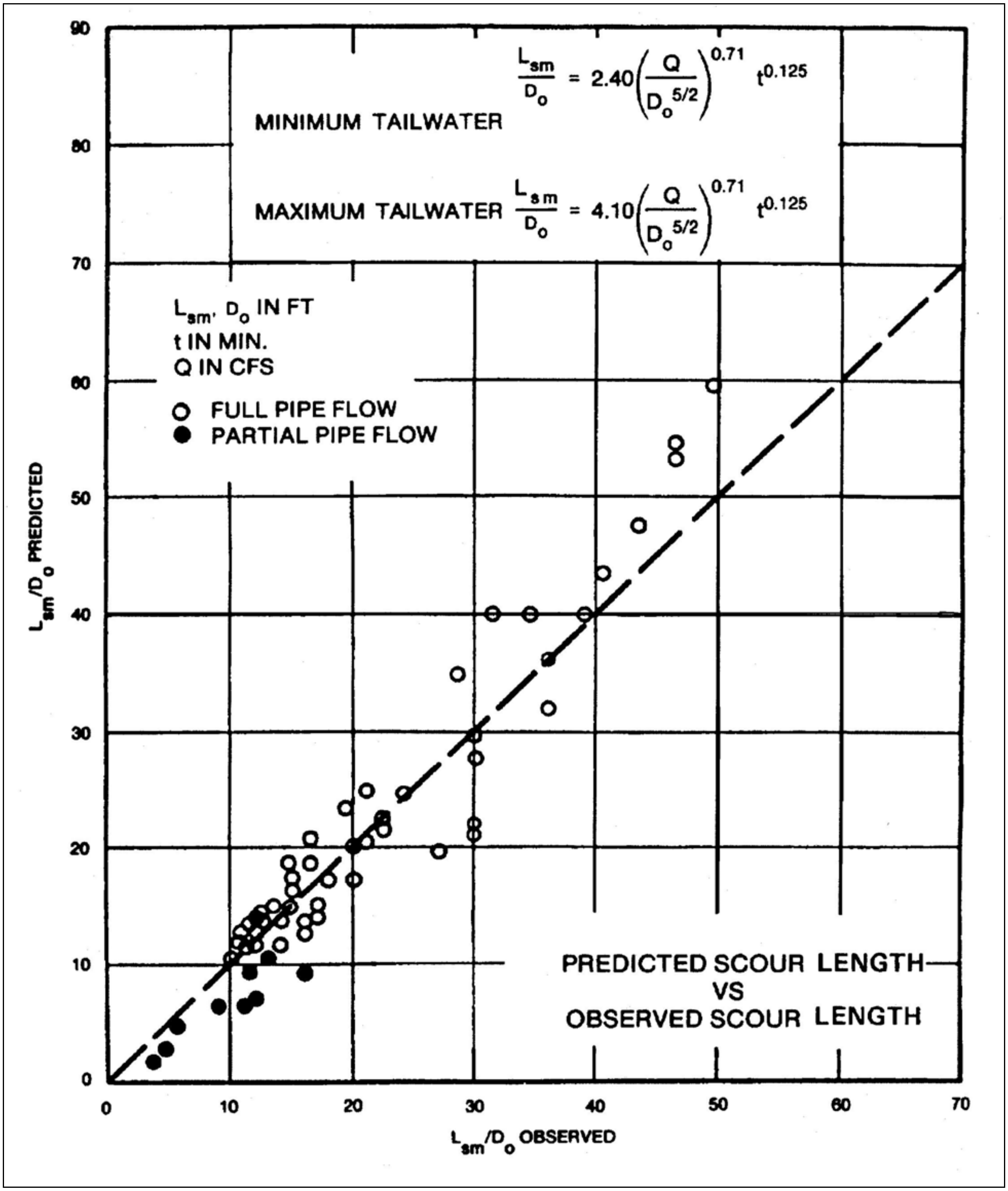


**Figure 4-18. Predicted Scour Volume Versus Observed Scour Volume**

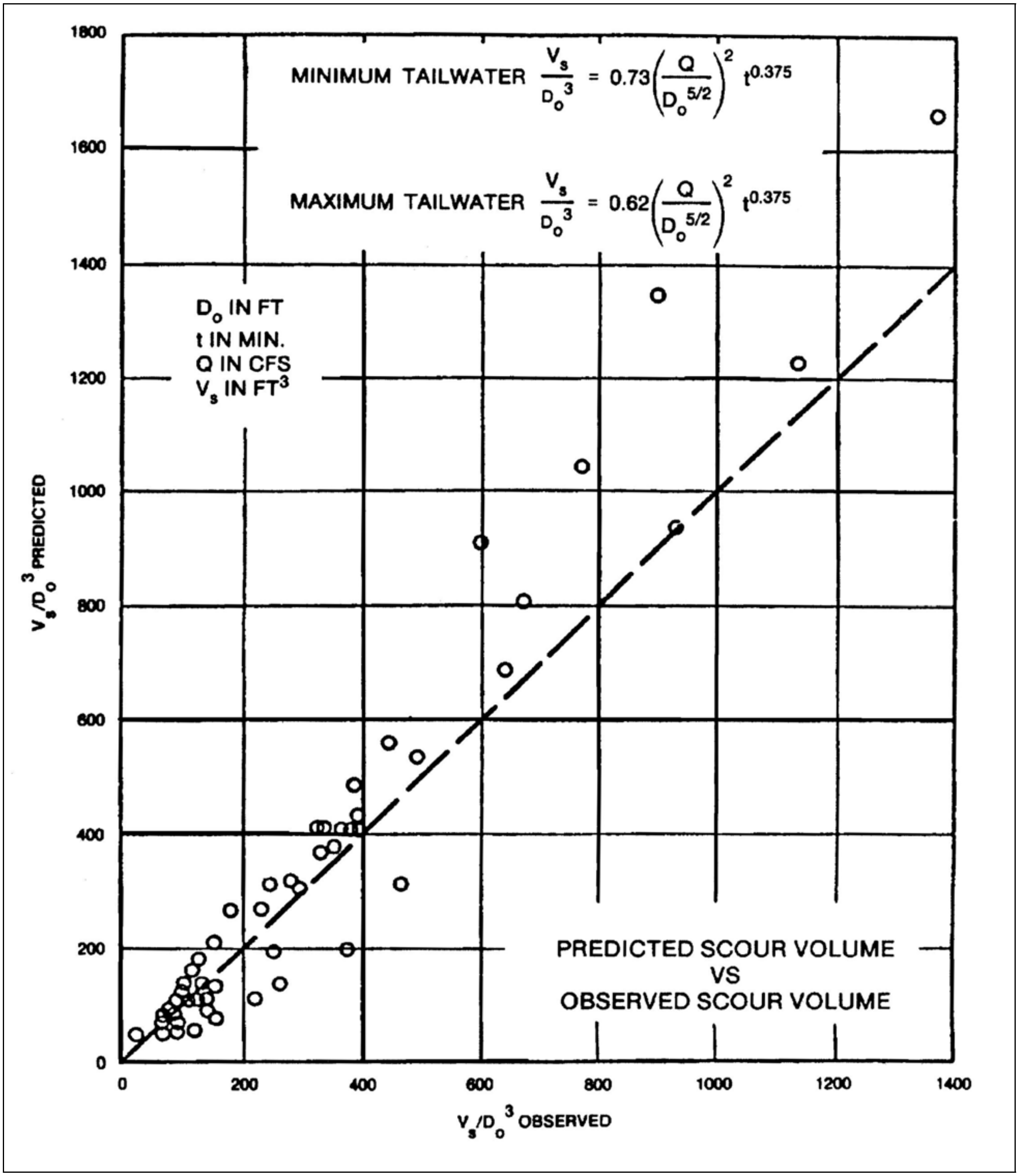


**Figure 4-19. Dimensionless Scour Hole Geometry for Minimum Tailwater**

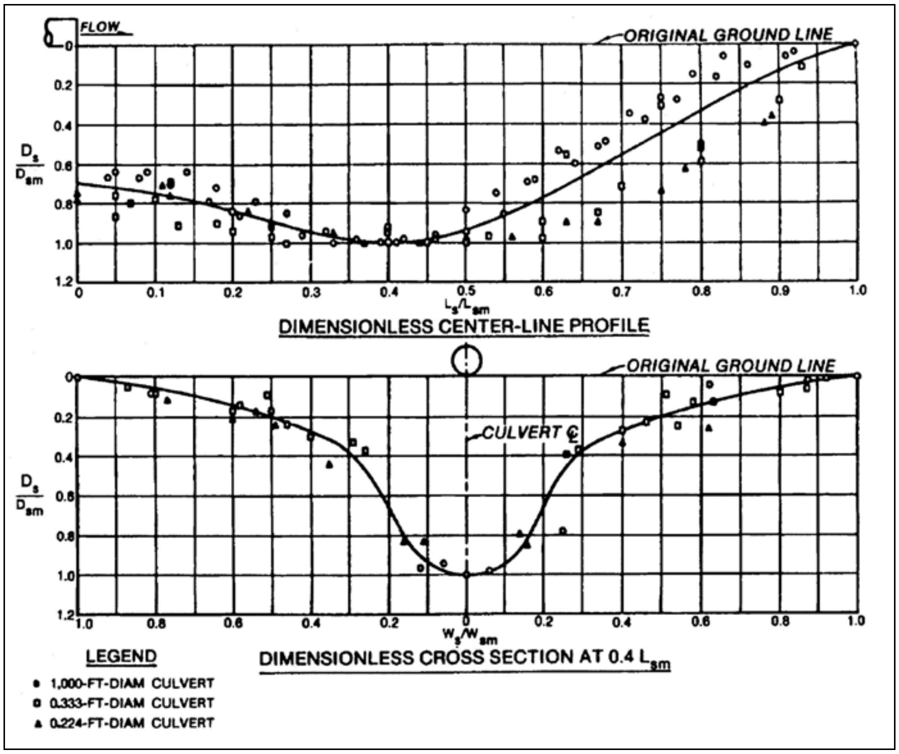



**Figure 4-20. Dimensionless Scour Hole Geometry for Maximum Tailwater**

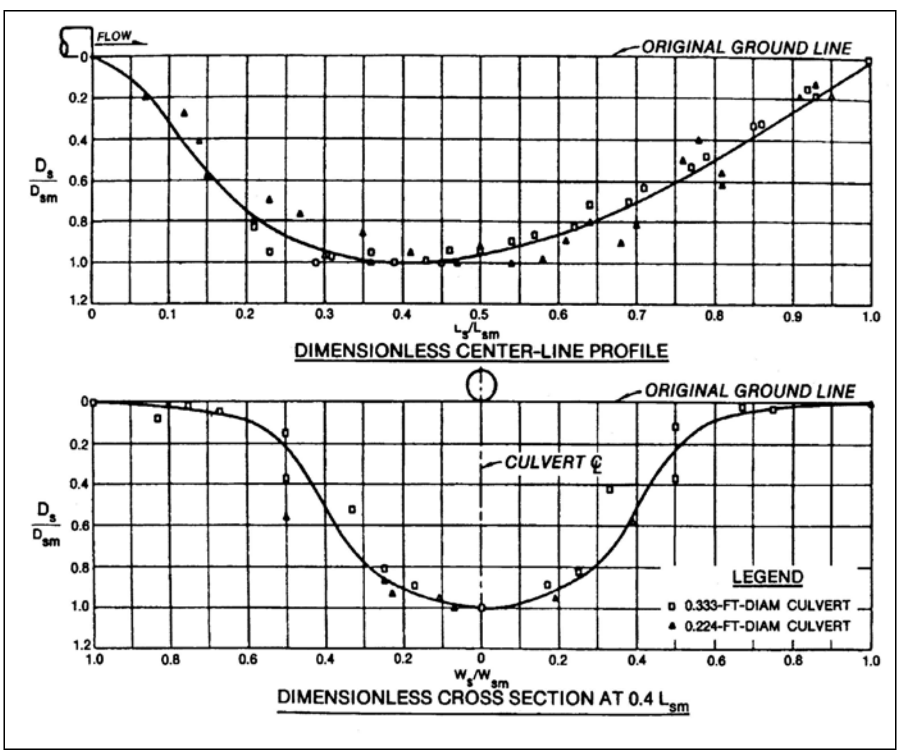
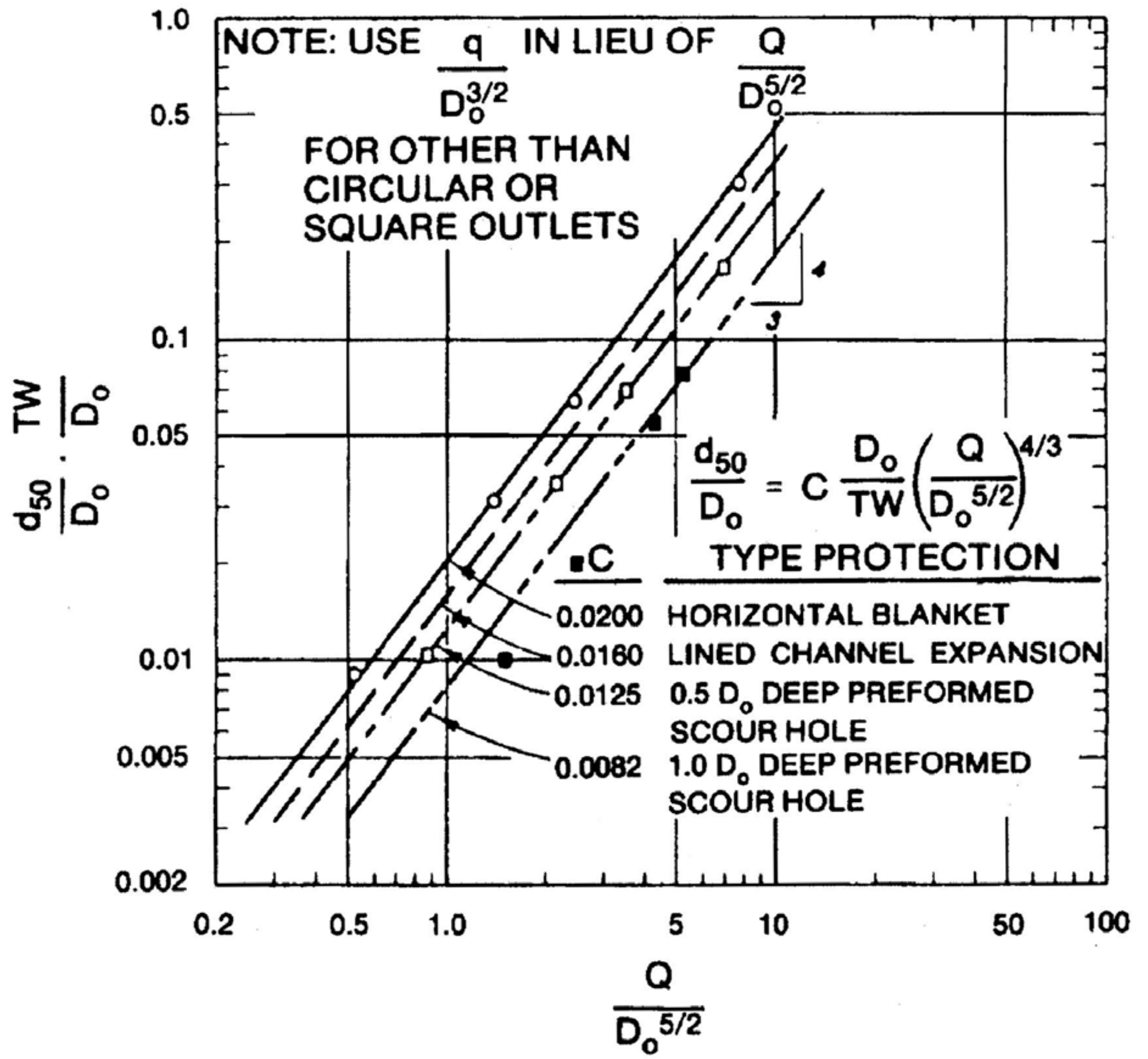


**Figure 4-21. Recommended Size of Protective Stone**




4-2.4.4.5 The maximum discharge parameters, $Q/D_o^{5/2}$ or $q/D_o^{3/2}$, of various schemes of protection can be calculated based on the above information; comparisons relative to the cost of each type of protection can then be made to determine the most practical design for providing effective drainage and erosion control facilities for a given site. There will be conditions where the design discharge and economical size of conduit will result in a value of the discharge parameter greater than the maximum value permissible thus requiring some form of energy dissipator.

**Figure 4-22. Length of Stone Protection, Horizontal Blanket**

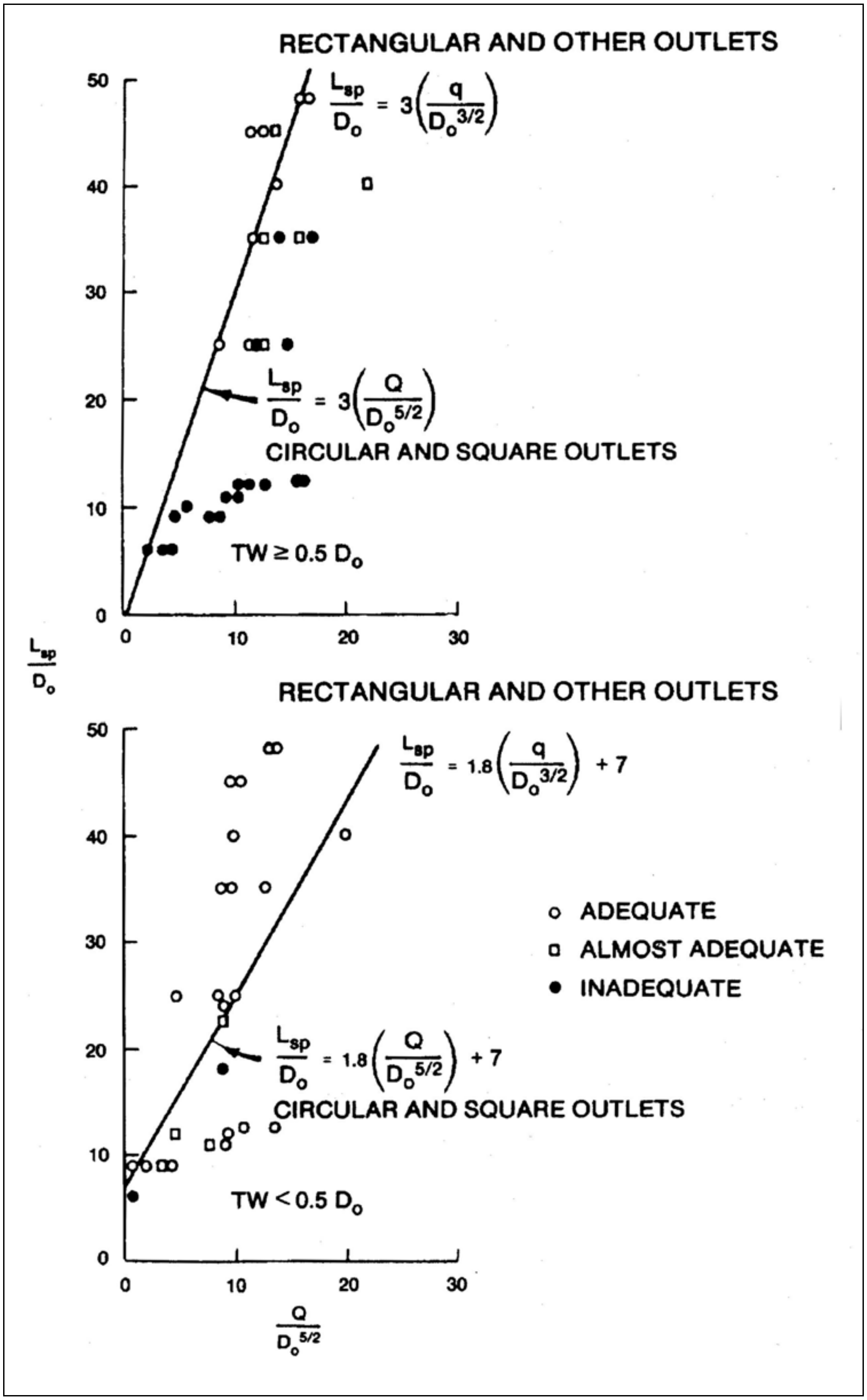


**Figure 4-23. Recommended Configuration of Riprap Blanket Subject to Minimum and Maximum Tailwaters**

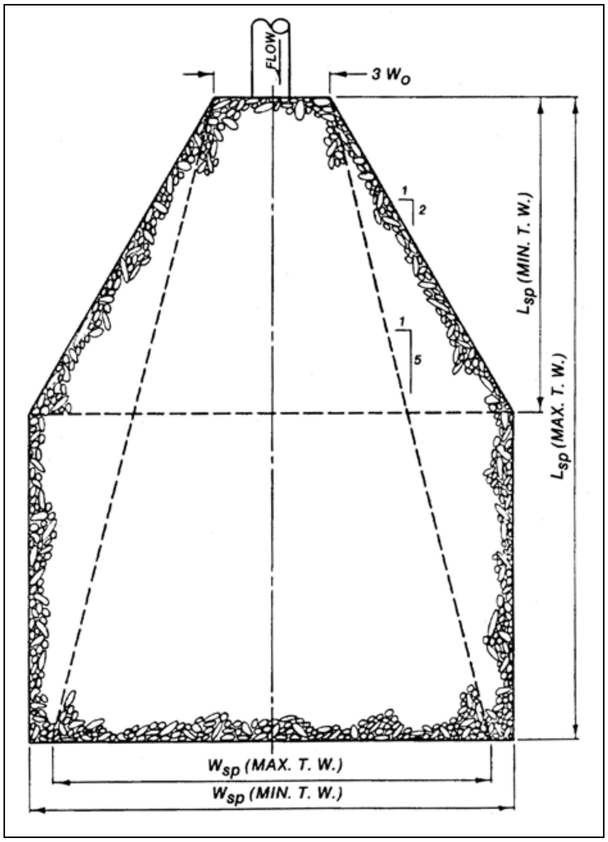



**Figure 4-24. Preformed Scour Hole**

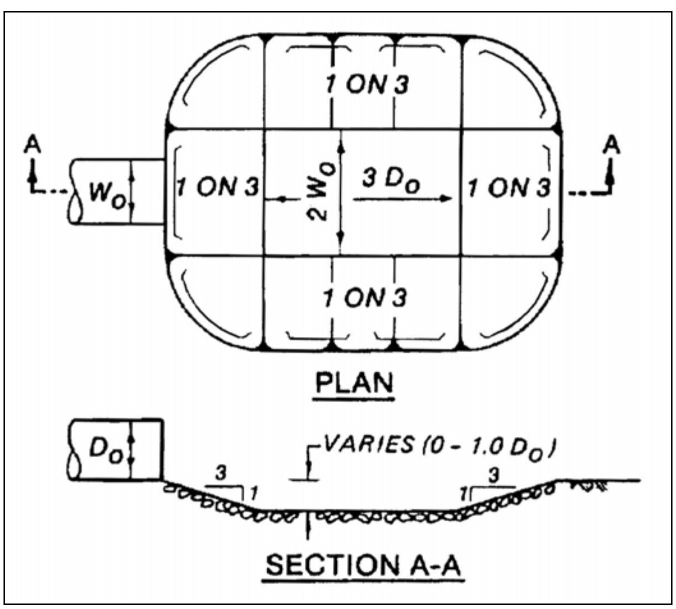


**Figure 4-25. Culvert Outlet Erosion Protection, Lined Channel Expansion**

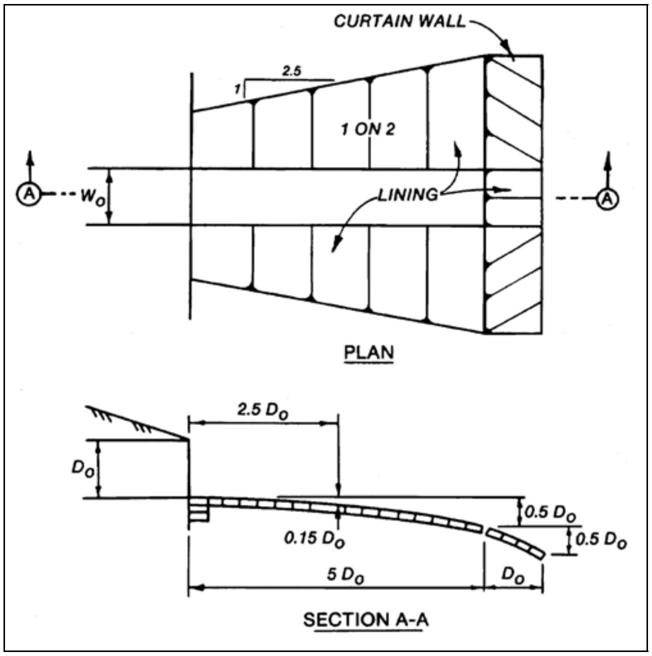
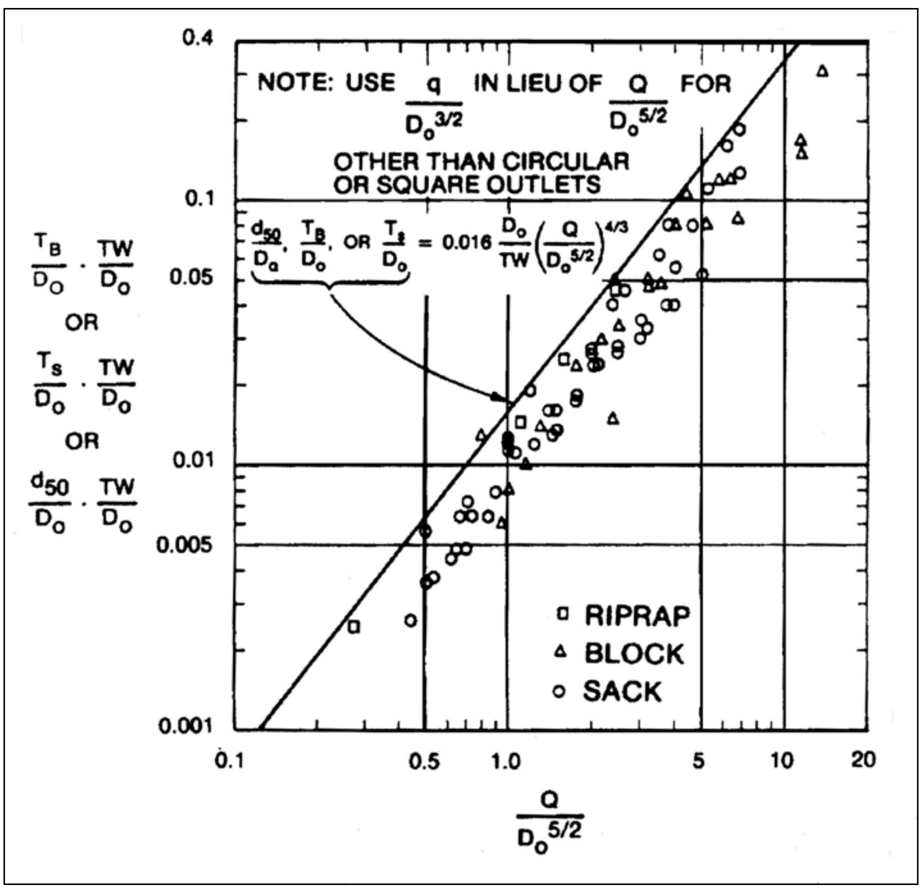



**Figure 4-26. Maximum Permissible Discharge for Lined Channel Expansions**



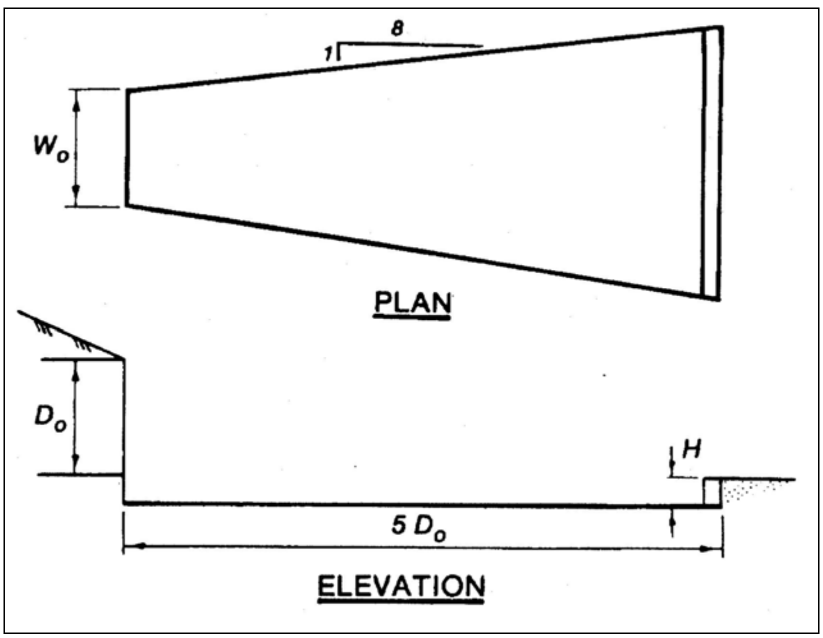
4-2.4.4.6 The simplest form of energy dissipator is the flared outlet transition. Protection is provided to the local area covered by the apron, and a portion of the kinetic energy of flow is reduced or converted to potential energy by hydraulic resistance provided by the apron. A typical flared outlet transition is shown in Figure 4-27. The flare angle of the walls should be 1 on 8. The length of transition needed for a given discharge conduit size and tailwater situation with the apron at the same elevation as the outlet invert (H = 0) can be calculated by the following equations.

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_0^{5/2}}\right)^{2.5(TW/D_o)^{1/3}} \quad \text{Circular and square outlets} \qquad \text{(eq. 4-1)}$$

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{q}{D_0^{3/2}}\right)^{2.5(TW/D_o)^{1/3}} \quad \text{Rectangular and other shaped outlets} \qquad \text{(eq. 4-2)}$$

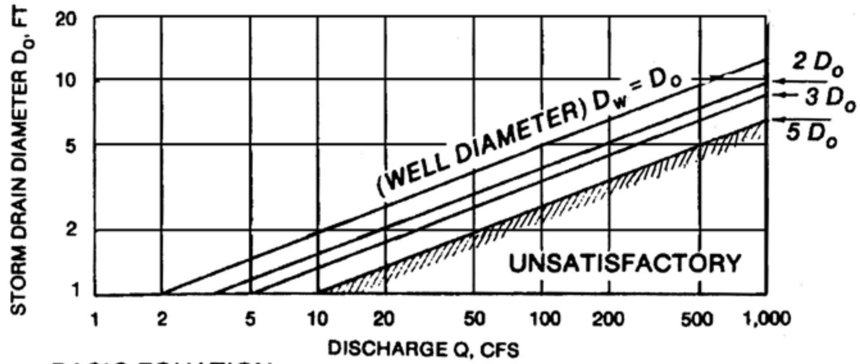
Recessing the apron and providing an end sill will not significantly improve energy dissipation.

**Figure 4-27. Flared Outlet Transition**




4-2.4.4.7 The flared transition is satisfactory only for low values of $Q/D_o^{5/2}$ or $q/D_o^{3/2}$ as will be found at culvert outlets. With higher values, however, as will be experienced at storm drain outlets, other types of energy dissipators will be required. Design criteria for three types of laboratory tested energy dissipators are presented in Figures 4-28 to 4-30. Each type has advantages and limitations. Selection of the optimum type and size is dependent upon local tailwater conditions, maximum expected discharge, and economic considerations.

**Figure 4-28. Stilling Well**

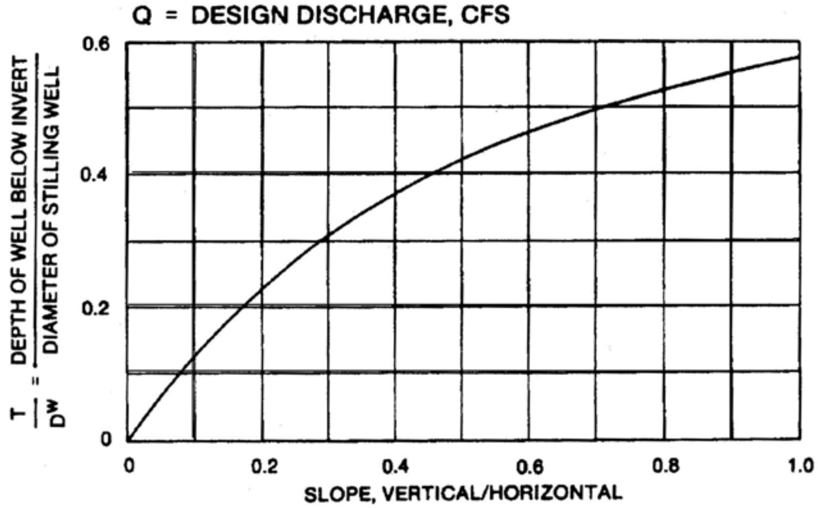


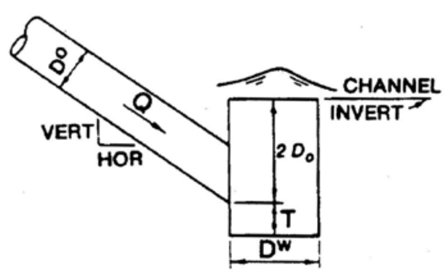

**BASIC EQUATION**

$$\frac{D_w}{D_o} = 0.53\left(\frac{Q}{D_o^{5/2}}\right) \text{ FOR } \frac{Q}{D_o^{5/2}} \leq 10$$

WHERE:

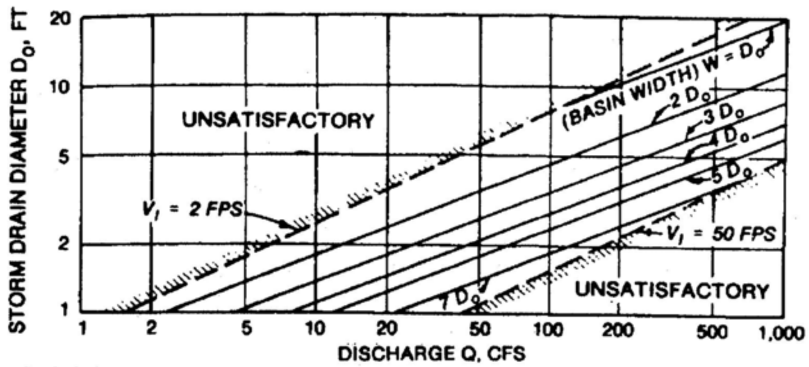
$D^w$ = STILLING WELL DIAMETER, FT
$D^o$ = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS




**ELEVATION**

**Figure 4-29. U.S. Bureau of Reclamation Impact Basin**

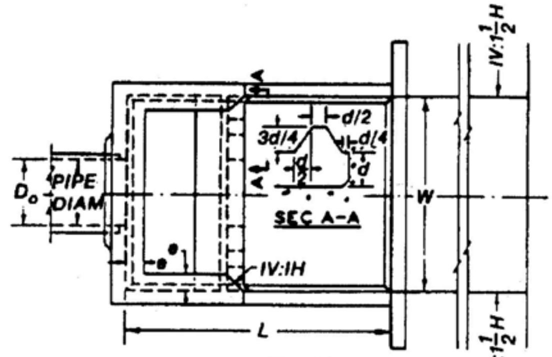



BASIC EQUATION

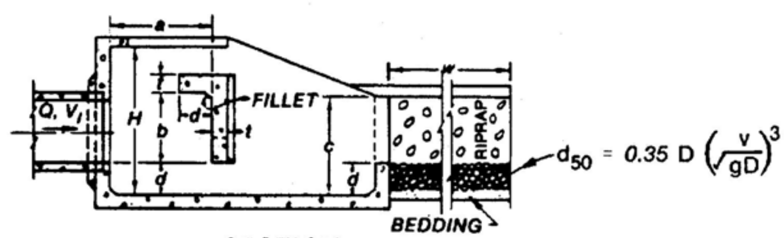
$$\frac{W}{D_o} = 1.3\left(\frac{Q}{D_o^{5/2}}\right)^{0.55} \text{ FOR } \frac{Q}{D_o^{5/2}} \leq 21$$

WHERE:

W = BASIN WIDTH, FT
$D^o$ = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS
$V_l$ = PIPE VELOCITY, FPS




PLAN




SECTION

STILLING BASIN DESIGN

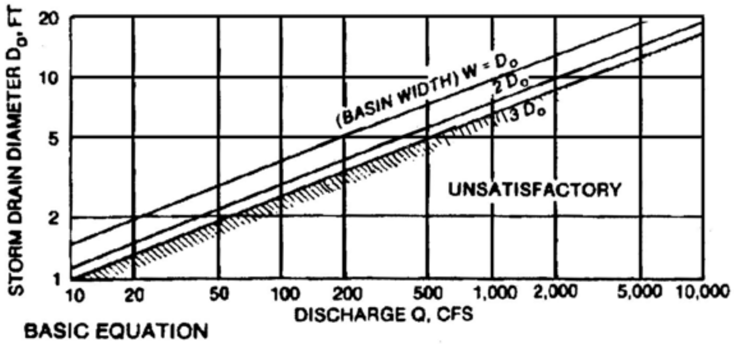
$H = \frac{3}{4}(W)$  $c = \frac{1}{2}(W)$

$L = \frac{4}{3}(W)$  $d = \frac{1}{6}(W)$

$a = \frac{1}{2}(W)$  $e = \frac{1}{12}(W)$

$b = \frac{3}{8}(W)$  $t = \frac{1}{12}(W)$, SUGGESTED MINIMUM

**Figure 4-30. Saint Anthony Falls Stilling Basin**



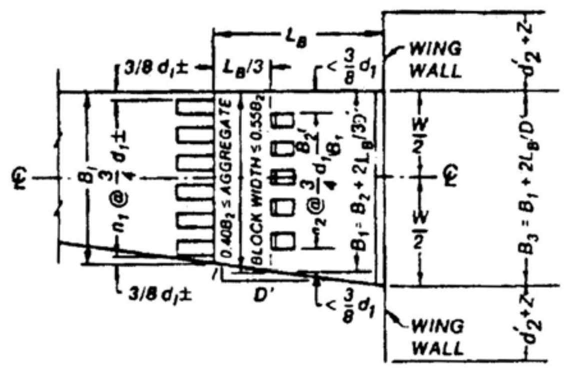

BASIC EQUATION

$$\frac{W}{D_o} = 0.3\frac{Q}{D_o^{5/2}} \quad \text{FOR} \quad \frac{Q}{D_o^{5/2}} \leq 9.5$$

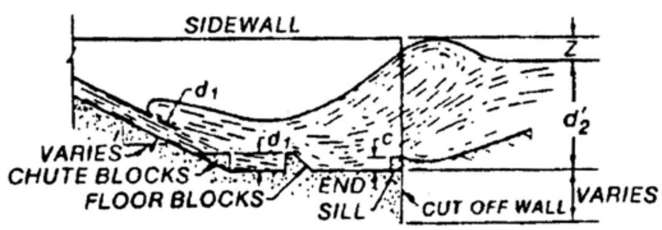
WHERE:

W = END SILL LENGTH, FT
$D^o$ = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS

STILLING BASIN HALF-PLAN




TRAPEZOIDAL STILLING BASIN HALF-PLAN




CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{gd_1} \qquad (1)$$

$$L_B = \frac{4.5\,d_2}{F^{0.38}} \qquad (4)$$

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{8F + 1}\right) \qquad (2)$$

$$Z = \frac{d_2}{3} \qquad (5)$$

$$c = 0.07d_2 \qquad (6)$$

F = 3 TO 30 $\quad d_2 = (1.10 - F/120)d_2$ (3a)

F = 30 TO 120 $\quad d_2 = 0.85d_2$ (3b)

F = 120 TO 300 $\quad d_2 = (1.00 - F/800)d_2$ (3c)

4-2.4.4.8 The stilling well shown in Figure 4-28 consists of a vertical section of circular pipe affixed to the outlet end of a storm sewer. The recommended depth of the well below the invert of the incoming pipe is dependent on the slope and diameter of the incoming pipe and can be determined from the plot in Figure 4-28. The recommended height above the invert of the incoming pipe is two times the diameter of the incoming pipe. The required well diameter can be determined from the equation in Figure 4-28. The top of the well should be located at the elevation of the invert of a stable channel or drainage basin. The area adjacent to the well may be protected by riprap or paving. Energy dissipation is accomplished without the necessity of maintaining a specified tailwater depth in the vicinity of the outlet. Use of the stilling well is not recommended with $Q/D_o^{5/2}$ greater than 10.

4-2.4.4.9 The U.S. Bureau of Reclamation (USBR) impact energy dissipator shown in Figure 4-29 is an efficient stilling device even with deficient tailwater. Energy dissipation is accomplished by the impact of the entering jet on the vertically hanging baffle and by the eddies that are formed following impact on the baffle. Excessive tailwater causes flow over the top of the baffle and should be avoided. The basin width required for good energy dissipation for a given storm drain diameter and discharge can be calculated from the information in Figure 4-29. The other dimensions of energy dissipator are a function of the basin width as shown in Figure 4-29. This basin can be used with $Q/D_o^{5/2}$ ratios up to 21.

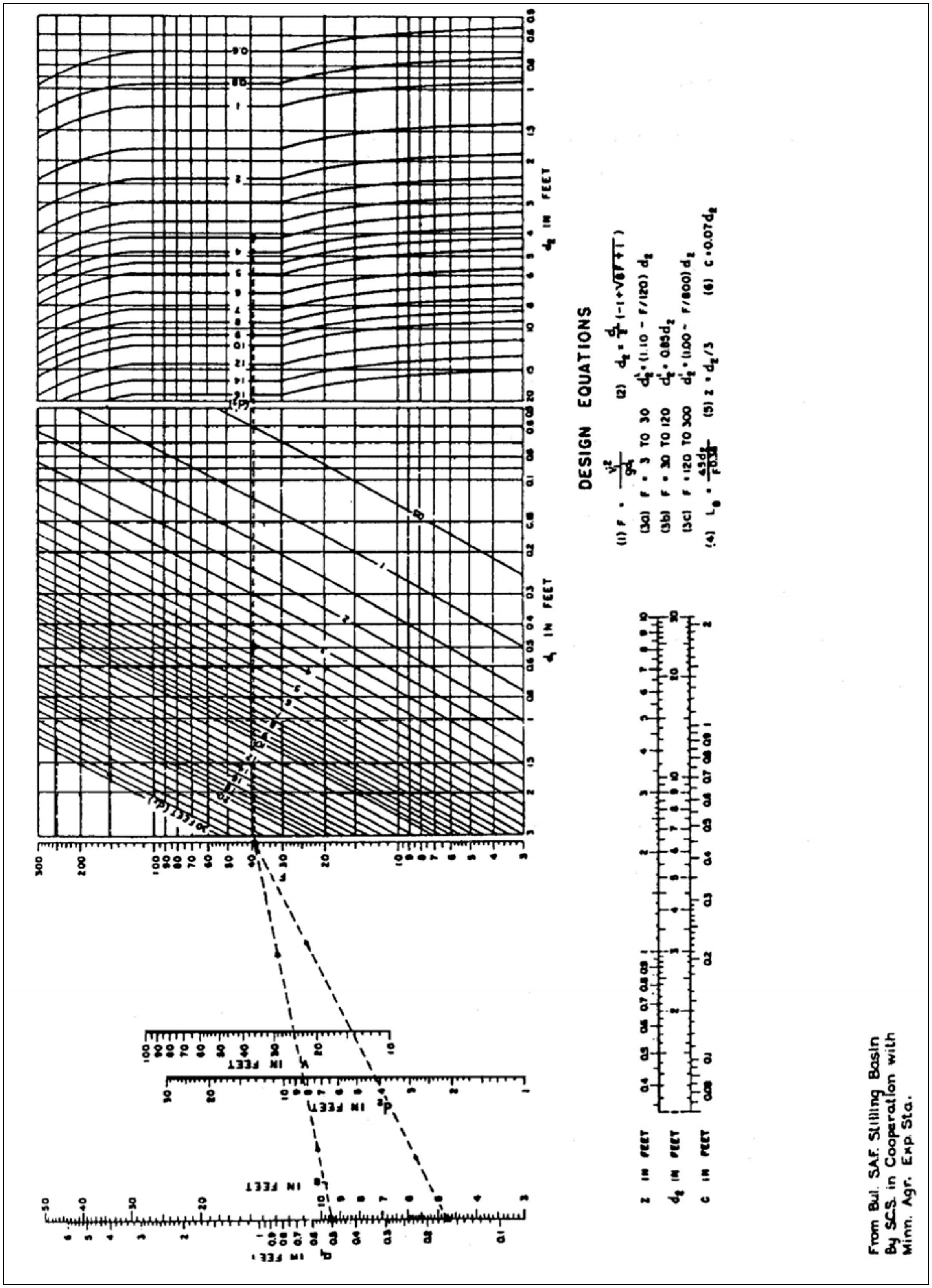
4-2.4.4.10 The Saint Anthony Falls (SAF) stilling basin shown in Figure 4-30 is a hydraulic jump energy dissipator. To function satisfactorily this basin must have sufficient tailwater to cause a hydraulic jump to form. Design equations for determining the dimensions of the structure in terms of the square of the Froude number of flow entering the dissipator are shown in this figure. Figure 4-31 is a design chart based on these equations. The width of basin required for good energy dissipation can be calculated from the equation in Figure 4-30. Tests used to develop this equation were limited to basin widths of three times the diameter of the outlet. But, other model tests indicate that this equation also applies to ratios greater than the maximum shown in Figure 4-30. However, outlet portal velocities exceeding 60 ft/sec are not recommended for design containing chute blocks. Parallel basin sidewalls are recommended for best performance. Transition sidewalls from the outlet to the basin should not flare more than 1 on 8.

4-2.4.4.11 Riprap will be required downstream from the above energy dissipators. The size of the stone can be estimated by the following equation.

$$d_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{or} \quad F = \left(d_{50} / D\right)^{1/3} \qquad \text{(eq. 4-3)}$$

This equation is also to be used for riprap subject to direct attack or adjacent to hydraulic structures such as inlets, confluences, and energy dissipators, where

**Figure 4-31. Design Chart for SAF Stilling Basin**



turbulence levels are high. The riprap should extend downstream for a distance approximately 10 times the theoretical depth of flow required for a hydraulic jump.

4-2.4.4.12 Smaller riprap sizes can be used to control channel erosion. Equation 4-4 is to be used for riprap on the banks of a straight channel where flows are relatively quiet and parallel to the banks.

Trapezoidal channels

$$d_{50} = .0.35D\left(\frac{V}{\sqrt{gD}}\right)^{3} \text{ or } F = 1.42\left(d_{50}/D\right)^{1/3} \qquad \text{(eq. 4-4)}$$

Equation 4-5 is to be used for riprap at the outlets of pipes or culverts where no preformed scour holes are made.

Wide channel bottom or horizontal scour hole

$$d_{50} = 0.15D\left(\frac{V}{\sqrt{gD}}\right)^{3} \text{ or } F = 1.88\left(d_{50}/D\right)^{1/3} \qquad \text{(eq. 4-5)}$$

½ D deep scour hole

$$d_{50} = 0.09D\left(\frac{V}{\sqrt{gD}}\right)^{3} \text{ or } F = 2.23\left(d_{50}/D\right)^{1/3} \qquad \text{(eq. 4-6)}$$

D deep scour hole

$$d_{50} = 0.055D\left(\frac{V}{\sqrt{gD}}\right)^{3} \text{ or } F = 2.63\left(d_{50}/D\right)^{1/3} \qquad \text{(eq. 4-7)}$$

These relationships are shown in Figures 4-32 and 4-33.

4-2.4.4.13 Examples of recommended application to estimate the extent of scour in a cohesionless soil and several alternate schemes of protection required to prevent local scour downstream from a circular and rectangular outlet are shown in Appendix C.

4-2.4.4.14 User-friendly computer programs are available to assist the designer with many of the design problems discussed in this chapter (Conversationally Oriented Real-Time Program Generating System (CORPS)). These programs are available from CEWES-LIB, U.S. Army Engineer Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180-0631.

**Figure 4-32. Recommended Riprap Sizes**

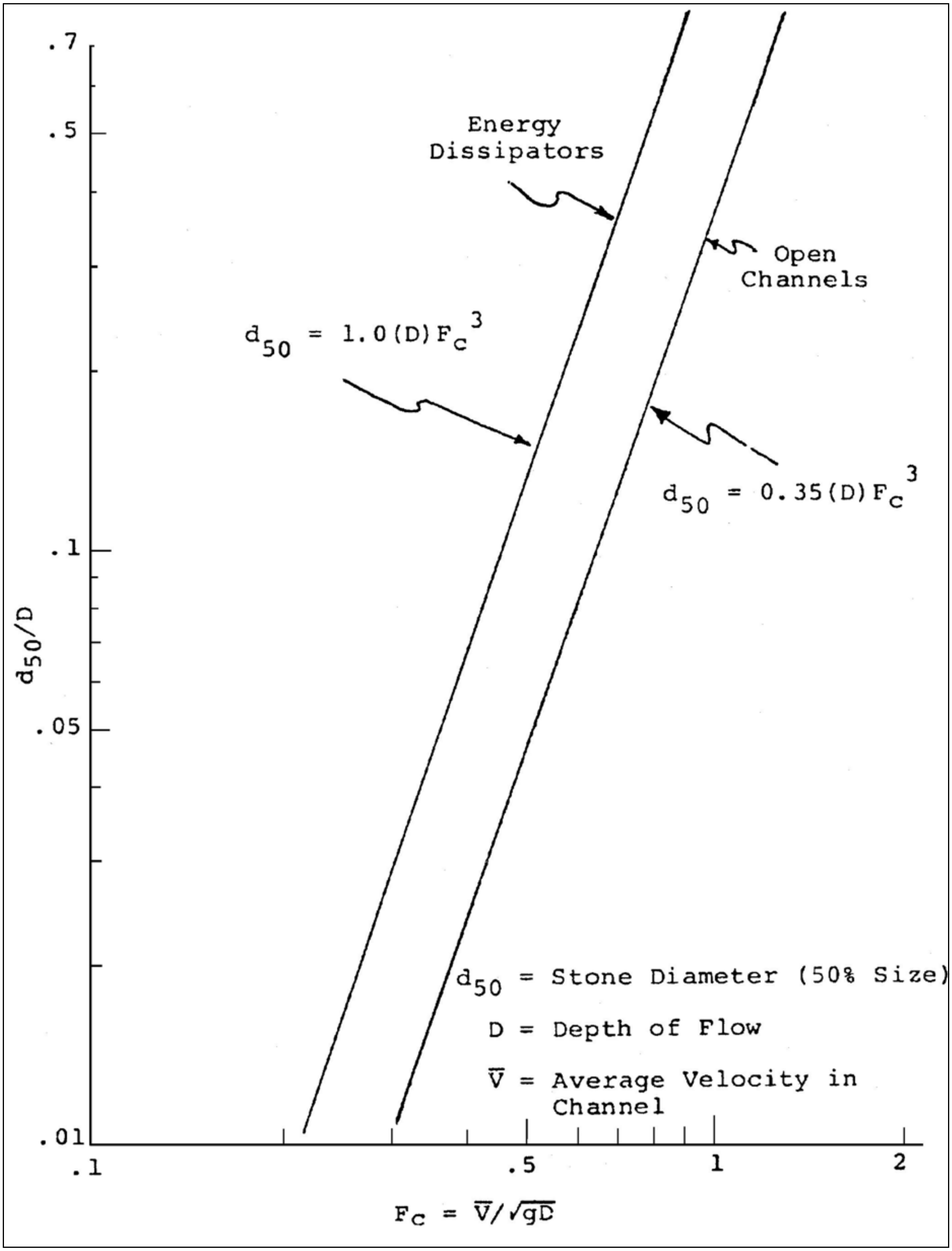
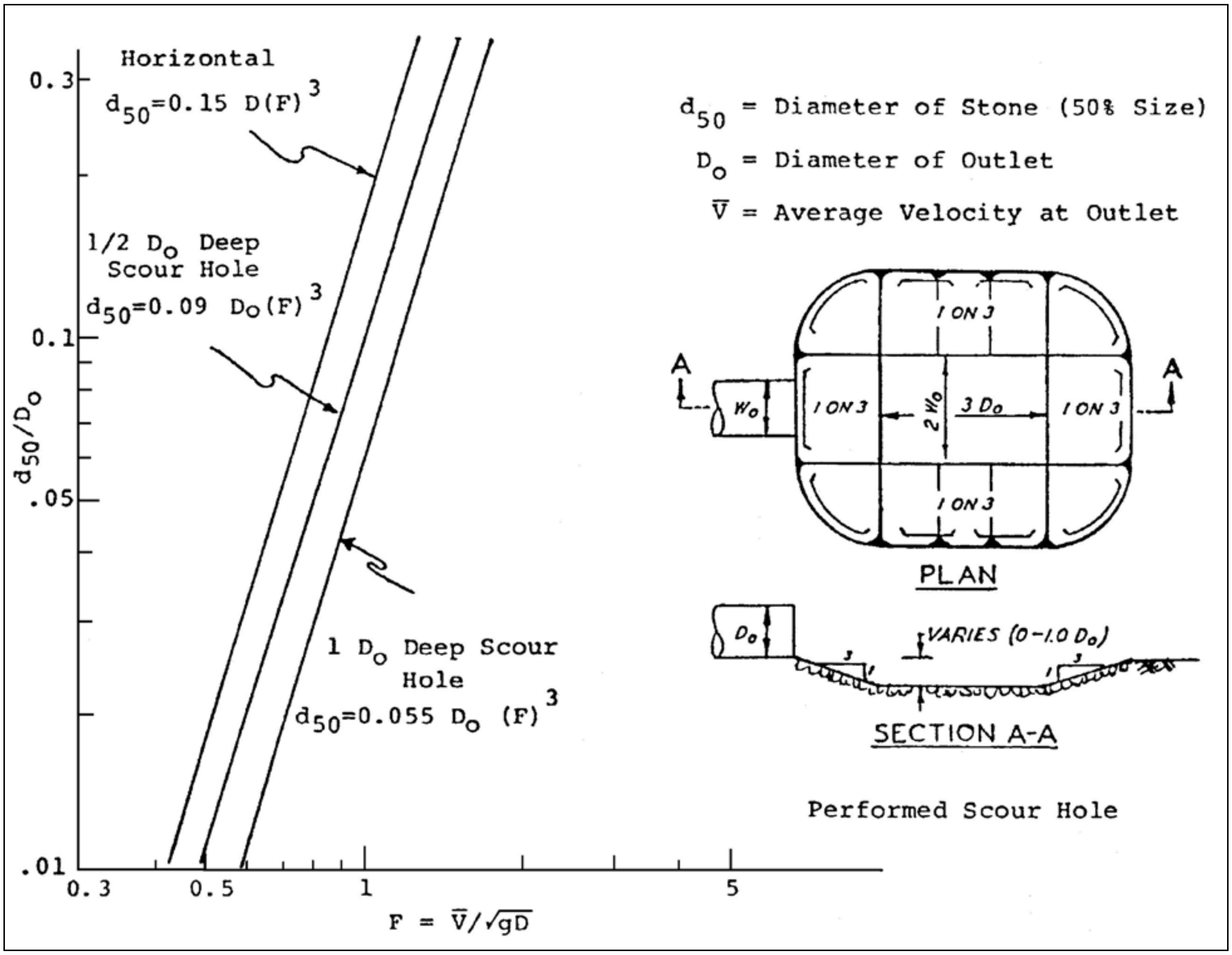


**Figure 4-33. Scour Hole Riprap Sizes**




## 4-2.5 Open Channels

4-2.5.1 **General**. One of the most difficult problems associated with surface drainage facilities is the design of effective, stable, natural, open channels that will not be subject to severe erosion and/or deposition. Tests show that performance is poorer and requires more costly and more frequent maintenance to provide effective drainage channels. Open channels which meet the airfield and heliport's safety and operational requirements will be used since they provide greater flexibility, a higher safety factor, and are more cost effective. Drop structures and check dams can be used to control the effective channel gradient.

4-2.5.2 **Channel design**. The following items merit special consideration in designing channels.

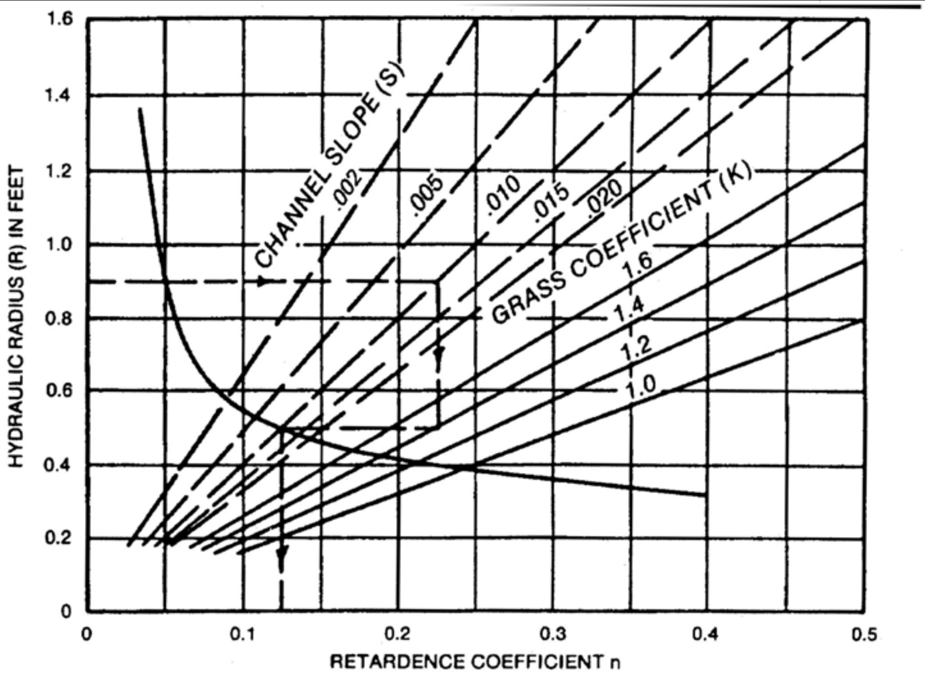
4-2.5.2.1 The hydraulic characteristics of the channel may be studied by using an open-channel formula such as Manning's. Suggested retardance coefficients and maximum permissible velocities for nonvegetated channels are given in Table 4-3. Retardance coefficients for turf-lined channels are a function of both the turf characteristics and the depth and velocity of flow and can be estimated by the graphical relations shown in Figure 4-34. It is suggested that maximum velocity in turf-lined channels not exceed 6 feet per second. In regions where runoff has appreciable silt load, particular care will be given to securing generally nonsilting velocities.

**Table 4-3. Suggested Coefficients of Roughness and Maximum Permissible Mean Velocities for Open Channels in Military Construction**

| Material | Manning's n | Maximum permissible mean velocity ft/sec |
|---|---|---|
| Concrete, with surfaces as indicated: | | |
| Formed, no finish | 0.014 | -- |
| Trowel finish | 0.012 | -- |
| Float finish | 0.012 | -- |
| Gunite, good section | 0.016 | 30 |
| Concrete, bottom float finish, sides as indicated: | | |
| Cement rubble masonry | 0.020 | 20 |
| Cement rubble masonry, plastered | 0.018 | 25 |
| Rubble lined, uniform section | 0.030-0.045 | 7-13 |
| Asphalt: | | |
| Smooth | 0.012 | 15 |
| Rough | 0.016 | 12 |
| Earth, uniform section: | | |
| Sandy silt, weathered | 0.020 | 2.0 |
| Silt clay | 0.020 | 3.5 |
| Soft shale | 0.020 | 3.5 |
| Clay | 0.020 | 6.0 |
| Soft sandstone | 0.020 | 8.0 |
| Gravelly soil, clean | 0.025 | 6.0 |
| Natural earth, with vegetation | 0.03-0.150 | 6.0 |
| Grass swales and ditches[1] | | 6.0 |
| [1] See Figure 4-34. | | 6.0 |

4-2.5.2.2 The selection of the channel cross section is predicted on several factors other than hydraulic elements. Within operational areas, the adopted section will conform with the grading criteria contained in AFR 86-8 or TM 5-803-4. Proposed maintenance methods affect the selection of side slopes for turfed channels since gang mowers cannot be used on slopes steeper than 1 vertical (V) to 3 horizontal (H), and hand cutting is normally required on steeper slopes. In addition, a study will be made of other factors that might affect the stability of the side slopes, such as soil characteristics, excessive ground-water inflow, and bank erosion from local surface-water inflow.

**Figure 4-34. Retardance Coefficients for Flow in Turfed Channels**




GRASS COEFFICIENTS (K) FOR DENSE AIRFIELD TURF

| GRASS SPECIES | AVG LENGTH OF GRASS IN INCHES | | |
|---|---|---|---|
| | < 6 | 6-12 | > 12 |
| BUFFALO | 1.6 | -- | -- |
| BLUE GRAMMA | 1.5 | 1.4 | 1.3 |
| BLUE GRASS | 1.4 | 1.3 | 1.2 |
| BERMUDA | 1.4 | 1.3 | 1.2 |
| LESPEDEZA SERICEA | 1.3 | 1.2 | 1.1 |

EXAMPLE:

DETERMINE n FOR 4-INCH BERMUDA GRASS CHANNEL WITH R = 0.9 and S = 0.010.

FROM TABLE k = 1.4 AND FROM GRAPH, FOLLOWING DASHED LINE, n IS EQUAL TO 0.125.

4-2.5.2.3 Earth channels normally require some type of lining such as that obtained by developing a strong turf of a species not susceptible to rank growth. In particularly erosive soils, special methods will be necessary to establish the turf quickly or to provide supplemental protection by mulching or similar means. For further discussion of turfing methods, see TM 5-803-13/AFM 126-8. Where excessive velocities are to be encountered or where satisfactory turf cannot be established and maintained, it may be necessary to provide a paved channel.

4-2.5.2.4 A channel design calling for an abrupt change in the normal flow pattern induces turbulence and causes excessive loss of head, erosion, or deposition of silt. Such a condition may result at channel transitions, junctions, storm-drain outlets, and reaches of excessive curvature, and special attention will be given to the design of structures at these locations.

4-2.5.2.5 Channel design (see Example C-5 in Appendix C) must include measures for preventing uncontrolled inflow from drainage areas adjacent to open channels. This local inflow has caused numerous failures and is particularly detrimental where, due to the normal irregularities experienced in grading operations, runoff becomes concentrated and results in excessive erosion as it flows over the sides of the channel. A berm at the top edge of the channel will prevent inflow except at designated points, where inlets properly protected against erosion are provided. The inlet may vary from a sodded or paved chute to a standard field inlet with a storm drain connection to the channel. Erosion resulting from inflow into shallow drainage ditches or swales with flat side slopes can be controlled by a vigorous turfing program supplemented by mulching where required. Where excavated material is wasted in a levee or dike parallel and adjacent to the channel, provision will be made for frequent openings through the levee to permit local inflow access to the channel. A suitable berm (minimum of 3 ft) will be provided between the levee and the top edge of the channel to prevent sloughing as a result of the spoil bank load and to minimize movement of excavated material back into the channel. Example problems in channel design are shown in Appendix C.

4-2.5.2.6 Field observations indicate that stable channels relatively free of deposition and/or erosion can be obtained provided the Froude number of flow in the channel is limited to a certain range depending upon the type of soil. An analysis of experimental data indicates that the Froude number of flow (based on average velocity and depth of flow) required to initiate transport of various diameters of cohesionless material, $d_{50}$ , in a relatively wide channel can be predicted by the empirical relation, $F = 1.88\ (d_{50}/D)^{1/3}$. The terms are defined in Section 4-2.8.

### 4-2.5.3 **Design procedure**

4-2.5.3.1 This design procedure is based on the premise that the above empirical relation can be used to determine the Froude number of flow in the channel required to initiate or prevent movement of various sizes of material. Relations based on the Manning formula can then be applied to determine the geometry and slope of a channel of practical proportion that will convey flows with Froude numbers within a desired range such that finer material will be transported to prevent deposition but larger material will not be transported to prevent erosion.

4-2.5.3.2 Appendix C contains an example problem for the design of a channel using this procedure. It will satisfy the conditions desired for the design discharge and one that will ensure ho deposition or erosion under these conditions.

4-2.5.4 **Drop structures and check dams**

4-2.5.4.1 Drop structures and check dams are designed to check channel erosion by controlling the effective gradient and to provide for abrupt changes in channel gradient by means of a vertical drop. They also provide satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding 5 ft and over embankments higher than 5 ft if the end sill of the drop structure extends beyond the toe of the embankment. The check dam is a modification of the drop structure used for erosion control in small channels where a less elaborate structure is permissible.

4-2.5.4.2 There are numerous types of drop and grade control structures. They can be constructed of concrete, metal piling, gabions, riprap, or a combination of materials. Design of many of these structures is beyond the scope of this manual, and if the designer needs design information for a specific type structure, the publications in the bibliography should be consulted.

4-2.5.4.3 Pertinent features of a typical drop structure are shown in Figure 4-35. The hydraulic design of these structures can be divided into two general phases: design of the weir and design of the stilling basin. It is emphasized that for a drop structure or check dam to be permanently and completely successful, the structure must be soundly designed to withstand soil and hydrostatic pressures and the effects of frost action, when necessary. Also, the adjacent ditches or channels must be completely stable. A stable grade for the channel must first be ascertained before the height and spacing of the various drop structures can be determined.

4-2.5.4.4 The following design rules are based on hydraulic considerations only. They are minimum standards subject to increase on the basis of other considerations such as structural requirements and special frost condition design.

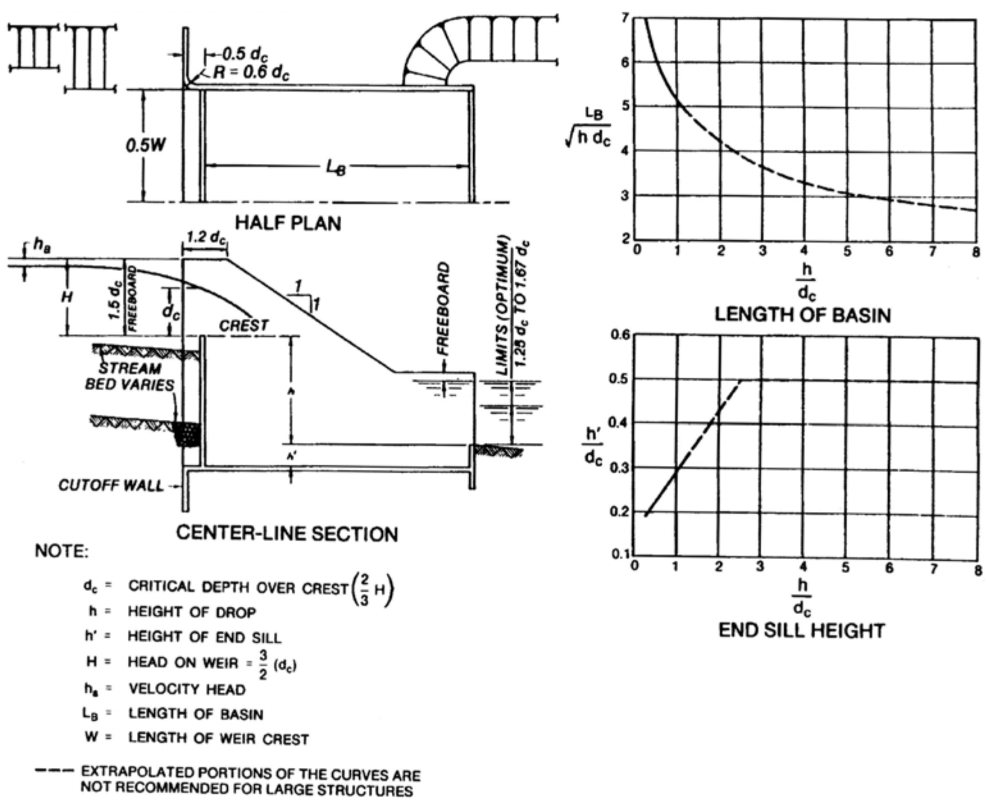
a. Discharge over the weir should be computed from the equation $Q = CWH^{3/2}$ using a C value of 3.0. To minimize erosion and obtain maximum use of the available channel cross section upstream from the structure, the length of the weir should be adjusted to maintain a head on the weir equivalent to the depth of flow in the channel. A trial-and-error procedure should be used to balance the crest height and width with the channel cross section.

b. The relation between the height of drop, h, critical depth at the drop, $d_c$, and the required stilling basin length, $L_B$, is defined by the equation

$$L_B = C_L \sqrt{h d_c} \qquad \text{(eq. 4-8)}$$

where $C_L$ is an empirical coefficient between 2 and 7, as shown in Figure 4-35. The stilling basin length and end sill height can be determined from the design curves in Figure 4-35. Optimum performance of the basin is obtained when the tailwater-critical depth ratio is 1.25 to 1.67. However, the basin will function satisfactorily with higher tailwaters if the depth of tailwater above the weir does not exceed 0.7 $d_c$. The stilling basin walls should be high enough to prevent the tailwater from reforming over the walls into the stilling basin. Riprap protection should be provided immediately downstream from the structure. Guidance provided in Section 4-2.4.4.11 can be used for design of the riprap.

**Figure 4-35. Details and Design Chart for Typical Drop Structure**




4-2.5.4.5 A design illustrating the use of the above information and Figure 4-35 is shown in the following example. Design a drop structure for a discharge of 250 $ft^3$/sec in a trapezoidal channel with a 10-ft base width and side slopes of 1V on 3H, and a depth of flow of 5 ft. The amount of drop required is 4 ft. If the crest is placed at invert of the channel, the head on the crest, H, will be equal to the depth of flow, 5 ft.

Width of Crest, W:

$$Q = CWH^{3/2} \tag{eq. 4-9}$$

$$W = \frac{250}{3 \times (5)^{3/2}} 7.5\ ft \qquad \text{(eq. 4-10)}$$

Since the base width of the channel is 10 ft, the weir crest should be made 10 ft long and raised up to maintain a depth of 5 ft upstream. If the width determined above would have been greater than 10 ft then the greater width would have had to be retained and the channel expanded to accommodate this width.

4-2.5.4.6 With width of crest equal to 10 ft, determine head on the crest:

$$Q = CWH^{3/2} \qquad \text{(eq. 4-11)}$$

$$H = (250/3 \times 10)^{2/3} = 4.1\,ft \qquad \text{(eq. 4-12)}$$

Thus, crest elevation will be 5 - 4.1 = 0.9 ft above channel invert and distance from crest to downstreams channel invert, h, will be 4 + 0.9 = 4.9 ft.

Critical depth, $d_c$:

$$d_c = \frac{2}{3}H = \frac{2}{3}(4.1) = 2.73\ ft \qquad \text{(eq. 4-13)}$$

$$\frac{h}{d_c} = \frac{4.9}{2.73} = 1.8 \qquad \text{(eq. 4-14)}$$

From Figure 4-35:

$$\frac{L_B}{\sqrt{hd_c}} = 4.4 \qquad \text{(eq. 4-15)}$$

$L_B$ = 16.09 ft (use 16.1 ft) (eq. 4-16)

$$\frac{h'}{d_c} = 0.4 \qquad \text{(eq. 4-17)}$$

$$h' = 0.4 \times 2.73 = 1.09\text{ ft (use 1.1 ft)} \qquad \text{(eq. 4-18)}$$

The tailwater depth will depend on the channel configuration and slope downstream from the structure. If these parameters are the same as those of the approach channel, the depth of tailwater will be 5 ft. Thus, the tailwater/$d_c$ ratio is 5/2.73 = 1.83 which is greater than 1.67 recommended for optimum energy dissipation. However, the tailwater depth above the crest (5.0 – 0.49 = 0.10) divided by critical depth (2.73) is (0.1/2.73=0.04) much less than 0.7 and the basin will function satisfactorily.

Riprap design:

$$d_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{(eq. 4-19)}$$

$$d_{50} = 5\left(\frac{5}{\sqrt{32.2 \times 5}}\right)^3 = 0.306 \text{ ft (use 4 in.)} \quad \text{(eq. 4-20)}$$

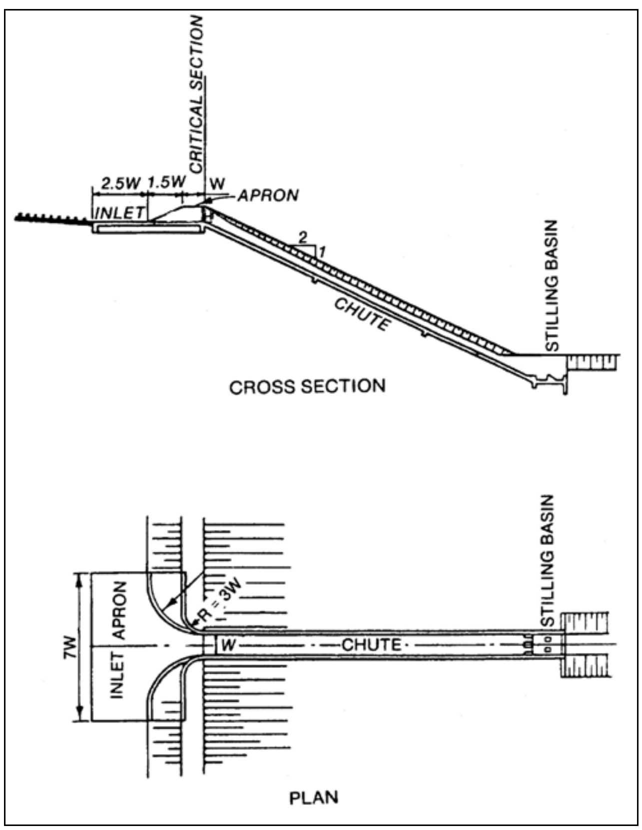
Riprap should extend approximately 10 times depth of flow downstream from structure (10 × 5 = 50 ft).

V = Discharge/area at end of basin = 250/10 × 5 = 5 ft/sec

## 4-2.6 Chutes

4-2.6.1 **General**. A chute is a steep open channel which provides a method of discharging accumulated surface runoff over fills and embankments. A typical design is shown in Figure 4-36. Frost penetration beneath the structure will be restricted to nonfrost-susceptible materials using procedures outlined in Section 4-2.1.6.2, since small increments of heave may seriously affect its drainage capacity and stability. The following features of the chute will be given special consideration in the preparation of the design.

**Figure 4-36. Details and Typical Drainage Chute**



4-2.6.1.1 The berm at the edge of the fill will have sufficient freeboard to prevent overtopping from discharges in excess of design runoff. A minimum height of wall of one and one-half times the computed depth of flow is suggested. Turfed berm slopes will not be steeper than 1V to 3H because they cannot be properly mowed with gang mowers.

4-2.6.1.2 A paved approach apron is desirable to eliminate erosion at the entrance to the chute. A cutoff wall should be provided around the upstream edge of the apron to prevent undercutting, and consideration should be given to effects of frost action in the design. Experience has shown that a level apron minimizes erosion of adjacent soil and is self-cleaning as a result of increased velocities approaching the critical section.

4-2.6.2 **Design**

4-2.6.2.1 The entrance to the chute can be level or a drop can be provided as shown in Figure 4-37. The advantage of providing the drop is to reduce the depth of headwater upstream. The dimensions of the structure can be determined from a known discharge and allowable head or width of chute by using the charts provided in Figure 4-38. The curve with D=0 is for a level approach to a drop. The following equation can be used to determine the discharge at given head and chute width when no drop is provided.

$$Q = 3.1W\ H^{1.5} \qquad \text{(eq. 4-21)}$$

All of the curves shown in Figure 4-38 were developed with the radius of an abutment equal to three times the width of the chute. If it becomes necessary to increase the radius of the abutments because of upstream embankments or other reasons, as will probably be the case for smaller chutes, the equation for D = 0 should be used for design since the radius of the abutments will have little effect on the discharge.

4-2.6.2.2 The depth of flow in the chute can be computed using Manning's equation

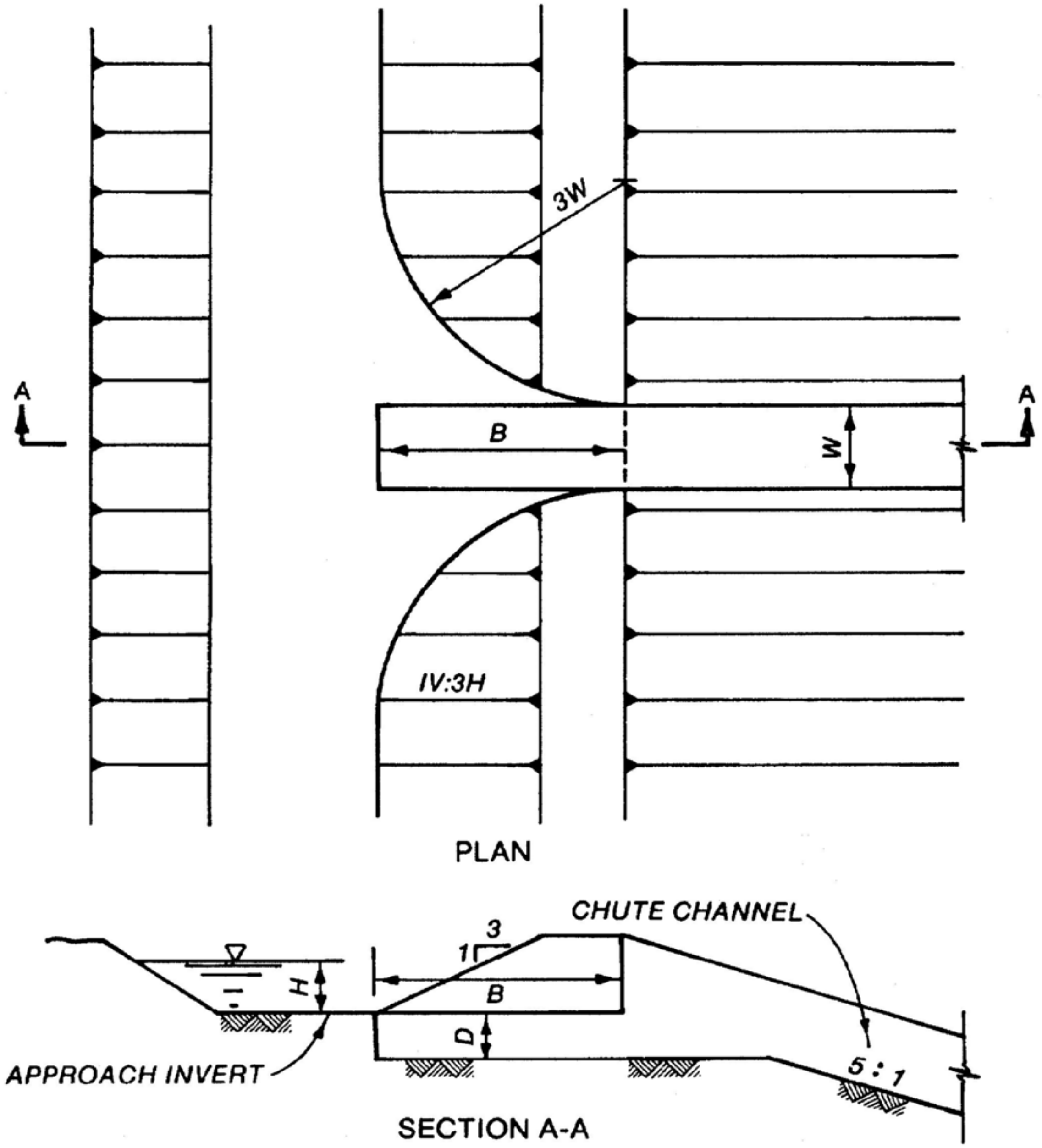
$$Q = \frac{1.486}{n} = A\,S^{1/2}R^{2/3} \qquad \text{(eq. 4-22)}$$

where:

Q = Discharge, ft$^3$/sec
n = Roughness factor
A = Area, ft$^2$
S = Slope, ft/ft
R = Hydraulic radius, ft

Air becomes entrained in flow through steep chutes causing the depth of flow to increase which necessitates increasing the side-wall height. The chart in Figure 4-39 can be used to determine the amount of air entrainment and thus the total depth of flow which is equal to the depth of air plus the depth of water.

**Figure 4-37. Details of Typical Drop Intake**




LEGEND

B = LENGTH OF DROP FEET
D = DEPTH OF DROP FEET
W = CHUTE WIDTH, FEET
H = UPSTREAM, HEADWATER DEPTH, FT.
Q = DISCHARGE, CFS

**Figure 4-38. Drop Structure Calibration Curve**

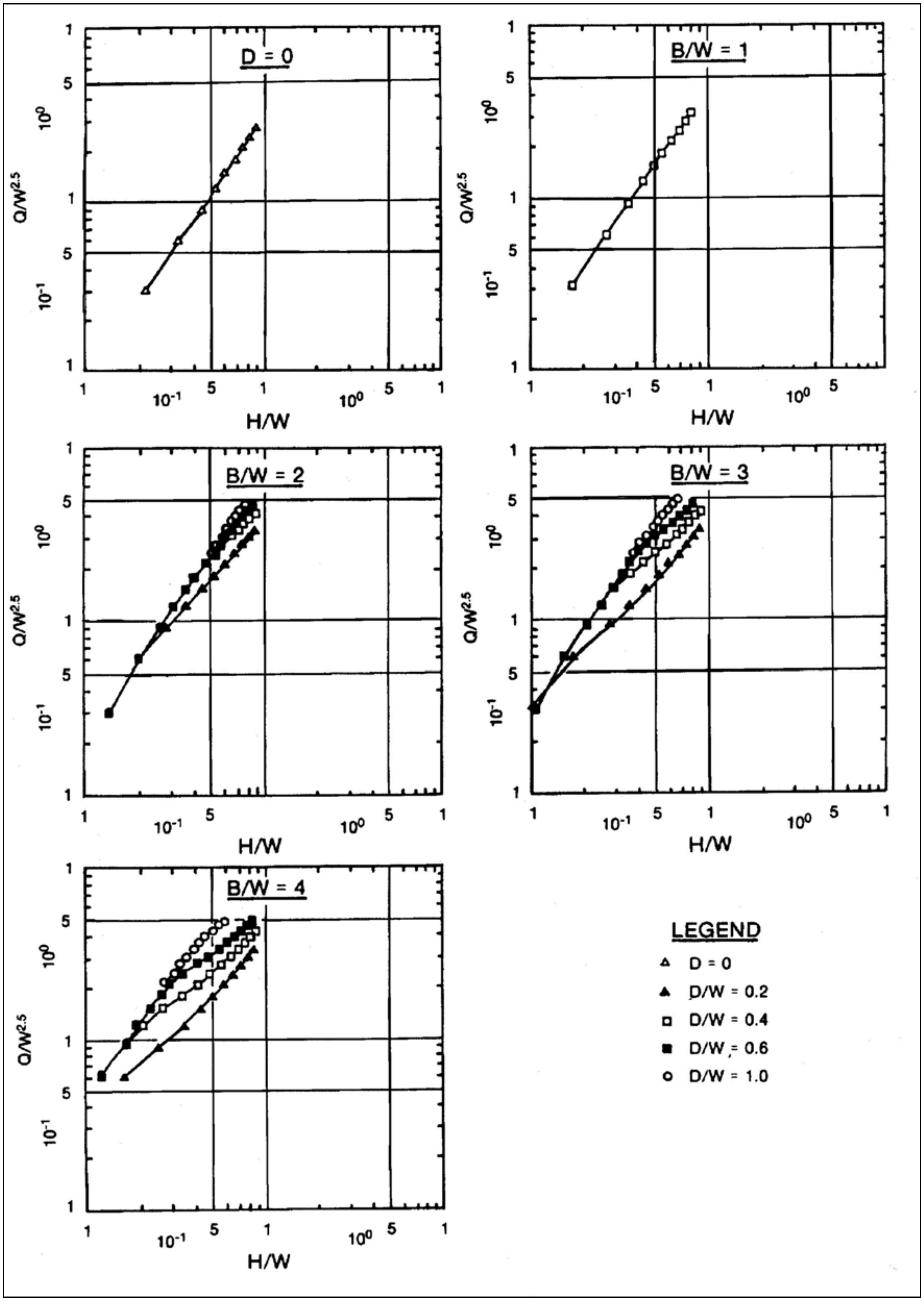
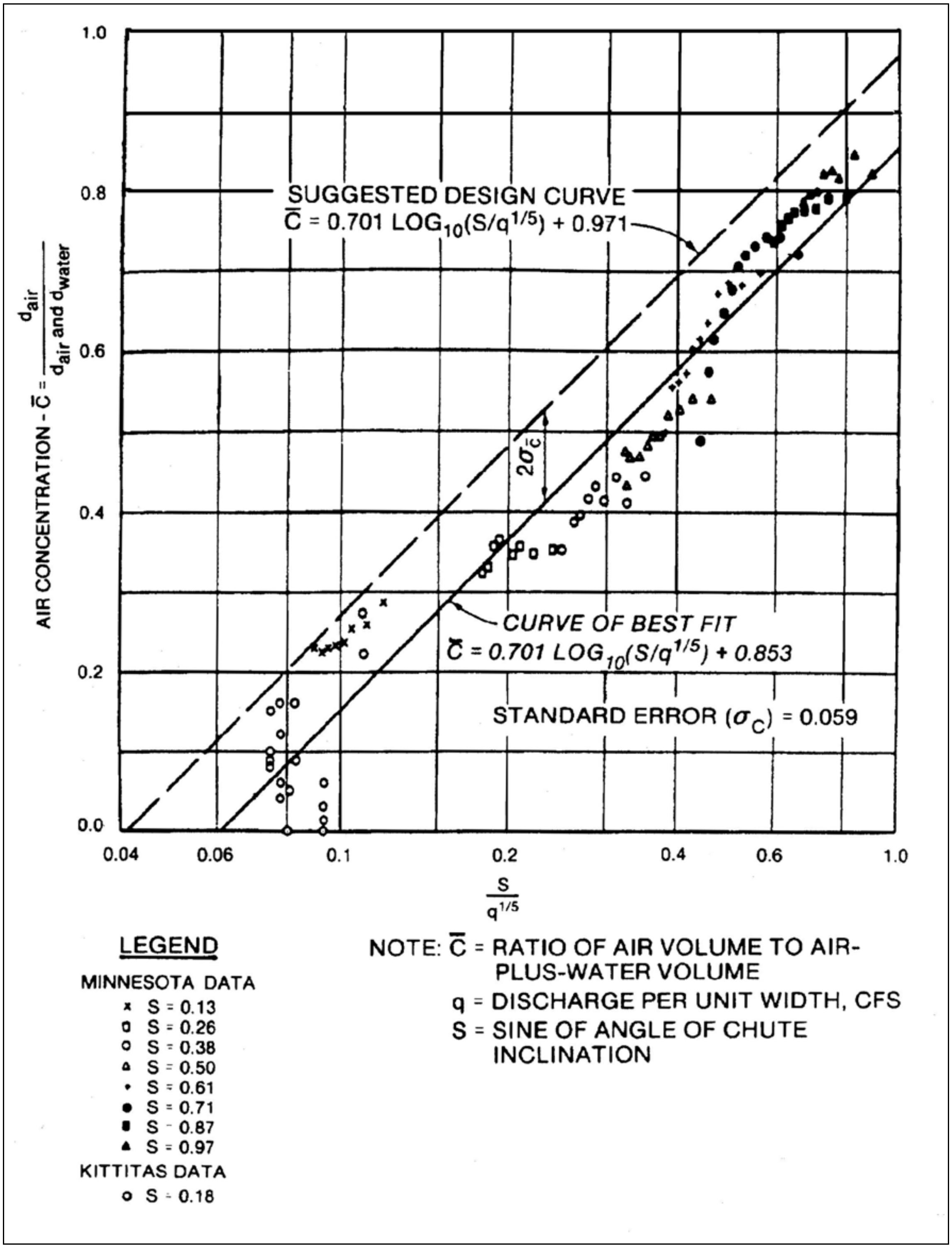


**Figure 4-39. Air Entrainment in Chute Flow**



4-2.6.2.3 Adequate freeboard is most important in the design of a concrete chute. The critical section where most failures have occurred is at the entrance where the structure passes through the berm. As indicated earlier, a minimum freeboard equal to one and one-half times the computed depth of flow is recommended. A minimum depth of 3 in. is suggested for the chute. Minor irregularities in the finish of the chute frequently result in major flow disturbances and may even cause overtopping of sidewalls and structural failure. Consequently, special care must be given to securing a uniform concrete finish and adequate structural design to minimize cracking, settlement, heaving, or creeping. A suitable means for energy dissipation or erosion prevention must be provided at the end of the chute.

## 4-2.7 **Construction Drainage**

4-2.7.1 **General**. Proper consideration of drainage during construction can frequently prevent costly delays and future failures. Delays can occur not only because of damaged or washed-out facilities but because of shut-down resulting from environmental considerations. Proper construction drainage is critical to efficient and timely completion of earthwork.

4-2.7.2 **Planning**. Efforts to control delays or damages caused by construction drainage must begin in the planning stage and carry through design and construction. Guide specifications have been developed by Division offices, but it is impractical to prescribe fixed rules to cover all eventualities. Protective measures cannot generally be reduced to biddable contract items.

4-2.7.3 **Environmental degradation**. Every construction activity can create environmental impacts to some degree. Although the effects are usually temporary, it is important to minimize damage by anticipating problems and applying protective standards of performance.

4-2.7.4 **Protective measures**. Control of runoff problems during construction can be costly. Consideration of the following items will aid in maintaining satisfactory drainage during the construction period.

4-2.7.4.1 Maximum use will be made of existing ditches and drainage features. Where possible, grading operations will proceed downhill, both for economic grading and to use natural drainage to the greatest extent.

4-2.7.4.2 Temporary ditches will be required to facilitate construction drainage. A particular effort will be made to drain pavement subgrade excavations and base courses to prevent detrimental saturation. Careful considerations will be given to the drainage of all construction roads, equipment areas, borrow pits, and waste areas.

4-2.7.4.3 Temporary retention structures will be required in areas where open excavation can lead to excessive erosion or discharge of turbid water to local streams.

4-2.7.4.4 Random excavation will be held to a minimum, and finished surfaces will be sodded or seeded immediately.

4-2.7.4.5 Installation of final storm drain facilities and backfilling operations will be planned and timed to render maximum use during the construction period.

### 4-2.8 Notation

| | |
|---|---|
| **A** | Cross-sectional area, $ft^2$ |
| **a** | Offset for weir notch ventilation, ft |
| **B** | Base width of channel, ft |
| $\mathbf{b_n}$ | Length of notch, ft |
| $\mathbf{B_s}$ | Bottom width of approach channel, ft |
| **C** | Coefficient |
| **D** | Depth of flow in channel, ft |
| $\mathbf{D_o}$ | Diameter of circular culverts, ft |
| $\mathbf{D_s}$ | Depth of scour, ft |
| $\mathbf{D_{sm}}$ | Maximum depth of scour, ft |
| $\mathbf{D_w}$ | Diameter of stilling well, ft |
| **d** | Depth of uniform flow in culvert, ft |
| $\mathbf{d_c}$ | Critical depth, ft |
| $\mathbf{d_s}$ | Depth of approach flow, ft |
| $\mathbf{d_1}$ | Depth of flow upstream of hydraulic pump, ft |
| $\mathbf{d_2}$ | Theoretical depth of flow required for hydraulic jump, ft |
| $\mathbf{d_{50}}$ | Diameter of average size stone, ft |
| **F** | Froude number |
| $\mathbf{F_{ch}}$ | Froude number of flow in channel, $F_{ch} = Q/gA^3/T$ |
| **g** | Acceleration due to gravity, ft-$sec^2$ |

**H** head, depth of recessed apron and height of end still, ft. Also, horizontal

**h** Height of fall or drop in structure, ft

**$h_1$** Height of longitudinal sill, ft

**$h_t$** Height of transverse end sill, ft

**h'** Height of end sill

**L** Gross perimeter of grate opening, length of flared outlet transition, length of apron, length of basin, ft

**$L_s$** Length of scour, ft

**$L_{sm}$** Maximum length of scour, ft

**$L_{sp}$** Length of stone protection

**n** Manning's roughness coefficient

**Q** Discharge, cfs

**q** Discharge per foot of width, cfs/ft

**S** Slope of channel bottom for partial pipe flow and slope of energy gradient for full pipe flow

**T** Depth of stilling well below invert of incoming pipe, ft

**TW** Tailwater depth above invert of culvert outlet, ft

**T** Top width of flow in channel, ft

**$T_s$** Thickness of sack revetment

**$T_B$** Thickness of cellular blocks

**t** Thickness of breast wall at notch, in and duration of flow, min

**V,v** Average velocity of flow, ft/sec. Also, vertical

**$V_s$** Volume of scour, $ft^3$

**W** Length of weir, width of flume, ft

**$W_s$** Width of scour from centerline of single circular or square outlet, ft

**$W_{sm}$** One-half maximum width of scour from centerline of single circular or square outlet, ft

**$W_{smr}$** One-half maximum width of scour from centerline of single rectangular outlet, ft

4-3 **FUEL/WATER SEPARATORS**. Fuel/water separators should be installed where there is an oil/water separation problem. The most common location for these units is in areas that contain vehicle washracks. Details on the selection and design of oil/water separators can be found in ETL 1110-3-466, dated 26 August 1994.

4-4 **AREAS OTHER THAN AIRFIELDS**

4-4.1 **General**. Hydraulic design of the required elements of a system for drainage or for protective works may be initiated after functional design criteria and basic hydrologic data have been determined. The hydraulic design continually involves two prime considerations, namely, the flow quantities to which the system will be subjected, and the potential and kinetic energy and the momentum that are present. These considerations require that the hydraulic grade line and, in many cases, the energy grade line for design and pertinent relative quantities of flow be computed, and that conditions whereby energy is lost or dissipated must be carefully analyzed. The phenomena that occur in flow of water at, above, or below critical depth and in change from one of these flow classes to another must be recognized. Water velocities must be carefully computed not only in connection with energy and momentum considerations, but also in order to establish the extent to which the drainage lines and water-courses may be subjected to erosion or deposition of sediment, thus enabling determination of countermeasures needed. The computed velocities and possible resulting adjustments to the basic design layout often affect certain parts of the hydrology. Manning's equation is most commonly used to compute the mean velocities of essentially horizontal flow that occurs in most elements of a system:

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

The terms are defined in Section 4-4.15. Values of n for use in the formula are listed in Section 4-2.1.

4-4.2 **Channels**.

4-4.2.1 Open channels on military installations range in form from graded swales and bladed ditches to large channels of rectangular or trapezoidal cross section. Swales are commonly used for surface drainage of graded areas around buildings and within housing developments. They are essentially triangular in cross section, with some bottom rounding and very flat side slopes, and normally no detailed computation of their flow-carrying capacity is required. Ditches are commonly used for collection of surface water in outlying areas and along roadway shoulders. Larger open channels, which may be either wholly within the ground or partly formed by levees, are used principally for perimeter drains, for upstream flow diversion or for those parts of the drainage system within a built-up area where construction of a covered drain would be unduly costly or otherwise impractical. They are also used for rainfall drainage disposal. Whether a channel will be lined or not depends on erosion characteristics, possible grades, maintenance requirements, available space, overall comparative costs, and other factors. The need for providing a safety fence not less than 4 ft high along the larger channels (especially those carrying water at high velocity) will be considered, particularly in the vicinity of housing areas.

4-4.2.2 The discussion that follows will not attempt to cover all items in the design of an open channel; however, it will cite types of structures and design features that require special consideration.

4-4.2.3 Apart from limitations on gradient imposed by available space, existing utilities, and drainage confluences is the desirability of avoiding flow at or near critical depths. At such depths, small changes in cross section, roughness, or sediment transport will cause instability, with the flow depth varying widely above and below critical. To insure reasonable flow stability, the ratio of invert slope to critical slope should be not less than 1.29 for supercritical flow and not greater than 0.76 for subcritical flow. Unlined earth channel gradients should be chosen that will produce stable subcritical flow at nonerosive velocities. In regions where mosquito-borne diseases are prevalent, special attention must be given in the selection of gradients for open channels to minimize formation of breeding areas; pertinent information on this subject is given in TM 5-632/AFM 91-16.

4-4.2.4 Recommended maximum permissible velocities and Froude numbers for nonerosive flow are given in Section 4-2.3. Channel velocities and Froude numbers of flow can be controlled by providing drop structures or other energy dissipators, and to a limited extent by widening the channel thus decreasing flow depths or by increasing roughness and depth. If nonerosive flows cannot be attained, the channel can be lined with turf, asphaltic or portland cement concrete, and ungrouted or grouted rubble; for small ditches, half sections of pipe can be used, although care must be taken to prevent entrance and side erosion and undermining and ultimate displacement of individual sections. The choice of material depends on the velocity, depth, and turbulence involved; on the quantities, availability, and cost of materials; and on evaluation of their maintenance. In choosing the material, its effect on flow characteristics may be an important factor. Further, if an impervious lining is to be used, the need for subdrainage and end protection must be considered. Where a series of drop structures is proposed, care must be taken to avoid placing them too far apart, and to insure that they will not be undermined by scour at the foot of the overpour. The design of energy dissipators and means for scour protection are discussed subsequently.

4-4.2.5 Side slopes for unlined earth channels normally will be no steeper than 1 on 3 in order to minimize maintenance and permit machine mowing of grass and weeds. Side-slope steepness for paved channels will depend on the type of material used, method of placement, available space, accessibility requirements of maintenance equipment, and economy. Where portland-cement concrete is used for lining, space and overall economic considerations may dictate use of a rectangular channel even though wall forms are required. Rectangular channels are particularly desirable for conveyance of supercritical channel flow. Most channels, however, will convey subcritical flow and be of trapezoidal cross section. For relatively large earth channels involving levees, side slopes will depend primarily on stability of materials used.

4-4.2.6 An allowance for freeboard above the computed water surface for a channel is provided so that during a design storm the channel will not overflow for such reasons as minor variations in the hydrology or future development, minor superelevation of flow at curves, formation of waves, unexpected hydraulic performance, embankment settlement, and the like. The allowance normally ranges from 0.5 to 3 ft, depending on the type of construction, size of channel, consequences of overflow, and degree of safety desired. Requirements are greater for leveed channels than those wholly within the ground because of the need to guard against overtopping and breaching of embankments where failure would cause a sudden, highly damaging release of water. For areas upstream of culverts and bridges, the freeboard allowance should include possible rises in water-surface elevation due to occurrence of greater-then-design, runoff, unforeseen, entrance conditions or blockage by debris. In high-velocity flows, the effect of entrained air on flow depth should be considered.

4-4.2.7 Whenever water flows in a curved alignment, superelevation of the water surface will occur, the amount depending on the velocity and degree of curvature. Further, if the water entering a curve is flowing at supercritical velocity, a wave will be formed on the surface at the initial point of change in direction, and this wave will be reflected back and forth across the channel in zigzag fashion throughout the curve and for a long distance along the downstream tangent. Where such rises in water surface are less than 0.5 ft, they may normally be ignored because the regular channel freeboard allowance is ample to contain them. Where the rises are substantial, channel wall heights can be held to a minimum and corresponding economy achieved by superelevating the channel bottom to fit the water-surface superelevation, and the formation of transverse waves (in supercritical flow) can be effectively eliminated by providing a spiral for each end of the curve. In superelevating the channel, the transition from horizontal to full tilt is accomplished in the spiral. Figure 4-40 is a chart indicating formulas pertinent for use in computing design wall heights under typical superelevation conditions. For practical reasons, the spirals generally used are a modified type consisting of a series of circular arcs of equal length and decreasing radius. Experience has shown that if the curve is to be superelevated, the length of the spiral transition $L_t$ may be short, a safe minimum being given by the following equation.

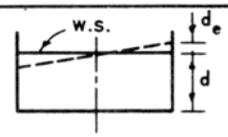
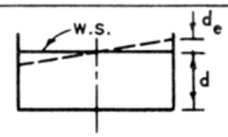
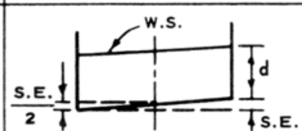
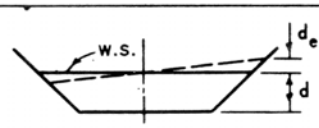
$$L_t = 15\frac{V^2T}{R_c g} \qquad \text{(eq. 4-23)}$$

If spirals are to be used in a non-superelevated channel, the minimum length of spiral $L_s$ required is:

$$L_s = \frac{1.82\,VT}{(gd)^{1/2}} \qquad \text{(eq. 4-24)}$$

The terms in both equations are defined in Section 4-4.15. The rise in water surface at the outside bank of a curved channel with a trapezoidal section can be estimated by the use of the preceding formulas.

**Figure 4-40. Superelevation Formulas**

| DEPTH > $d_c$ SUBCRITICAL FLOW | SECTION | DEPTH < $d_c$ SUPERCRITICAL FLOW |
|---|---|---|
| $d_e = \frac{V^2T}{2gR_c}$ <br> $Ht = d + F.B. + d_e$ |   HORIZONTAL INVERT NO SPIRAL | $d_e' = \frac{V^2T}{gR_c}$ <br> $Ht = d + F.B. + d_e$ |
| $d_e = \frac{V^2T}{2gR_c}$ <br> $Ht = d + F.B. + d_e$ |   HORIZONTAL INVERT SPIRAL TRANSITION | $d_e' = \frac{V^2T}{2gR_c}$ <br> $Ht = d + F.B. + d_e$ |
| $S.E. = \frac{V^2T}{gR_c}$ <br> $Ht = d + F.B.$ |   SUPERELEVATED INVERT SPIRAL TRANSITION | $S.E. = \frac{V^2T}{gR_c}$ <br> $Ht = d + F.B.$ |
| $d_e = \frac{V^2T}{2gR_c}$ <br> $Ht = d + F.B. + d_e$ |   HORIZONTAL INVERT WITH OR WITHOUT SPIRAL TRANSITION † | $d_e' = \frac{V^2T}{gR_c}$ <br> $Ht = d + F.B. + d_e$ |

LEGEND

- F.B. FREEBOARD IN FEET
- V VELOCITY IN FEET PER SECOND
- d DEPTH IN FEET
- $d_e$ RISE ABOVE d DUE TO CENTRIFUGAL FORCE IN FEET
- $d_e'$ RISE ABOVE d DUE TO CENTRIFUGAL FORCE AND TRANSVERSE WAVES IN FEET
- S.E. DIFFERENCE IN ELEVATION OF WATER SURFACE BETWEEN WALLS IN FEET
- T TOP WIDTH AT WATER SURFACE IN FEET
- $R_c$ RADIUS OF CURVATURE CENTER LINE OF CHANNEL IN FEET
- Ht WALL HEIGHT IN FEET
- g ACCELERATION OF GRAVITY IN FEET PER SECOND$^2$

NOTE: WHEN SUPERELEVATION IS LESS THAN 0.5 FOOT NEGLECT THE SUPERELEVATION OF THE INVERT, BUT LET Ht = DEPTH + FREEBOARD + SUPERELEVATION.

† IF MODEL STUDIES INDICATE THAT THE SPIRAL TRANSITION CURVE ELIMINATES THE TRANSVERSE WAVES FOR SUPERCRITICAL FLOW, USE $d_e$ INSTEAD OF $d_e'$.

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4-4.2.8 For most open channel confluences, proper design can be accomplished satisfactorily by computations based on the principle of conservation of momentum. If the channel flows are supercritical, excessive waves and turbulence are likely to occur unless a close balance of forces is achieved. In such confluences, minimum disturbances will result if the tributary inflow is made to enter the main channel in a direction parallel to the main flow, and if the design depth and velocity of the tributary inflow are made equal to those in the main channel. Further, even though minimum disturbances appear likely under such design conditions, it must be remembered that natural flood-flows are highly variable, both in magnitude and distribution. Since this variability leads to unbalanced forces and accompanying turbulence, a need may well exist for some additional wall height or freeboard allowance at and downstream from the confluence structure.

4-4.2.9 Side inflows to channels generally enter over the tops of the walls or in covered drains through the walls. If the main channel is earth, erosion protection frequently is required at (and perhaps opposite) the point of entry. If the sides of a channel through an erosible area are made of concrete or other durable materials and inflows are brought in over them, care must be taken to insure positive entry. There are two methods of conducting storm water into a concrete-lined channel. Entry of large flows over the top is provided by a spillway built as an integral part of the side slope while smaller flows are admitted to the channel by a conduit through the side slope. Gating of conduit is not required at this location because any ponding is brief and not damaging. Where covered tributary drains enter, examination must be made to see whether the water in the main channel, if full, would cause damaging backflooding of the tributary area, which would be more damaging than temporary stoppage of the tributary flow. If so, means for precluding backflow must be employed; this can often be accomplished by a flap gate at the drain outfall, and if positive closure is required, a slide gate can be used. If flow in the main channel is supercritical, the design of side inlet structures may require special provisions to minimize turbulence effects.

### 4-4.3 Bridges

4-4.3.1 A bridge is a structure, including supports, erected over a depression or an obstruction, such as water, a highway, or a railway, having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 ft between undercopings of abutments or spring lines of arches, or extreme ends of the openings for multiple boxes; it may include multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening.

4-4.3.2 Sufficient capacity will be provided to pass the runoff from the design storm determined in accordance with principles given in Section 2-9. Normally such capacity is provided entirely in the waterway beneath the bridge. Sometimes this is not practical, and it may be expedient to design one or both approach roadways as overflow sections for excess runoff. In such an event, it must be remembered that automobile traffic will be impeded, and will be stopped altogether if the overflow depth is much more than 6 in.

However, for the bridge proper, a waterway opening smaller than that required for 10-yr storm runoff will be justifiable.

4-4.3.3 In general, the lowest point of the bridge superstructure shall clear the design water surface by not less than 2 ft for average flow and trash conditions. This may be reduced to as little as 6 in. if the flow is quiet, with low velocity and little or no trash. More than 2 ft will be required if flows are rough or large-size floating trash is anticipated.

4-4.3.4 The bridge waterway will normally be aligned to result in the least obstruction to streamflow, except that for natural streams consideration will be given to realignment of the channel to avoid costly skews. To the maximum extent practicable, abutment wings will be aligned to improve flow conditions. If a bridge is to span an improved trapezoidal channel of considerable width, the need for overall economy may require consideration of the relative structural and hydraulic merits of on-bank abutments with or without piers and warped channel walls with vertical abutments.

4-4.3.5 To preclude failure by underscour, abutment and pier footings will usually be placed either to a depth of not less than 5 ft below the anticipated depth of scour, or on firm rock if such is encountered at a higher elevation. Large multispan structures crossing alluvial streams may require extensive pile foundations. To protect the channel against the increased velocities, turbulence, and eddies expected to occur locally, revetment of channel sides or bottom consisting of concrete, grouted rock, loose riprap, or sacked concrete will be placed as required. Criteria for selection of revetment are given in Chapter 5.

4-4.3.6 Where flow velocities are high, bridges should be of clear span, if at all practicable, in order to preclude serious problems attending debris lodgment and to minimize channel construction and maintenance costs.

4-4.3.7 It is important that storm runoff be controlled over as much of the contributing watershed as practicable. Diversion channels, terraces, check dams, and similar conventional soil conserving features will be installed, implemented, or improved to reduce velocities and prevent silting of channels and other downstream facilities. When practicable, unprotected soil surfaces within the drainage area will be planted with appropriate erosion-resisting plants. These parts of the drainage area which are located on private property or otherwise under control of others will be considered fully in the planning stages, and coordinated efforts will be taken to assure soil stabilization both upstream and downstream from the construction site.

4-4.3.8 Engineering criteria and design principles related to traffic, size, load capacity, materials, and structural requirements for highway and railroad bridges are given in Chapter 6, and in AASHTO Standard Specifications for Highway Bridges, design manuals of the different railroad companies, and recommended practices of AREA Manual for Railway Engineering.

### 4-4.4 Curb-and-Gutter Sections

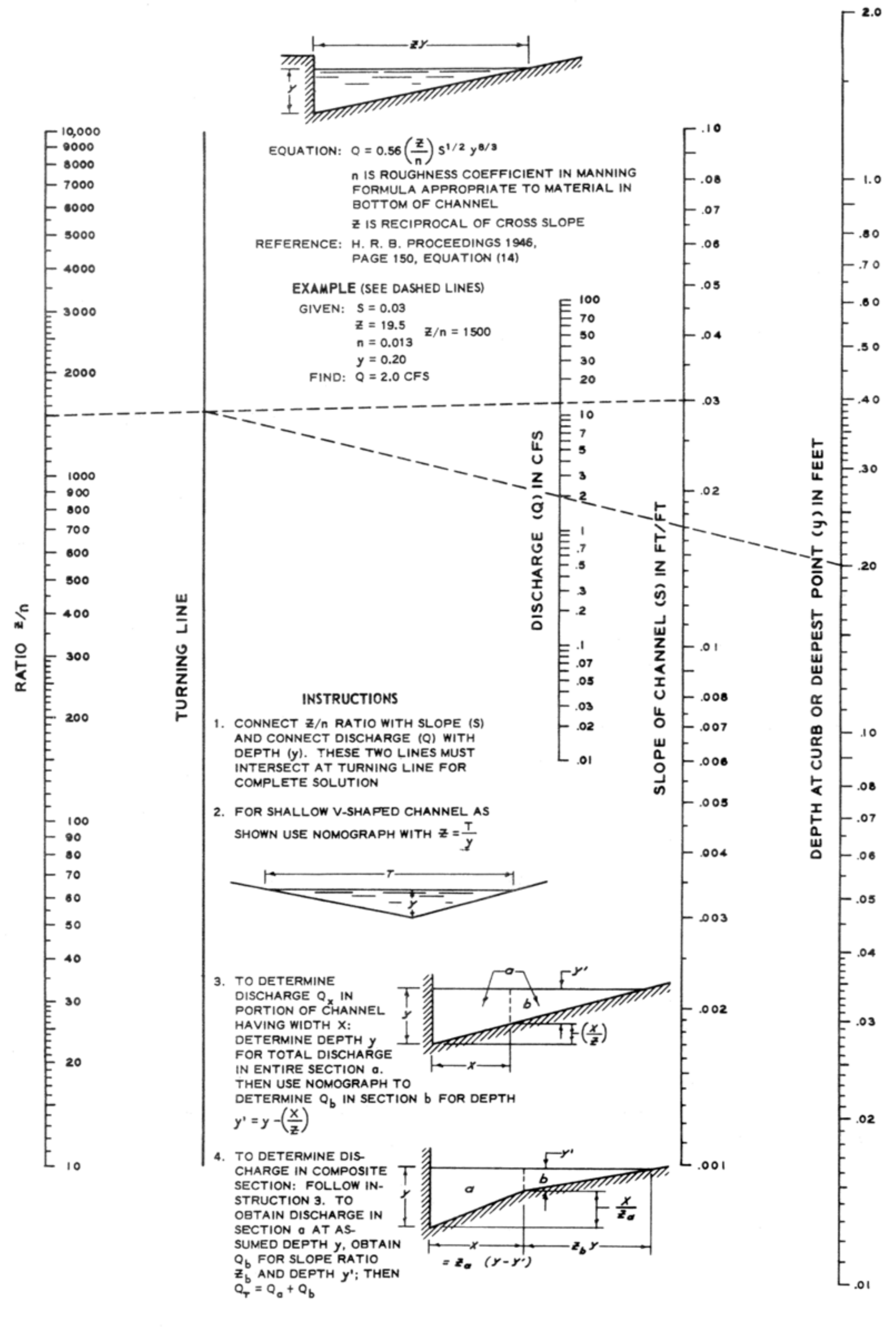
4-4.4.1 Precipitation which occurs upon city streets and adjacent areas must be rapidly and economically removed before it becomes a hazard to traffic. Water falling on the pavement surface itself is removed from the surface and concentrated in the gutters by the provision of an adequate crown. The surface channel formed by the curb and gutter must be designed to adequately convey the runoff from the pavement and adjacent areas to a suitable collection point. The capacity can be computed by using the nomograph for flow in a triangular channel, Figure 4-41. This figure can also be used for a battered curb face section, since the battering has negligible effect on the cross sectional area. Limited data from field tests with clear water show that a Manning's n of 0.013 is applicable for pavement. The n value should be raised when appreciable quantities of sediment are present. Figure 4-41 also applies to composite sections comprising two or more rates of cross slope.

4-4.4.2 Good roadway drainage practice requires the extensive use of curb-and-gutter sections in combination with spillway chutes or inlets and downspouts for adequate control of surface runoff, particularly in hilly and mountainous terrain where it is necessary to protect roadway embankments against formation of rivulets and channels by concentrated flows. Materials used in such construction include portland-cement concrete, asphaltic concrete, stone rubble, sod checks, and prefabricated concrete or metal sections. Typical of the latter are the entrance tapers and embankment protectors made by manufacturers of corrugated metal products. Downspouts as small as 8 in. in diameter may be used, unless a considerable trash problem exists, in which case a large size will be required. When frequent mowing is required, consideration will be given to the use of buried pipe in lieu of open paved channels or exposed pipe. The hydrologic and hydraulic design and the provision of outfall erosion protection will be accomplished in accordance with principles outlined for similar component structures discussed in this manual.

4-4.4.3 Curbs are used to deter vehicles from leaving the pavement at hazardous points as well as to control drainage. The two general classes of curbs are known as barrier and mountable and each has numerous types and detail designs. Barrier curbs are relatively high and steep faced and designed to inhibit and to at least discourage vehicles from leaving the roadway. They are considered undesirable on high-speed arterials. Mountable curbs are designed so that vehicles can cross them with varying degrees of ease.

4-4.4.4 Curbs, gutters, and storm drains will not be provided for drainage around tank-car or tank-truck unloading areas, tank-truck loading stands, and tanks in bulk-fuel-storage areas. Safety requires that fuel spillage must not be collected in storm or sanitary sewers. Safe disposal of fuel spillage of this nature may be facilitated by provision of ponded areas for drainage so that any fuel spilled can be removed from the water surface.

**Figure 4-41. Nomograph for Flow in Triangular Channels**

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### 4-4.5 **Culverts**

4-4.5.1 A drainage culvert is defined as any structure under the roadway with a clear opening of twenty feet or less measured along the center of the roadway. Culverts are generally of circular, oval, elliptical, arch, or box cross section and may be of single or multiple construction, the choice depending on available headroom and economy. Culvert materials for permanent-type installations include plain concrete, reinforced concrete, corrugated metal, asbestos cement, and clay. Concrete culverts may be either precast or cast in place, and corrugated metal culverts may have either annular or helical corrugations and be constructed of steel or aluminum. For the metal culverts, different kinds of coatings and linings are available for improvement of durability and hydraulic characteristics. The design of economical culverts involves consideration of many factors relating to requirements of hydrology, hydraulics, physical environment, imposed exterior loads, construction, and maintenance. With the design discharge and general layout determined, the design requires detailed consideration of such hydraulic factors as shape and slope of approach and exit channels, allowable head at entrance (and ponding capacity, if appreciable), tailwater levels, hydraulic and energy grade lines, and erosion potential. A selection from possible alternative designs may depend on practical considerations such as minimum acceptable size, available materials, local experience concerning corrosion and erosion, and construction and maintenance aspects. If two or more alternative designs involving competitive materials of equivalent merit appear to be about equal in estimated cost, plans will be developed to permit contractor's options or alternate bids, so that the least construction cost will result.

4-4.5.2 In most localities, culvert pipe is available in sizes to 36 in. diameter for plain concrete, 144 in. or larger for reinforced concrete, 120 in. for standard and helically corrugated metal (plain, polymer coated, bituminous coated, part paved, and fully paved interior), 36 in. for asbestos cement or clay, and 24 in. for corrugated polyethylene pipe. Concrete elliptical in sizes up to 116 H 180 in., concrete arch in sizes up to 107 H 169 in. and reinforced concrete box sections in sizes from 3 H 2 ft to 12 H 12 ft are available. Structural plate, corrugated metal pipe can be fabricated with diameters from 60 to 312 in. or more. Corrugated metal pipe arches are generally available in sizes to 142 by 91 in., and corrugated, structural plate pipe arches in spans to 40 ft. Reinforced concrete vertical oval (elliptical) pipe is available in sizes to 87 by 136 in., and horizontal oval (elliptical) pipe is available in sizes to 136 by 87 in. Designs for extra large sizes or for special shapes or structural requirements may be submitted by manufacturers for approval and fabrication. Short culverts under sidewalks (not entrances or driveways) may be as small as 8 in. in diameter if placed so as to be comparatively free from accumulation of debris or ice. Pipe diameters or pipe-arch rises should be not less than 18 in. A diameter or pipe-arch of not less than 24 in. should be used in areas where wind-blown materials such as weeds and sand may tend to block the waterway. Within the above ranges of sizes, structural requirements may limit the maximum size that can be used for a specific installation.

4-4.5.3 The selection of culvert materials to withstand deterioration from corrosion or abrasion will be based on the following considerations:

4-4.5.3.1 Rigid culvert is preferable where industrial wastes, spilled petroleum products, or other substances harmful to bituminous paving and coating in corrugated metal pipe are apt to be present. Concrete pipe generally should not be used where soil is more acidic than pH 5.5 or where the fluid carried has a pH less than 5.5 or higher than 9.0. Polyethylene pipe is unaffected by acidic or alkaline soil conditions. Concrete pipe can be engineered to perform very satisfactorily in the more severe acidic or alkaline environments. Type II or Type V cements should be used where soils and/or water have a moderate or high sulfate concentration, respectively; criteria are given in Federal Specification SS-C-1960/GEN. High-density concrete pipe is recommended when the culvert will be subject to tidal drainage and salt-water spray. Where highly corrosive substances are to be carried, the resistive qualities of vitrified clay pipe or plastic lined concrete pipe should be considered.

4-4.5.3.2 Flexible culvert such as corrugated-steel pipe will be galvanized and generally will be bituminous coated for permanent installations. Bituminous coating or polymeric coating is recommended for corrugated steel pipe subjected to stagnant water; where dense decaying vegetation is present to form organic acids; where there is continuous wetness or continuous flow; and in well-drained, normally dry, alkali soils. The polymeric coated pipe is not damaged by spilled petroleum products or industrial wastes. Asbestos-fiber treatment with bituminous coated or a polymeric coated pipe is recommended for corrugated-steel pipe subjected to highly corrosive soils, cinder fills, mine drainage, tidal drainage, salt-water spray, certain industrial wastes, and other severely corrosive conditions; or where extra-long life is desirable. Cathodic protection is rarely required for corrugated-steel-pipe installations; in some instances, its use may be justified. Corrugated-aluminum-alloy pipe, fabricated in all of the shapes and sizes of the more familiar corrugated-steel pipe, evidences corrosion resistance in clear granular materials even when subjected to sea water. Corrugated-aluminum pipe will not be installed in soils that are highly acid (pH less than 5) or alkaline (pH greater than 9), or in metallic contact with other metals or metallic deposits, or where known corrosive conditions are present or where bacterial corrosion is known to exist. Similarly, this type pipe will not be installed in material classified as OH or OL according to the Unified Soil Classification System as presented in MIL-STD 619. Although bituminous coatings can be applied to aluminum-alloy pipe, such coatings do not afford adequate protection (bituminous adhesion is poor) under the aforementioned corrosive conditions. Suitable protective coatings for aluminum alloy have been developed, but are not economically feasible for culverts or storm drains. For flow carrying debris and abrasives at moderate to high velocity, paved-invert pipe may be appropriate. When protection from both corrosion and abrasion is required, smooth-interior corrugated-steel pipe may be desirable, since in addition to providing the desired protection, improved hydraulic efficiency of the pipe will usually allow a reduction in pipe size. When considering a coating for use, performance data from users in the area can be helpful. Performance history indicates various successes or failures of coatings and their probable cause and are available from local highway departments.

4-4.5.4 The capacity of a culvert is determined by its ability to admit, convey, and discharge water under specified conditions of potential and kinetic energy upstream and downstream. The hydraulic design of a culvert for a specified design discharge involves selection of a type and size, determination of the position of hydraulic control, and hydraulic computations to determine whether acceptable headwater depths and outfall conditions will result. In considering what degree of detailed refinement is appropriate in selecting culvert sizes, the relative accuracy of the estimated design discharge should be taken into account. Hydraulic computations will be carried out by standard methods based on pressure, energy, momentum, and loss considerations. Appropriate formulas, coefficients, and charts for culvert design are given in Section 4-4.5.9.

4-4.5.5 Rounding or beveling the entrance in any way will increase the capacity of a culvert for every design condition. Some degree of entrance improvement should always be considered for incorporation in design. A headwall will improve entrance flow over that of a projecting culvert. They are particularly desirable as a cutoff to prevent saturation sloughing and/or erosion of the embankment. Provisions for drainage should be made over the center of the headwall to prevent scouring along the sides of the walls. A mitered entrance conforming to the fill slope produces little if any improvement in efficiency over that of the straight, sharp-edged, projecting inlet, and may be structurally unsafe due to uplift forces. Both types of inlets tend to inhibit the culvert from flowing full when the inlet is submerged. The most efficient entrances incorporate such geometric features as elliptical arcs, circular arcs, tapers, and parabolic drop-down curves. In general elaborate inlet designs for culverts are justifiable only in unusual circumstances.

4-4.5.6 Outlets and endwalls must be protected against undermining, bottom scour, damaging lateral erosion and degradation of the downstream channel. The presence of tailwater higher than the culvert crown will affect the culvert performance and may possibly require protection of the adjacent embankment against wave or eddy scour. Endwalls (outfall headwalls) and wingwalls should be used where practical, and wingwalls should flare one on eight from one diameter width to that required for the formation of a hydraulic jump and the establishment of a Froude number in the exit channel that will insure stability. Two general types of channel instability can develop downstream of a culvert. The conditions are known as either gully scour or a localized erosion referred to as a scour hole. Gully scour is to be expected when the Froude number of flow in the channel exceeds that required for stability. Erosion of this type maybe of considerable extent depending upon the location of the stable channel section relative to that of the outlet in both the vertical and downstream directions. A scour hole can be expected downstream of an outlet even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. See Chapter 5 for additional information on erosion protection.

4-4.5.7 In the design and construction of any drainage system it is necessary to consider the minimum and maximum earth cover allowable in the underground conduits to be placed under both flexible and rigid pavements. Minimum-maximum cover requirements for asbestos-cement pipe, corrugated-steel pipe, reinforced concrete culverts and storm drains, standard strength clay and non-reinforced concrete pipe are given in Section 4-4.9. The cover depths recommended are valid for average bedding and backfill conditions. Deviations from these conditions may result in significant minimum cover requirements.

4-4.5.8 Infiltration of fine-grained soils into drainage pipelines through joint openings is one of the major causes of ineffective drainage facilities. This is particularly a problem along pipes on relatively steep slopes such as those encountered with broken back culverts. Infiltration of backfill and subgrade material can be controlled by watertight flexible joint materials in rigid pipe and with watertight coupling bands in flexible pipe. The results of laboratory research concerning soil infiltration through pipe joints and the effectiveness of gasketing tapes for waterproofing joints and seams are available.

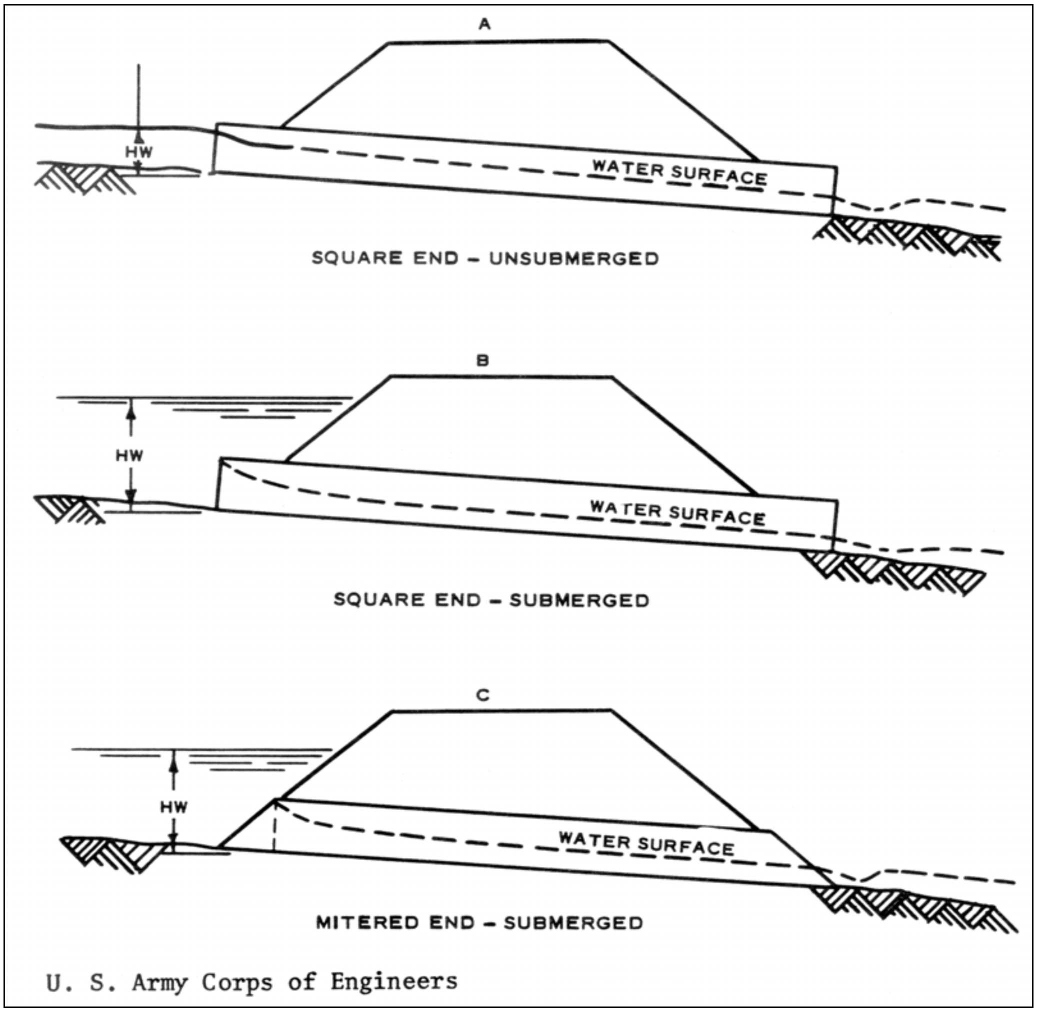
4-4.5.9 **Hydraulic design data for culverts**

4-4.5.9.1 **General**. This section presents diagrams, charts, coefficients, and related information useful in design of culverts. The information largely has been obtained from the U.S. Department of Transportation, Federal Highway Administration (formerly, Bureau of Public Roads), supplemented, or modified as appropriate by information from various other sources and as required for consistency with design practice of the Corps of Engineers.

4-4.5.9.2 Laboratory tests and field observations show two major types of culvert flow: flow with inlet control and flow with outlet control. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness, and length of the culvert barrel. The type of flow or the location of the control is dependent on the quantity of flow, roughness of the culvert barrel, type of inlet, flow pattern in the approach channel, and other factors. In some instances the flow control changes with varying discharges, and occasionally the control fluctuates from inlet control to outlet control and vice versa for the same discharge. Thus, the design of culverts should consider both types of flow and should be based on the more adverse flow condition anticipated.

4-4.5.10 **Inlet control**. The discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HIV) and the entrance geometry, including the area, slope, and type of inlet edge. Types of inlet-controlled flow for unsubmerged and submerged entrances are shown at A and B in Figure 4-42. A mitered entrance (Figure 4-42) produces little if any improvement in efficiency over that of the straight, sharp-edged, projecting inlet. Both types of inlets tend to inhibit the culvert from flowing full when the inlet is submerged. With inlet control the roughness and length of the culvert barrel and outlet conditions (including depths of tailwater) are not factors in determining culvert capacity. The effect of the barrel slope on inlet-control flow in conventional culverts is negligible. Nomography for determining culvert capacity for inlet control were developed by the Division of Hydraulic Research, Bureau of Public Roads. (See Hydraulics of Bridge Waterways.) These nomography (Figures 4-43 through 4-50) give headwater-discharge relations for most conventional culverts flowing with inlet control.

**Figure 4-42. Inlet Control**




4-4.5.11 **Outlet control**

4-4.5.11.1 Culverts flowing with outlet control can flow with the culvert barrel full or partially full for part of the barrel length or for all of it (Figure 4-51). If the entire barrel is filled (both cross section and length) with water, the culvert is said to be in full flow or flowing full (Figure 4-51A and B). The other two common types of outlet-control flow are shown in Figure 4-51C and D. The procedure given in this appendix for outlet-control flow does not give an exact solution for a free-water-surface condition throughout the barrel length shown in Figure 4-51D. An approximate solution is given for this case when the headwater, HW, is equal to or greater than 0.75D, where D is the height of the culvert barrel. The head, H, required to pass a given quantity of water through a culvert flowing full with control at the outlet is made up of three major parts.

**Figure 4-43. Headwater Depth for Concrete Pipe Culverts with Inlet Control**

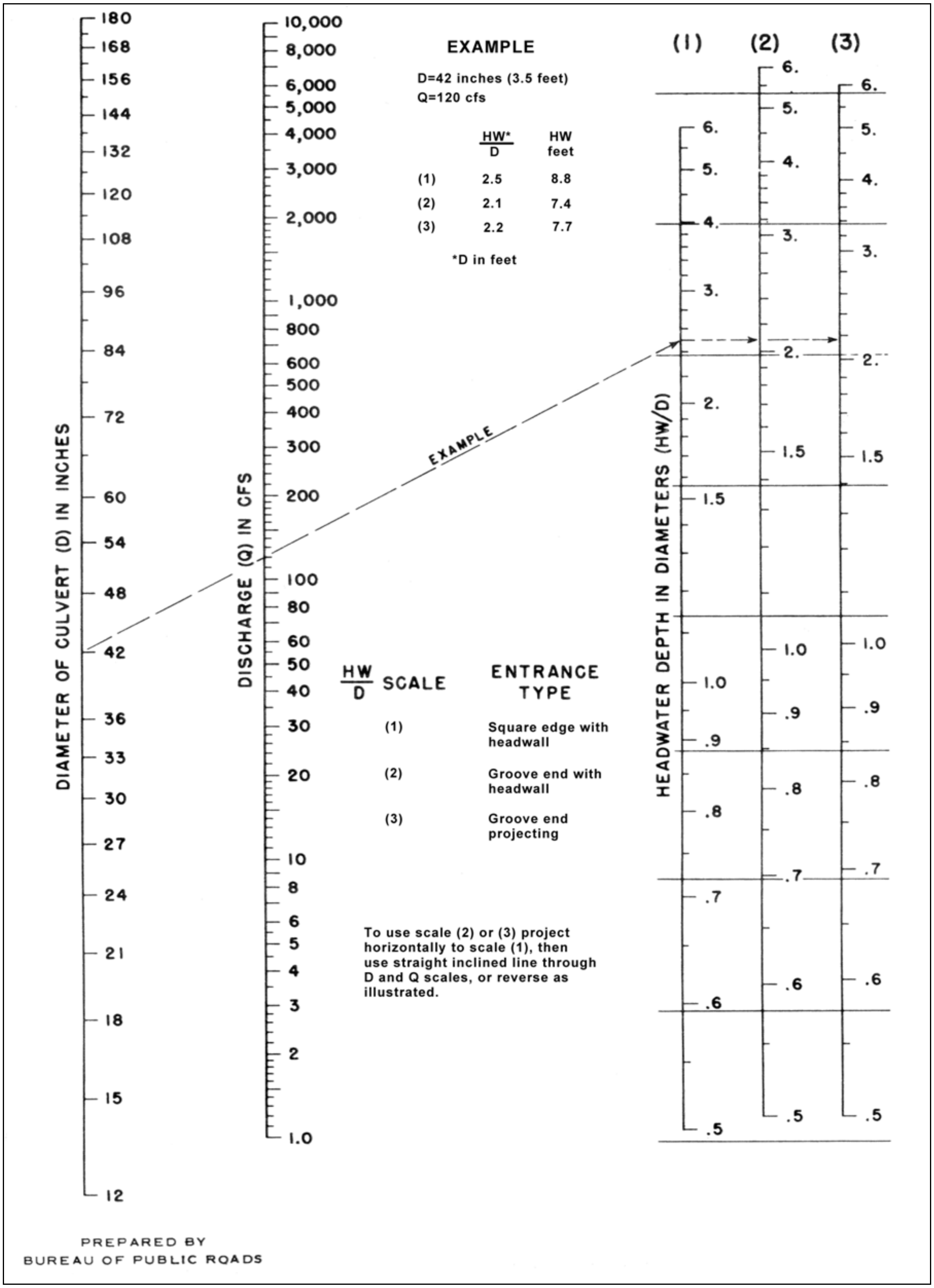


**Figure 4-44. Headwater Depth for Oval Concrete Pipe Culverts Long Axis Vertical with Inlet Control**

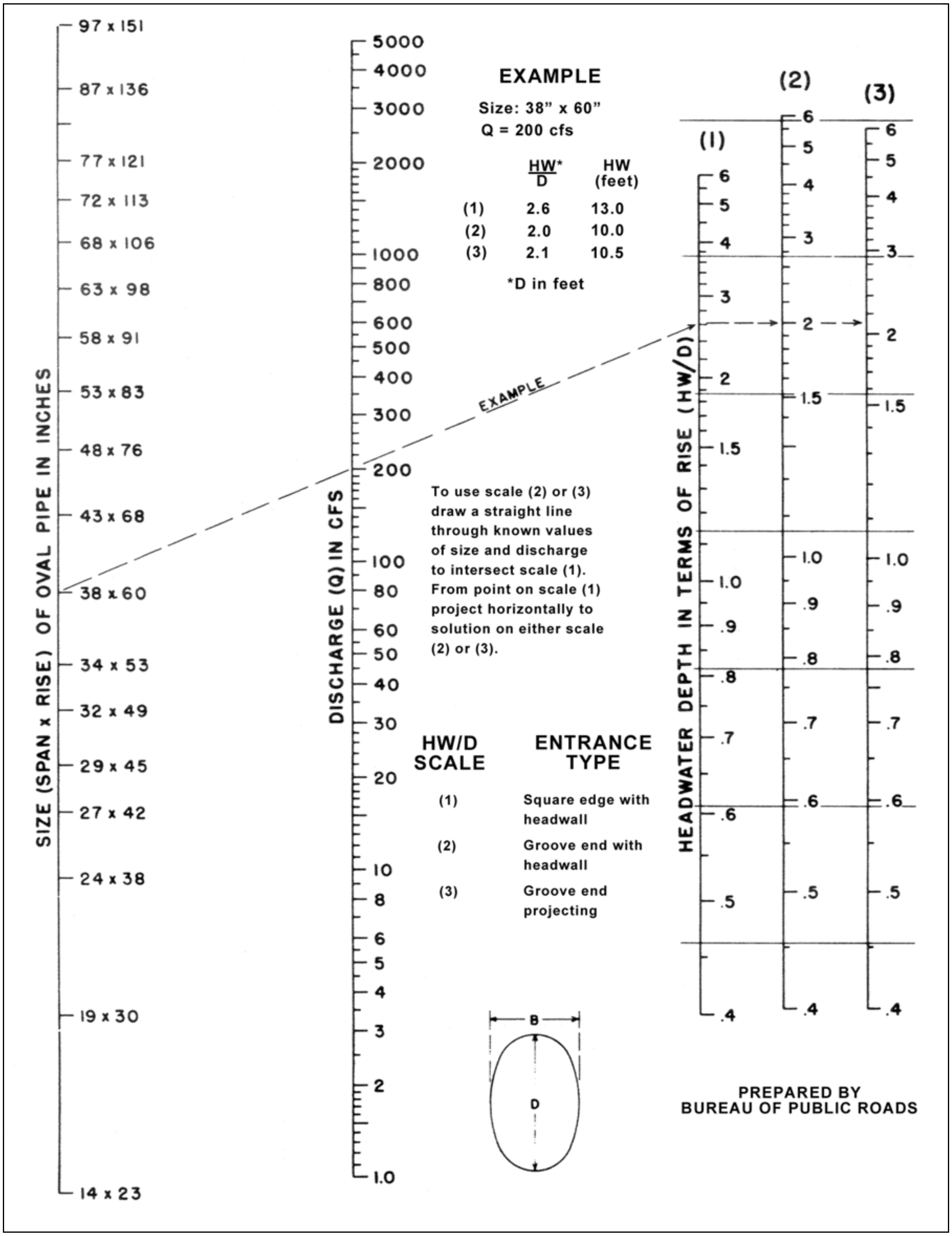


**Figure 4-45. Headwater Depth for Oval Concrete Pipe Culverts Long Axis Horizontal with Inlet Control**

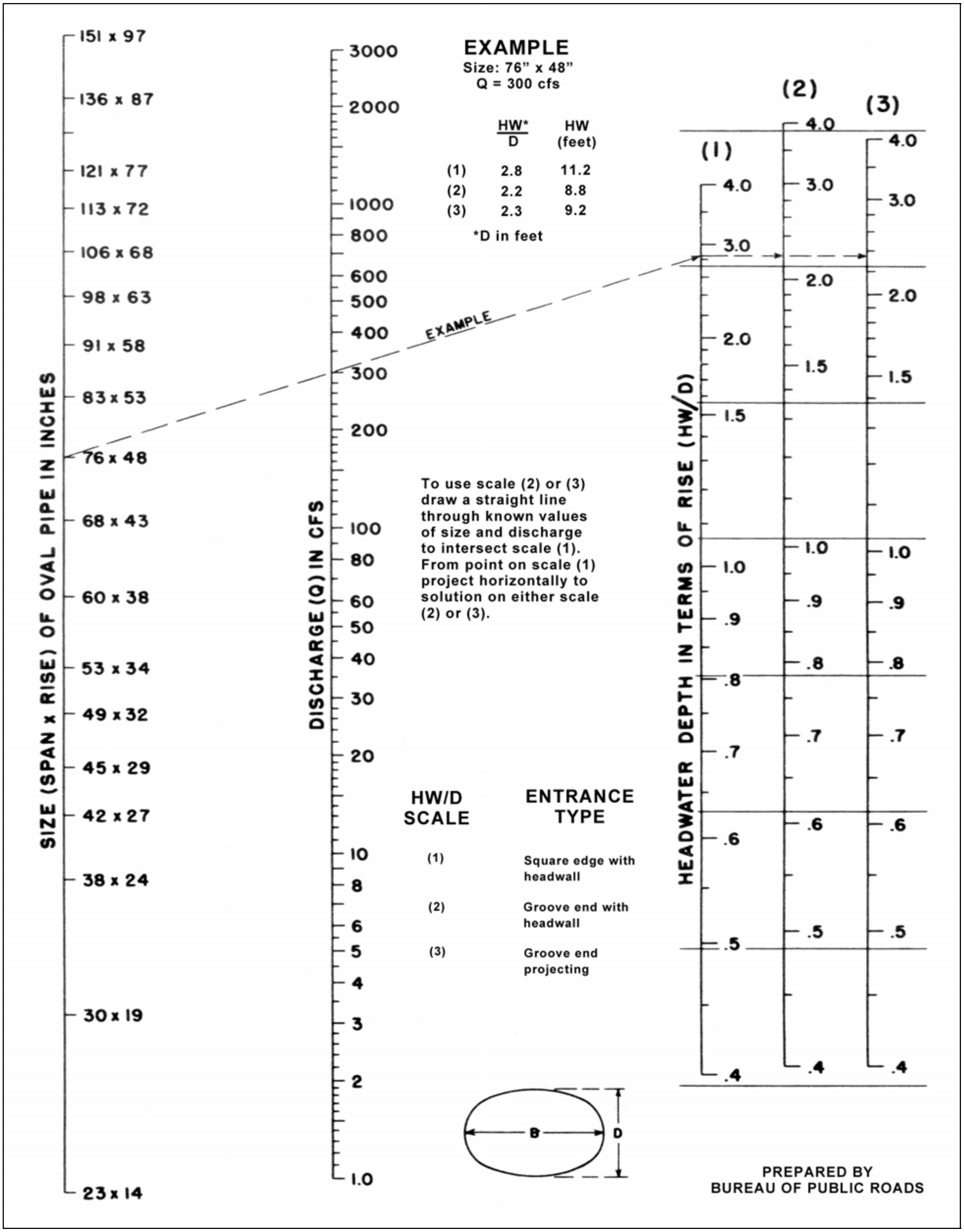


**Figure 4-46. Headwater Depth for Corrugated Metal Pipe Culverts with Inlet Control**

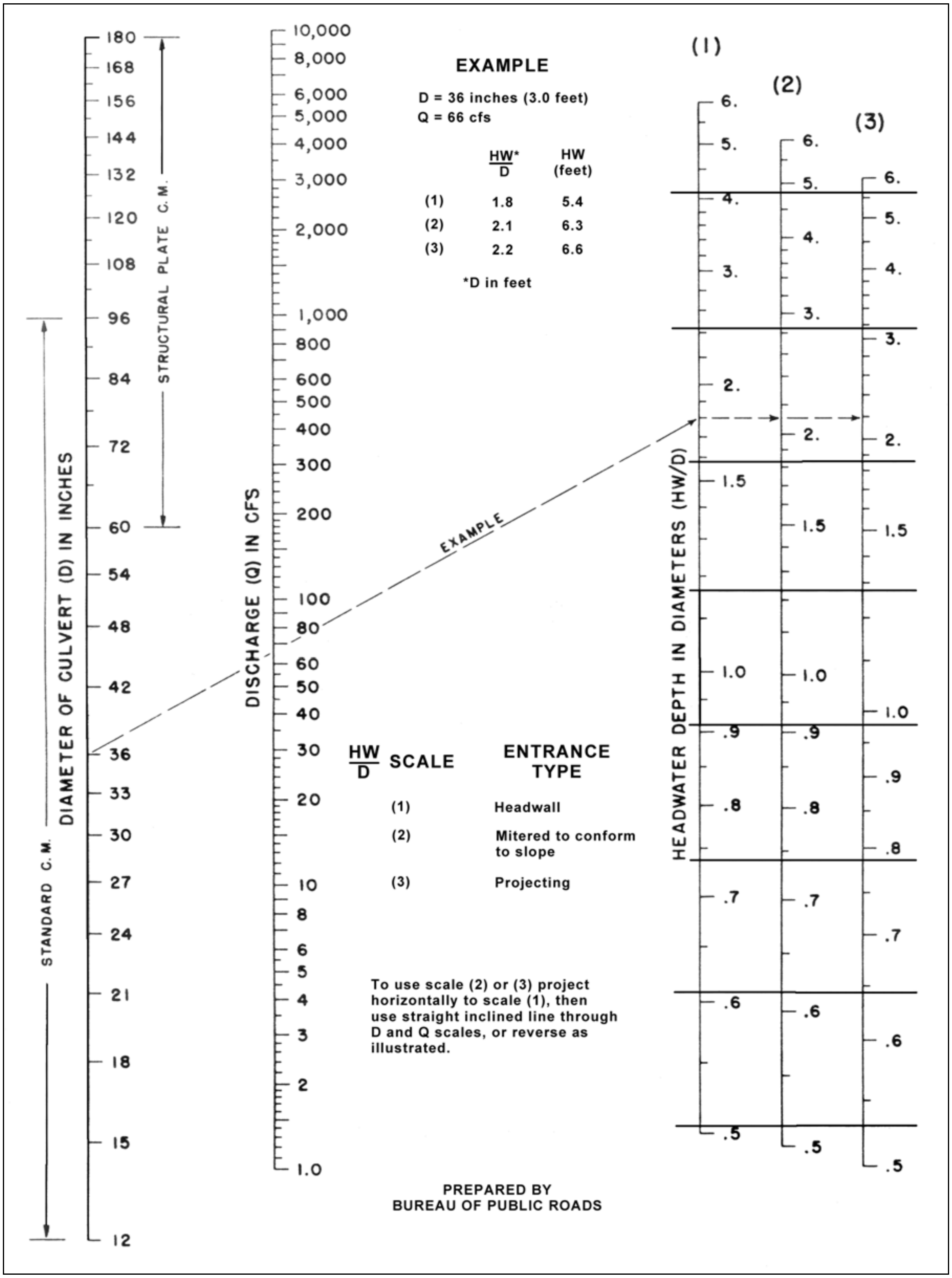


**Figure 4-47. Headwater Depth for Structural Plate and Standard Corrugated Metal Pipe-Arch Culverts with Inlet Control**

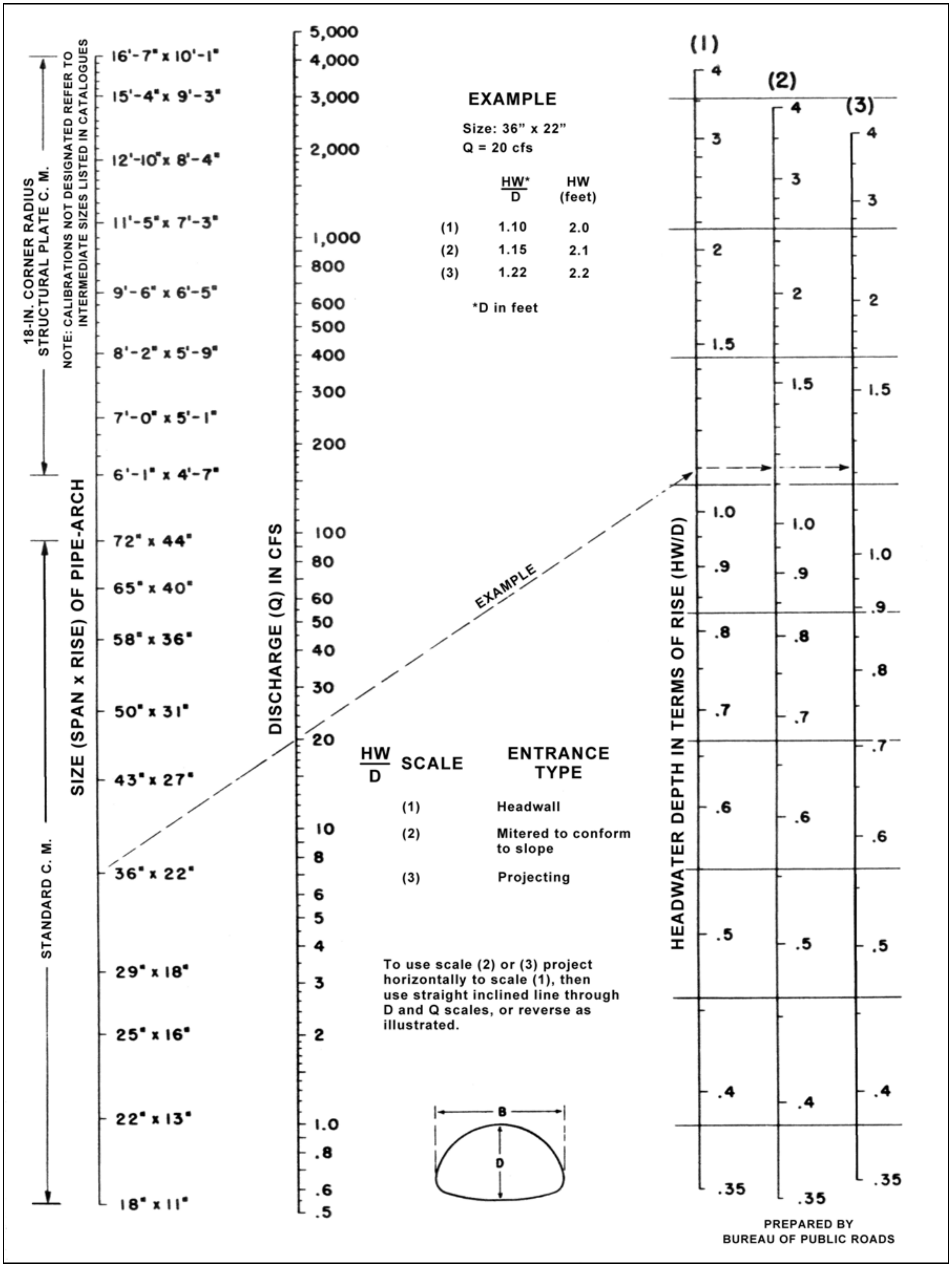


**Figure 4-48. Headwater Depth for Box Culverts with Inlet Control**

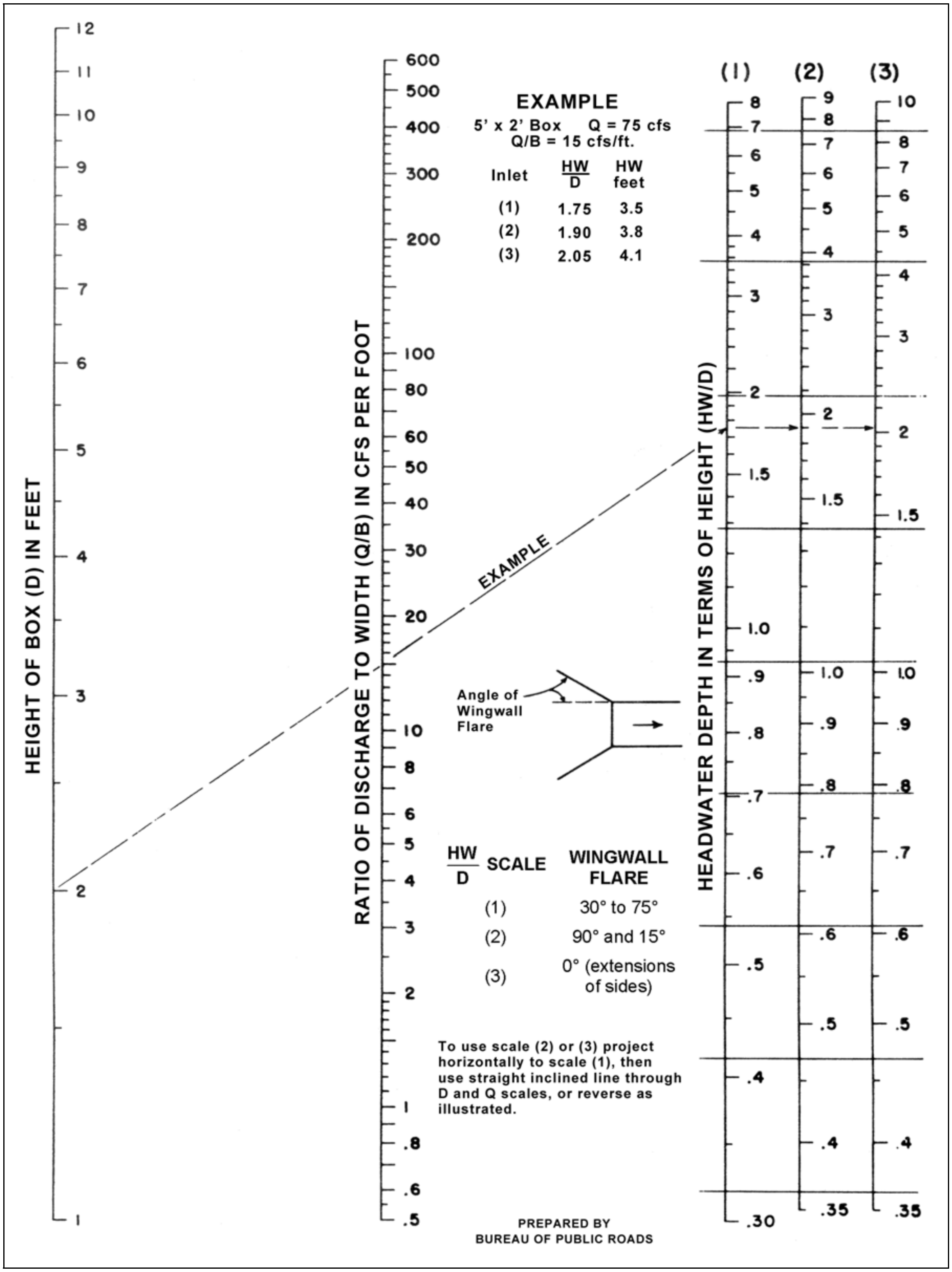


**Figure 4-49. Headwater Depth for Corrugated Metal Pipe Culverts with Tapered Inlet-Inlet Control**

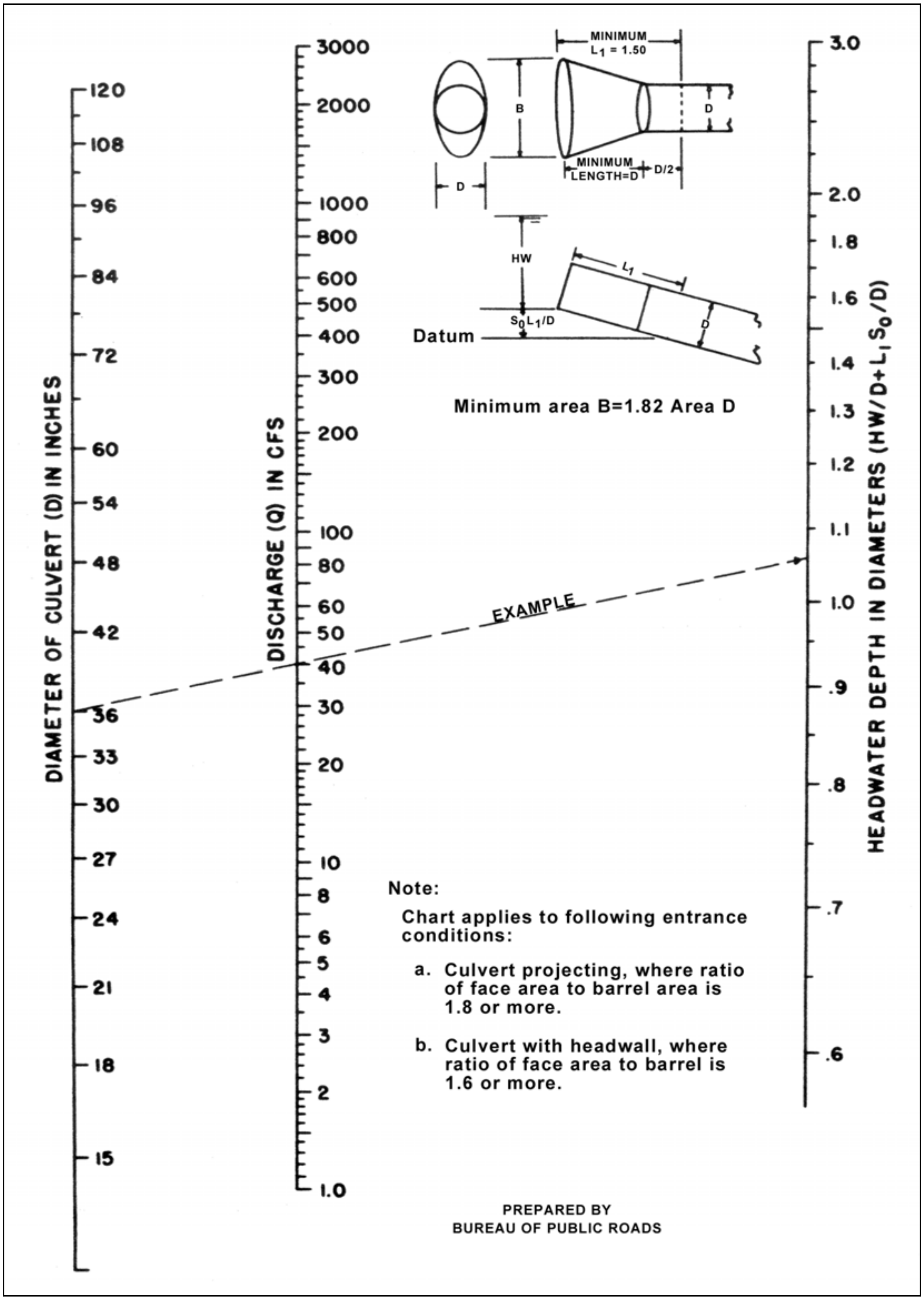


**Figure 4-50. Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control**

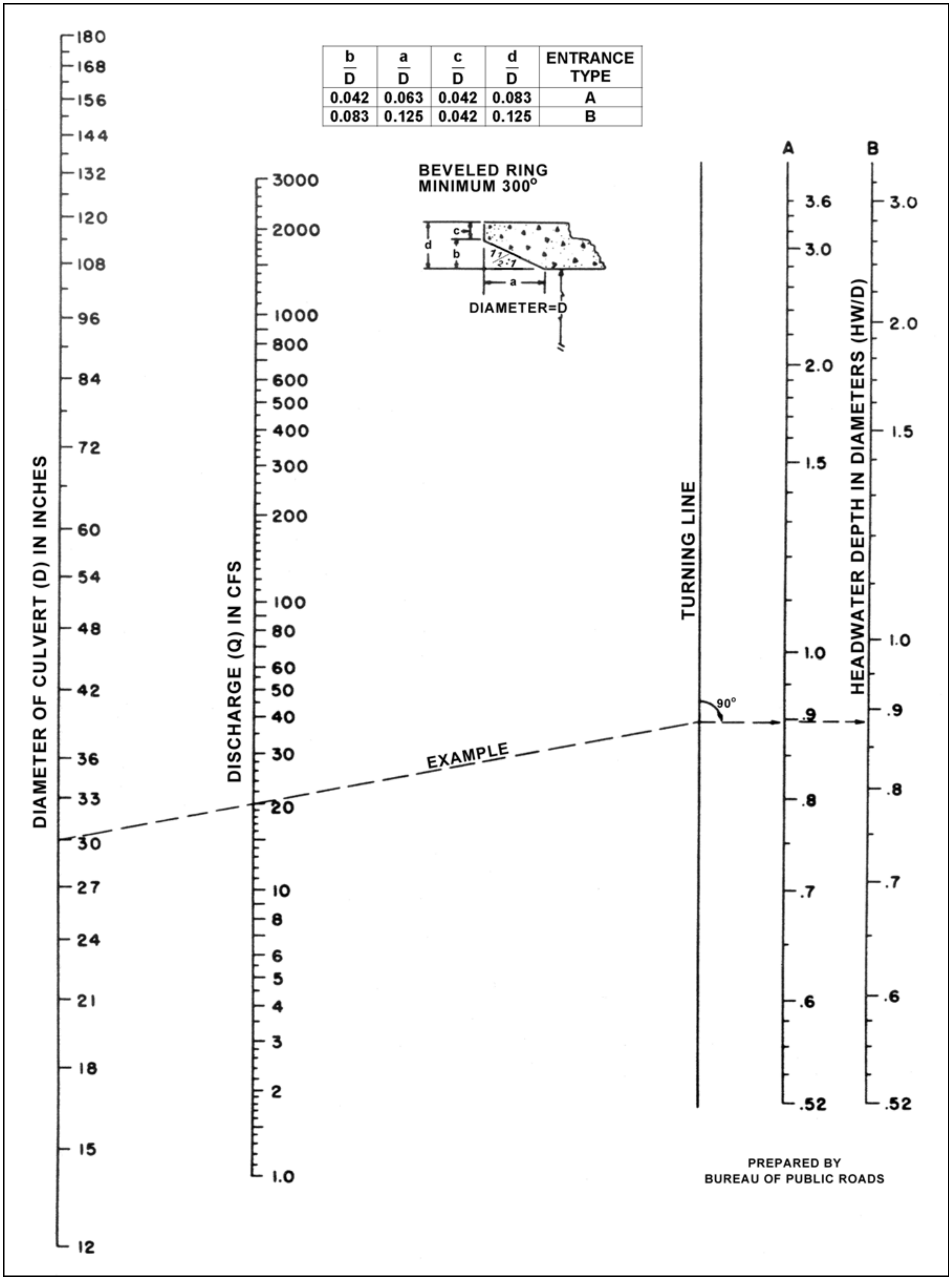
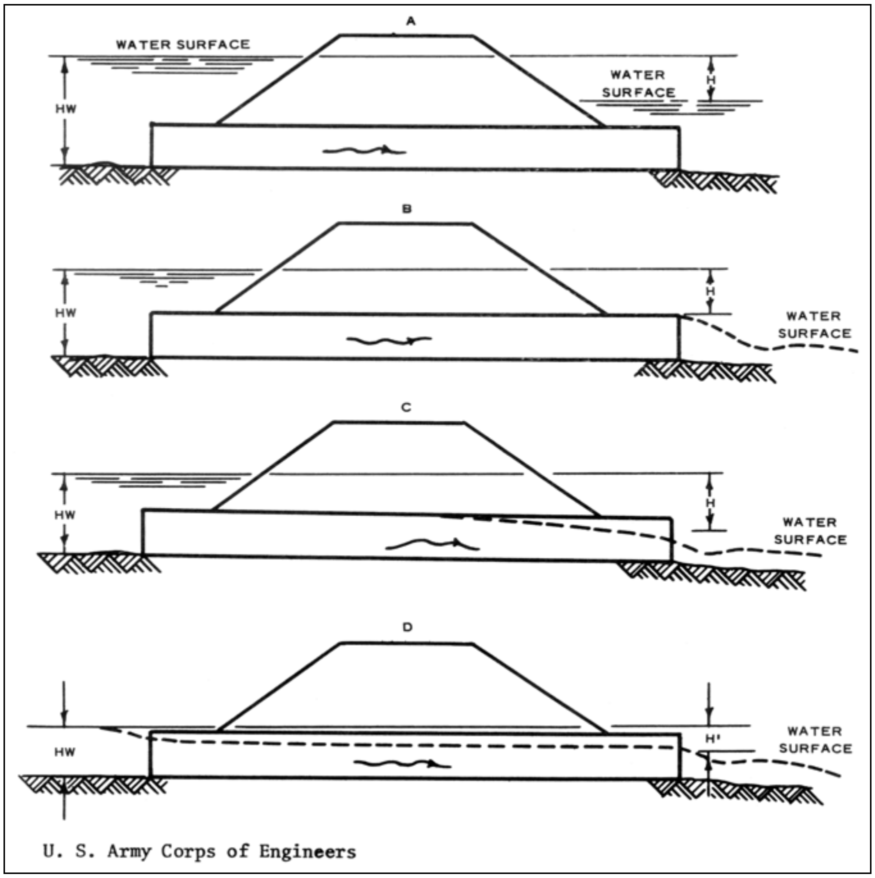


**Figure 4-51. Outlet Control**




These three parts are usually expressed in feet of water and include a velocity head, an entrance loss, and a friction loss. The velocity head (the kinetic energy of the water in the culvert barrel) equals $\frac{V^2}{2g}$. The entrance loss varies with the type or design of the culvert inlet and is expressed as a coefficient times the velocity head or $K_e \frac{V^2}{2g}$. Values of $K_e$ for various types of culvert entrances are given in Table 4-4. The friction loss, $H_f$, is the energy required to overcome the roughness of the culvert barrel and is usually expressed in terms of Manning's n and the following expression:

$$H_f = \left(\frac{29n^2L}{R^{1.333}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(eq. 4-25)}$$

Variables in the equation are defined in Section 4-4.15.

Adding the three terms and simplifying, yields for full pipe, outlet control flow the following expression:

$$H = \left(1 + K_e + \frac{29n^2L}{R^{1.333}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(eq. 4-26)}$$

This equation can be solved readily by the use of the full-flow nomography, Figures 4-52 through 4-58. The equations shown on these nomography are the same as Equation 1 expressed in a different form. Each nomograph is drawn for a single value of n as noted in the respective figure. These nomography may be used for other values of n by modifying the culvert length as directed in Section 4-4.5.14 of this chapter, which describes use of the outlet-control nomography. The value of H must be measured from some “control” elevation at the outlet which is dependent on the rate of discharge or the elevation of the water surface of the tailwater. For simplicity, a value $h_o$ is used as the distance in feet from the culvert invert (flow line) at the outlet to the control elevation. The following equation is used to compute headwater in reference to the inlet invert:

$$HW = h_o + H - LS_o \qquad \text{(eq. 4-27)}$$

4-4.5.11.2 **Tailwater elevation at or above the top of the culvert barrel outlet (Figure 4-51A)**. The tailwater (TW) depth is equal to $h_o$, and the relation of headwater to other terms in Equation 4-27 is illustrated in Figure 4-59.

4-4.5.11.3 **Tailwater elevation below the top or crown of the culvert barrel outlet.** Figure 4-513B, C, and D are three common types of flow for outlet control with this low tailwater condition. In these cases $h_o$ is found by comparing two values, TW depth in the outlet channel and $\frac{d_c + D}{2}$, and setting $h_o$ equal to the larger value. The fraction $\frac{d_c + D}{2}$ is a simplified mean of computing $h_o$ when the tailwater is low and the discharge does not fill the culvert barrel at the outlet. In this fraction, dc is critical depth as determined from Figures 4-61 through 4-66 and D is the culvert height. The value of D should never exceed D, making the upper limit of this fraction equal to D. Figure 4-62 shows the terms of Equation 4-27 for the cases discussed above. Equation 4-27 gives accurate answers if the culvert flows full for a part of the barrel length as illustrated by Figure 4-66. This condition of flow will exist if the headwater, as determined by Equation 4-27, is equal to or greater than the quantity:

$$HW \geq D + (1 + K_e)\frac{V^2}{2g} \qquad \text{(eq. 4-28)}$$

**Table 4-4. Entrance Loss Coefficients, Outlet Control, Full or Partly Full**

**Entrance Head Loss,** $H_e = K_e \frac{V^2}{2g}$

| Type 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 |
| Rounded (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° or 45° bevels | 0.2 |
| Side- or sloped-tapered inlet | 0.2 |
| Pipe, or Pipe-Arch, Corrugated Metal | |
| Projecting from fill (no headwall) | 0.9 |
| Headwall or headwall and wingwalls, square-edge | 0.5 |
| Mitered to conform to fill slope, paved or unpaved slope | 0.7 |
| *End section conforming to fill slope | 0.5 |
| Beveled edges, 33.7° or 45° bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| Box, Reinforced Concrete | |
| Headwall parallel to embankment (no wingwalls) | |
| Square-edged on 3 edges | 0.5 |
| Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides | 0.2 |
| Wingwalls at 30° to 75° to barrel | |
| Square-edged at crown | 0.4 |
| Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge | 0.2 |
| Wingwall at 10° to 25° barrel | |
| Square-edged at crown | 0.7 |
| Wingwalls parallel (extension of sides) | |
| Square-edged at crown | 0.7 |
| Side- or slope-tapered inlet | 0.2 |
| * Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet. | |

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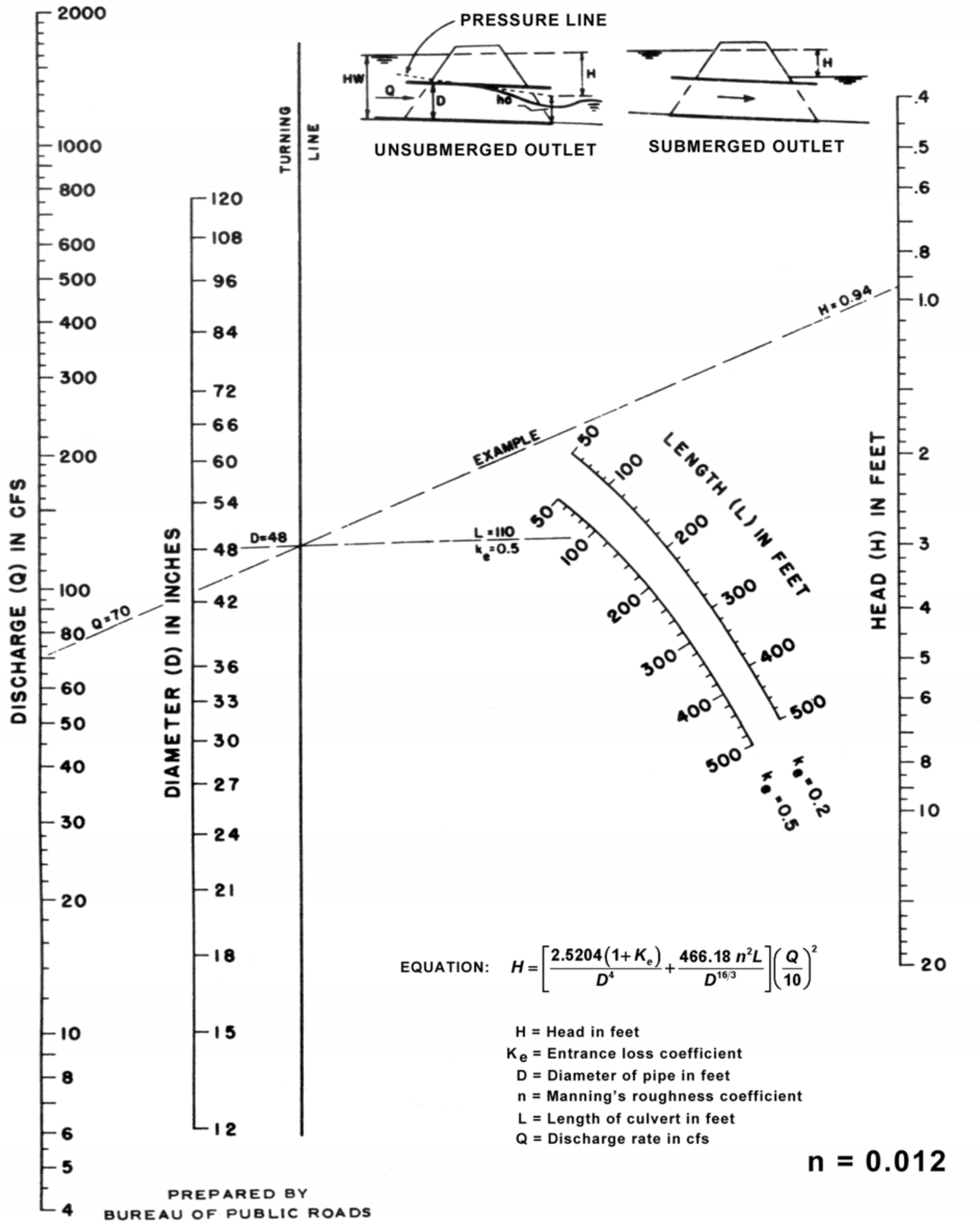
**Figure 4-52. Head for Circular Pipe Culverts Flowing Full, n = 0.012**




EQUATION: $$H = \left[\frac{2.5204(1+K_e)}{D^4} + \frac{466.18\, n^2 L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

H = Head in feet
$K_e$ = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Discharge rate in cfs

**n = 0.012**

**Figure 4-53. Head for Oval Circular Pipe Culverts Long Axis Horizontal or Vertical Flowing Full, n = 0.012**

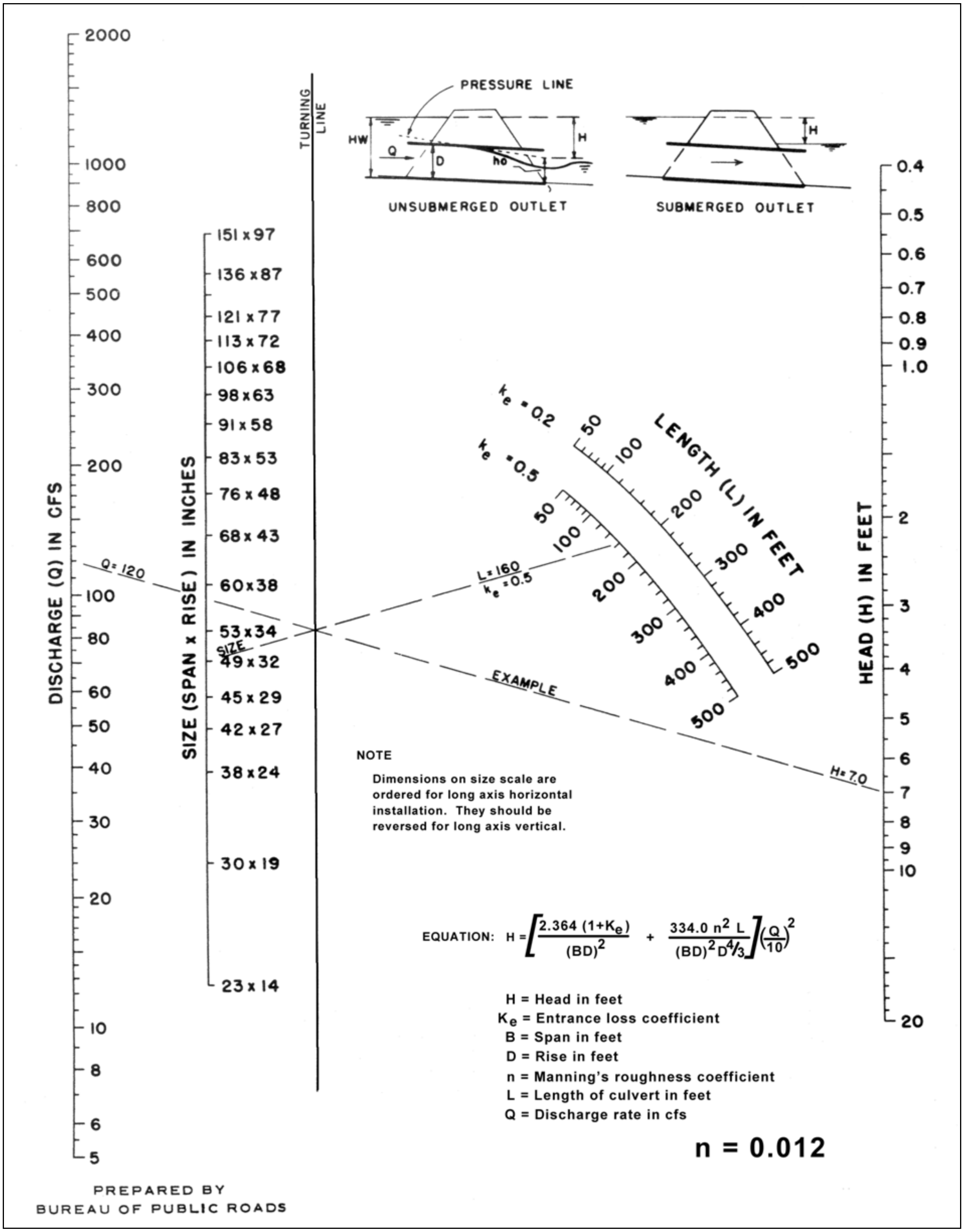


**Figure 4-54. Head for Circular Pipe Culverts Flowing Full, n = 0.024**

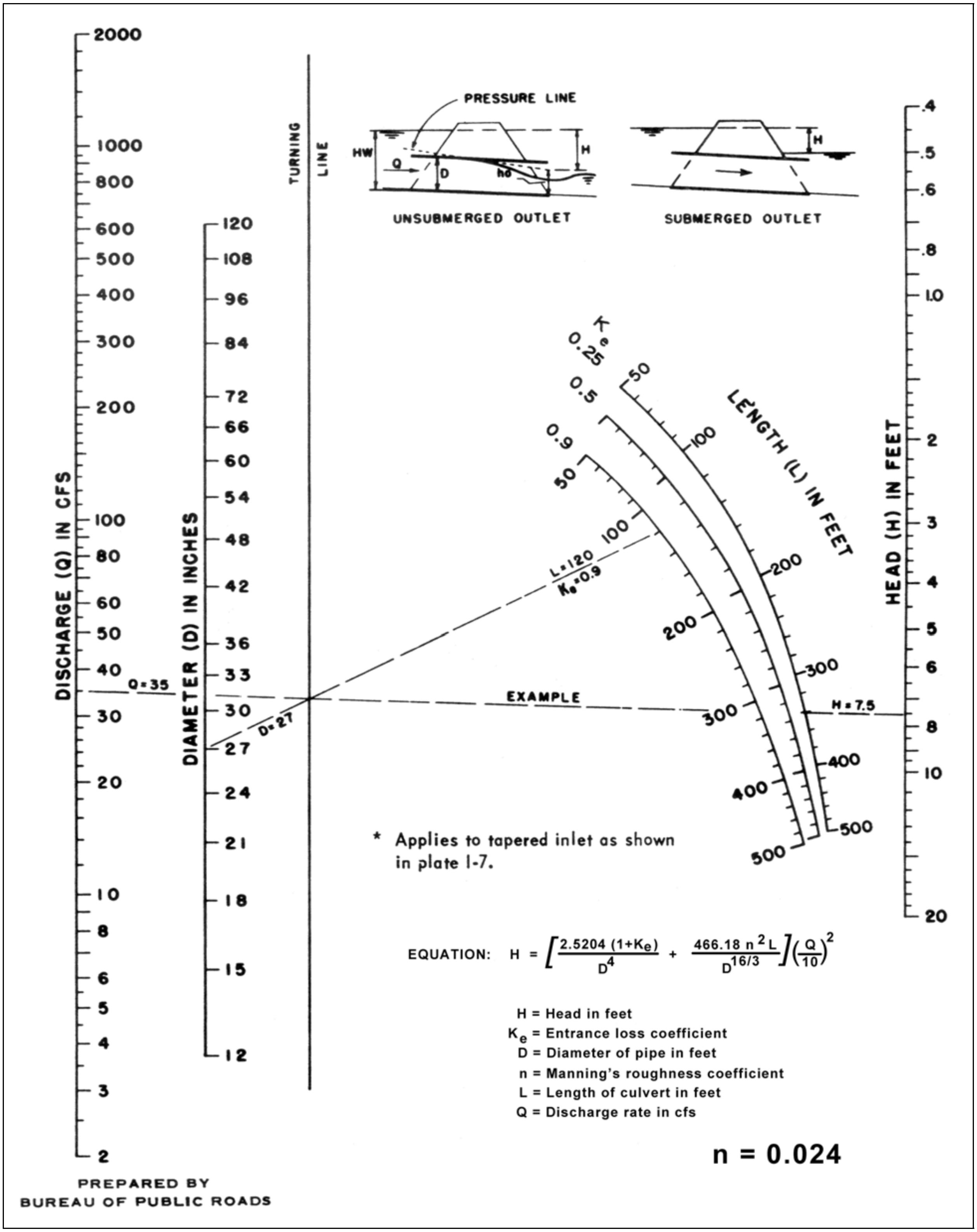


**Figure 4-55. Head for Circular Pipe Culverts Flowing Full, n = 0.0328 to 0.0302**

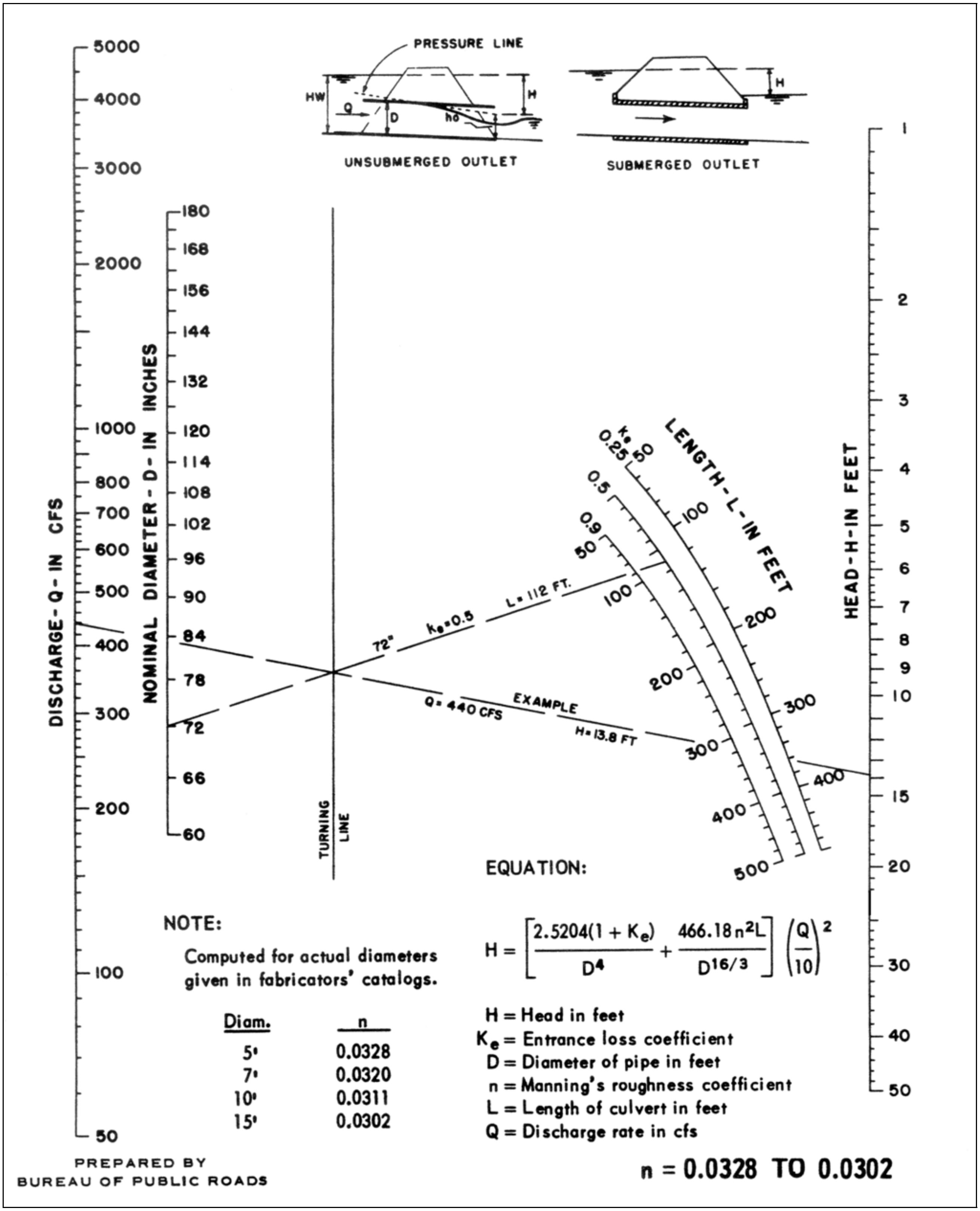


**Figure 4-56. Head for Standard Corrugated Metal Pipe-Arch Culverts Flowing Full, n = 0.024**

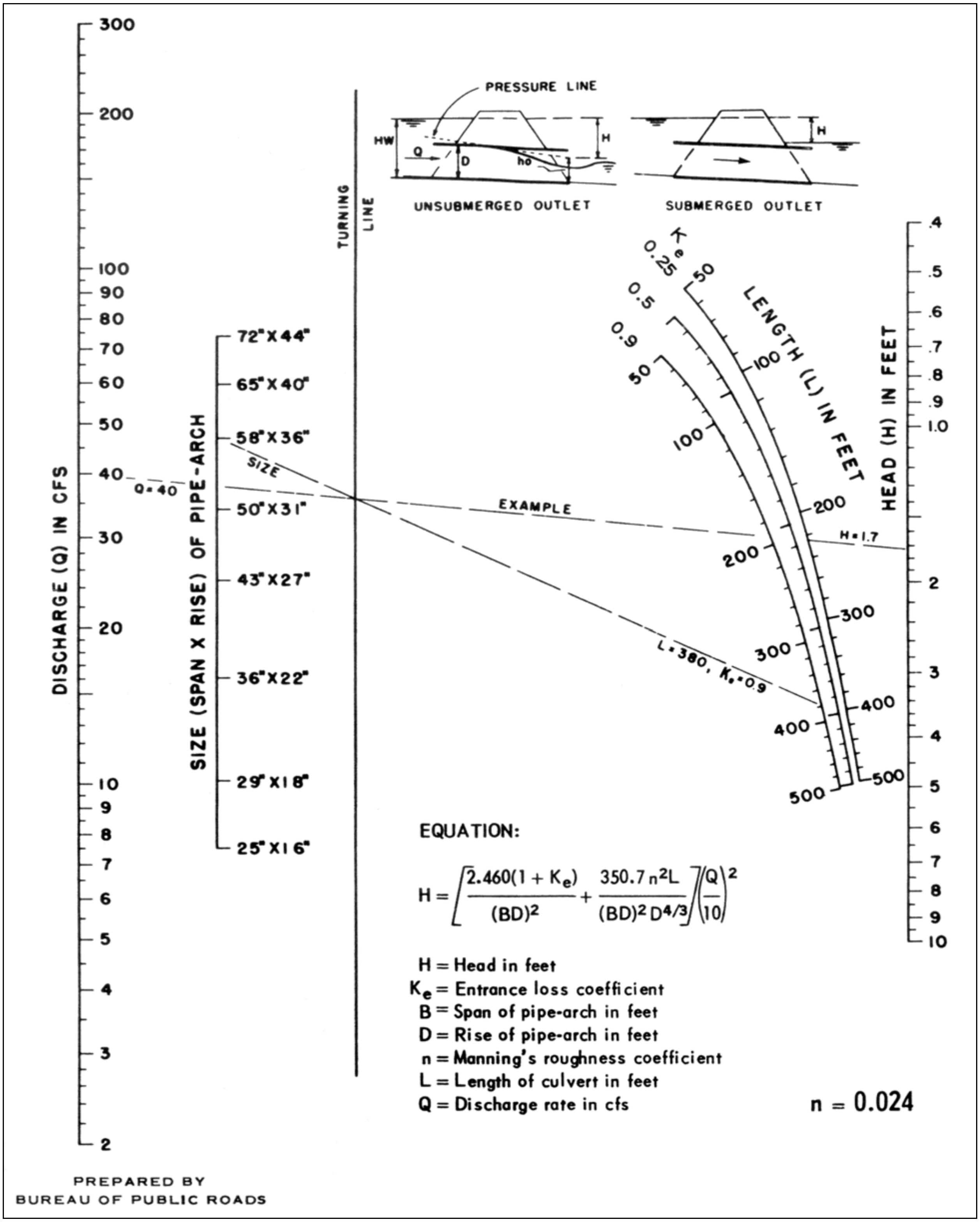
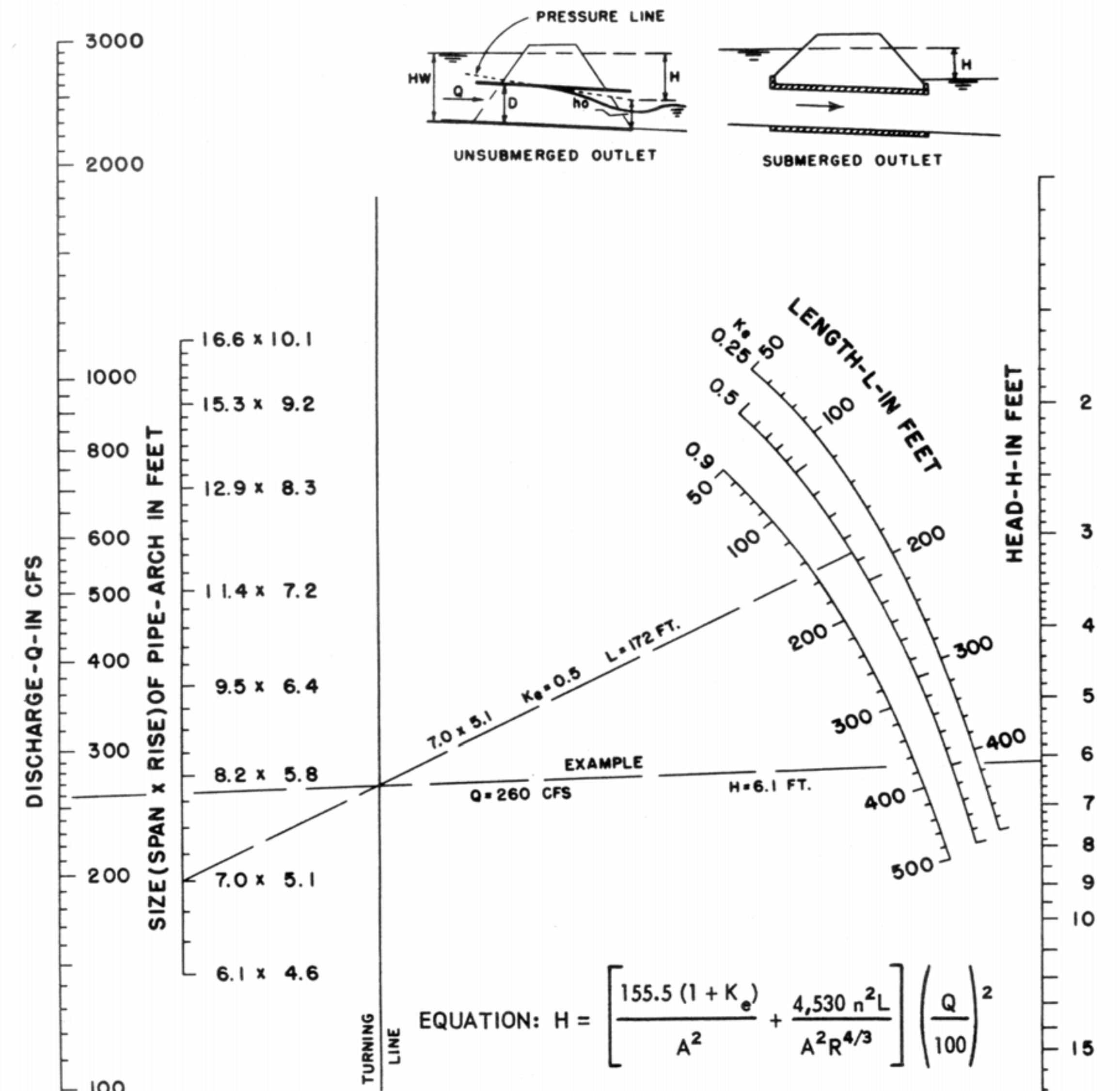


**Figure 4-57. Head for Field-Bolted Structural Plate Pipe-Arch Culverts 18 in. Corner Radius Flowing Full, n = 0.0327 to 0.0306**




$$H = \left[\frac{155.5\,(1 + K_e)}{A^2} + \frac{4{,}530\, n^2 L}{A^2 R^{4/3}}\right]\left(\frac{Q}{100}\right)^2$$

EQUATION: as above.

$H$ = Head in feet
$K_e$ = Entrance loss coefficient
$A$ = Area of pipe-arch opening in square feet
$n$ = Manning's roughness coefficient
$L$ = Length of culvert in feet
$R$ = Hydraulic radius in feet
$Q$ = Design discharge rate in cfs

NOTE:
Additional sizes not dimensioned here are given in fabricators catalogs.

| Size | n |
|---|---|
| 6.1 × 4.6 | 0.0327 |
| 8.2 × 5.8 | 0.0321 |
| 11.4 × 7.2 | 0.0315 |
| 16.6 × 10.1 | 0.0306 |

**n = 0.0327 TO 0.0306**

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**Figure 4-58. Head for Concrete Box Culverts Flowing Full, n = 0.012**

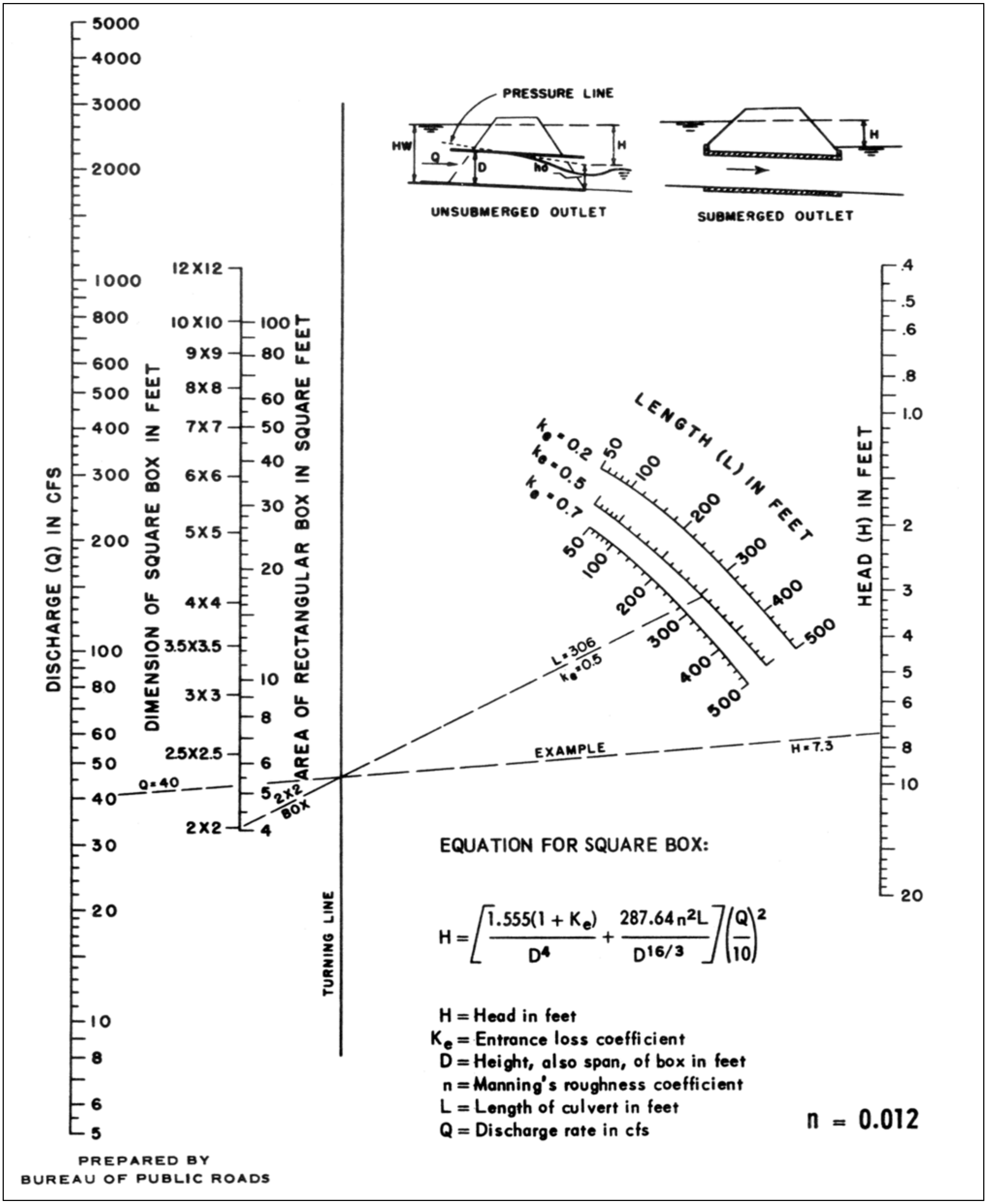
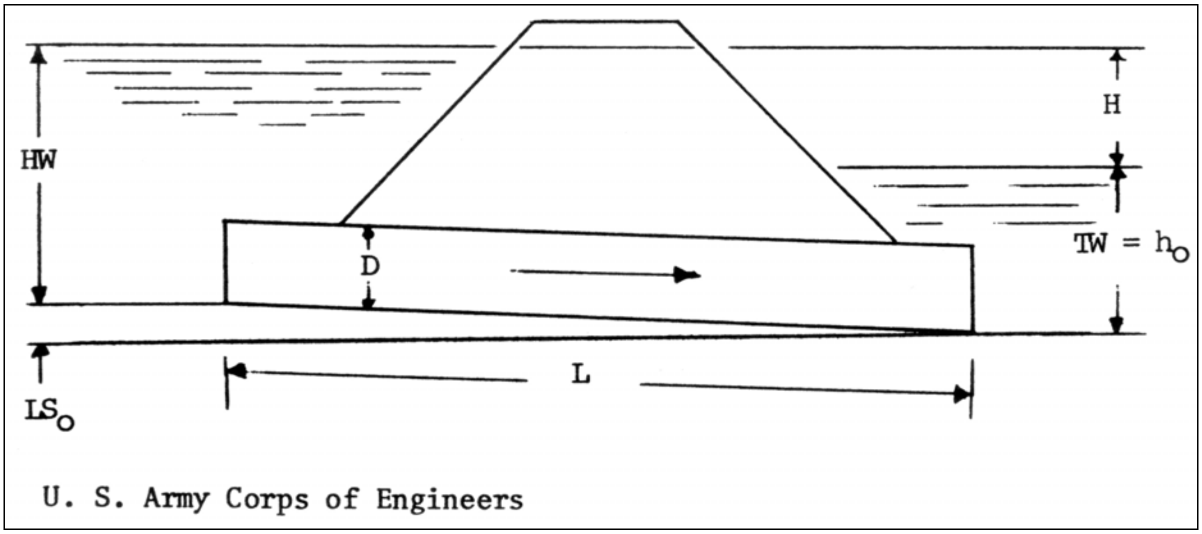


**Figure 4-59. Tailwater Elevation at or Above Top of Culvert**




If the headwater drops below this point the water surface will be free throughout the culvert barrel as in Figure 4-51D, and Equation 4-27 yields answers with some error since the only correct method of finding headwater in this case is by a backwater computation starting at the culvert outlet. However, Equation 4-27 will give answers of sufficient accuracy for design purposes if the headwater is limited to values greater than 0.75D. HN is used in Figure 4-53D to show that the head loss here is an approximation of H. No solution is given for headwater less than 0.75D. The depth of tailwater is important in determining the hydraulic capacity of culverts flowing with outlet control. In many cases the downstream channel is of considerable width and the depth of water in the natural channel is less than the height of water in the outlet end of the culvert barrel, making the tailwater ineffective as a control, so that its depth need not be computed to determine culvert discharge capacity or headwater. There are instances, however, where the downstream water-surface elevation is controlled by a downstream obstruction or backwater from another stream. A field inspection of all major culvert locations should be made to evaluate downstream controls and determine water stages. An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation, $V = \frac{1.486}{n} R^{2/3} S^{1/2}$, if the channel is reasonably uniform in cross section, slope, and roughness. Values of n for natural streams in Manning's formula are given in Table 4-5. If the water surface in the outlet channel is established by downstream controls other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relation of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

**Table 4-5. Manning's n for Natural Stream Channels (Surface Width at Flood Stage Less Than 100 ft)**

| | |
|---|---|
| Fairly regular section: | |
| Some grass and weeds, little or no brush ........ | 0.030-0.035 |
| Dense growth of weeds, depth of flow materially greater than weed height ........ | 0.035-0.05 |
| Some weeds, light brush on banks ........ | 0.035-0.05 |
| Some weeds, heavy brush on banks ........ | 0.05-0.07 |
| Some weeds, dense willows on banks ........ | 0.06-0.08 |
| For trees within the channel, with branches submerged at high stage, increase all above values by ........ | 0.01-0.02 |
| Irregular sections with pools, slight channel meander; increase values given above about ........ | 0.01-0.02 |
| Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage: | |
| Bottom of gravel, cobbles, and few boulders ........ | 0.04-0.05 |
| Bottom of cobbles, with large boulders ........ | 0.05-0.07 |

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**Figure 4-60. Tailwater Below the Top of the Culvert**

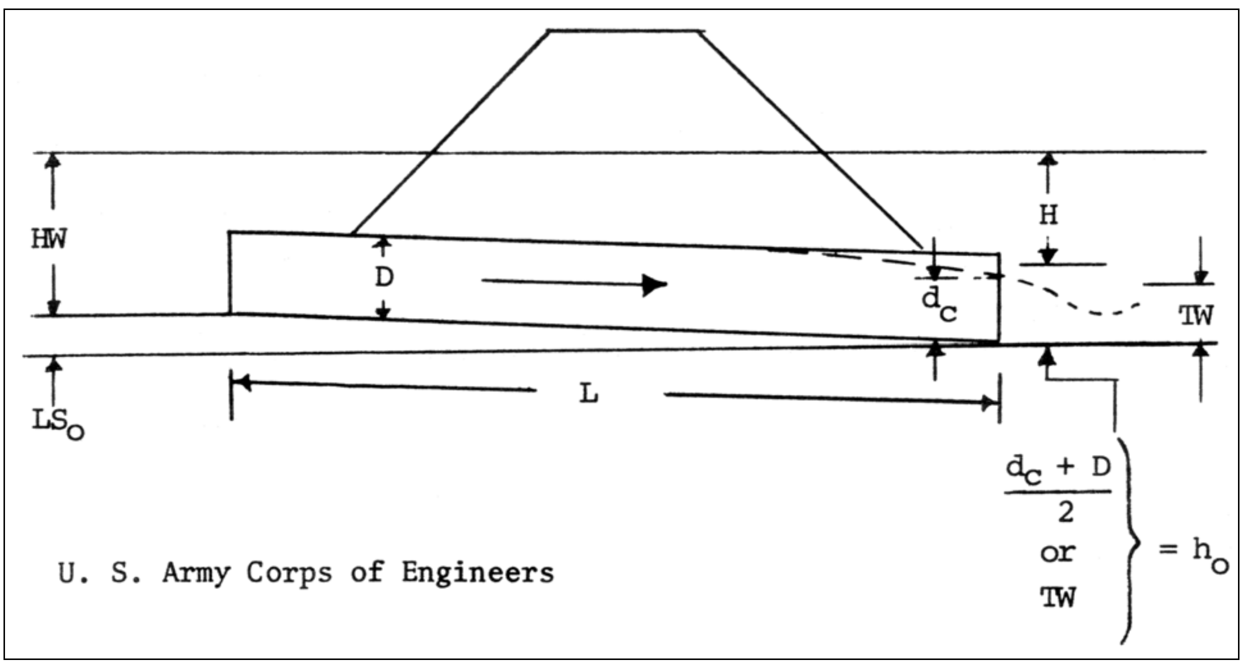


**Figure 4-61. Circular Pipe—Critical Depth**

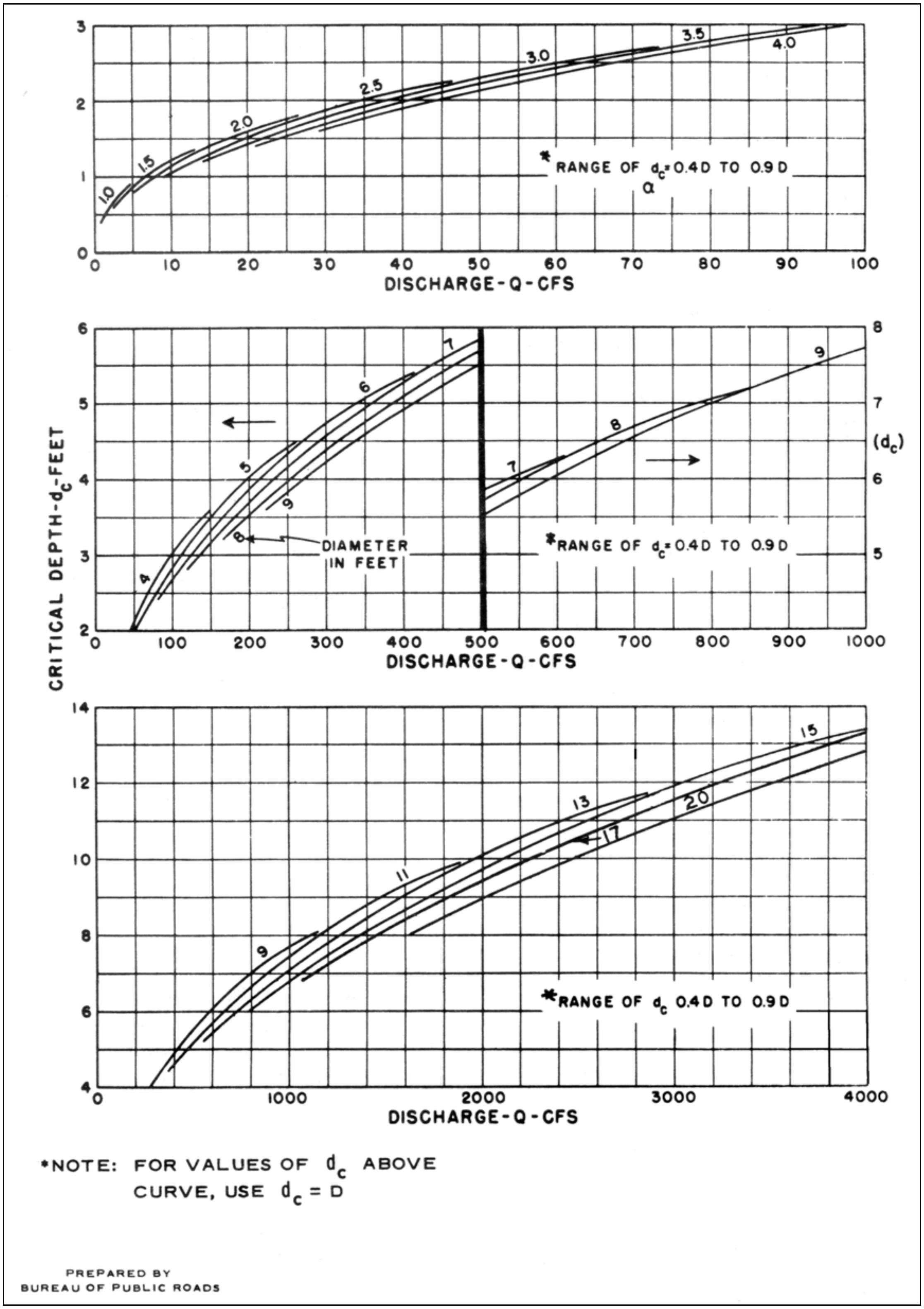


**Figure 4-62. Oval Concrete Pipe Long Axis Horizontal Critical Depth**

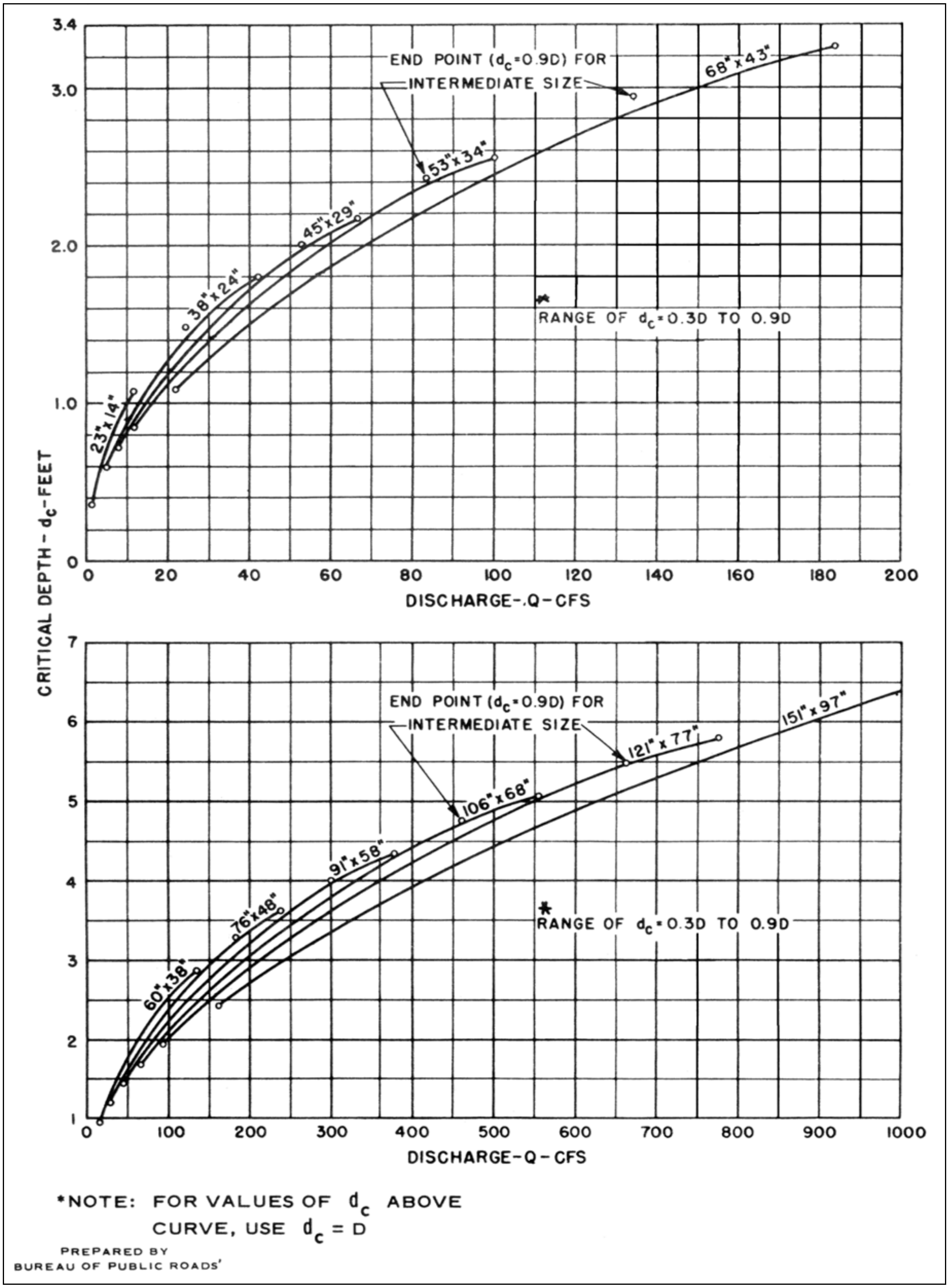


**Figure 4-63. Oval Concrete Pipe Long Axis Vertical Critical Depth**

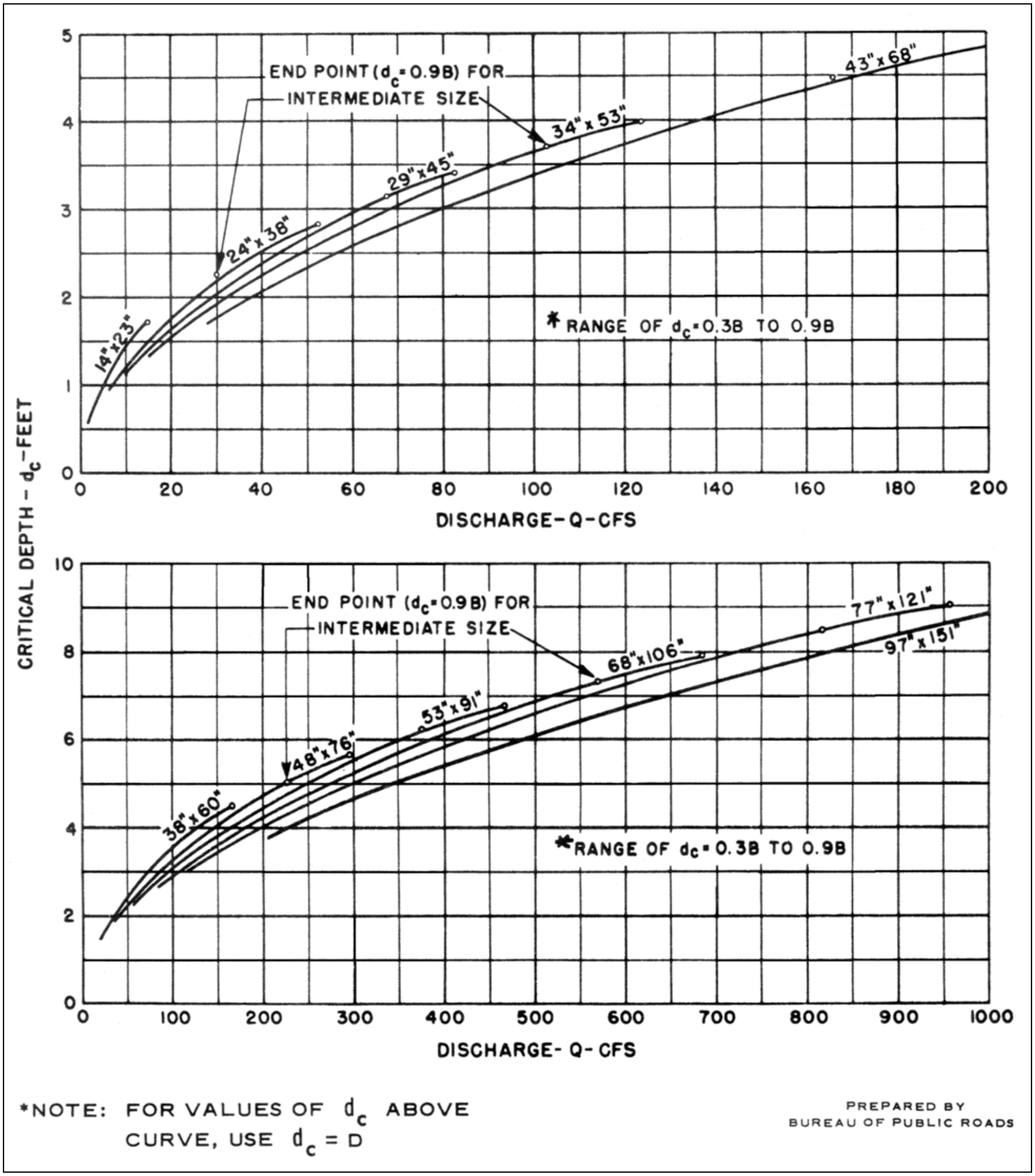
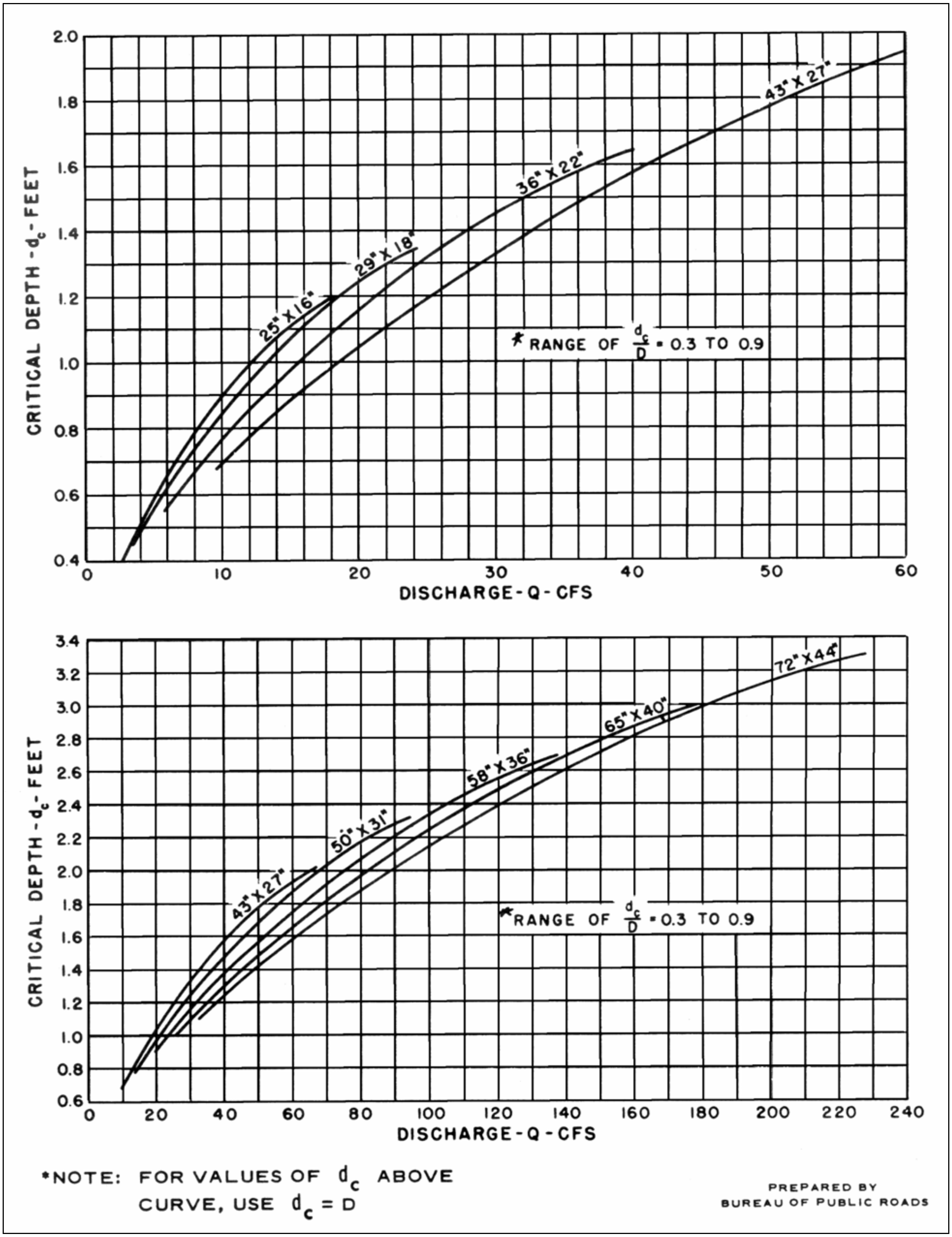


**Figure 4-64. Standard Corrugated Metal Pipe-Arch Critical Depth**



4-4.5.12 **Procedure for selection of culvert size**

4-4.5.12.1 Select the culvert size by the following steps:

a. Step 1: List given data.

(1) Design discharge, Q, in $ft^3$/sec.

(2) Approximate length of culvert, in feet.

(3) Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flow line) at entrance to the water-surface elevation permissible in the approach channel upstream from the culvert.

(4) Type of culvert, including barrel material, barrel cross-sectional shape, and entrance type.

(5) Slope of culvert. (If grade is given in percent, convert to slope in feet per foot.)

(6) Allowable outlet velocity (if scour is a problem).

b. Step 2: Determine a trial-size culvert.

(1) Refer to the inlet-control nomograph (Figures 4-43 through 4-50) for the culvert type selected.

(2) Using an $\frac{HW}{D}$ of approximately 1.5 and the scale for the entrance type to be used, find a trial-size culvert by following the instructions for use of these nomographs. If reasons for less or greater relative depth of headwater in a particular case should exist, another value of $\frac{HW}{D}$ may be used for this trial selection.

(3) If the trial size for the culverts is obviously too large because of limited height of embankment or availability of size, try a $\frac{HW}{D}$ value or multiple culverts by dividing the discharge equally for the number of culverts used. Raising the embankment height or using pipe arch and box culverts with width greater than height should be considered. Selection should be based on an economic analysis.

c. Step 3: Find headwater depth for the trial-size culvert.

(1) Determine and record headwater depth by use of the appropriate inlet-control nomograph (Figures 4-43 through 4-50). Tailwater conditions

are to be neglected in this determination. Headwater in this case is found by simply multiplying $\frac{HW}{D}$ obtained from the nomograph by D.

(2) Compute and record headwater for outlet control as instructed below:

(a) Approximate the depth of tailwater for the design flood condition in the outlet channel. The tailwater depth may also be due to backwater caused by another stream or some control downstream.

(b) For tailwater depths equal to or above the depth of the culvert at the outlet, set tailwater equal to $h_o$ and find headwater by the following equation:

$$HW = h_o + H - S_o L$$

(c) For tailwater elevations below the crown of culvert at the outlet, use the following equation to find headwater:

$$HW = h_o + H - S_o L$$

where $h_o = \frac{d_c + D}{2}$ or TW, whichever is greater. When $d_c$ (Figures 4-61 through 4-66) exceeds rectangular section, $h_o$ should be set equal to D.

(3) Compare the headwater found in Step 3a and Step 3b (inlet control and outlet control). The higher headwater governs and indicates the flow control existing under the given conditions.

(4) Compare the higher headwater above with that allowable at the site. If headwater is greater than allowable, repeat the procedure using a larger culvert. If headwater is less than allowable, repeat the procedure to investigate the possibility of using a smaller size.

d. Step 4: Check outlet velocities for size selected.

(1) If outlet control governs in Step 3c, outlet velocity equals Q/A, where A is the cross-sectional area of flow at the outlet. If $d_c$ or TW is less than the height of the culvert barrel, use cross-sectional area corresponding to $d_c$ or TW depth, whichever gives the greater area of flow.

(2) If inlet control governs in Step 3c, outlet velocity can be assumed to equal normal velocity in open-channel flow as computed by Manning's equation for the barrel size, roughness, and slope of culvert selected.

e. Step 5: Try a culvert of another type or shape and determine size and headwater by the above procedure.

f. Step 6: Record final selection of culvert with size, type, outlet velocity, required headwater, and economic justification.

**Figure 4-65. Structural Plate Pipe-Arch Critical Depth**

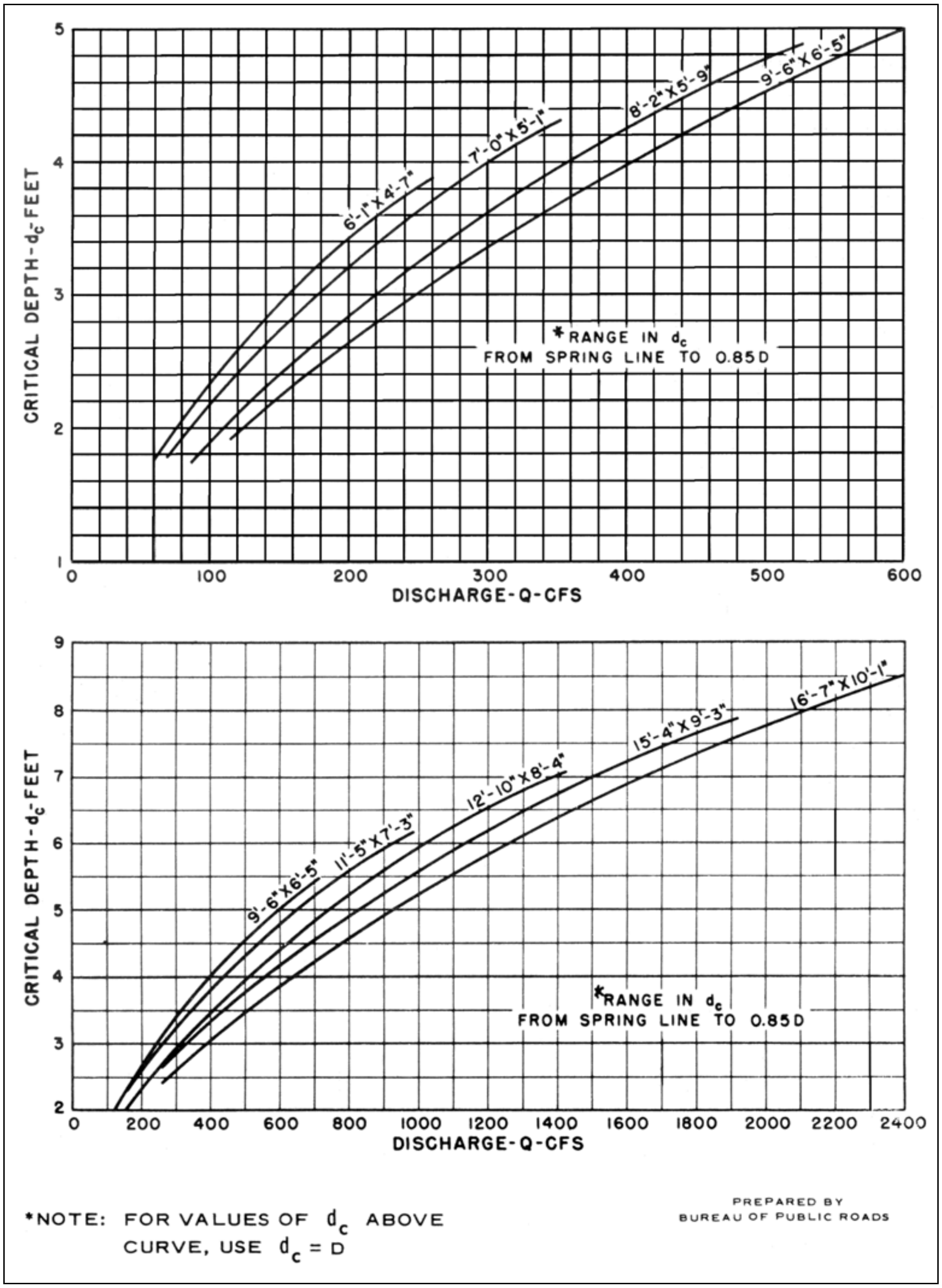
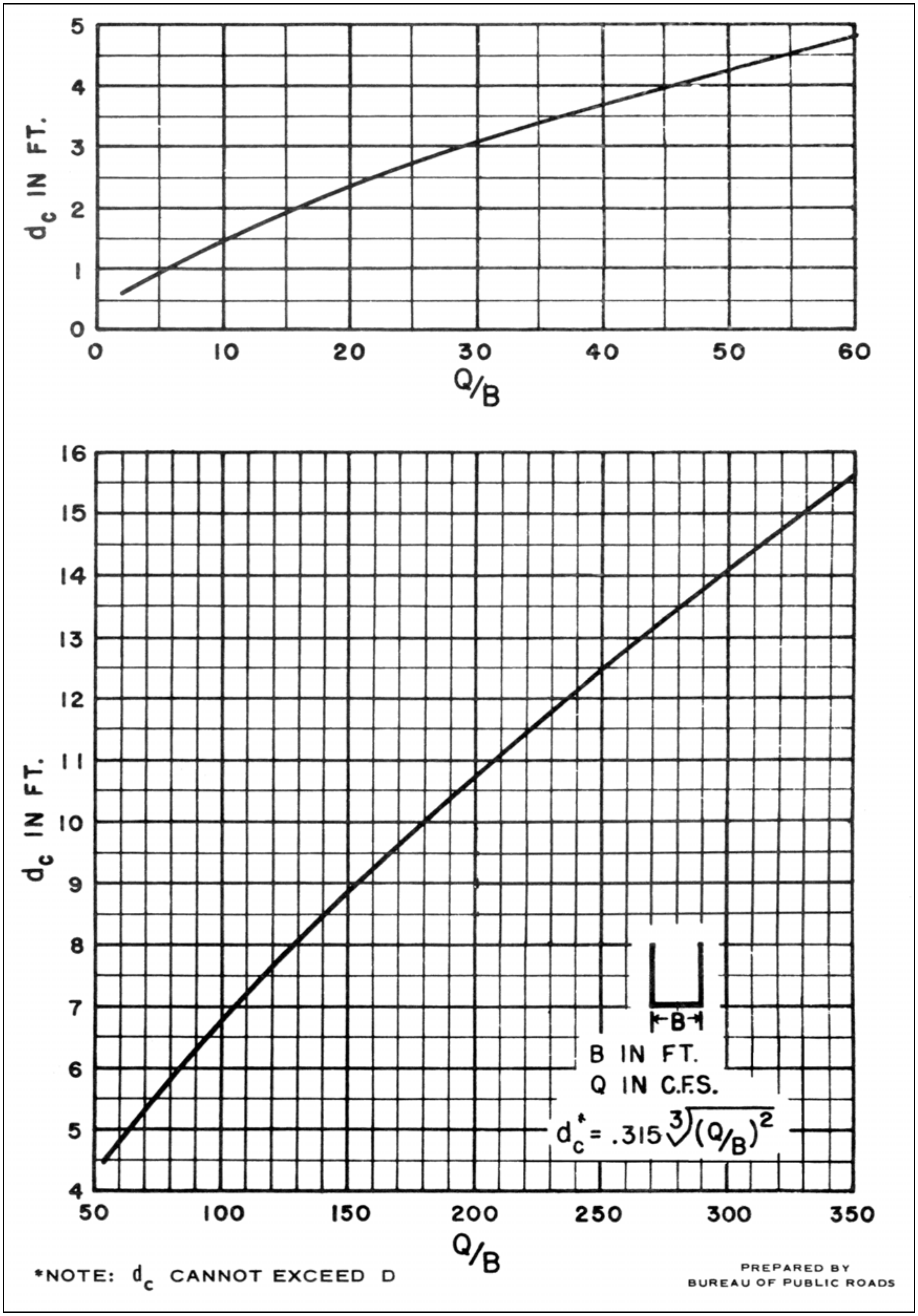


**Figure 4-66. Critical Depth Rectangular Section**



4-4.5.13 **Instructions for use of inlet-control nomographs (Figures 4-43 through 4-50)**

4-4.5.13.1 To determine headwater.

a. Connect with a straight edge the given culvert diameter or height, D, and the discharge, Q, or Q/B for box culverts; mark intersection of straight edge on $\frac{HW}{D}$ scale 1.

b. If $\frac{HW}{D}$ scale 1 represents entrance type used, read $\frac{HW}{D}$ on scale 1. If some other entrance type is used extend the point of intersection ((a) above) horizontally to scale 2 or 3 and read $\frac{HW}{D}$.

c. Compute headwater by multiplying $\frac{HW}{D}$ by D.

4-4.5.13.2 To determine culvert size.

a. Given an $\frac{HW}{D}$ value, locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale 2 or 3 is used, extend $\frac{HW}{D}$ point horizontally to scale 1.

b. Connect point on $\frac{HW}{D}$ scale 1 as found in (a) above to given discharge and read diameter, height, or size of culvert required.

4-4.5.13.3 To determine discharge.

a. Given HW and D, locate $\frac{HW}{D}$ on scale for appropriate entrance type. Continue as in 4-4.5.13.2(a) above.

b. Connect point on $\frac{HW}{D}$ scale 1 as found in (a) above and the size of culvert on the left scale and read Q or Q/B on the discharge scale.

c. If Q/B is read multiply B to find Q.

4-4.5.14 **Instruction for use of outlet-control nomography**

4-4.5.14.1 Figures 4-52 through 4-58 are nomography to solve for head when culverts flow full with outlet control. They are also used in approximating the head for some partially full flow conditions with outlet control. These nomography do not give a complete solution for finding headwater. (See Section 4-4.5.12)

a. Locate appropriate nomograph for type of culvert selected.

b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scale, follow instructions below:

(1) If the n value of the nomograph corresponds to that of the culvert being used, find the proper $K_e$ from Table 4-4 and on the appropriate nomograph locate starting point on length curve for the $K_e$. If a $K_e$ curve is not shown for the selected $K_e$, and (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

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

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

$$L_1 = L\left(\frac{n_1}{n}\right)^2 \qquad \text{(eq. 4-29)}$$

(See Section 4-4.5.14.2 for n values.)

c. Using a straight edge, connect point on length scale to size of culvert barrel and mark the point of crossing on the “turning line.” See Section 4-4.5.14.3 for size considerations for rectangular box culvert.

d. Pivot the straight edge on this point on the turning line and-connect given discharge rate. Read head in feet on the head scale. For values beyond the limit of the chart scales, find H by solving equation given in nomograph or by H = KQ2 where K is found by substituting values of H and Q from chart.

4-4.5.14.2 Table 4-1 is used to find the n value for the culvert selected.

4-4.5.14.3 To use the box-culvert nomograph (Figure 4-58) for full flow for other than square boxes:

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

Note: The area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 2/3D. For other box proportions use equation shown in nomograph for more accurate results.

b. Connect proper point (see Section 4-4.5.14.2 of this chapter) on length scale to barrel area and mark point on turning line.

c. Pivot the straight edge on this point on the turning line and connect given discharge rate. Read head in feet on the head scale.

4-4.5.15 **Culvert capacity charts.** Figures 4-67 through 4-84, prepared by the Bureau of Public Roads, present headwater discharge relations convenient for use in design of culverts of the most common types and sizes. The solid-line curve for each type and size represents for a given length: slope ratio the culvert capacity with control at the inlet; these curves are based generally on model data. For those culvert types for which a dashed-line curve is shown in addition to a solid-line curve, the dashed line represents for a given length: slope ratio the discharge capacity for free flow and control at the outlet; these curves are based on experimental data and backwater computations. The length: slope ratio is L/100 $S_o$ given on the solid line curve and in each case is the value at which the discharge with outlet control equals the discharge with inlet control. For culverts with free flow and control at the outlet, interpolation and extrapolation for different L/100 $S_o$ values is permitted in the range of headwater depths equal to or less than twice the barrel height. The upper limit of this range of headwater depths is designated by a horizontal dotted line on the charts. Values of L/100 $S_o$ less than those given in the chart do not impose any limitation; merely read the solidline curves. The symbol AHW means allowable headwater depth. The charts permit rapid selection of a culvert size to meet a given headwater limitation for various entrance conditions and types and shapes of pipe. One can enter with a given discharge and read vertically upward to the pipe size that will carry the flow to satisfy the headwater limitation of the design criteria. The major restriction on the use of the charts is that free flow must exist at the outlet. In most culvert installations free flow exists, i.e., flow passes through critical depth near the culvert outlet. For submerged flow conditions the solution can be obtained by use of the outlet control nomographs.

## 4-4.6 Underground Hydraulic Design

4-4.6.1 The storm-drain system will have sufficient capacity to convey runoff from the design storm (usually a 10-yr frequency for permanent installations) within the barrel of the conduit. Design runoff will be computed by the methods indicated in Section 2-9. Concentration times will increase and average rainfall intensities will decrease as the design is carried to successive downstream points. In general, the incremental concentration times and the point-by-point totals should be estimated to the nearest minute. These totals should be rounded to the nearest 5 min in selecting design intensities from the intensity duration curve. Advantage will be taken of any permanently available surface ponding areas, and their effectiveness determined, in order to hold design discharges and storm-drain sizes to a minimum. Experience indicates that it is feasible and practical in the actual design of storm drains to adopt minimum values of concentration times of 10 min for paved areas and 20 min for turfed areas. Minimum times of concentration should be selected by weighting for combined paved and turfed areas.

**Figure 4-67. Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 18 in. to 66 in.**

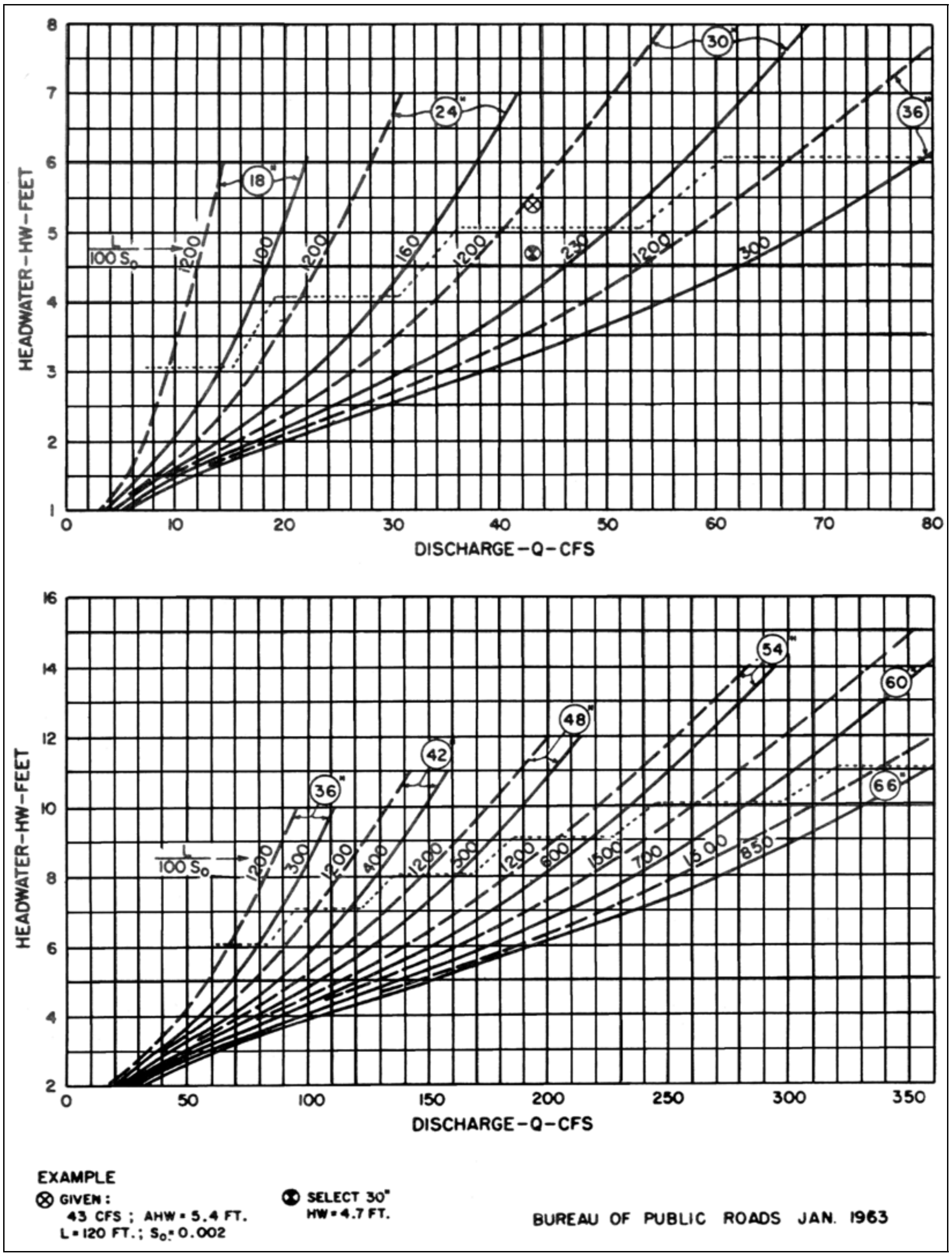


**Figure 4-68. Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 60 in. to 180 in.**

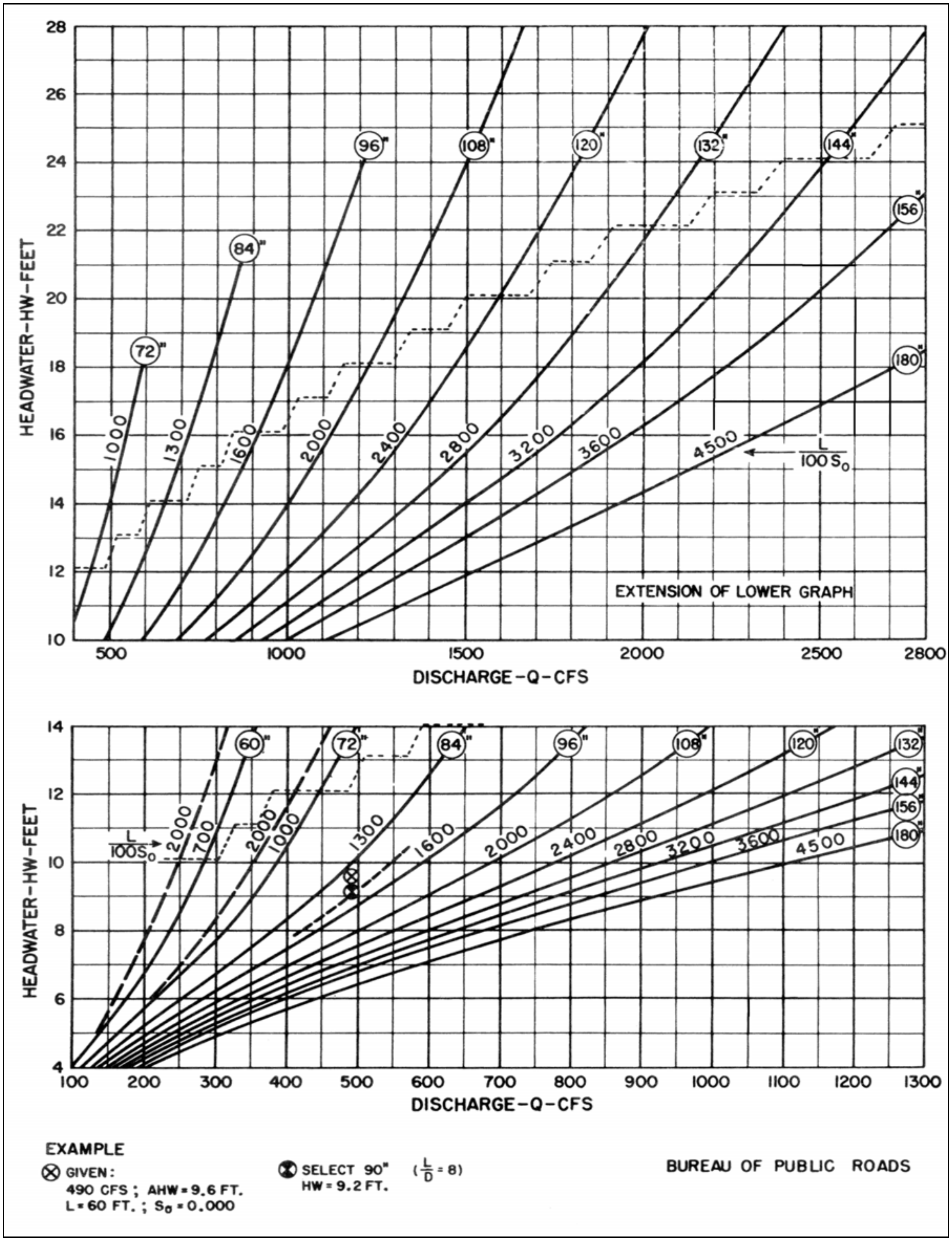


**Figure 4-69. Culvert Capacity Standard Circular Corrugations Metal Pipe Projecting Entrance 18 in. to 36 in.**

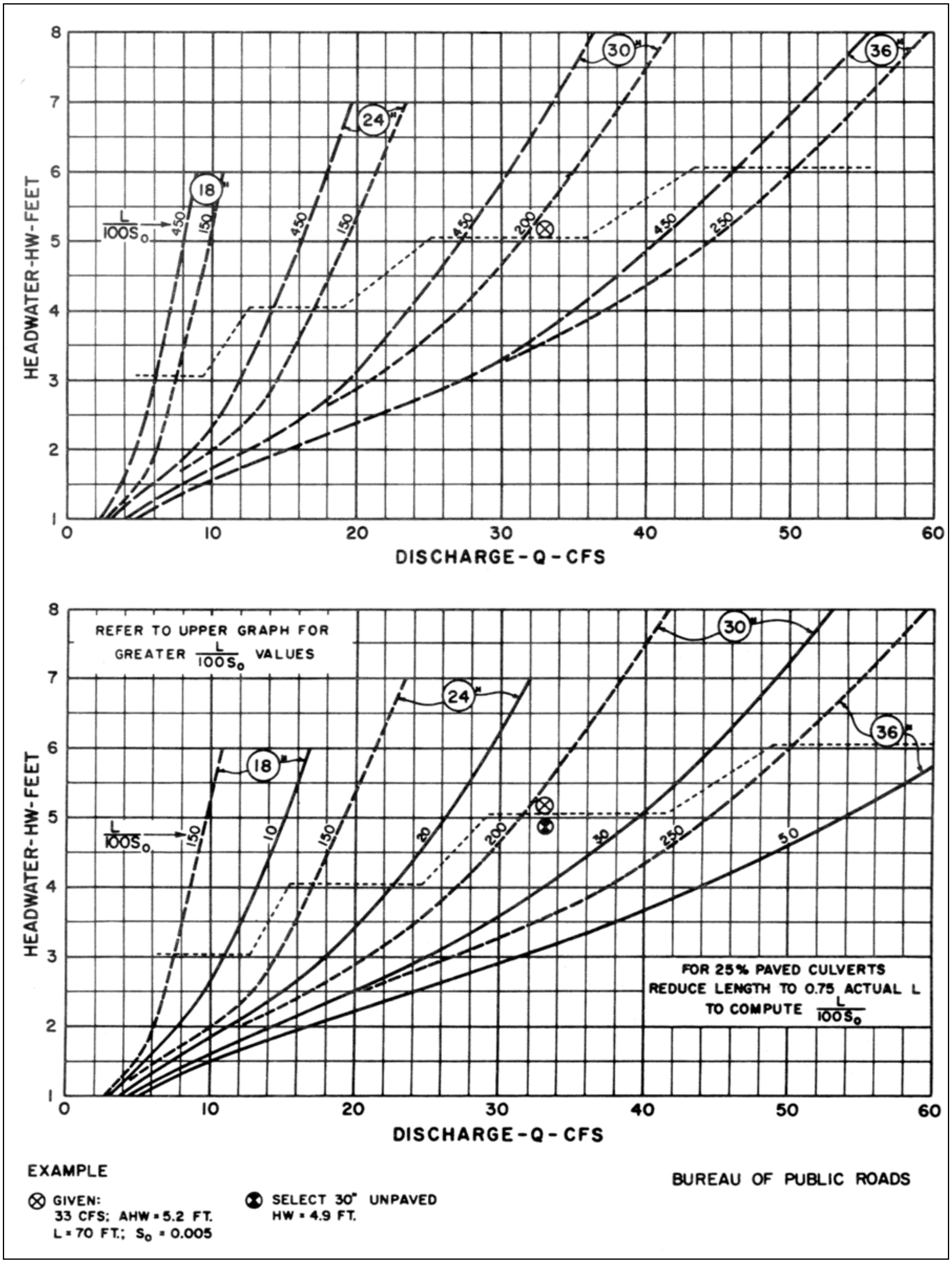


**Figure 4-70. Culvert Capacity Standard Circular Corrugations Metal Projecting Entrance 36 in. to 66 in.**

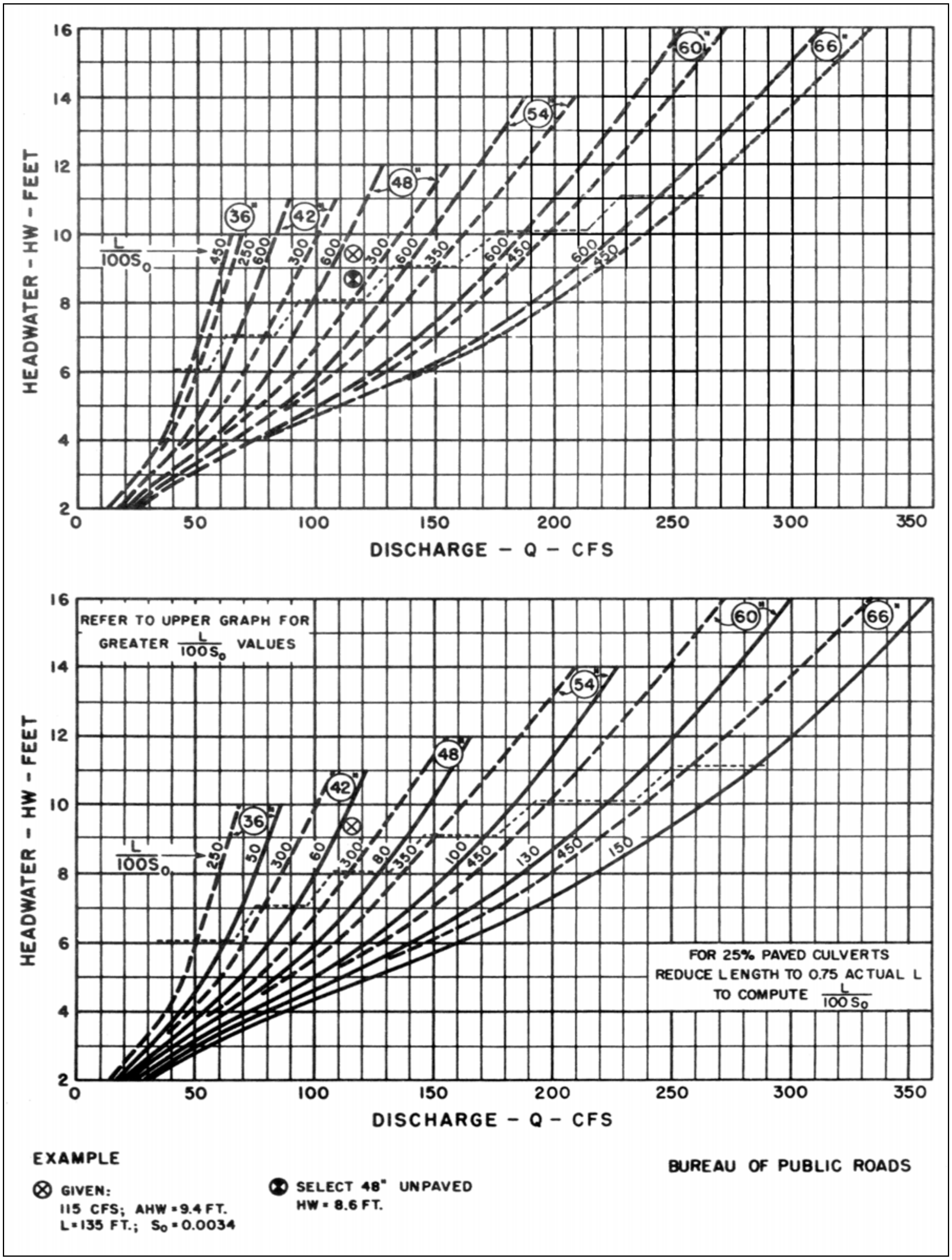


**Figure 4-71. Culvert Capacity Standard Circular Corrugations Metal Headwall Entrance 18 in. to 36 in.**

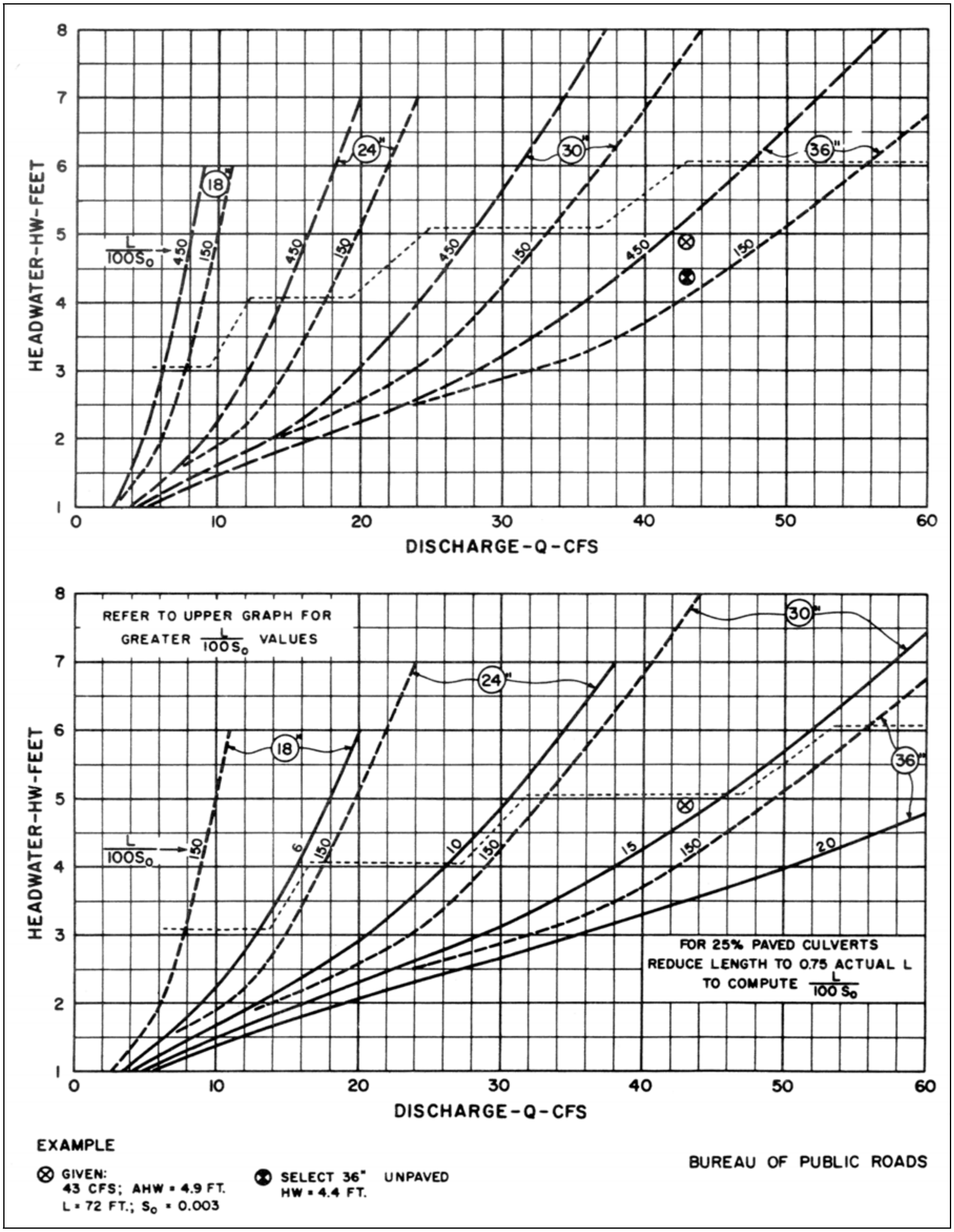


**Figure 4-72. Culvert Capacity Standard Circular Corrugations Metal Headwall Entrance 36 in. to 66 in.**

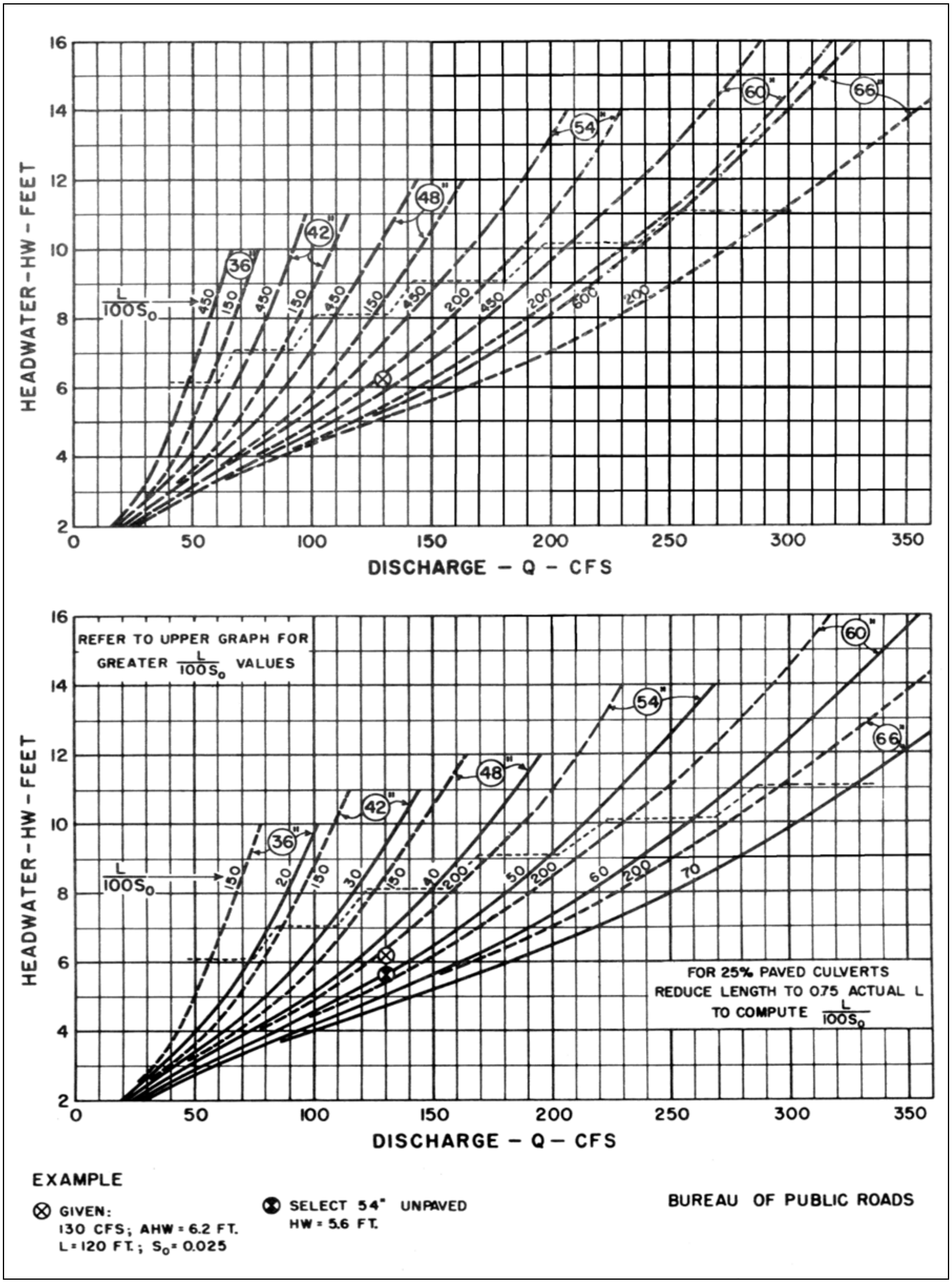


**Figure 4-73. Culvert Capacity Standard Corrugations Metal Pipe-Arch Projecting Entrance 25 in. by 16 in. to 43 in. by 27 in.**

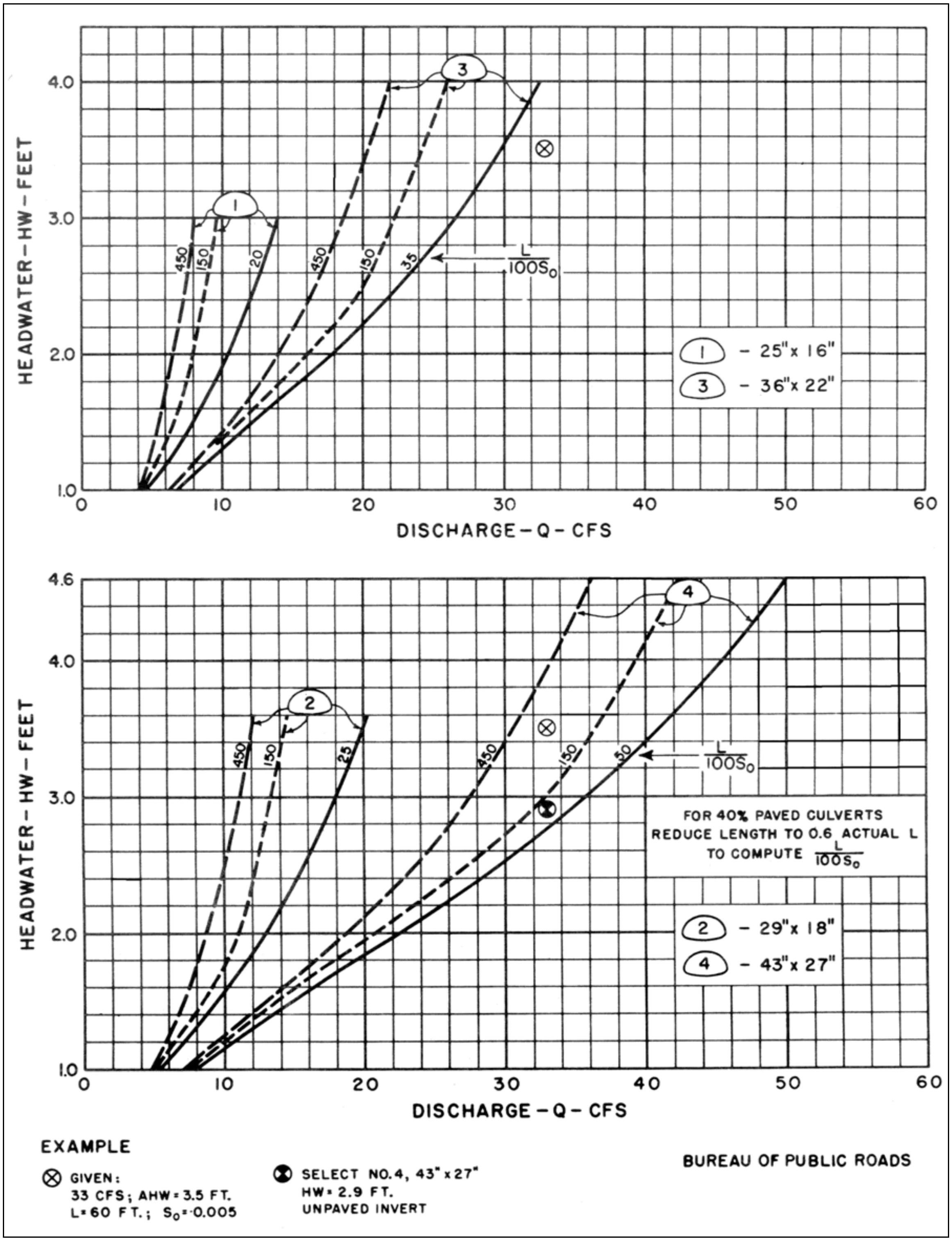


**Figure 4-74. Culvert Capacity Standard Corrugations Metal Pipe-Arch Projecting Entrance 50 in. by 31 in. to 72 in. by 44 in.**

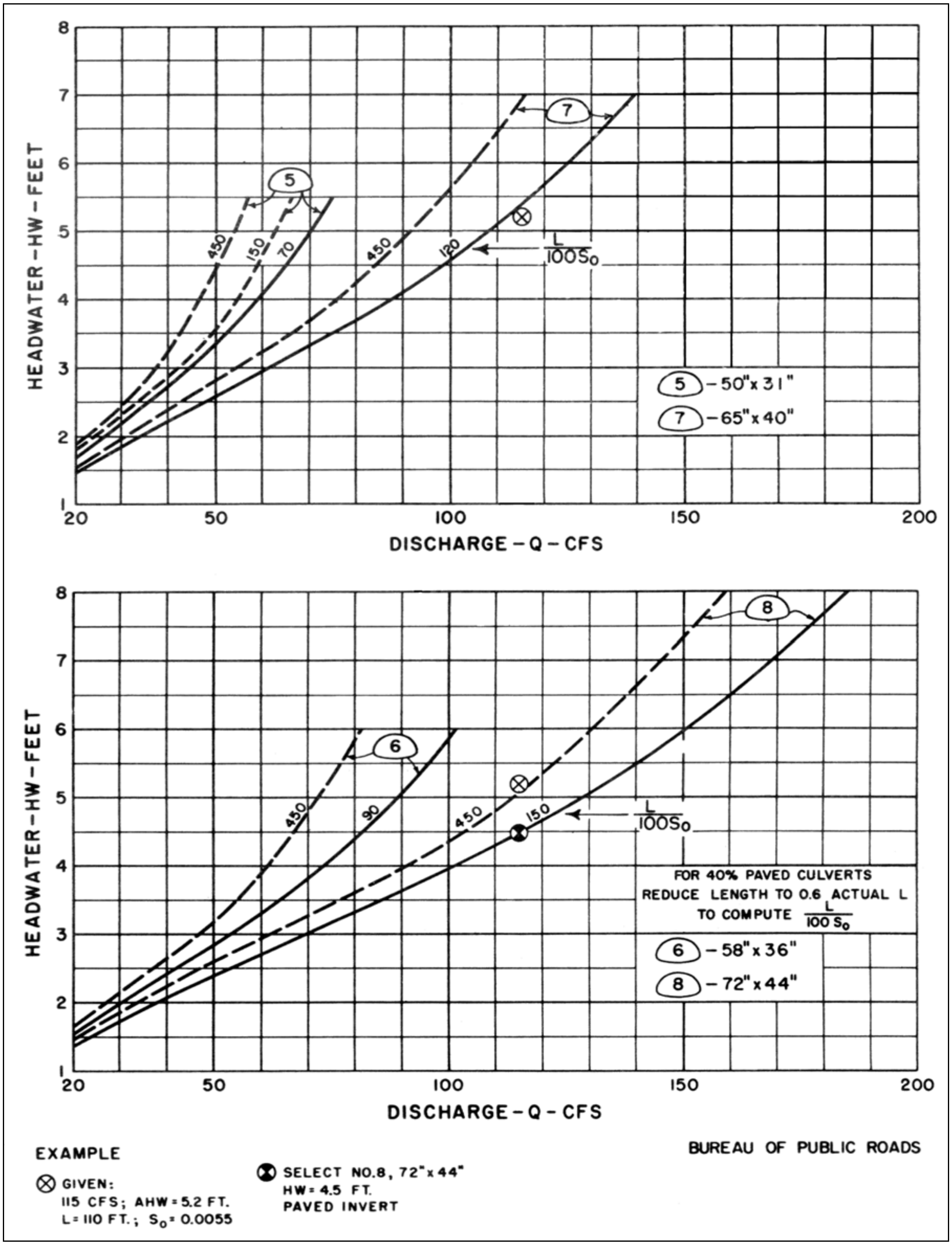


**Figure 4-75. Culvert Capacity Standard Corrugations Metal Pipe-Arch Headwall Entrance 25 in. by 16 in. to 43 in. by 27 in.**

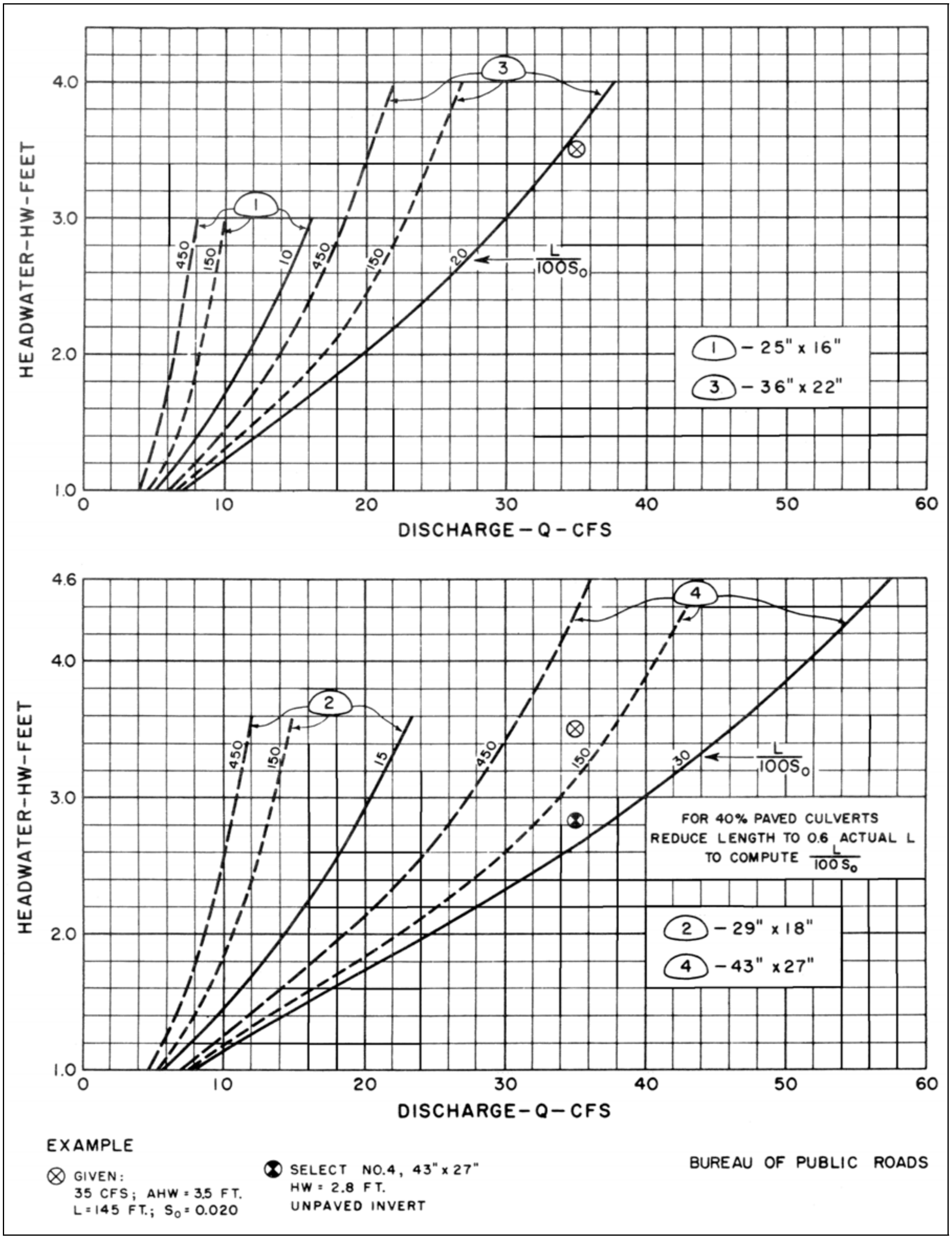


**Figure 4-76. Culvert Capacity Standard Corrugations Metal Pipe-Arch Headwall Entrance 50 in. by 31 in. to 72 in. by 44 in.**

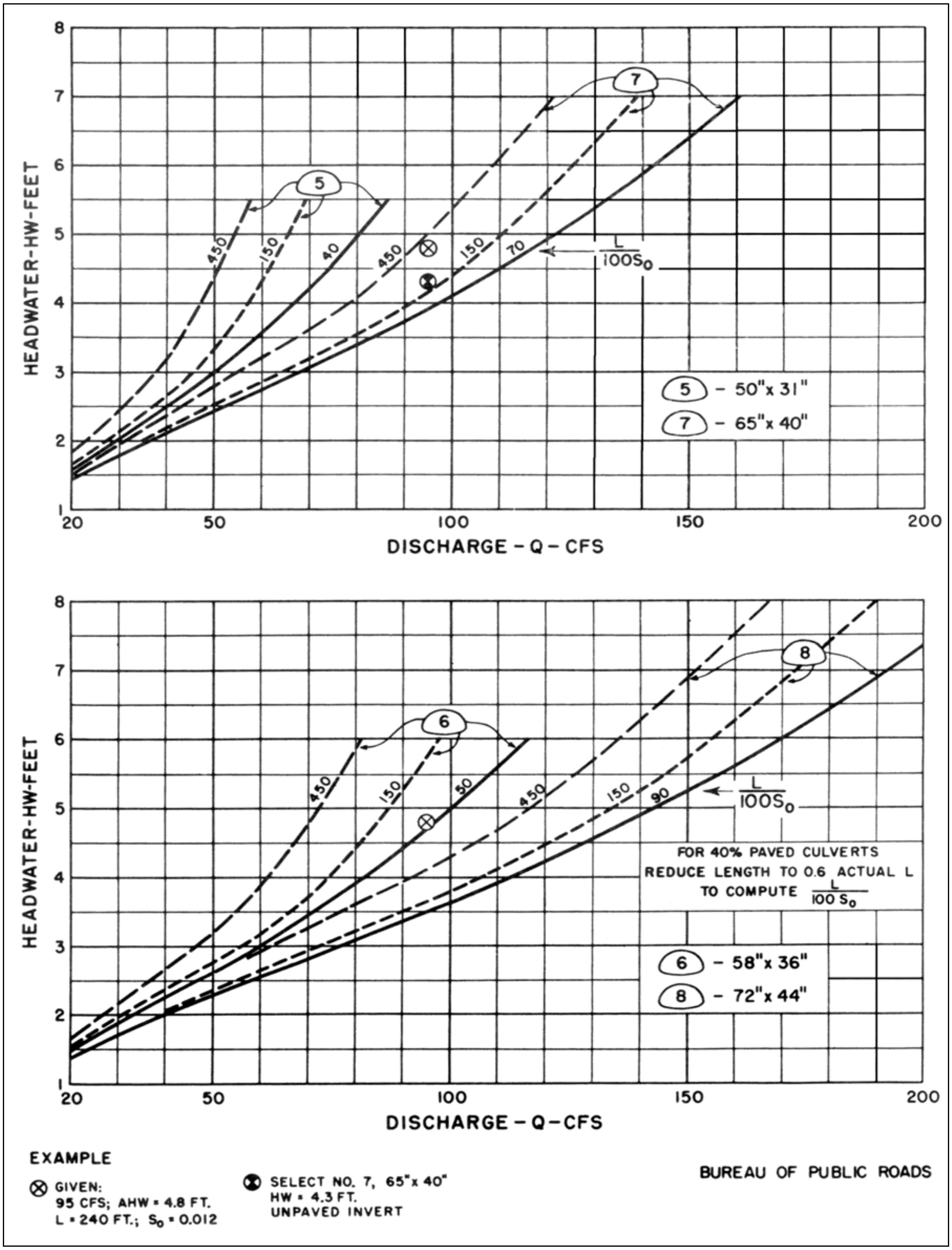


**Figure 4-77. Culvert Capacity Square Concrete Box 90 Degree and 15 Degree Wingwall Flare 1.5 ft by 1.5 ft to 7 ft by 7 ft**

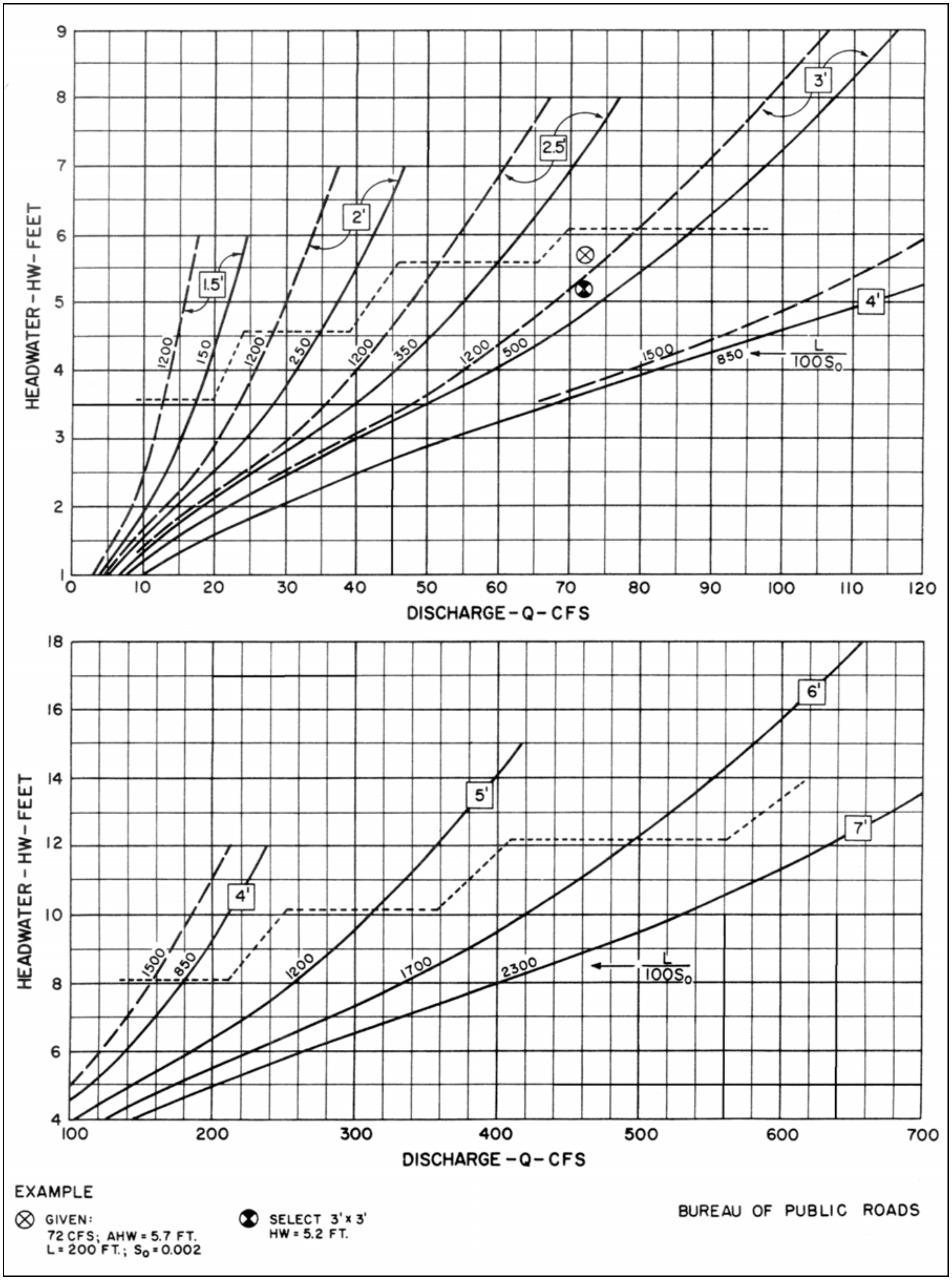


**Figure 4-78. Culvert Capacity Square Concrete Box 30 Degree and 75 Degree Wingwall Flare 1.5 ft by 1.5 ft to 7 ft by 7 ft**

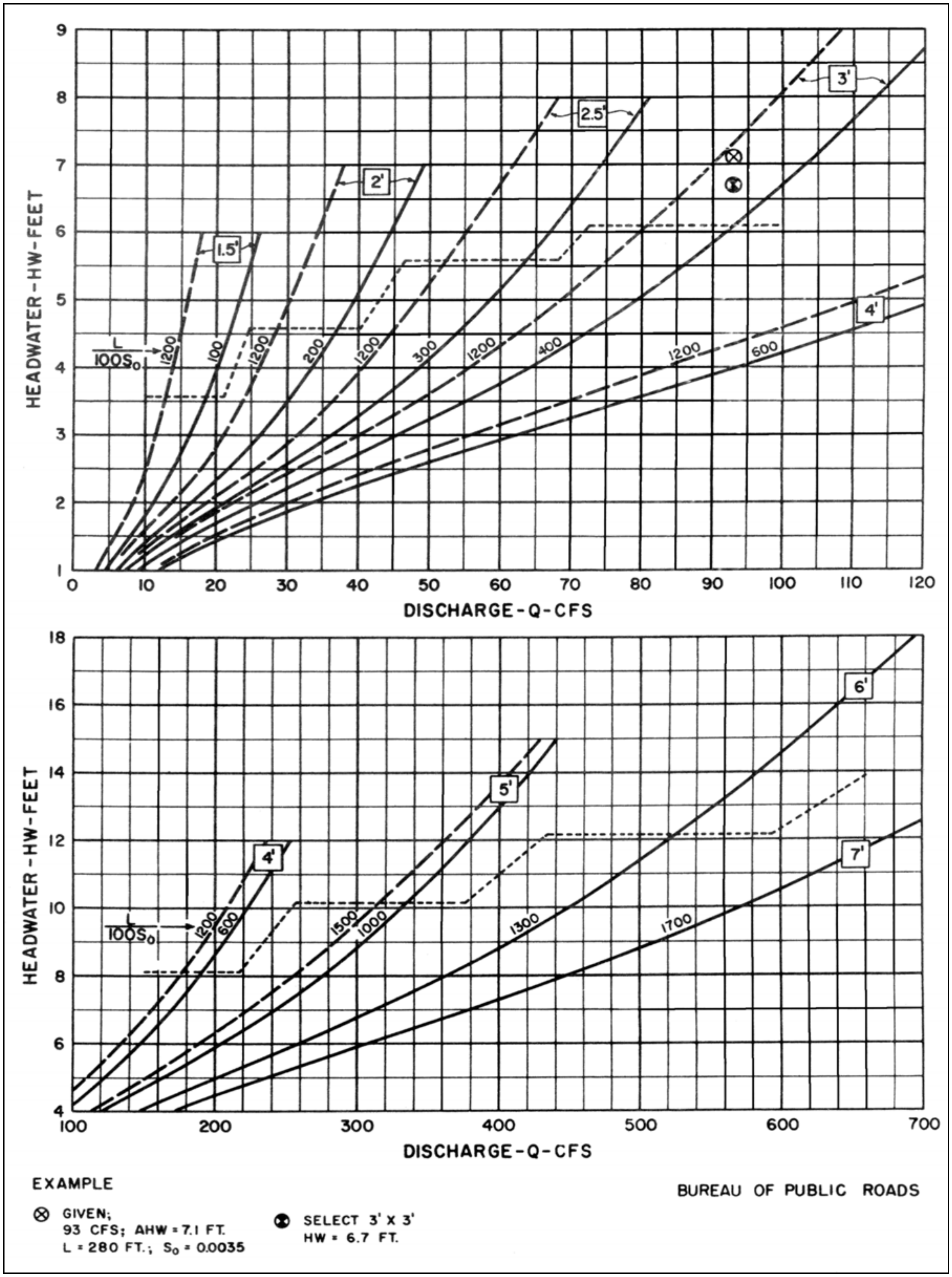


**Figure 4-79. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 1.5 ft, 2.0 ft, and 2.5 ft Heights**

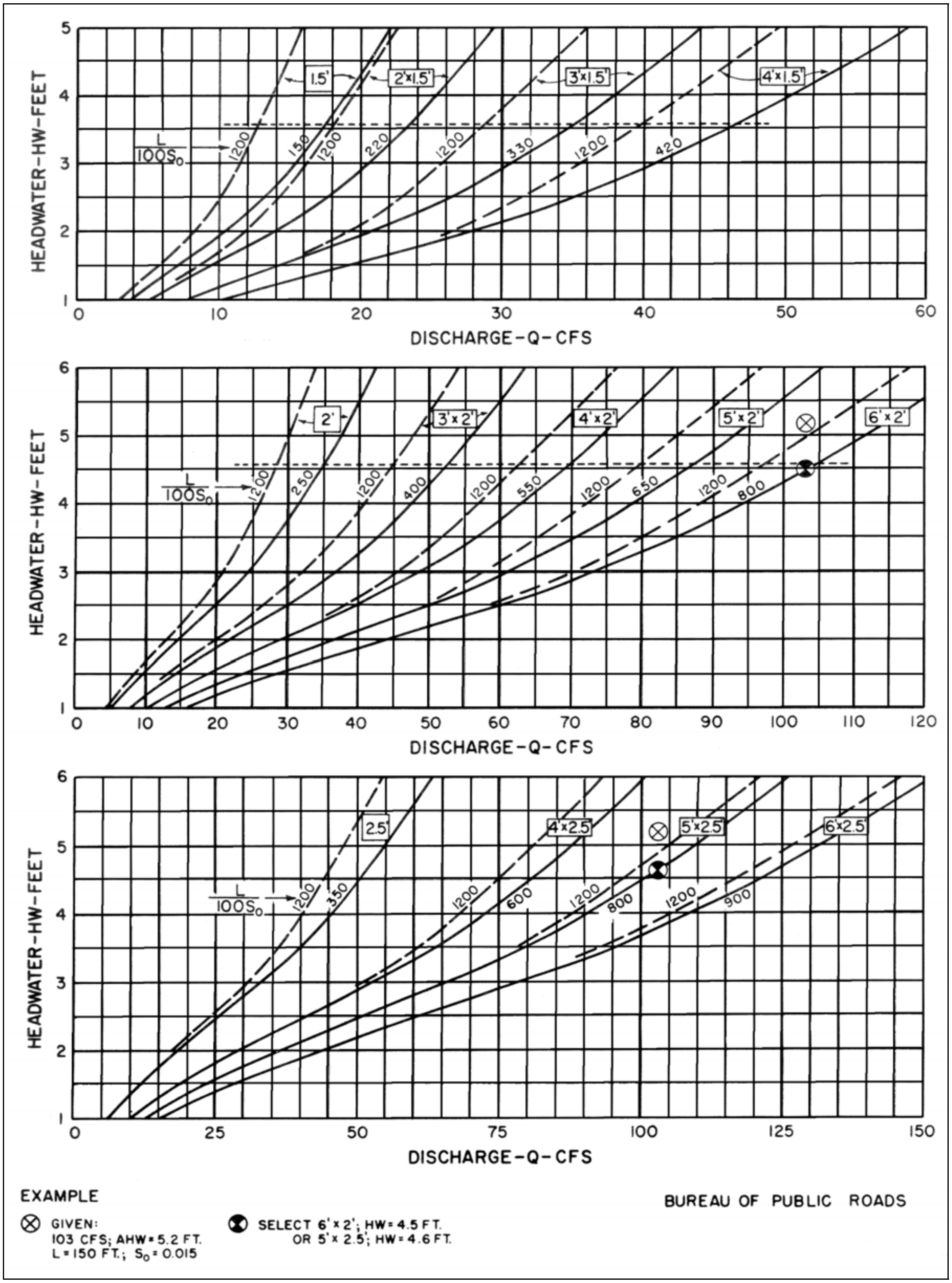


**Figure 4-80. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 3 ft and 4 ft Heights**

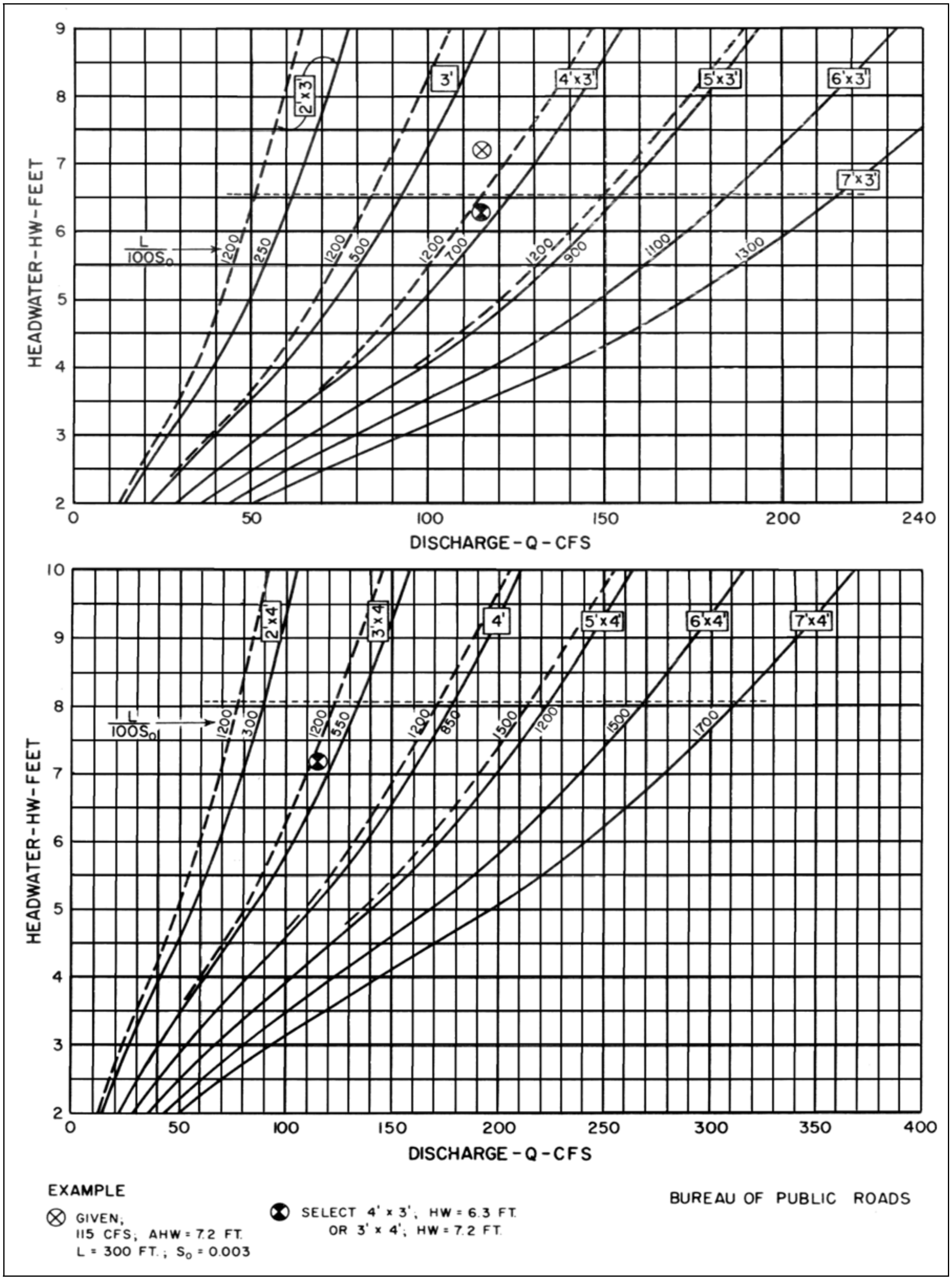


**Figure 4-81. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 5 ft and 6 ft Heights**

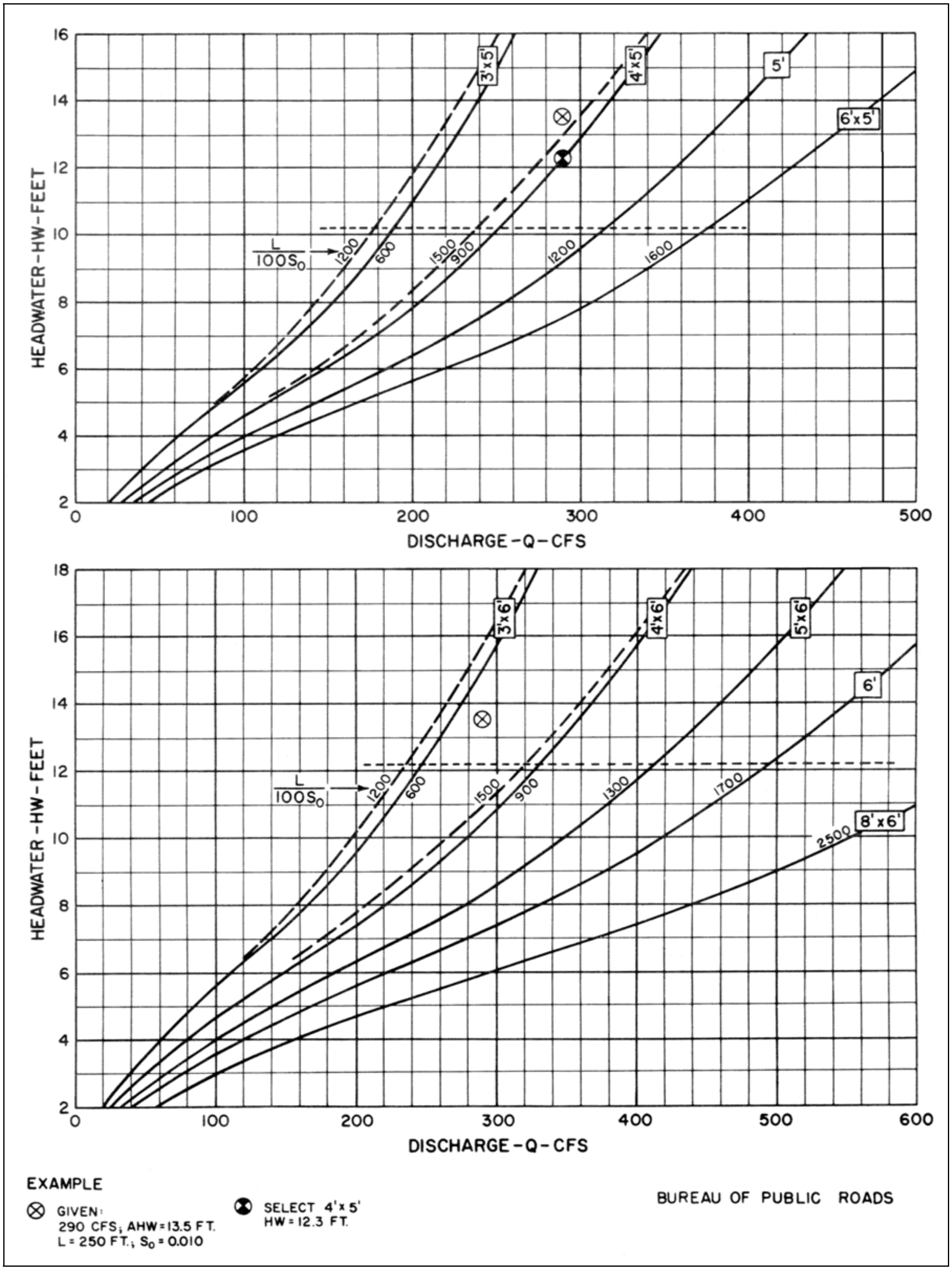


**Figure 4-82. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 1.5 ft, 2.0 ft, and 2.5 ft Heights**

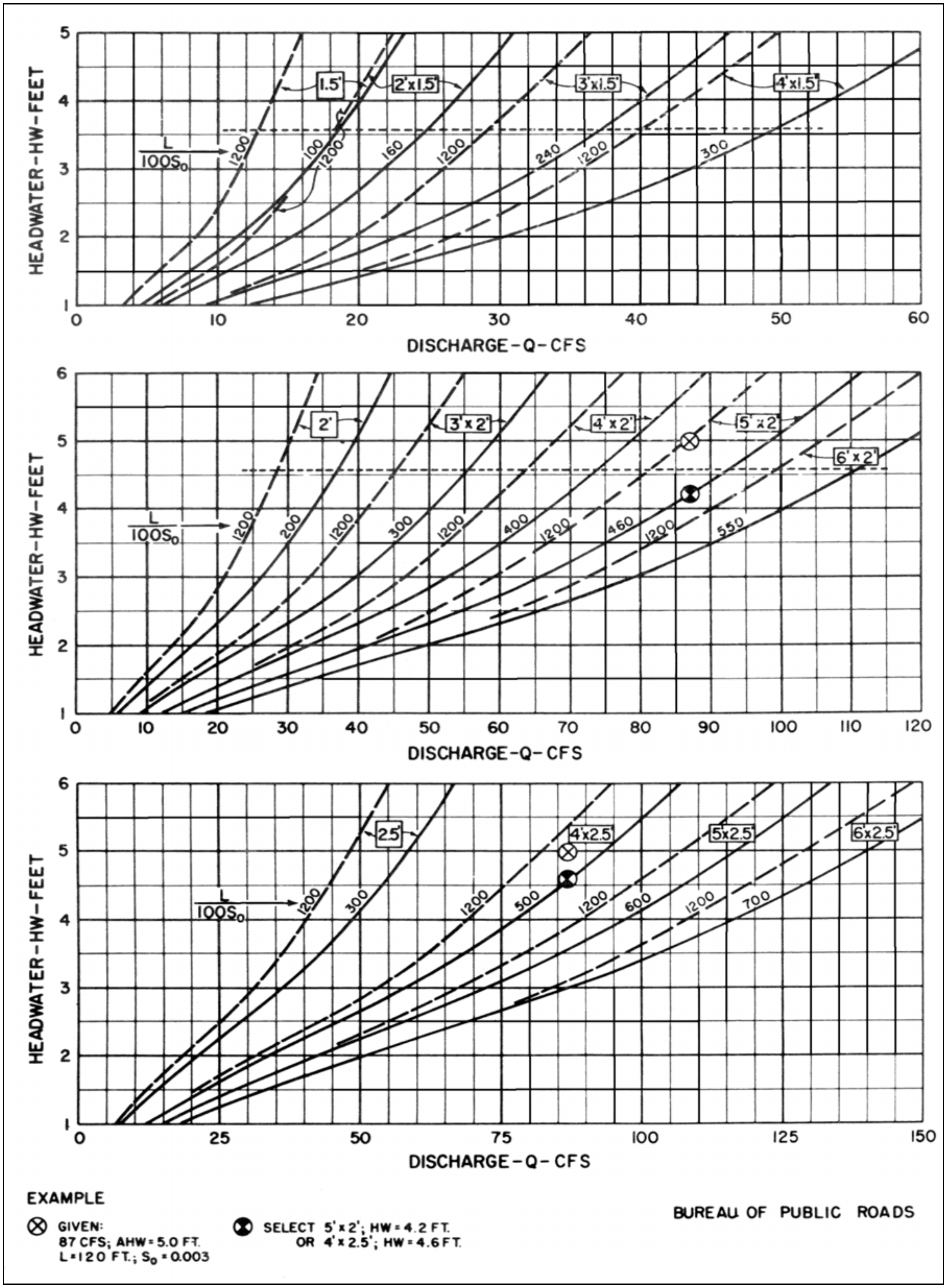


**Figure 4-83. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 3 ft and 4 ft Heights**

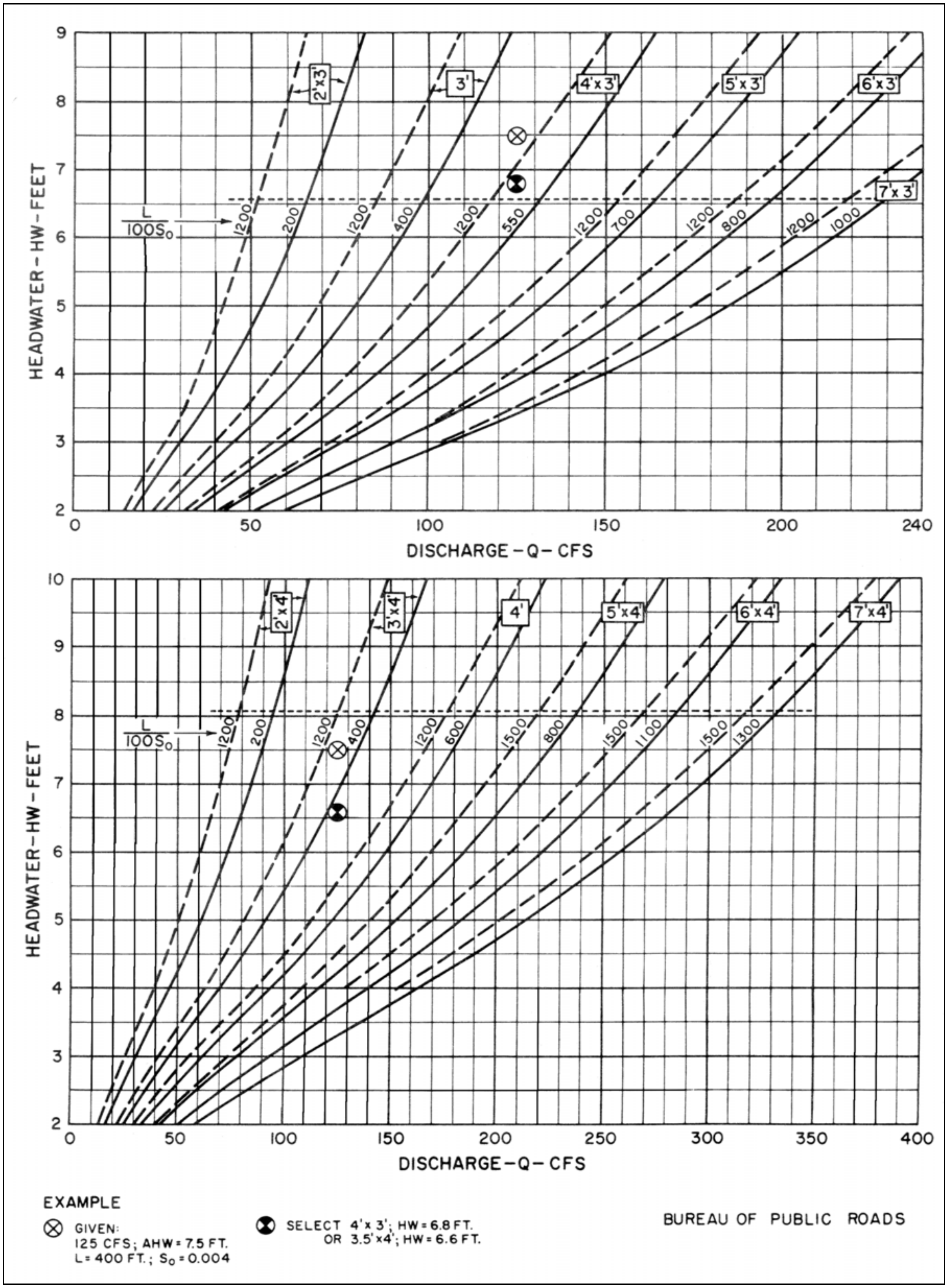
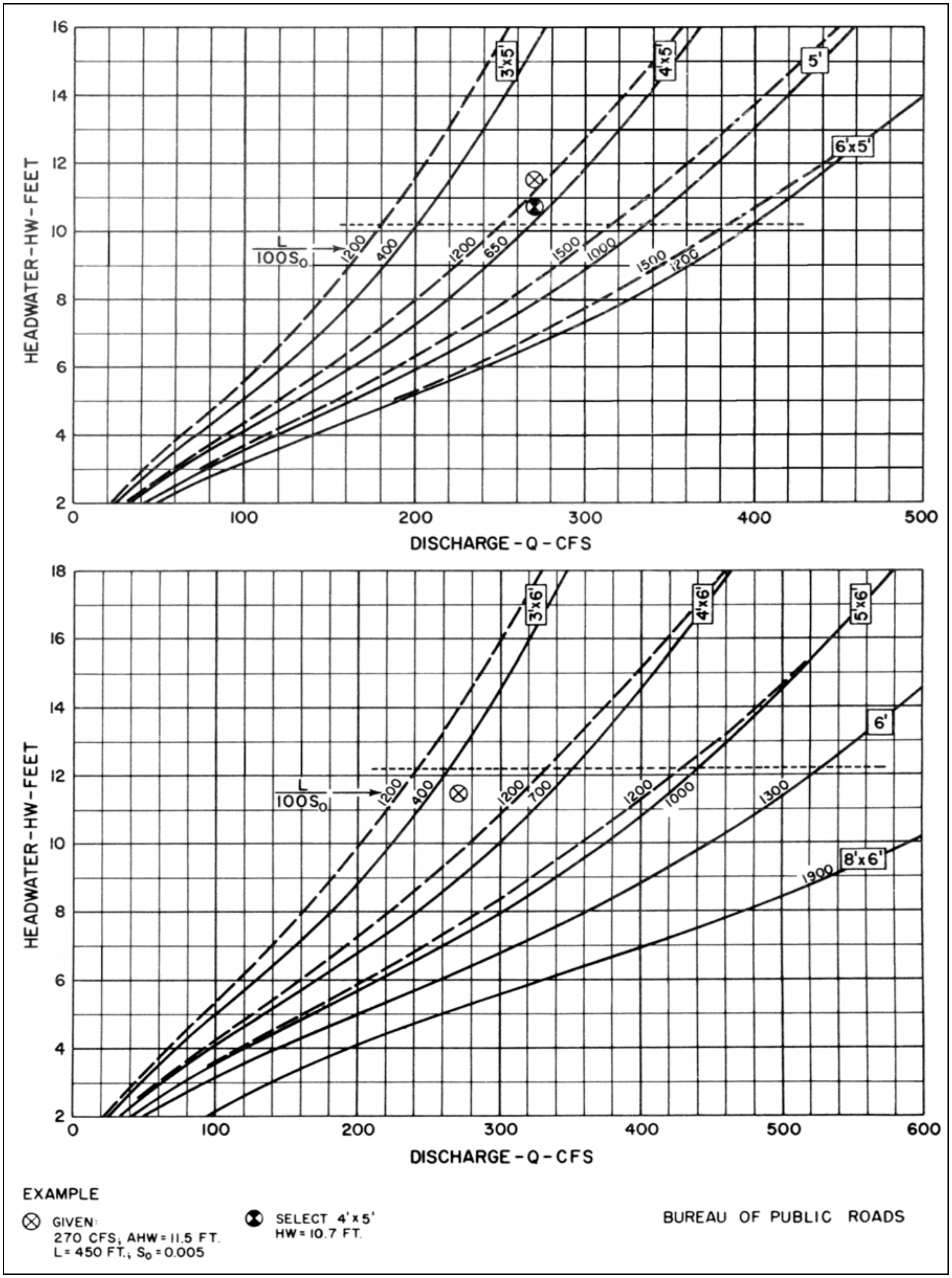


**Figure 4-84. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 5 ft and 6 ft Heights**



4-4.6.2 Storm-drain systems will be so designed that the hydraulic gradeline for the computed design discharge in as near optimum depth as practicable and velocities are not less than 2.5 ft/sec (nominal minimum for cleansing) when the drains are one-third or more full. To minimize the possibility of clogging and to facilitate cleaning, the minimum pipe diameter or box section height will generally be not less than 12 in.; use of smaller size must be fully justified. Tentative size selections for capacity flow may be made from the nomography for computing required size of circular drains in Section 3-11. Problems attending high-velocity flow should be carefully analyzed, and appropriate provisions made to insure a fully functional project.

4-4.6.3 Site topography will dictate the location of possible outlets and the general limiting grades for the system. Storm drain depths will be held to the minimum consistent with limitations imposed by cover requirements, proximity of other structures, interference with other utilities, and velocity requirements because deep excavation is expensive. Usually in profile, proceeding downstream, the crowns of conduits whose sizes progressively increase will be matched, the invert grade dropping across the junction structure; similarly, the crowns of incoming laterals will be matched to that of the main line. If the downstream conduit is smaller as on a steep slope, its invert will be matched to that of the upstream conduit. Some additional lowering of an outgoing pipe may be required to compensate for pressure loss within a junction structure.

4-4.6.4 Manholes or junction boxes usually will be provided at points of change in conduit grade or size, at junctions with laterals of branches and wherever entry for maintenance is required. Distance between points of entry will be not more than approximately 300 ft for conduits with a minimum dimension smaller than 30 in. If the storm drain will be carrying water at a velocity of 20 ft/sec or greater, with high energy and strong forces present, special attention must be given such items as alignment, junctions, anchorage requirements, joints, and selection of materials.

### 4-4.7 **Inlets**

4-4.7.1 Storm-drain inlet structures to intercept surface flow are of three general types: drop, curb, and combination. Hydraulically, they may function as either weirs or orifices depending mostly on the inflowing water. The allowable depth for design storm conditions and consequently the type, size and spacing of inlets will depend on the topography of the surrounding area, its use, and consequences of excessive depths. Drop inlets, which are provided with a grated entrance opening, are in general more efficient than curb inlets and are useful in sumps, roadway sags, swales, and gutters. Such inlets are commonly depressed below the adjacent grade for improved interception or increased capacity. Curb inlets along sloping gutters require a depression for adequate interception. Combination inlets may be used where some additional capacity in a restricted space is desired. Simple grated inlets are most susceptible to blocking by trash. Also, in housing areas, the use of grated drop inlets should be kept to a reasonable minimum, preference being given to the curb type of opening. Where an abnormally high curb opening is needed, pedestrian safety may require one or more protective bars across the opening. Although curb openings are less susceptible to blocking by trash, they are also less efficient for interception on hydraulically steep slopes, because of the difficulty of turning the flow into them. Assurance of satisfactory performance by any system of inlets requires careful consideration of the several factors involved. The final selection of inlet types will be based on overall hydraulic performance, safety requirements, and reasonableness of cost for construction and maintenance.

4-4.7.2 In placing inlets to give an optimum arrangement for flow interception, the following guides apply:

4-4.7.2.1 At street intersections and crosswalks, inlets are usually placed on the upstream side. Gutters to transport flow across streets or roadways will not be used.

4-4.7.2.2 At intermediate points on grades, the greatest efficiency and economy commonly result if either grated or curb inlets are designed to intercept only about three-fourths of the flow.

4-4.7.2.3 In sag vertical curves, three inlets are often desirable, one at the low point and one on each side of the low point where the gutter grade is about 0.2 ft above the low point. Such a layout effectively reduces pond buildup and deposition of sediment in the low area.

4-4.7.2.4 Large quantities of surface runoff flowing toward main thoroughfares normally should be intercepted before reaching them.

4-4.7.2.5 At a bridge with curbed approaches, gutter flow should be intercepted before it reaches the bridge, particularly where freezing weather occurs.

4-4.7.2.6 Where a road pavement on a continuous grade is warped in transitions between superelevated and normal sections, surface water should normally be intercepted upstream of the point where the pavement cross slope begins to change, especially in areas where icing occurs.

4-4.7.2.7 On roads where curbs are used, runoff from cut slopes and from off-site areas should, wherever possible, be intercepted by ditches at the tops of slopes or in swales along the shoulders and not be allowed to flow onto the roadway. This practice minimizes the amount of water to be intercepted by gutter inlets and helps to prevent mud and debris from being carried onto the pavement.

4-4.7.3 Inlets placed in sumps have a greater potential capacity than inlets on a slope because of the possible submergence in the sump. Capacities of grated, curb, and combination inlets in sumps will be computed as outlined below. To allow for blockage by trash, the size of inlet opening selected for construction will be increased above the computed size by 100 percent for grated inlets and 25 to 75 percent, depending on trash conditions, for curb inlets and combination inlets.

4-4.7.3.1 **Grated type (in sump)**

a. For depths of water up to 0.4 ft use the weir formula:

$$Q = 3.0LH^{3/2} \quad \text{(eq. 4-30)}$$

If one side of a rectangular grate is against a curb, this side must be omitted in computing the perimeter.

b. For depths of water above 1.4 ft use the orifice formula:

$$Q = 0.6A\sqrt{2gH} \quad \text{(eq. 4-31)}$$

c. For depths between 0.4 and 1.4 ft, operation is indefinite due to vortices and other disturbances. Capacity will be somewhere between those given by the preceding formulas.

d. Problems involving the above criteria may be solved graphically by use of Figure 4-85.

4-4.7.3.2 **Curb type (in sump)**. For a curb inlet in a sump, the above listed general concepts for weir and orifice flow apply, the latter being in effect for depths greater than about 1.4 h (where h is the height of curb opening entrance). Figure 4-86 presents a graphic method for estimating capacity.

4-4.7.3.3 **Combination type (in sump)**. For a combination inlet in a sump no specific formulas are given. Some increase in capacity over that provided singly by either a grated opening or a curb opening may be expected, and the curb opening will operate as a relief opening if the grate becomes clogged by debris. In estimating the capacity, the inlet will be treated as a simple grated inlet, but a safety factor of 25 to 75 percent will be applied.

4-4.7.3.4 **Slotted drain type**. For a slotted drain inlet in a sump, the flow will enter the slot as either all orifice type or all weir type, depending on the depth of water at the edge of the slot. If the depth is less than .18 ft, the length of slot required to intercept total flow is equal to:

$$\frac{Q}{3.125\, d^{3}/2} \quad \text{(eq. 4-32)}$$

If the depth is greater than .18 feet, the length of slot required to intercept total flow is equal to:

$$\frac{Q}{.5\, w\, \sqrt{2gd}} \quad \text{(eq. 4-33)}$$

d = depth of flow-in.

w = width of slot-.146 ft

**Figure 4-85. Capacity of Grate Inlet in Sump Water Pond on Grate**

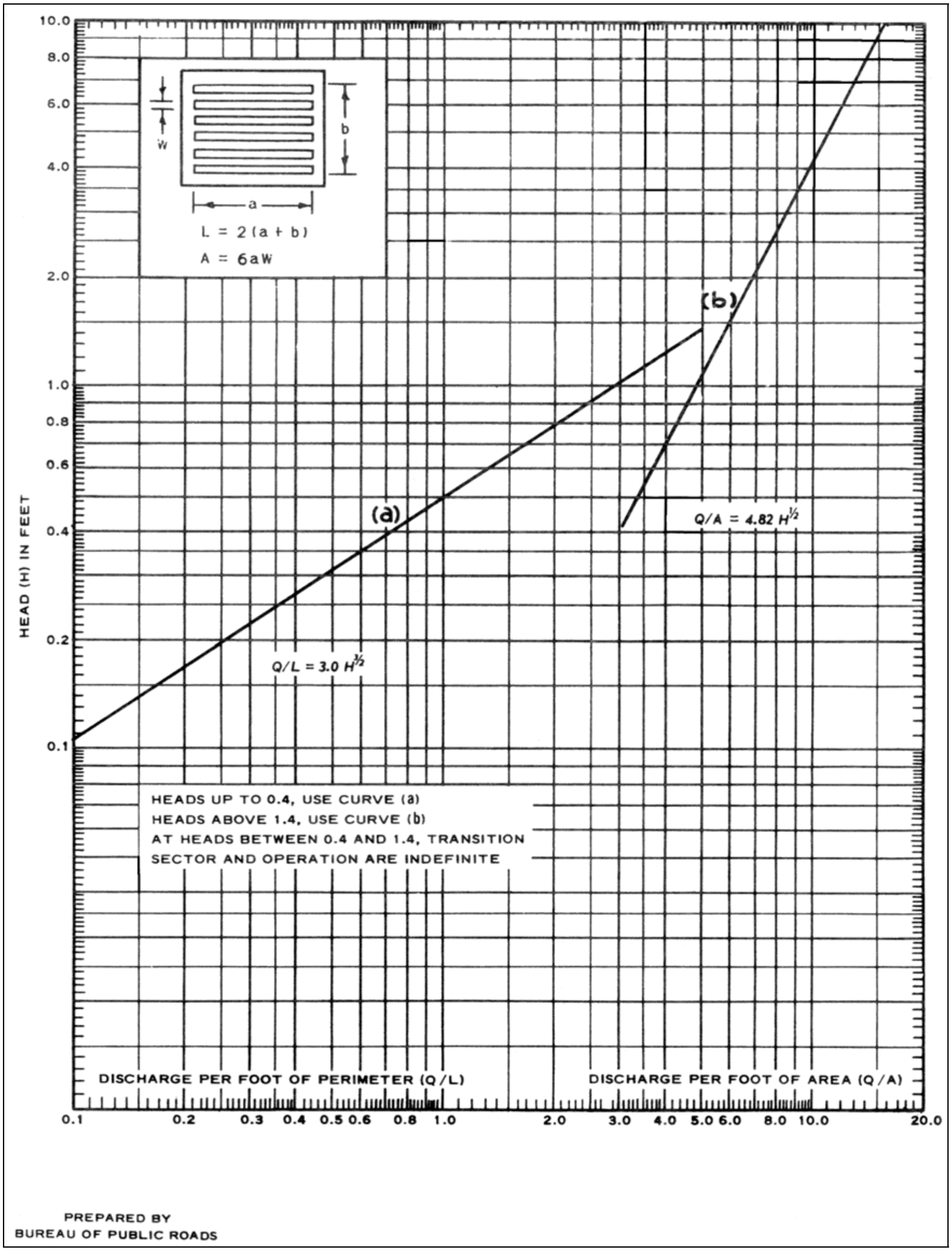
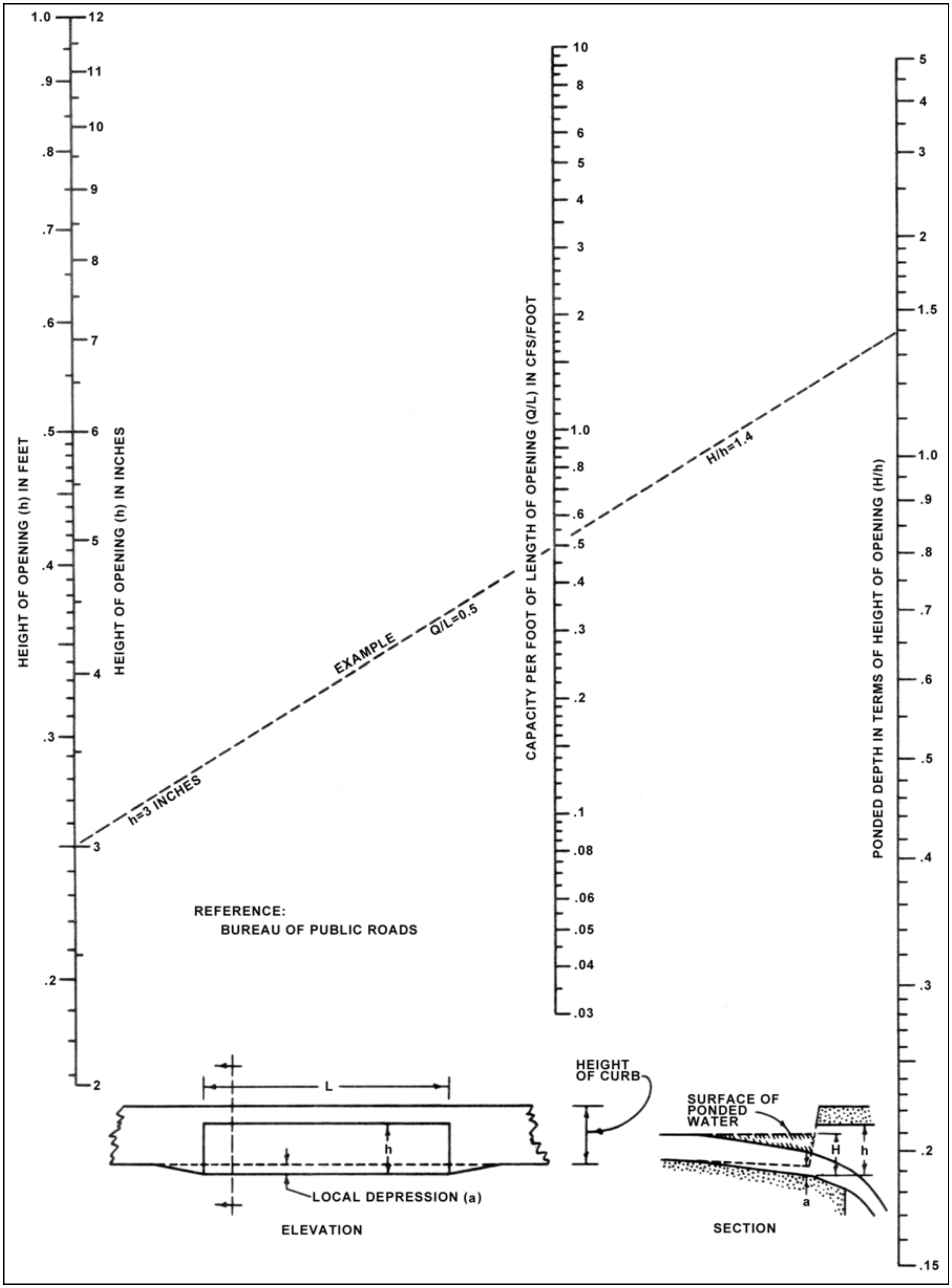


**Figure 4-86. Capacity of Curb Opening Inlet at Low Point in Grade**



4-4.7.4 Each of a series of inlets placed on a slope is usually, for optimum efficiency, designed to intercept somewhat less than the design gutter flow, the remainder being passed to downstream inlets. The amount that must be intercepted is governed by whatever width and depth of bypassed flow can be tolerated from a traffic and safety viewpoint. Such toleration levels will nearly always be influenced by costs of drainage construction. With the flat street crowns prevalent in modern construction, many gutter flows are relatively wide and in built-up areas some inconveniences are inevitable, especially in regions of high rainfall, unless an elaborate inlet system is provided. The achievement of a satisfactory system at reasonable cost requires careful consideration of use factors and careful design of the inlets themselves. However, it must also be remembered that a limitation on types and sizes for a given project is also desirable, for standardization will lead to lower construction costs. Design of grated, curb, and combination inlets on slopes will be based on principles outlined below.

4-4.7.4.1 **Grated type (on slope)**. A grated inlet placed in a sloping gutter will provide optimum interception of flow if the bars are placed parallel to the direction of flow, if the openings total at least 50 percent of the width of the grate (i.e., normal to the direction of flow), and if the unobstructed opening is long enough (parallel to the direction of flow) that the water falling through will clear the downstream end of the opening. The minimum length of clear opening required depends on the depth and velocity of flow in the approach gutter and the thickness of the grate at the end of the slot. This minimum length may be estimated by the partly empirical formula:

$$L = \frac{V}{2}\sqrt{y + d} \qquad \text{(eq. 4-34)}$$

A rectangular grated inlet in a gutter on a continuous grade can be expected to intercept all the water flowing in that part of the gutter cross section that is occupied by the grating plus an amount that will flow in along the exposed sides. However, unless the grate is over 3 ft long or greatly depressed (extreme warping of the pavement is seldom permissible), any water flowing outside the grate width can be considered to bypass the inlet. The quantity of flow in the prism intercepted by such a grate can be computed by following instruction 3 in Figure 4-41. For a long grate the inflow along the side can be estimated by considering the edge of the grate as a curb opening whose effective length is the total grate length (ignoring crossbars) reduced by the length of the jet directly intercepted at the upstream end of the grate. To attain the optimum capacity of an inlet consisting of two grates separated by a short length of paved gutter, the grates should be so spaced that the carryover from the upstream grate will move sufficiently toward the curb to be intercepted by the downstream grate.

4-4.7.4.2 **Curb type (on slope)**. In general, a curb inlet placed on a grade is a hydraulically inefficient structure for flow interception. A relatively long opening is required for complete interception because the heads are normally low and the direction of oncoming flows is not favorable. The cost of a long curb inlet must be weighed against that of a drop type with potentially costly grate. The capacity of a curb inlet intercepting all the flow can be calculated by an empirical equation. The equation is a function of length of clear opening of the inlet, depth of depression of flow line at inlet in feet, and the depth of flow in approach gutter in feet. Depression of the inlet flow line is an essential part of good design, for a curb inlet with no depression is very inefficient. The flow intercepted may be markedly increased without changing the opening length if the flow line can be depressed by one times the depth of flow in the approach gutter. The use of long curb openings with intermediate supports should generally be avoided because of the tendency for the supports to accumulate trash. If supports are essential, they should be set back several inches from the gutter line.

4-4.7.4.2 **Combination type (on slope)**. The capacity of a combination inlet on a continuous grade is not much greater than that of the grated portion itself, and should be computed as a separate grated inlet except in the following situations. If the curb opening is placed upstream from the grate, the combination inlet can be considered to operate as two separate inlets and the capacities can be computed accordingly. Such an arrangement is sometimes desirable, for in addition to the increased capacity the curb opening will tend to intercept debris and thereby reduce clogging of the grate. If the curb opening is placed downstream from the grate, effective operation as two separate inlets requires that the curb opening be sufficiently downstream to allow flow bypassing the grate to move into the curb opening. The minimum separation will vary with both the cross slope and the longitudinal slope.

4-4.7.5 Structural aspects of inlet construction should generally be as indicated in Figures 4-87, 4-88, and 4-89 which show respectively, standard circular grate inlets, types A and B; typical rectangular grate combination inlet, type C; and curb inlet, type D. It will be noted that the type D inlet provides for extension of the opening by the addition of a collecting trough whose backwall is cantilevered to the curb face. Availability of gratings and standards of municipalities in a given region may limit the choice of inlet types. Grated inlets subject to heavy wheel loads will require grates of precast steel or of built-up, welded steel. Steel grates will be galvanized or bituminous coated. Unusual inlet conditions will require special design.

## 4-4.8 Vehicular Safety and Hydraulically Efficient Drainage Practice

4-4.8.1 Some drainage structures are potentially hazardous and, if located in the path of an errant vehicle, can substantially increase the probability of an accident. Inlets should be flush with the ground, or should present no obstacle to a vehicle that is out of control. End structures or culverts should be placed outside the designated recovery area wherever possible. If grates are necessary to cover culvert inlets, care must be taken to design the grate so that the inlet will not clog during periods of high water. Where curb inlet systems are used, setbacks should be minimal, and grates should be designed for hydraulic efficiency and safe passage of vehicles. Hazardous channels or energy dissipating devices should be located outside the designated recovery area or adequate guardrail protection should be provided.

4-4.8.2 It is necessary to emphasize that liberties should not be taken with the hydraulic design of drainage structures to make them safer unless it is clear that their function and efficiency will not be impaired by the contemplated changes. Even minor changes at culvert inlets can seriously disrupt hydraulic performance.

**Figure 4-87. Standard Type "A" and Type "B" Inlets**

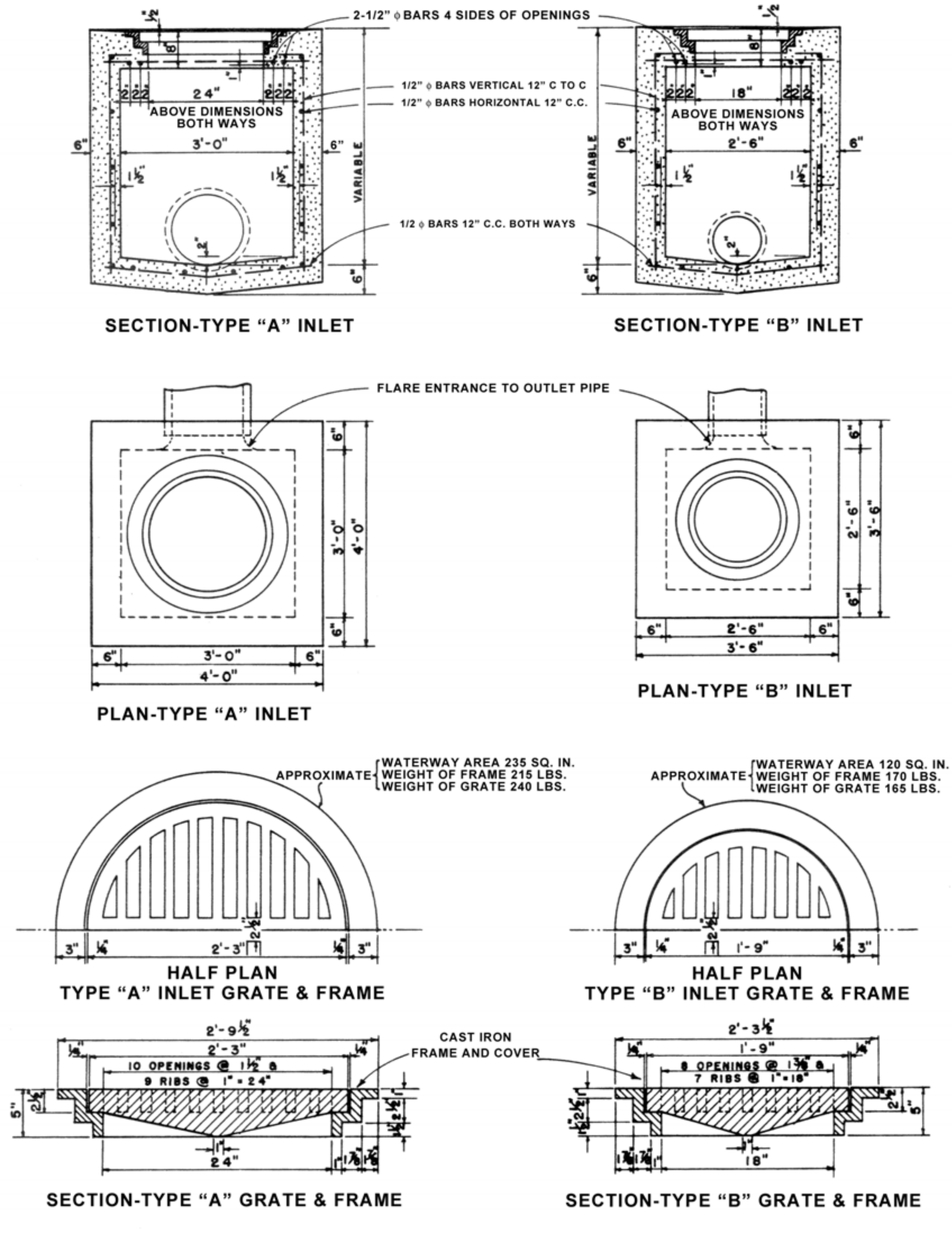



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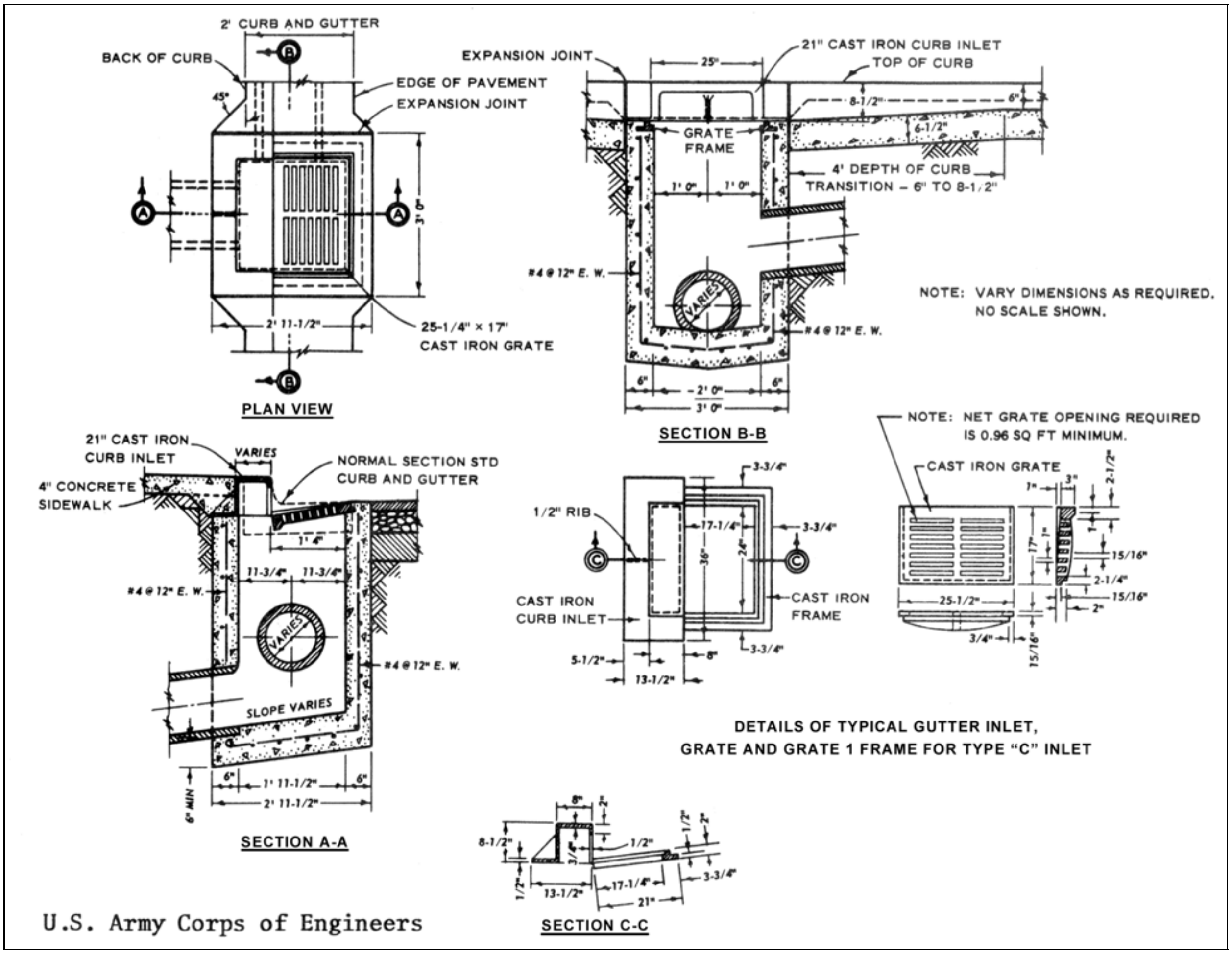
**Figure 4-88. Type "C" Inlet—Square Grating**

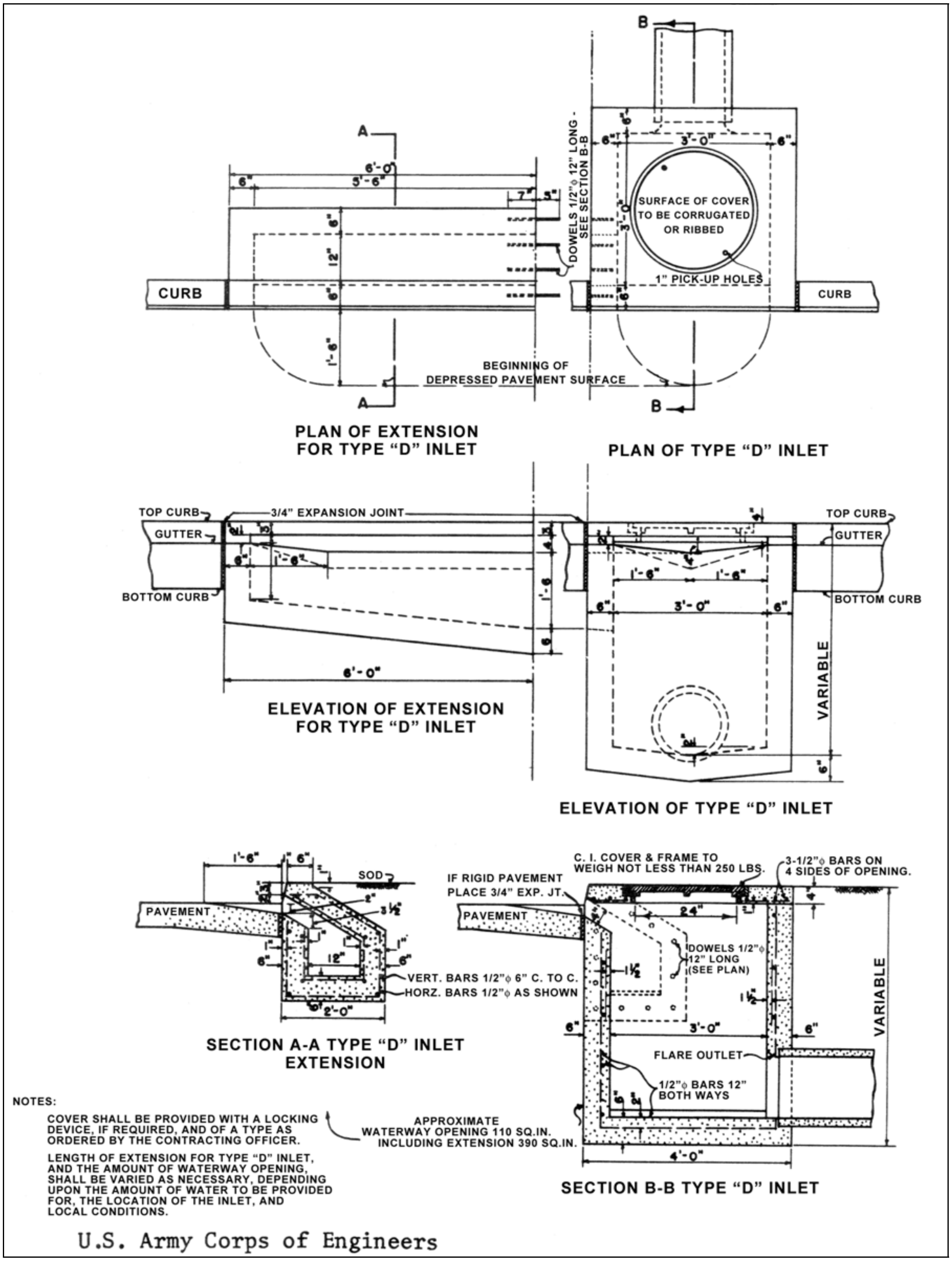



4-4.9 **Pipe Strength, Cover, and Bedding**. A drainage pipe is defined as a structure (other than a bridge) to convey water through a trench or under a fill or some other obstruction. Materials for permanent-type installations include non-reinforced concrete, reinforced concrete, corrugated steel, asbestos-cement, clay, corrugated aluminum alloy, and structural plate steel pipe.

4-4.9.1 **Selection of type of pipe**

4-4.9.1.1 The selection of a suitable construction conduit will be governed by the availability and suitability of pipe materials for local conditions with due consideration of economic factors. It is desirable to permit alternates so that bids can be received with contractor's options for the different types of pipe suitable for a specific installation. Allowing alternates serves as a means of securing bidding competition. When alternate designs are advantageous, each system will be economically designed, taking advantage of full capacity, best slope, least depth, and proper strength and installation provisions for each material involved. Where field conditions dictate the use of one pipe material in preference to others, the reasons will be clearly presented in the design analysis.

**Figure 4-89. Standard Type "D" Inlet**



4-4.9.1.2 Several factors should be considered in selecting the type of pipe to be used in construction. The factors include strength under either maximum or minimum cover being provided, pipe bedding and backfill conditions, anticipated loadings, length of pipe sections, ease of installation, resistance to corrosive action by liquids carried or surrounding soil materials, suitability of jointing methods, provisions for expected deflection without adverse effect on the pipe structure or on the joints or overlying materials, and cost of maintenance. Although it is possible to obtain an acceptable pipe installation to meet design requirements by establishing special provisions for several possible materials, ordinarily only one or two alternates will economically meet the individual requirements for a proposed drainage system.

4-4.9.2 **Selection of n values.** A designer is continually confronted with what coefficient of roughness n to use in a given situation. The question of whether n should be based on the new and ideal condition of a pipe or on anticipated condition at a later date is difficult to answer. Sedimentation or paved pipe can affect the coefficient of roughness. Table 4-1 gives the n values for smooth interior pipe of any size, shape, or type and for annular and helical corrugated metal pipe both unpaved and 25 percent paved. When n values other than those listed are selected, such values will be amply justified in the design analysis.

4-4.9.3 **Restricted use of bituminous-coated pipe.** Corrugated-metal pipe with any percentage of bituminous coating will not be installed where solvents can be expected to enter the pipe. Polymeric coated corrugated steel pipe is recommended where solvents might be expected.

4-4.9.4 **Minimum cover**

4-4.9.4.1 In the design and construction of the drainage system it will be necessary to consider both minimum and maximum earth cover allowable on the underground conduits to be placed under both flexible and rigid pavements. Underground conduits are subject to two principal types of loads: dead loads (DL) caused by embankment or trench backfill plus superimposed stationary surface loads, uniform or concentrated; and live or moving loads (LL), including impact. Live loads assume increasing importance with decreasing fill height.

4-4.9.4.2 AASHTO Standard Specifications for Highway Bridges should be used for all H-20 Highway Loading Analyses. AREA Manual for Railway Engineering should be used for all Cooper's E 80 Railway Loadings. Appropriate pipe manufacturer design manuals should be used for maximum cover analyses.

4-4.9.4.3 Drainage systems should be designed in order to provide an ultimate capacity sufficient to serve the planned installation, Addition to, or replacement of, drainage lines following initial construction is costly.

4-4.9.4.4 Investigations of in-place drainage and erosion control facilities at 50 military installations were made during the period 1966 to 1972. The facilities observed varied from one to more than 30 years of age. The study revealed that buried conduits and associated storm drainage facilities installed from the early 1940s until the mid-1960s appeared to be in good to excellent structural condition. However, many reported failures of buried conduits occurred during construction. Therefore, it should be noted that minimum conduit cover requirements are not always adequate during construction. When construction equipment, which may be heavier than live loads for which the conduit has been designed, is operated over or near an already in-place underground conduit, it is the responsibility of the contractor to provide any additional cover during construction to avoid damage to the conduit. Major improvements in the design and construction of buried conduits in the two decades mentioned include, among other items, increased strength of buried pipes and conduits, increased compaction requirements, and revised minimum cover tables.

4-4.9.4.5 The necessary minimum cover in certain instances may determine pipe grades. A safe minimum cover design requires consideration of a number of factors including selection of conduit material, construction conditions and specifications, selection of pavement design, selection of backfill material and compaction, and the method of bedding underground conduits. Emphasis on these factors must be carried from the design stage through the development of final plans and specifications.

4-4.9.4.6 Tables 4-6 through 4-11 identify certain suggested cover requirements for storm drains and culverts which should be considered as guidelines only. Cover requirements have been formulated for asbestos-cement pipe, reinforced and non-reinforced concrete pipe, corrugated-aluminum-alloy pipe, corrugated-steel pipe, structural-plate-aluminum- alloy pipe, and structural-plate-steel pipe. The different sizes and materials of conduit and pipe have been selected to allow the reader an appreciation for the many and varied items which are commercially available for construction purposes. The cover depths listed are suggested only for average bedding and backfill conditions. Deviations from average conditions may result in significant minimum cover requirements and separate cover analyses must be made in each instance of a deviation from average conditions. Specific bedding, backfill, and trench widths may be required in certain locations; each condition deviating from the average condition should be analyzed separately. Where warranted by design analysis the suggested maximum cover may be exceeded.

4-4.9.5 **Classes of bedding and installation**. Figures 4-1 through 4-4 indicate the classes of bedding for conduits. Figure 4-5 is a schematic representation of the subdivision of classes of conduit installation which influences loads on underground conduits.

4-4.9.6 **Strength of pipe**. Pipe shall be considered of ample strength when it meets the conditions specified for the loads indicated in Tables 4-6 through 4-13. When railway or vehicular wheel loads or loads due to heavy construction equipment (live loads, LL) impose heavier loads, or when the earth (or dead loads, DL) vary materially from those normally encountered, these tables cannot be used for pipe installation design and separate analyses must be made. The suggested minimum and maximum cover shown in the tables pertain to pipe installations in which the back fill material is compacted to at least 90 percent of CE55 (MIL-STD-621) or AASHTO-T99 density (100 percent for cohesionless sands and gravels). This does not modify requirements for any greater degree of compaction specified for other reasons. It is emphasized that proper bedding, backfilling, compaction, and prevention of infiltration of backfill material into pipe are important not only to the pipe, but also to protect overlying and nearby structures. When in doubt about minimum and maximum cover for local conditions, a separate cover analysis must be performed.

4-4.9.7 **Rigid pipe**. Tables 4-6 and 4-7 indicate maximum and minimum cover for trench conduits employing asbestos-cement pipe and concrete pipe. If positive projecting conduits are employed, they are those which are installed in shallow bedding with a part of the conduit projecting above the surface of the natural ground and then covered with an embankment. Due allowance will be made in amounts of minimum and maximum cover for positive projecting conduits. Table 4-14 suggests guidelines for minimum cover to protect the pipe during construction and the minimum finished height of cover.

4-4.9.8 **Flexible pipe**. Suggested maximum cover for trench and positive projecting conduits are indicated in Tables 4-8 through 4-11 for corrugated-aluminum-alloy pipe, corrugated-steel pipe, structural-plate-aluminum-alloy pipe, and structural-plate-steel pipe. Conditions other than those stated in the tables, particularly other loading conditions will be compensated for as necessary. For unusual installation conditions, a detailed analysis will be made so that ample safeguards for the pipe will be provided with regard to strength and resistance to deflection due to loads. Determinations for deflections of flexible pipe should be made if necessary. For heavy live loads and heavy loads due to considerable depth of cover, it is desirable that a selected material, preferably bank-run gravel or crushed stone where economically available, be used for backfill adjacent to the pipe. Table 4-14 suggests guidelines for minimum cover to protect the pipe during construction and the minimum finished height of cover.

4-4.9.9 **Bedding of pipe (culverts and storm drains)**. The contact between a pipe and the foundation on which it rests is the pipe bedding. It has an important influence on the supporting strength of the pipe. For drainpipes at military installations, the method of bedding shown in Figure 4-3 is generally satisfactory for both trench and positive projecting (embankment) installations. Some designs standardize and classify various types of bedding in regard to the shaping of the foundation, use of granular material, use of concrete, and similar special requirements. Although such refinement is not considered necessary, at least for standardized cover requirements, select, fine granular material can be used as an aid in shaping the bedding, particularly where foundation conditions are difficult. Also, where economically available, granular materials can be used to good advantage for backfill adjacent to the pipe. When culverts or storm drains are to be installed in unstable or yielding soils, under great heights of fill, or where pipe will be subjected to very heavy live loads, a method of bedding can be used in which the pipe is set in plain or reinforced concrete of suitable thickness extending upward on each side of the pipe. In some instances, the pipe may be totally encased in concrete or concrete may be placed along the side and over the

**Table 4-6. Suggested Maximum Cover Requirements for Asbestos-Cement Pipe H-20 Highway Loading**

| Diameter in. | Suggested Maximum Cover Above Top of Pipe, ft Circular Section Class | | | | |
|---|---|---|---|---|---|
| | 1500 | 2000 | 2500 | 3000 | 3750 |
| 12 | 9 | 13 | 16 | 19 | 24 |
| 15 | 10 | 13 | 17 | 19 | 24 |
| 18 | 10 | 13 | 17 | 20 | 25 |
| 21 | 10 | 13 | 17 | 20 | 25 |
| 24 | 10 | 14 | 17 | 20 | 25 |
| 27 | 10 | 14 | 17 | 20 | 25 |
| 30 | 11 | 14 | 17 | 21 | 24 |
| 33 | 11 | 14 | 17 | 21 | 26 |
| 36 | 11 | 14 | 17 | 21 | 26 |
| 42 | 11 | 14 | 17 | 21 | 26 |

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Notes:
1. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. The number in the class designation for asbestos-cement pipe is the minimum 3-edge test load to produce failure in pounds per linear foot. It is independent of pipe diameter. An equivalent to the D-load can be obtained by dividing the number in the class designation by the internal pipe diameter in feet.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then coyer depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-7. Suggested Maximum Cover Requirements for Concrete Pipe, Reinforced Concrete, H-20 Highway Loading**

| Diameter in. | Suggested Maximum Cover Above Top of Pipe, ft | | | | |
|---|---|---|---|---|---|
| | Circular Section | | | | |
| | Class | | | | |
| | 1500 | 2000 | 2500 | 3000 | 3750 |
| 12 | 9 | 13 | 16 | 19 | 24 |
| 24 | 10 | 13 | 17 | 19 | 24 |
| 36 | 10 | 13 | 17 | 20 | 25 |
| 48 | 10 | 13 | 17 | 20 | 25 |
| 60 | 10 | 14 | 17 | 20 | 25 |
| 72 | 10 | 14 | 17 | 20 | 25 |
| 84 | 11 | 14 | 17 | 21 | 24 |
| 108 | 11 | 14 | 17 | 21 | 26 |

**Non-Reinforced Concrete**

| Diameter in. | Suggested Maximum Cover Above Top of Pipe, ft | | |
|---|---|---|---|
| | Circular Section | | |
| | I | II | III |
| 12 | 14 | 14 | 17 |
| 24 | 13 | 13 | 14 |
| 36 | 9 | 12 | 12 |

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Notes:
1. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. “D” loads listed for the various classes of reinforced-concrete pipe are the minimum required 3-edge test loads to produce ultimate failure in pounds per linear foot of interval pipe diameter.
7. Each diameter pipe in each class designation of non-reinforced concrete has a different D-load value which increases with wall thickness.
8. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
9. See Table 4-14 for suggested minimum cover requirements.

**Table 4-8. Suggested Maximum Cover Requirements for Corrugated-Aluminum-Alloy Pipe, Riveted, Helical, or Welded Fabrication 2-2/3-in. Spacing, 1/2-in.-Deep Corrugations, H-20 Highway Loading**

| Diameter in. | Suggested Maximum Cover Above Top of Pipe, ft | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Circular Section | | | | | Vertically Elongated Section | | | | |
| | Thickness, in. | | | | | Thickness, in. | | | | |
| | .060 | .075 | .105 | .135 | .164 | .060 | 0.075 | .105 | .135 | .164 |
| 12 | 50 | 50 | 86 | 90 | 93 | | | | | |
| 15 | 40 | 40 | 69 | 72 | 74 | | | | | |
| 18 | 33 | 33 | 57 | 60 | 62 | | | | | |
| 24 | 25 | 25 | 43 | 45 | 46 | | | | | |
| 30 | 20 | 20 | 34 | 36 | 37 | | | | | |
| 36 | 16 | 16 | 28 | 30 | 31 | | | | | |
| 42 | 16 | 16 | 28 | 30 | 31 | | | 50 | 52 | 53 |
| 48 | | | 28 | 30 | 31 | | | 43 | 45 | 47 |
| 54 | | | 28 | 30 | 31 | | | | | |
| 60 | | | | 30 | 31 | | | | | |
| 66 | | | | | 31 | | | | | |
| 72 | | | | | 31 | | | | | |

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Notes:
1. Corrugated-aluminum-alloy pipe will conform to the requirements of Federal Specification WW-P-402.
2. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load. Cooper E-80 railway loadings should be independently made.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. Vertical elongation will be accomplished by shop fabrication and will generally be 5 percent of the pipe diameter.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-9. Suggested Maximum Cover Requirements for Corrugated-Steel-Pipe, 2-2/3-in. Spacing, 1/2-in.-Deep Corrugations**

| H-20 Highway Loading | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Maximum Cover Above Top of Pipe, feet | | | | | | | | | | | | |
| Diameter, in. | Riveted – Thickness, in. | | | | | | Helical – Thickness, in. | | | | | |
| | .052 | .064 | .079 | .109 | .138 | .168 | .052 | .064 | .079 | .109 | .138 | .168 |
| 12 | 92 | 92 | 101 | 130 | | | 170 | 213 | 266 | 372 | | |
| 15 | 74 | 74 | 80 | 104 | | | 136 | 170 | 212 | 298 | | |
| 18 | 61 | 61 | 67 | 86 | | | 113 | 142 | 173 | 212 | | |
| 21 | 53 | 53 | 57 | 74 | | | 97 | 121 | 139 | 164 | | |
| 24 | 46 | 46 | 50 | 65 | 68 | | 85 | 106 | 120 | 137 | 155 | |
| 27 | 41 | 41 | 44 | 57 | 60 | | 75 | 94 | 109 | 120 | 133 | |
| 30 | 37 | 37 | 40 | 52 | 54 | | 68 | 85 | 101 | 110 | 119 | |
| 36 | 30 | 30 | 33 | 43 | 45 | | 56 | 71 | 88 | 98 | 103 | |
| 42 | 34 | 34 | 47 | 74 | 77 | 81 | 48 | 60 | 76 | 92 | 95 | 99 |
| 48 | | 30 | 41 | 65 | 68 | 71 | | 53 | 66 | 88 | 91 | 93 |
| 54 | | | 36 | 57 | 60 | 63 | | | 59 | 82 | 88 | 90 |
| 60 | | | | 52 | 54 | 57 | | | | 74 | 86 | 87 |
| 66 | | | | | 49 | 51 | | | | | 85 | 86 |
| 72 | | | | | 45 | 47 | | | | | 79 | 85 |
| 78 | | | | | | 43 | | | | | | 84 |
| 84 | | | | | | 40 | | | | | | 75 |

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the tables are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-10. Suggested Maximum Cover Requirements for Structural-Plate-Aluminum-Alloy Pipe, 9-in. Spacing, 2-1/2-in. Corrugations**

| H-20 Highway Loading | | | | | | | |
|---|---|---|---|---|---|---|---|
| Diameter, in. | Suggested Maximum Cover Above Top of Pipe, ft | | | | | | |
| | Circular Section | | | | | | |
| | Thickness, in. | | | | | | |
| | 0.10 | 0.125 | 0.15 | 0.175 | 0.20 | 0.225 | 0.250 |
| 72 | 24 | 32 | 41 | 48 | 55 | 61 | 64 |
| 84 | 20 | 27 | 35 | 41 | 47 | 52 | 55 |
| 96 | 18 | 24 | 30 | 36 | 41 | 45 | 50 |
| 108 | 16 | 21 | 27 | 32 | 37 | 40 | 44 |
| 120 | 14 | 19 | 24 | 29 | 33 | 36 | 40 |
| 132 | 13 | 17 | 22 | 26 | 30 | 33 | 36 |
| 144 | 12 | 16 | 20 | 24 | 27 | 30 | 33 |
| 156 | | 14 | 18 | 22 | 25 | 28 | 30 |
| 168 | | 13 | 17 | 20 | 23 | 26 | 28 |
| 180 | | | 16 | 19 | 22 | 24 | 26 |

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Notes:
1. Structural-plate-aluminum-alloy pipe will conform to the requirements of Federal Specification WW-P-402.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. The number in the class designation for asbestos-cement pipe is the minimum 3-edge test load to produce failure in pounds per linear foot. It is independent of pipe diameter. An equivalent to the D-load can be obtained by dividing the number in the class designation by the internal pipe diameter in feet.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-11. Suggested Maximum Cover Requirements for Corrugated Steel Pipe, 125-mm Span, 25-mm-Deep Corrugations**

| H-20 Highway Loading | | | | | |
|---|---|---|---|---|---|
| Diameter, in. | Maximum cover above top of pipe, ft | | | | |
| | Helical—thickness, in. | | | | |
| | .064 | .079 | .109 | .138 | .168 |
| 48 | 54 | 68 | 95 | 122 | 132 |
| 54 | 48 | 60 | 84 | 109 | 117 |
| 60 | 43 | 54 | 76 | 98 | 107 |
| 66 | 39 | 49 | 69 | 89 | 101 |
| 72 | 36 | 45 | 63 | 81 | 96 |
| 78 | 33 | 41 | 58 | 75 | 92 |
| 84 | 31 | 38 | 54 | 70 | 85 |
| 90 | 29 | 36 | 50 | 65 | 80 |
| 96 | | 34 | 47 | 61 | 75 |
| 102 | | 32 | 44 | 57 | 70 |
| 108 | | | 42 | 54 | 66 |
| 114 | | | 40 | 51 | 63 |
| 120 | | | 38 | 49 | 60 |

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-12. Suggested Maximum Cover Requirements for Structural Plate Steel Pipe, 6-in. Span, 2-in.-Deep Corrugations**

| H-20 Highway Loading | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Maximum Cover Above Top of Pipe, ft | | | | | | |
| | Thickness, in. | | | | | | |
| Diameter, ft | .109 | .138 | .168 | .188 | .218 | .249 | .280 |
| 5.0 | 46 | 68 | 90 | 103 | 124 | 146 | 160 |
| 5.5 | 42 | 62 | 81 | 93 | 113 | 133 | 145 |
| 6.0 | 38 | 57 | 75 | 86 | 103 | 122 | 133 |
| 6.5 | 35 | 52 | 69 | 79 | 95 | 112 | 123 |
| 7.0 | 33 | 49 | 64 | 73 | 88 | 104 | 114 |
| 7.5 | 31 | 45 | 60 | 68 | 82 | 97 | 106 |
| 8.0 | 29 | 43 | 56 | 64 | 77 | 91 | 100 |
| 8.5 | 27 | 40 | 52 | 60 | 73 | 86 | 94 |
| 9.0 | 25 | 38 | 50 | 57 | 69 | 81 | 88 |
| 9.5 | 24 | 36 | 47 | 54 | 65 | 77 | 84 |
| 10.0 | 23 | 34 | 45 | 51 | 62 | 73 | 80 |
| 10.5 | 22 | 32 | 42 | 49 | 59 | 69 | 76 |
| 11.0 | 21 | 31 | 40 | 46 | 56 | 66 | 72 |
| 11.5 | 20 | 29 | 39 | 44 | 54 | 63 | 69 |
| 12.0 | 19 | 28 | 37 | 43 | 51 | 61 | 66 |
| 12.5 | 18 | 27 | 36 | 41 | 49 | 58 | 64 |
| 13.0 | 17 | 26 | 34 | 39 | 47 | 56 | 61 |
| 13.5 | 17 | 25 | 33 | 38 | 46 | 54 | 59 |
| 14.0 | 16 | 24 | 32 | 36 | 44 | 52 | 57 |
| 14.5 | 16 | 23 | 31 | 35 | 42 | 50 | 55 |
| 15.0 | 15 | 22 | 30 | 34 | 41 | 48 | 53 |
| 15.5 | 15 | 22 | 29 | 33 | 40 | 47 | 51 |
| 16.0 | | 21 | 28 | 32 | 38 | 45 | 50 |
| 16.5 | | 20 | 27 | 31 | 37 | 44 | 48 |
| 17.0 | | 20 | 26 | 30 | 36 | 43 | 47 |
| 17.5 | | 19 | 25 | 29 | 35 | 41 | 45 |
| 18.0 | | | 25 | 28 | 34 | 40 | 44 |
| 18.5 | | | 24 | 27 | 33 | 39 | 43 |
| 19.0 | | | 23 | 27 | 32 | 38 | 42 |
| 19.5 | | | 23 | 26 | 31 | 37 | 41 |
| 20.0 | | | | 25 | 31 | 36 | 40 |
| 20.5 | | | | 25 | 30 | 35 | 39 |
| 21.0 | | | | | 29 | 34 | 38 |
| 21.5 | | | | | 28 | 34 | 37 |
| 22.0 | | | | | 28 | 33 | 36 |
| 22.5 | | | | | 27 | 32 | 35 |
| 23.0 | | | | | | 31 | 34 |
| 23.5 | | | | | | 31 | 34 |
| 24.0 | | | | | | 30 | 33 |
| 24.5 | | | | | | | 32 |
| 25.0 | | | | | | | 32 |
| 25.5 | | | | | | | 31 |

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Notes:

1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

**Table 4-13. Suggested Maximum Cover Requirements for Corrugated Steel Pipe, 3-in. Span, 1-in. Corrugations**

| H-20 Highway Loading | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Diameter, in. | Maximum Cover Above Top of Pipe, ft | | | | | | | | | |
| | Riveted - Thickness, in. | | | | | Helical – Thickness, in. | | | | |
| | .064 | .079 | .109 | .138 | .168 | .064 | .079 | .109 | .138 | .168 |
| 36 | 53 | 66 | 98 | 117 | 130 | 81 | 101 | 142 | 178 | 201 |
| 42 | 45 | 56 | 84 | 101 | 112 | 69 | 87 | 122 | 142 | 157 |
| 48 | 39 | 49 | 73 | 88 | 98 | 61 | 76 | 107 | 122 | 132 |
| 54 | 35 | 44 | 65 | 78 | 87 | 54 | 67 | 95 | 110 | 117 |
| 60 | 31 | 39 | 58 | 70 | 78 | 48 | 61 | 85 | 102 | 107 |
| 66 | 28 | 36 | 53 | 64 | 71 | 44 | 55 | 77 | 97 | 101 |
| 72 | 26 | 33 | 49 | 58 | 65 | 40 | 50 | 71 | 92 | 96 |
| 78 | 24 | 30 | 45 | 54 | 60 | 37 | 47 | 65 | 84 | 93 |
| 84 | 22 | 28 | 42 | 50 | 56 | 34 | 43 | 61 | 78 | 91 |
| 90 | 21 | 26 | 39 | 47 | 52 | 32 | 40 | 57 | 73 | 89 |
| 96 | | 24 | 36 | 44 | 49 | | 38 | 53 | 69 | 84 |
| 102 | | 23 | 34 | 41 | 46 | | 35 | 50 | 64 | 79 |
| 108 | | | 32 | 39 | 43 | | | 47 | 61 | 75 |
| 114 | | | 30 | 37 | 41 | | | 45 | 58 | 71 |
| 120 | | | 29 | 35 | 39 | | | 42 | 55 | 67 |

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

### Table 4-14. Suggested Guidelines for Minimum Cover

| H-20 Highway Loading | | | |
|---|---|---|---|
| | Minimum Cover to Protect Pipe | | Minimum Finished Height of Cover (From Bottom of Subbase, toTop of Pipe) |
| | Pipe Diameter, in. | Height of Cover During Construction, ft | |
| Asbestos-Cement Pipe | 12 to 42 | Diameter/2 or 3.0 ft whichever is greater | Diameter/2 or 2.0 ft whichever is greater |
| Concrete Pipe Reinforced | 12 to 108 | Diameter/2 or 3.0 ft whichever is greater | Diameter/2 or 2.0 ft whichever is greater |
| Non-Reinforced | 12 to 36 | Diameter/2 or 3.0 ft whichever is greater | Diameter/2 or 2.0 ft whichever is greater |
| Corrugated Aluminum Pipe 2-2/3 in. by 1/2 in. | 12 to 24<br>30 and over | 1.5 ft Diameter | Diameter/2 or 1.0 ft whichever is greater<br>Diameter/2 |
| Corrugated Steel Pipe 3 in. by 1 in. | 12 to 30<br>36 and over | 1.5 ft Diameter | Diameter/2 or 1.0 ft whichever is greater<br>Diameter/2 |
| Structural Plate Aluminum Alloy Pipe 9 in. by 2-1/2 in. | 72 and over | Diameter/2 | Diameter/4 |
| Structural Plate Steel 6 in. by 2 in. | 60 and over | Diameter/2 | Diameter/4 |

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Notes:
1. All values shown above are for average conditions and are to be considered as guidelines only.
2. Calculations should be made for minimum cover for all individual pipe installation for pipe underlying roads, streets and open storage areas subject to H-20 live loads.
3. Calculations for minimum cover for all pipe installations should be separately made for all Cooper E-80 railroad live loading.
4. In seasonal frost areas, minimum pipe cover must meet requirements of Table 2-3 of TM 5-820-3 for protection of storm drains.
5. Pipe placed under rigid pavement will have minimum cover from the bottom of the subbase to the top of pipe of 1.0 ft for pipe up to 60 in. and greater than 1.0 ft for sizes above 60 in. if calculations so indicate.
6. Trench widths depend upon varying conditions of construction but may be as wide as is consistent with space required to install the pipe and as deep as can be managed from practical construction methods.
7. Non-reinforced concrete pipe is available in sizes up to 36 in.
8. See Tables 4-6 through 4-13 for suggested minimum cover requirements.

top of the pipe (top or arch encasement) after proper bedding and partial backfilling. Pipe manufacturers will be helpful in recommending type and specific requirements for encased, partially encased, or specially reinforced pipe in connection with design for complex conditions.

4-4.10 **Manholes and Junction Boxes**. Drainage systems require a variety of appurtenances to assure proper operations. Most numerous appurtenances are manholes and junction boxes. Manholes and junction boxes are generally constructed of any suitable materials such as brick, concrete block, reinforced concrete, precast reinforced-concrete sections, or preformed corrugated metal sections. Manholes are located at intersections, changes in alignment or grade, and at intermediate joints in the system up to every 500 ft. Junction boxes for large pipes are located as necessary to assure proper operation of the drainage system. Inside dimensions of manholes will not be less than 2.5 ft. Inside dimensions of junction boxes will provide for not less than 3 in. of wall on either side of the outside diameter of the largest pipes involved. Manhole frames and cover will be provided as required; rounded manhole and box covers are preferred to square covers. Slab top covers will be provided for large manholes and junction boxes too shallow to permit corbeling of the upper part of the structure. A typical large box drain cover is shown in Figure 4-10. Fixed ladders will be provided depending on the depth of the structures. Access to manhole and junction boxes without fixed ladders will be by portable ladders. Manhole and junction box design will insure minimum hydraulic losses through them. Typical manhole and junction box construction is shown in Figures 4-90 through 4-92.

4-4.11 **Detention Pond Storage**. Hydrologic studies of the drainage area will reveal if detention ponds are required. Temporary storage or ponding may be required if the outflow from a drainage area is limited by the capacity of the drainage system serving a given area. A full discussion of temporary storage or ponding design will be found in Section 3-11. Ponding areas should be designed to avoid creation of a facility that would be unsightly, difficult to maintain, or a menace to health or safety.

4-4.12 **Outlet Energy Dissipators**

4-4.12.1 Most drainage systems are designed to operate under normal free outfall conditions. Tailwater conditions are generally absent. However, it is possible for a discharge resulting from a drainage system to possess kinetic energy in excess of that which normally occurs in waterways. To reduce the kinetic energy, and thereby reduce downstream scour, outfalls may sometimes be required to reduce streambed scour. Scour may occur in the streambed if discharge velocities exceed the values listed in Table 4-15. These values are to be used only as guides; studies of local materials must be made prior to a decision to install energy dissipation devices. Protection against scour may be provided by plain outlets, transitions and stilling basins. Plain outlets provide no protective works and depend on natural material to resist erosion. Transitions provide little or no dissipation of energy themselves, but by spreading the effluent jet to approximately the flow cross-section of the natural channel, the energy is greatly reduced prior to releasing the effluent into the outlet channel. Stilling basins dissipate the high kinetic energy of flow by a hydraulic jump or other means. Riprap may be required at any of the three types of outfalls.

**Table 4-15. Maximum Permissible Mean Velocities to Prevent Scour**

| Material | Maximum Permissible Mean Velocity |
|---|---|
| Uniform graded sand and cohesionless silts | 1.5 fps |
| Well-graded sand | 2.5 fps |
| Silty sand | 3.0 fps |
| Clay | 4.0 fps |
| Gravel | 6.0 fps |

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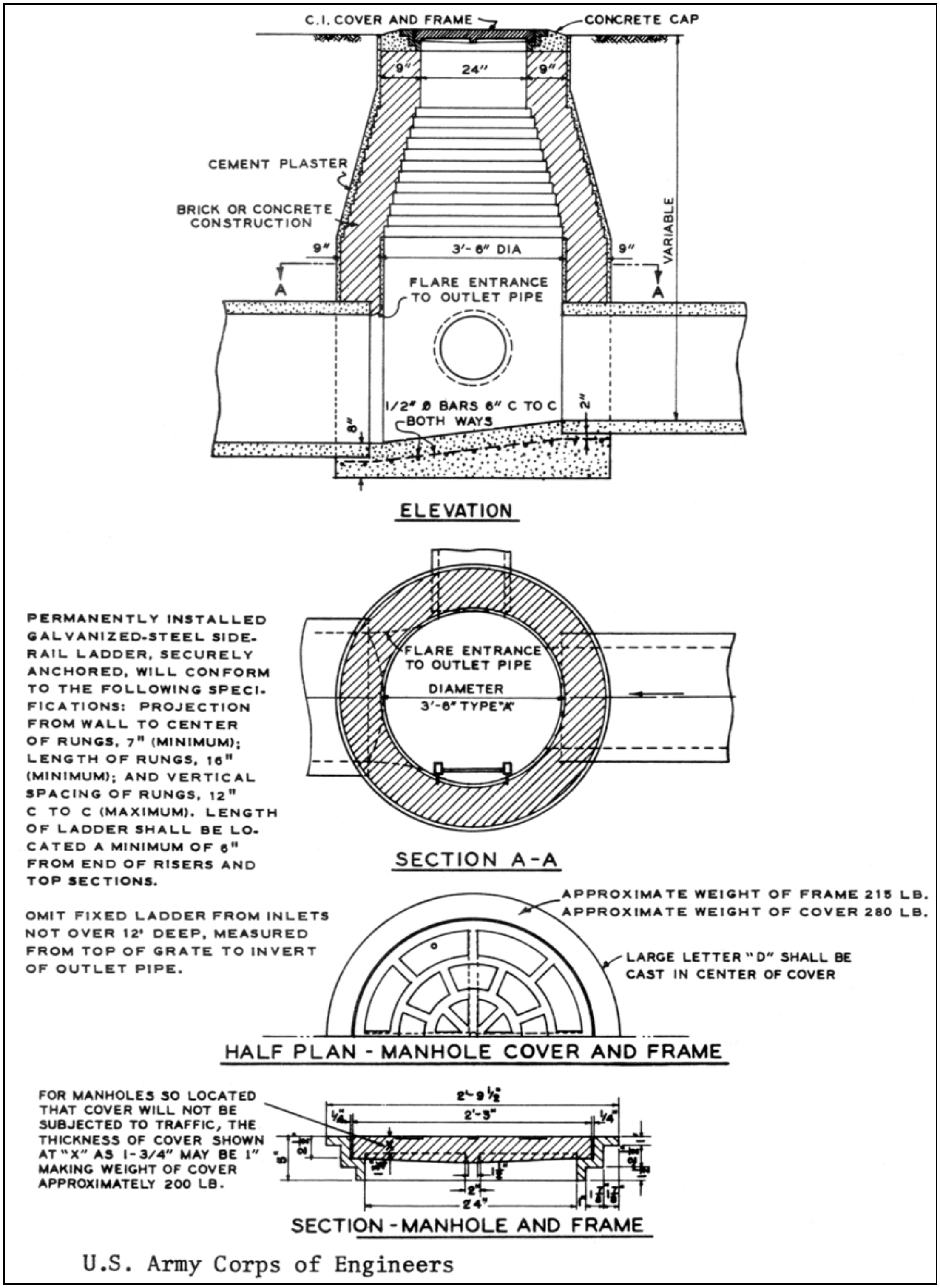
4-4.12.1.1 **Plain type**

a. If the discharge channel is in rock or a material highly resistant to erosion, no special erosion protection is required. However, since flow from the culvert will spread with a resultant drop in water surface and increase in velocity, this type of outlet should be used without riprap only if the material in the outlet channel can withstand velocities about 1.5 times the velocity in the culvert. At such an outlet, side erosion due to eddy action or turbulence is more likely to prove troublesome than is bottom scour.

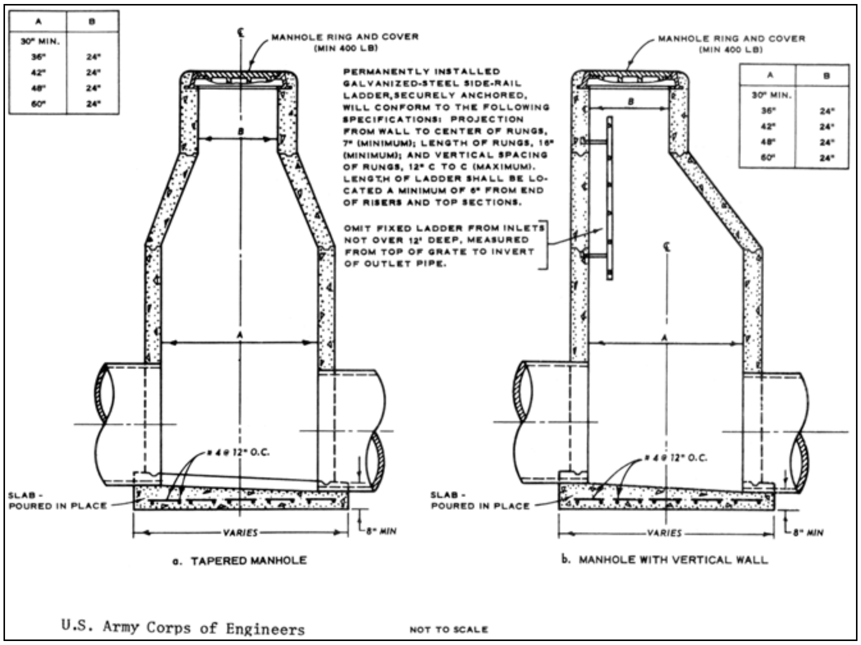
b. Cantilevered culvert outlets may be used to discharge a free-falling jet onto the bed of the outlet channel. A plunge pool will be developed, the depth and size of which will depend on the energy of the falling jet at the tailwater and the erodibility of the bed material.

4-4.12.1.2 **Transition type.** Endwalls (outfall headwalls) serve the dual purpose of retaining the embankment and limiting the outlet transition boundary. Erosion of embankment toes usually can be traced to eddy attack at the ends of such walls. A flared transition is very effective, if proportioned so that eddies induced by the effluent jet do not continue beyond the end of the wall or overtop a sloped wall. As a guide, it is suggested that the product of velocity and flare angle should not exceed 150. That is, if effluent velocity is 5 ft/sec each wingwall may flare 30 degrees; but if velocity is 15 ft/sec, the flare should not exceed 10 degrees. Unless wingwalls can be anchored on a stable foundation, a paved apron between the wingwalls is required. Special care must be taken in design of the structure to preclude undermining. A newly excavated channel may be expected to degrade, and proper allowance for this action should be included in establishing the apron elevation and depth of cutoff wall. Warped endwalls provide excellent transitions in that they result in the release of flow in a trapezoidal section, which generally approximates the cross section of the outlet channel. If a warped transition is placed at the end of a curved section below a culvert, the transition is made at the end of the curved section to minimize the possibility of overtopping due to superelevation of the water surface. A paved apron is required with warped endwalls. Riprap usually is required at the end of a transition-type outlet.

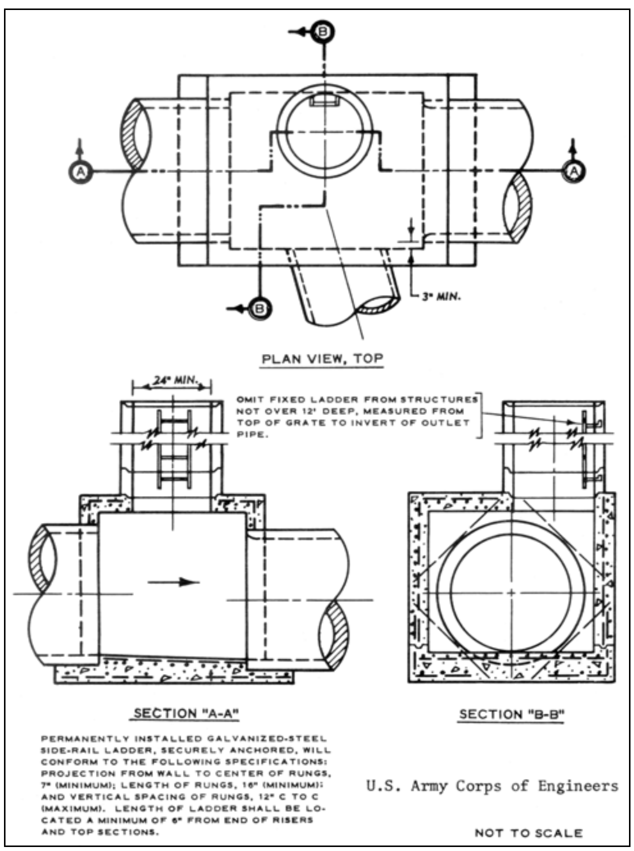
**Figure 4-90. Standard Storm Drain Manhole**



## Figure 4-91. Standard Precast Manholes




## Figure 4-92. Junction Details for Large Pipes



4-4.12.1.3 **Stilling basins.** A detailed discussion of stilling basins for circular storm drain outlets can be found in Section 4-2.6.

4-4.12.2 Improved channels, especially the paved ones, commonly carry water at velocities higher than those prevailing in the natural channels into which they discharge. Often riprap will suffice for dissipation of excess energy. A cutoff wall may be required at the end of a paved channel to preclude undermining. In extreme cases a flared transition, stilling basin, or impact device may be required.

4–4.13 **Drop Structures and Check Dams.** Drop structures and check dams are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. The structures also provide satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 ft and over embankments higher than 5 ft provided the end sill of the drop structure extends beyond the toe of the embankment. The check dam is a modification of the drop structure used for erosion control in small channels where a less elaborate structure is permissible. Pertinent design features are covered in Section 4-2.4.

4-4.14 **Miscellaneous Structures**

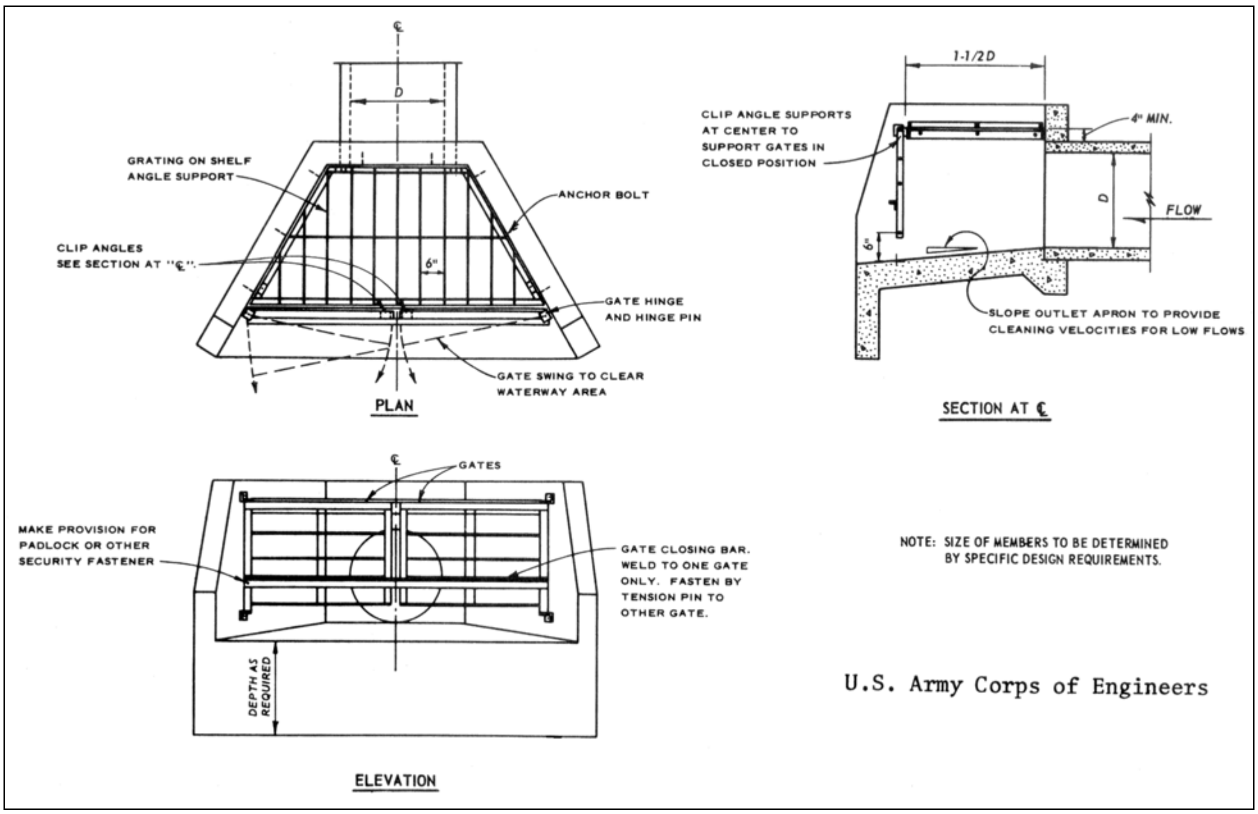
4-4.14.1 A chute is a steep open channel which provides a method of discharging accumulated surface runoff over fills and embankments. A typical design is included in Section 4-2.5.

4-4.14.2 When a conduit or channel passes through or beneath a security fence and forms an opening greater than 96 in.$^2$ in area a security barrier must be installed. Barriers are usually of bars, grillwork, or chain-link screens, parallel bars used to prevent access will be spaced not more than 6 in. apart, and will be of sufficient strength to preclude bending by hand after assembly.

4-4.14.2.1 Where fences enclose maximum security areas such as exclusion and restricted areas, drainage channels, ditches, and equalizers will, wherever possible, be carried under the fence in one or more pipes having an internal diameter of not more than 10 in. Where the volume of flow is such that the multipipe arrangement is not feasible, the conduit or culvert will be protected by a security grill composed of 3/4-in.-diameter rods or 1/2-in. bars spaced not more than 6 in. on center, set and welded in an internal frame. Where rods or bars exceed 18 in. in length, suitable spacer bars will be provided at not more than 18 in. on center, welded at all intersections. Security grills will be located inside the protected area. Where the grill is on the downstream end of the culvert, the grill will be hinged to facilitate cleaning and provided with a latch and padlock, and a debris catcher will be installed in the upstream end of the conduit or culvert. Elsewhere the grill will be permanently attached to the culvert. Security regulations normally require the guard to inspect such grills at least once every shift. For culverts in rough terrain, steps will be provided to the grill to facilitate inspection and cleaning.

4-4.14.2.2 For culverts and storm drains, barriers at the intakes would be preferable to barriers at the outlets because of the relative ease of debris removal. However, barriers at the outfalls are usually essential; in these cases consideration should be given to placing debris interceptors at the inlets. Bars constituting a barrier should be placed in a horizontal position, and the number of vertical members should be limited in order to minimize clogging; the total clear area should be at least twice the area of the conduit or larger under severe debris conditions. For large conduits an elaborate cagelike structure may be required. Provisions to facilitate cleaning during or immediately after heavy runoff should be made. Figure 4-93 shows a typical barrier for the outlet of a pipe drain. It will be noted that a 6-in. underclearance is provided to permit passage of normal bedload material, and that the apron between the conduit outlet and the barrier is placed on a slope to minimize deposition of sediment on the apron during ordinary flow. Erosion protection, where required, is placed immediately downstream from the barrier.

**Figure 4-93. Outlet Security Barrier**




4-4.14.2.3 If manholes must be located in the immediate vicinity of a security fence their covers must be so fastened as to prevent unauthorized opening.

4-4.14.2.4 Open channels may present special problems due to the relatively large size of the waterway and the possible requirements for passage of large floating debris. For such channels, a barrier should be provided that can be unfastened and opened or lifted during periods of heavy runoff or when clogged. The barrier is hinged at the top and an empty tank is welded to it at the bottom to serve as a float. Open channels or swales which drain relatively small areas and whose flows carry only minor quantities of debris may be secured merely by extending the fence down to a concrete sill set into the sides and across the bottom of the channel.

4-4.15 **Notation**

**A** Drainage area, acres, total area of clear opening, or cross-sectional area of flow, $ft^2$

**AHW** Allowable Headwater depth, ft

**B** Width, ft

**C** Coefficient

**D** Height of culvert barrel, ft

**d** Depth of thickness of grate, ft

$\mathbf{d_c}$ Critical depth, ft

**F** Infiltration rate, in/hr

**g** Acceleration due to gravity, $ft/sec^2$

**H** Depth of water, ft

$\mathbf{H_f}$ Headloss due to friction, ft

**HW** Headwater, ft

$\mathbf{h_0}$ Distance from culvert invert at the outlet to the control elevation, ft

**I** Rainfall intensity, in./hr

**i** Hydraulic gradient

**K** Constant

$\mathbf{K_e}$ Coefficient

**k** Coefficient of permeability

**L** Length of slot or gross perimeter of grate opening, or length, ft

$\mathbf{L_1}$ Adjusted length, ft

| | |
|---|---|
| **$L_s$** | Length of spiral, ft (nonsuperelevated channel) |
| **$L_t$** | Length of spiral, ft (superelevated channel) |
| **n** | Manning's roughness coefficient |
| **Q** | Discharge or peak rate of runoff, cfs |
| **R** | Hydraulic radius, ft |
| **$R_c$** | Radius of curvature center line of channel, ft |
| **S** | Slope of energy gradient, ft/ft |
| **$S_0$** | Slope of flow line, ft/ft |
| **T** | Top width at water surface, ft |
| **TW** | Tailwater, ft |
| **V** | Mean velocity of flow, ft/sec |
| **v** | Discharge velocity in Darcy's law, ft/sec |
| **y** | Depth of water, ft |

# CHAPTER 5

# EROSION CONTROL AND RIPRAP PROTECTION

## 5-1 PLANNING AND DESIGN CONSIDERATIONS

5-1.1 Hydraulic structures discharging into open channels will be provided with riprap protection to prevent erosion. Two general types of channel instability can develop downstream from a culvert and storm drain outlet. The conditions are known as either gully scour or a localized erosion referred to as a scour hole. Distinction between the two conditions of scour and prediction of the type to be anticipated for a given field situation can be made by a comparison of the original or existing slope of the channel or drainage basin downstream of the outlet relative to that required for stability.

5-1.2 Gully scour is to be expected when the Froude number of flow in the channel exceeds that required for stability. It begins at a point downstream where the channel is stable and progresses upstream. If sufficient differential in elevation exists between the outlet and the section of stable channel, the outlet structure will be completely undermined. Erosion of this type may be of considerable extent depending upon the location of the stable channel section relative to that of the outlet in both the vertical and downstream directions.

5-1.3 A scour hole or localized erosion is to be expected downstream of an outlet even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. In some instances, the extent of the scour hole may be insufficient to produce either instability of the embankment or structural damage to the outlet. However, in many situations flow conditions produce scour of the extent that embankment erosion as well as structural damage of the apron, end wall, and culvert are evident.

5-1.4 The results of research conducted at U.S. Army Engineer Waterways Experiment Station to determine the extent of localized scour that may be anticipated downstream of culvert and storm-drain outlets has also been published. Empirical equations were developed for estimating the extent of the anticipated scour hole based on knowledge of the design discharge, the culvert diameter, and the duration and Froude number of the design flow at the culvert outlet. These equations and those for the maximum depth, width, length and volume of scour and comparisons of predicted and observed values are discussed in Section 4-2.4.3. Examples of recommended application to estimate the extent of scour in a cohesionless soil and several alternate schemes of protection required to prevent local scour downstream of a circular and rectangular outlet are illustrated in Practical Guidance for Design of Lined Channel Expansions at Culvert Outlets, Technical Report H-74-9.

## 5-2 RIPRAP PROTECTION

5-2.1 Riprap protection should be provided adjacent to all hydraulic structures placed in erosive materials to prevent scour at the ends of the structure. The protection is required on the bed and banks for a sufficient distance to establish velocity gradients and turbulence levels at the end of the riprap approximating conditions in the natural channel. Riprap can also be used for lining the channel banks to prevent lateral erosion and undesirable meandering. Consideration should be given to providing an expansion in either or both the horizontal and vertical direction immediately downstream from hydraulic structures such as drop structures, energy dissipators, culvert outlets or other devices in which flow can expand and dissipate its excess energy in turbulence rather than in a direct attack on the channel bottom and sides.

5-2.2 There are three ways in which riprap has been known to fail: movement of the individual stones by a combination of velocity and turbulence; movement of the natural bed material through the riprap resulting in slumping of the blanket; and undercutting and raveling of the riprap by scour at the end of the blanket. Therefore, in design, consideration must be given to selection of an adequate size stone, use of an adequately graded riprap or provision of a filter blanket, and proper treatment of the end of the riprap blanket.

5–3 **SELECTION OF STONE SIZE**. There are curves available for the selection of stone size required for protection as a function of the Froude number. (See Figures 4-21 through 4-23). Two curves are given, one to be used for riprap subject to direct attack or adjacent to hydraulic structures such as side inlets, confluences, and energy dissipators, where turbulence levels are high, and the other for riprap on the banks of a straight channel where flows are relatively quiet and parallel to the banks. With the depth of flow and average velocity in the channel known, the Froude number can be computed and a stone size determined from the appropriate curve. Curves for determining the riprap size required to prevent scour downstream from culvert outlets with scour holes of various depths are also available. The thickness of the riprap blanket should be equal to the longest dimension of the maximum size stone or 1.5 times the stone diameter (50 percent size), whichever is greater. When the use of very large rock is desirable but impractical, substitution of a grouted reach of smaller rock in areas of high velocities or turbulence maybe appropriate. Grouted riprap should be followed by an ungrouted reach.

5-4 **RIPRAP GRADATION**. A well-graded mixture of stone sizes is preferred to a relatively uniform size of riprap. In certain locations the available material may dictate the gradation of riprap to be used. In such cases the gradation should resemble as closely as possible the recommended mixture. Consideration should be given to increasing the thickness of the riprap blanket when locality dictates the use of gradations with larger percents of small stone than recommended. If the gradation of the available riprap is such that movement of the natural material through the riprap blanket would be likely, a filter blanket of sand, crushed, rock, gravel, or synthetic cloth must be placed under the riprap. The usual blanket thickness is 6 in., but greater thickness is sometimes necessary.

5-5 **RIPRAP DESIGN**. An ideal riprap design would provide a gradual reduction in riprap size until the downstream end of the blanket blends with the natural bed material. This is seldom justified. However, unless this is done, turbulence caused by the riprap is likely to develop a scour hole at the end of the riprap blanket. It is suggested that the thickness of the riprap blanket be doubled at the downstream end to protect against undercutting and unraveling. An alternative is to provide a constant-thickness rubble blanket of suitable length dipping below the natural streambed to the estimated depth of bottom scour.

# CHAPTER 6

# DESIGN OF SUBSURFACE DRAINAGE SYSTEMS

## 6-1 INTRODUCTION

6-1.1 **Purpose.** This chapter provides guidance for the design and construction of subsurface drainage facilities for airfields, roads, streets, parking lots and other paved areas.

6-1.2 **Scope.** The criteria within this chapter applies to paved areas such as airfields, roads, streets and parking lots having a relatively impervious surface such as asphalt concrete or Portland cement concrete. The criteria is limited to situations where the surface water can be drained by gravity flow and is mainly concerned with elimination of water which enters the pavement through the surface.

6-1.3 **Definitions.** This chapter uses a number of terms that have unique usage within the chapter or which may not be in common usage. The definitions of these terms are described below.

6-1.3.1 **Apparent Opening Size (AOS).** A measure of the opening size of a geotextile. AOS is the sieve number corresponding to the sieve size at which 95 percent of the single-size glass beads pass the geotextile ($O_{95}$) when tested in accordance with ASTM D 4751, Determining Apparent Opening Size (AOS) of a Geotextile.

6-1.3.2 **Coefficient of permeability (k).** A measure of the rate at which water passes through a unit area of material in a given amount of time under a unit hydraulic gradient.

6-1.3.3 **Choke Stone.** A small size stone used to stabilize the surface of an OGM. For a choke stone to be effective, the ratio of $d_{15}$ of the coarse aggregate to the $d_{15}$ of the choke stone must be less than 5, and the ratio of the $d_{50}$ of the coarse aggregate to $d_{50}$ of the choke stone must be greater than 2.

6-1.3.4 **Drainage Layer.** A layer in the pavement structure that is specifically designed to allow rapid horizontal drainage of water from the pavement structure. The layer is also considered to be a structural component of the pavement and may serve as part of the base or subbase.

6-1.3.5 **Effective Porosity.** The effective porosity is defined as the ratio of the volume of voids that will drain under the influence of gravity to the total volume of a unit of aggregate. The difference between the porosity and the effective porosity is the amount of water that will be held by the aggregate. For materials such as the RDM and OGM, the water held by the aggregate will be small; thus, the difference between the porosity and effective porosity will be small (less than 10 percent). The effective porosity may be estimated by computing the porosity from the unit dry weight of the aggregate and the specific gravity of the solids which then should be reduced by 5 percent to allow for water retention on the aggregate.

6-1.3.6 **Geocomposite Edge Drain.** A manufactured product using geotextiles, geogrids, geonets, and/or geomembranes in laminated or composite form, which can be used as an edge drain in place of trench-pipe construction.

6-1.3.7 **Geotextile.** A permeable textile used in geotechnical projects. For this manual geotextile will refer to a nonwoven needle punch fabric that meets the requirements of the apparent opening size (AOS), grab strength and puncture strength specified for the particular application.

6-1.3.8 **Open Graded Material (OGM).** A granular material having a very high permeability (greater than 1,500 m/day (5,000 ft/day)) which may be used for a drainage layer. Such a material will normally require stabilization for construction stability or for structural strength to serve as a base in a flexible pavement.

6-1.3.9 **Pavement Structure**. Pavement structure is the combination of subbase, base, and surface layers constructed on a subgrade.

6-1.3.10 **Permeable Base**. An open-graded granular material with most of the fines removed (e.g., less than 10 percent passing the No. 8 sieve) to provide high permeability (1,000 ft/day or more) for use in a drainage layer.

6-1.3.11 **Porosity**. The amount of voids in a material, expressed as the ratio of the volume of voids to the total volume.

6-1.3.12 **Rapid Draining Material (RDM).** A granular material having a sufficiently high permeability (300 to 1,500 m/day (1,000 to 5,000 ft/day)) to serve as a drainage layer and also having the stability to support construction equipment and the structural strength to serve as a base and/or a subbase.

6-1.3.13 **Separation Layer.** A layer provided directly beneath the drainage layer to prevent fines from infiltration or pumping into the drainage layer and to provide a working platform for construction and compaction of the drainage layer.

6-1.3.14 **Stabilization.** Stabilization refers to either mechanically or chemically stabilizing the drainage layer to increase the stability and strength to withstand construction traffic and/or design traffic. Mechanical stabilization is accomplished by the use of a choke stone and compaction. Chemical stabilization is accomplished by the use of either portland cement or asphalt.

6-1.3.15 **Subsurface Drainage**. Collection and removal of water from a pavement surface or subgrade. Subsurface drainage systems are categorized into two functional categories: one for draining surface infiltration water, and the other for controlling groundwater.

6-1.4 **Bibliography.** In recent years subsurface drainage has received increasing attention, particularly in the area of highway design. A number of studies have been conducted by State Highway Agencies and by the Federal Highway Administration that have resulted in a large number of publications on the subject of subsurface drainage. Appendix B contains a list of publications which contain information pertaining to the design of subsurface drainage for pavements.

6-1.5 **Effects of Subsurface Water.** Water has a detrimental effect on pavement performance, primarily by either weakening subsurface materials or erosion of material by free water movement. For flexible pavements the weakening of the base, subbase or subgrade when saturated with water is one of the main causes of pavement failures. In rigid pavement free water, trapped between the rigid concrete surface and an impermeable layer directly beneath the concrete, moves due to pressure caused by loadings. This movement of water (referred to as pumping) erodes the subsurface material creating voids under the concrete surface. In frost areas subsurface water will contribute to frost damage by heaving during freezing and loss of subgrade support during thawing. Poor subsurface drainage can also contribute to secondary damage such as 'D' cracking or swelling of subsurface materials. Water is contained above an impervious stratum and hence the infiltration water is prevented from reaching a groundwater table at a lower elevation. The upper body of water is called perched groundwater and its free surface is called a perched water table.

6-1.6 **Sources of Water**

6-1.6.1 **General.** The two sources of water to be considered are from infiltration and subterranean water. Infiltration is the most important source of water and is the source of most concern in this document. Subterranean water is important in frost areas and areas of very high water table or areas of artesian water. In many areas perched water may develop under pavements due to a reduced rate of evaporation of the water from the surface. In frost areas free water collects under the surface by freeze/thaw action.

6-1.6.2 **Infiltration.** Infiltration is surface water which enters the pavement from the surface through cracks or joints in the pavement, through the joint between the pavement and shoulder, through pores in the pavement, by movement from ditches and surface channels near the pavement, and through shoulders and adjacent areas. Since surface infiltration is the principal source of water, it is the source needing greatest control measures. Groundwater tables rise and fall depending upon the relation between infiltration, absorption, evaporation and groundwater flow. Seasonal fluctuations are normal because of differences in the amount of precipitation and maybe relatively large in some localities. Prolonged drought or wet periods will cause large fluctuations in the groundwater level.

6-1.6.3 **Subterranean water.** Subterranean water can be a source of water from a high water table, capillary forces, artesian pressure, and freeze-thaw action. This source of water is particularly important in areas of frost action when large volumes of water can be drawn into the pavement structure during the formation of ice lenses. For large paved areas the evaporation from the surface is greatly reduced which causes saturation of the subgrade by capillary forces. Also, if impervious layers exist beneath the pavement, perched water can be present or develop from water entering the pavement through infiltration. This perched water then becomes a subterranean source of water.

6-1.6.4 **Classification of subdrainage facilities.** Subdrainage facilities can be categorized into two functional categories, one to control infiltration, and one to control groundwater. An infiltration control system is designed to intercept and remove water that enters the pavement from precipitation or surface flow. An important function of this system is to keep water from being trapped between impermeable layers. A groundwater control system is designed to reduce water movement into subgrades and pavement sections by controlling the flow of groundwater or by lowering the water table. Often, subdrainage is required to perform both functions, and the two subdrainage functions can be combined into a single subdrainage system. Figures 6-1 and 6-2 illustrate examples of infiltration and groundwater control systems.

**Figure 6-1. Collector Drain Used to Remove Infiltration Water**

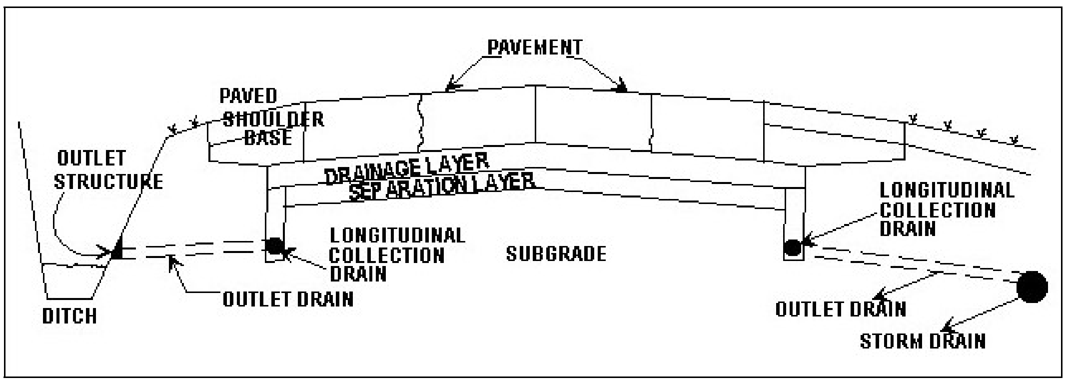
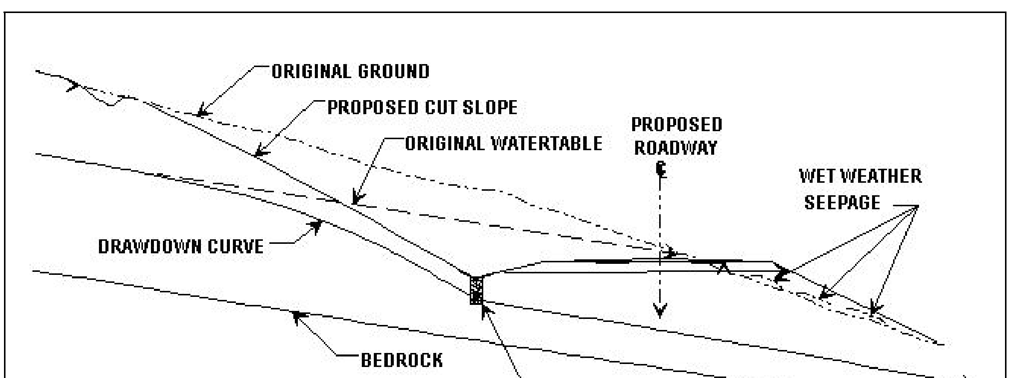



**Figure 6-2. Collector Drain to Intercept Seepage and Lower the Ground-Water Table**



6-1.7 **Subsurface Drainage Requirements.** The determination of the subsurface soil properties and water condition is a prerequisite for the satisfactory design of a subsurface drainage system. Field explorations and borings made in connection with the project design should include the following investigations pertinent to subsurface drainage. A topographic map of the proposed area and the surrounding vicinity should be prepared indicating all streams, ditches, wells, and natural reservoirs. The analysis of aerial photographs of the areas selected for construction may furnish valuable information on general soil and groundwater conditions. An aerial photograph presents a graphic record of the extent, boundaries, and surface features of soil patterns occurring at the surface of the ground. The presence of vegetation, the slopes of a valley, the colorless monotony of sand plains, the farming patterns, the drainage pattern, gullies, eroded lands, and evidences of the works of man are revealed in detail by aerial photographs. The use of aerial photographs may supplement both the detail and knowledge gained in topographic survey and ground explorations. The sampling and exploratory work can be made more rapid and effective after analysis of aerial photographs has developed the general soil features. The location and depth of permanent and perched groundwater tables may be sufficiently shallow to influence the design. The season of the year and rainfall cycle will measurably affect the depth to the water table. In many locations, information may be obtained from residents of the surrounding areas regarding the behavior of wells and springs and other evidences of subsurface water. The soil properties investigated for other purposes in connection with the design will supply information that can be used for the design of the drainage system. It may be necessary to supplement these explorations at locations of subsurface drainage structures and in areas where soil information is incomplete for design of the drainage system.

6-1.8 **Laboratory Tests.** The design of subsurface drainage structures requires knowledge of the following soil properties of the principal soils encountered: strength, compressibility, swell and dispersion characteristics, the in situ and compacted unit dry weights, the coefficient of permeability, the in situ water content, specific gravity, grain-size distribution, and the effective void ratio. These soil properties may be satisfactorily determined by experienced soil technicians through laboratory tests. The final selected soil properties for design purposes may be expressed as a range, one extreme representing a maximum value and the other a minimum value. The true value should be between these two extremes, but it may approach or equal one or the other, depending upon the variation within a soil stratum.

6-1.9 **Drainage of Water from Soil.** The quantity of water removed by a drain will vary depending on the type of soil and location of the drain with respect to the groundwater table. All of the water contained in a given specimen cannot be removed by gravity flow since water retained as thin films adhering to the soil particles and held in the voids by capillarity will not drain. Consequently, to determine the volume of water that can be removed from a soil in a given time, the effective porosity as well as the permeability must be known. The effective porosity is defined as the ratio of the volume of the voids that can be drained under gravity flow to the total volume of soil. Limited effective porosity test data for well-graded base course materials, such as bank-run sands and gravels, indicate a value for effective porosity of not more than 0.15. Uniformly graded soils such as medium coarse sands, may have an effective porosity of not more than 0.25.

## 6-2 PRINCIPLES OF PAVEMENT DRAINAGE

6-2.1 **Flow of Water Through Soils.** The flow of water through soils is expressed by Darcy's empirical law which states that the velocity of flow (v) is directly proportional to the hydraulic gradient (i). This law can be expressed as:

$$v = ki_1 \qquad \text{(eq. 6-1)}$$

where k is the coefficient of proportionality known as the coefficient of permeability. Equation 6-1 can be expanded to obtain the rate of flow through an area of soil (A). The equation for the rate of flow (Q) is:

$$Q = kiA_2 \qquad \text{(eq. 6-2)}$$

According to Darcy's law, the velocity of flow and the quantity of discharge through a porous media are directly proportional to the hydraulic gradient. For this condition to be true, flow must be laminar or nonturbulent. Investigations have indicated that Darcy's law is valid for a wide range of soils and hydraulic gradients. However, in developing criteria for subsurface drainage, liberal margins have been applied to allow for turbulent flow. The criteria and uncertainty depend heavily on the permeability of the soils involved in the pavement structure. It is therefore useful to examine the influence of various factors on the permeability of soils. In examining permeability of soils in regard to pavement drainage, the materials of most concern are base and subbase aggregate and aggregate used as drainage layers.

### 6-2.2 Factors Affecting Permeability

6-2.2.1 **Coefficient of permeability.** The value of permeability depends primarily on the characteristics of the permeable materials, but it is also a function of the properties of the fluid. An equation (after Taylor) demonstrating the influence of the soil and pore fluid properties on permeability was developed based on flow through porous media similar to flow through a bundle of capillary tubes. This equation is as follows:

$$k = D_s^2 \frac{\gamma \; e^3}{\mu \, (1-e)} C \qquad \text{(eq. 6-3)}$$

where

k = the coefficient of permeability
$D_s$ = some effective particle diameter
( = unit weight of pore fluid
: = viscosity of pore fluid

e = void ratio
C = shape factor

6-2.2.2 **Effect of pore fluid and temperature.** In the design of subsurface drainage systems for pavements, the primary pore fluid of concern is water. Therefore, when permeability is mentioned in this chapter, water is assumed to be the pore fluid. Equation 6-3 indicates that the permeability is directly proportional to the unit weight of water and inversely proportional to the viscosity. The unit weight of water is essentially constant, but the viscosity of water will vary with temperature. Over the widest range in temperatures ordinarily encountered in seepage problems, viscosity varies about 100 percent. Although this variation seems large, it can be insignificant when considered in the context of the variations which can occur with changes in material properties.

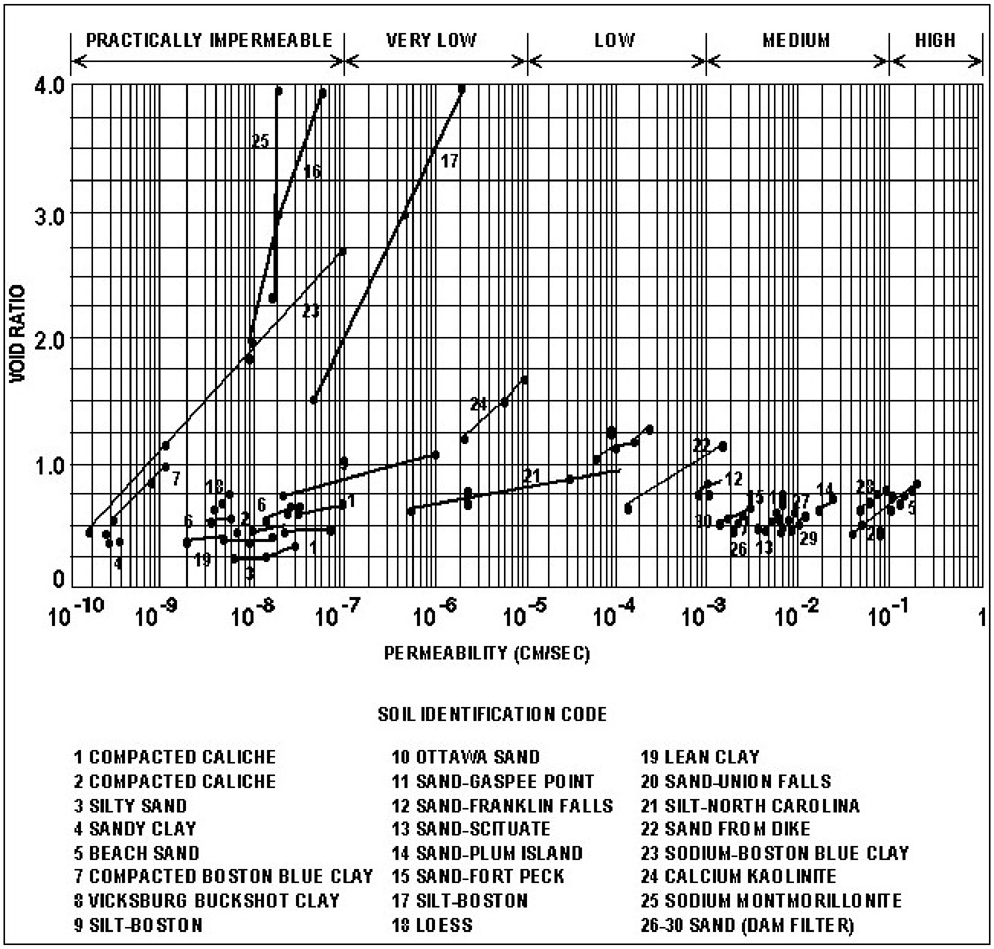
6-2.2.3 **Effect of grain size.** Equation 6-3 suggests that permeability varies with the square of the particle diameter. It is logical that the smaller the grain size the smaller the voids that constitute the flow channels, and hence the lower the permeability. Also, the shape of the void spaces has a marked influence on the permeability. As a consequence, the relationships between grain size and permeability are complex. Intuition and experimental test data suggest that the finer particles in a soil have the most influence on permeability. The coefficient of permeability of sand and gravel materials, graded between limits usually specified for pavement bases and subbases, depends principally upon the percentage by weight of particles passing the 0.075 mm (No. 200) sieve. Table 6-1 provides estimates of the permeability for these materials for various amounts of material finer than the 0.075 mm (No. 200) sieve.

**Table 6-1. Coefficient of Permeability for Sand and Gravel Materials.**
**Coefficient of 55**

| Percent by Weight Passing 0.075 mm (No. 200) Sieve | Permeability for Remolded Samples | |
|---|---|---|
| | mm/sec | ft/min |
| 3 | 5 H $10^{-1}$ | $10^{-1}$ |
| 5 | 5 H $10^{-2}$ | $10^{-2}$ |
| 10 | 5 H $10^{-3}$ | $10^{-3}$ |
| 15 | 5 H $10^{-4}$ | $10^{-4}$ |
| 20 | 5 H $10^{-5}$ | $10^{-5}$ |

6-2.2.4 **Effect of void ratio.** The void ratio or porosity of soils, though less important than grain size and soil structure, often has a substantial influence on permeability. The void ratio of a soil will also dictate the amount of fluid that can be held within the soil. The more dense a soil, the lower the soil permeability and the lesser the amount of water that can be retained in the soil. Figure 6-3 presents the permeability for different soils as a function of the void ratio. The amount of water that can be

**Figure 6-3. Permeability Test Data (from Lambe and Whitman, with permission)**




contained in a soil will directly relate to the void ratio. Not all water contained in a soil can be drained by gravity flow since water retained as thin films adhering to the soil particles and held by capillarity will not drain. Consequently, to determine the volume of water that can be removed from a soil the effective porosity ($n_e$) must be known. The effective porosity is defined as the ratio of the volume of the voids that can be drained under gravity flow to the total volume of soil, and can be expressed mathematically as

$$n_e = 1 - \frac{\gamma_d}{G_s \gamma_w}(1 + G_s W_e) \qquad \text{(eq. 6-4)}$$

where

$\gamma_d$ = dry density of the soil
$G_s$ = specific gravity of solids
$\gamma_w$ = unit weight of water
$W_e$ = effective water content (after the soil has drained) expressed as a decimal fraction relative to dry weight

Limited effective porosity test data for well-graded base-course materials, such as bank-run sands and gravels, indicate a value for effective porosity of not more than 0.15. Uniformly graded medium or coarse sands, may have an effective porosity of not more than 0.25 while for a uniformly graded aggregate, such as would be used in a drainage layer, the effective porosity may be above 0.30.

6-2.2.5 **Effect of structure and stratification.** Generally, in situ soils show a certain amount of stratification or a heterogeneous structure. Water deposited soils usually exhibit a series of horizontal layers that vary in grain-size distribution and permeability, and generally these deposits are more permeable in the horizontal than in the vertical direction. In pavement construction the subgrade, subbase, and base materials are placed and compacted in horizontal layers which result in having a different permeability in the vertical direction than in the horizontal direction. The vertical drainage of water from a pavement can be disrupted by a single relatively impermeable layer. For most pavements the subgrades have a very low permeability compared to the base and subbase materials. Therefore, water in the pavement structure can best be removed by horizontal flow. For a layered pavement system the effective horizontal permeability is obtained from a weighted average of the layer permeability by the formula

$$k=(k_1 d_1+k_2 d_2+k_3 d_3+\ldots)/(d_1+d_2+d_3+\ldots) \qquad \text{(eq. 6-5)}$$

where

$k$ = the effective horizontal permeability
$k_1,k_2,k_3,\ldots$ = the coefficients of horizontal permeability of individual layers
$d_1,d_2,d_3,\ldots$ = thicknesses of the individual layers

When a drainage layer is employed in the pavement section, the permeability of the drainage material will likely be several orders of magnitude greater than the other materials in the section. Since water flow is proportional to permeability, the flow of water from the pavement section can be computed based only on the characteristics of the drainage layer.

## 6-2.3 Quantity and Rate of Subsurface Flow

6-2.3.1 **General.** Water flowing from the pavement section may come from infiltration through the pavement surface and groundwater. Normally groundwater flows into collector drains from the subgrade and will be an insufficient flow compared to the flow coming from infiltration. The computation of the groundwater flow is beyond the scope of this manual and should it be necessary to compute the groundwater flow, a textbook on groundwater flow should be consulted. The volume of infiltration water flow from the pavement will depend on factors such as type and condition of surface, length and intensity of rainfall, properties of the drainage layer, hydraulic gradient, time allowed for drainage and the drained area. In the design of the subsurface drainage system all of these factors must be considered.

6-2.3.2 **Effects of pavement surface.** The type and condition of the pavement surface will have considerable influence on the volume of water entering the pavement structure. In the design of surface drainage facilities all rain falling on paved surfaces is assumed to be runoff. For new well designed and constructed pavements, the assumption of 100 percent runoff is probably a good conservative assumption for the design of surface drainage facilities. For design of the subsurface drainage facilities, the design should be based on the infiltration rate for a deteriorated pavement. Studies have shown that for badly deteriorated pavements well over 50 percent of the rainfall can flow through the pavement surface.

6-2.3.3 **Effects of rainfall.** It is only logical that the volume of water entering the pavement will be directly proportional to the intensity and length of the rainfall. Relatively low intensity rainfalls can be used for designing the subsurface drainage facilities because high intensity rainfalls do not greatly increase the adverse effect of water on pavement performance. The excess rainfall would, once the base and subbase are saturated, run off as surface drainage. For this reason a seemingly unconservative design rainfall can be selected.

6-2.3.4 **Capacity of drainage layers.** If water enters the pavement structure at a greater rate than the discharge rate, the pavement structure becomes saturated. The design of horizontal drainage layers for the pavement structure is based, in part, on the drainage layer serving as a reservoir for the excess water entering the pavement. The capacity of the drainage layer as a reservoir is a function of the storage capacity of the drainage layer plus the amount of water which drains from the layer during a rain event. The storage capacity of the drainage layer will be a function of the effective porosity of the drainage material and the thickness of the drainage layer. The storage capacity of the drainage layer ($q_s$) in terms of depth of water per unit area is computed by:

$$q_S = (n_e)(h) \qquad \text{(eq. 6-6)}$$

where

$n_e$ = the effective porosity
$h$ = the thickness of the drainage layer

In the equation the dimensions of the $q_s$ will be the same as the dimensions of the h. If it is considered that not all the water will be drained from the drainage layer, then the storage capacity will be reduced by the amount of water in the layer at the start of the rain event. The criterion for design of the drainage layer calls for 85 percent of the water to be drained from the drainage layer within 24 hr; therefore it is conservatively assumed that only 85 percent of the storage volume will be available at the beginning of a rain event. To account for the possibility of water in the layer at the beginning of a rain event, equation 6-6 is modified to be:

$$q_S = 0.85(n_e)(h) \qquad \text{(eq. 6-7)}$$

The amount of water ($q_d$) which will drain from the drainage layer during the rain event may be estimated using the equation

$$q_d = \frac{(t)(k)(i)(h)}{2} \qquad \text{(eq. 6-8)}$$

where

t = duration of the rain event
k = permeability of the drainage layer
i = slope of the drainage layer
h = thickness of the drainage layer

In these equations the dimensions of $q_s$, $q_d$, t, k, and h should be consistent. The total capacity (q) of the drainage layer will be the sum of $q_s$ and $q_d$ resulting in the following equation for the capacity

$$q = 0.85(n_e)(h) + \frac{(t)(k)(i)(h)}{2} \qquad \text{(eq. 6-9)}$$

Knowing the water entering the pavement, equation 6-9 can be used to estimate the thickness of the drainage layer such that the drainage layer will have the capacity for a given design rain event. For most situations the amount of water draining from the drainage layer will be small compared to the storage capacity. Therefore, in most cases, equation 6-7 can be used in estimating the thickness required for the drainage layer.

6-2.3.5 **Time for drainage.** It is desirable that the water be drained from the base and subbase layers as rapidly as possible. The time for drainage of these layers is a function of the effective porosity, length of the drainage path, thickness of the layers, slope of the drainage path, and permeability of the layers. Past criterion has specified that the base and subbase obtain a degree of 50 percent drainage within 10 days. The equation for computing time for 50 percent drainage is:

$$T_{50} = \frac{(n_e D)}{2kH_o} \qquad \text{(eq. 6-10)}$$

where

$T_{50}$ = time for 50 percent drainage
$n_e$ = effective porosity of the soil
k = coefficient of permeability
D and H = base- and subbase geometry dimensions (illustrated in Figure 6-4)

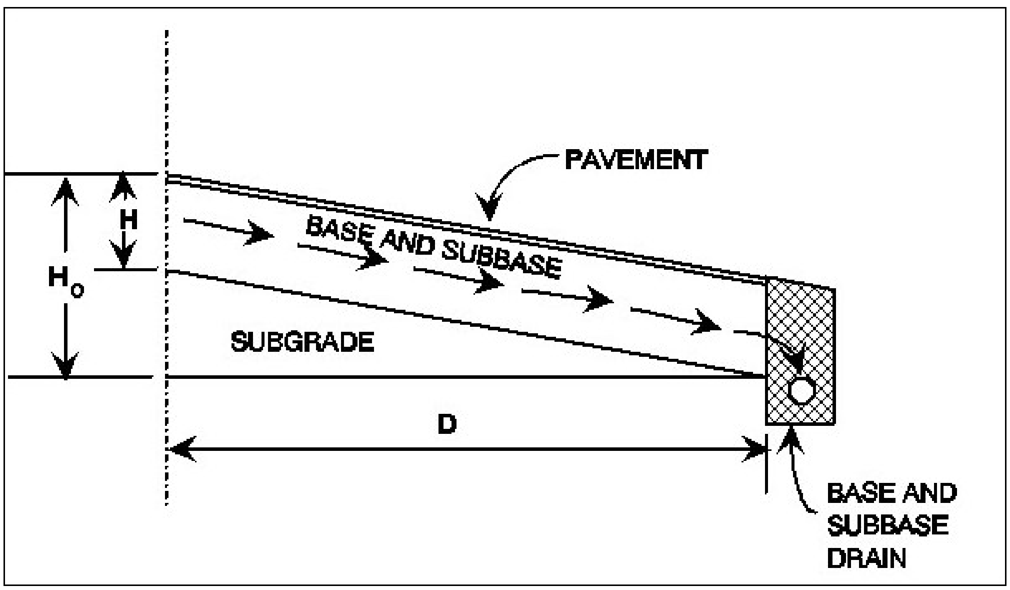
The dimensions of time, k, $H_o$, and D must be consistent. In Figure 6-4 the slope (i) of the drainage path is $D/H_o$; therefore equation 6-10 can be written

$$T_{50} = (n_e)(D)/2ik \quad \text{(eq. 6-11)}$$

Experience has shown that base and subbase materials, when compacted to densities required in pavement construction, seldom have sufficient permeability to meet the 10 day drainage criterion. In such pavements the base and subbase materials become saturated causing a reduced pavement life. When a drainage layer is incorporated into the pavement structure to improve pavement drainage, the criterion for design of the drainage layer shall be that the drainage layer shall reach a degree of drainage of 85 percent within 24 hr. The time for 85 percent drainage is approximately twice the time for 50 percent drainage. The time for 85 percent drainage ($T_{85}$) is computed by

$$T_{85} = (n_e)(D)/ik \quad \text{(eq. 6-12)}$$

**Figure 6-4. Pavement Geometry for Computation of Time for Drainage**




6-2.3.6 **Length and slope of the drainage path.** As can be seen in equation 6-10, the time for drainage is a function of the square of the length of drainage path. For this reason and the fact that for most pavement designs the length of the drainage path can be controlled, the drainage path length is an important parameter in the design of the drainage system. The length of the drainage path (L) may be computed from the following equation

$$L = \frac{L_t \sqrt{i_t^2 + i_e^2}}{i_t} \quad \text{(eq. 6-13)}$$

where

$L_t$ = the length of the transverse slope of the drainage layer

$i_t$ = the transverse slope of the drainage layer
$i_e$ = the longitudinal slope of the drainage layer

The slope of the drainage path (i) is a function of the transverse slope and longitudinal slope of the drainage layer and is computed by the equation

$$i = \sqrt{i_t^2 + i_e^2} \qquad \text{(eq. 6-14)}$$

6-2.3.7 **Rate of flow.** The edge drains for pavements having drainage layers shall be designed to handle the maximum rate of flow from the drainage layer. This maximum rate of flow will be obtained when the drainage layer is flowing full and may be estimated using equation 6-2.

6-2.4 **Use of Drainage Layers**

6-2.4.1 **Purpose of drainage layers.** Special drainage layers may be used to promote horizontal drainage of water from pavements, prevent the buildup of hydrostatic water pressure, and facilitate the drainage of water generated by cycles of freeze-thaw.

6-2.4.2 **Placement of drainage layers.** In rigid pavements the drainage layer will generally be placed directly beneath the concrete slab. In this location the drainage layer will intercept water entering through cracks and joints, and permit rapid drainage of the water away from the bottom of the concrete slab. In flexible pavements the drainage layer will normally be placed beneath the base. In placing the drainage layer beneath the base the stresses on the drainage layer will be reduced to an acceptable level and drainage will be provided for the base course.

6-2.4.3 **Permeability requirements for the drainage layer.** The material for drainage layers in pavements must be of sufficient permeability to provide rapid drainage and rapidly dissipate water pressure and yet provide sufficient strength and stability to withstand load induced stresses. There is a trade off between strength or stability and permeability; therefore the material for the drainage layers should have the minimum permeability for the required drainage application. For most applications a material with a permeability of 300 m/day (1,000 ft/day) will provide sufficient drainage.

6-2.5 **Use of Filters**

6-2.5.1 **Purpose of filters in pavement structures.** The purpose of filters in pavement structures is to prevent the movement of soil (piping) yet allow the flow of water from one material to another. The need for a filter is dictated by the existence of water flow from a fine grain material to a coarse gain material generating a potential for piping of the fine grain material. The principal location in the pavement structure where a flow from a fine grain material into a coarse grain material is water flowing from the base, subbase, or subgrade into the coarse aggregate surrounding the drain pipe. Thus, the principal use of a filter in a pavement system will be in preventing piping into the drain pipe. Although rare, the possibility exists for hydrostatic head forcing a flow of water upward from the subbase or subgrade into the pavement drainage layer. For such a condition it would be necessary to design a filter to separate the drainage layer from the finer material.

6-2.5.2 **Piping criteria.** The criteria for preventing movement of particles from the soil or granular material to be drained into the drainage material are:

$$\frac{\text{15 percent size of drainage or filter material}}{\text{85 percent size of material to be drained}} \leq 5$$

and

$$\frac{\text{50 percent size of drainage or filter material}}{\text{50 percent size of material to be drained}} \leq 25$$

The criteria given above will be used when protecting all soils except clays without sand or silt particles. For these soils, the 15 percent size of drainage or filter material may be as great as 0.4 mm and the $d_{50}$ criteria will be disregarded.

6-2.5.3 **Permeability requirements.** To assure that the filter material is sufficiently permeable to permit passage of water without hydrostatic pressure buildup, the following requirement should be met:

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of material to be drained}} \geq 5$$

### 6-2.6 Use of Separation Layers

6-2.6.1 **Purpose of separation layers.** When drainage layers are used in pavement systems, the drainage layers must be separated from fine grain subgrade materials to prevent penetration of the drainage material into the subgrade or pumping of fines from the subgrade into the drainage layer. The separation layer is different from a filter in that there is no requirement, except during frost thaw, to protect against water flow through the layer.

6-2.6.2 **Requirements for separation layers.** The main requirements of the separation layer are that the material for the separation layer have sufficient strength to prevent the coarse aggregate of the drainage layer from being pushed into the fine material of the subgrade and that the material have sufficient permeability to prevent buildup of hydrostatic pressure in the subgrade. To satisfy the strength requirements the material of the separation layer should have a minimum CBR of 50. To allow for release of hydrostatic pressure in the subgrade, the permeability of the separation layer should have a permeability greater than that of the subgrade. This would not normally be a problem because the permeability of subgrades are orders of magnitude less than the permeability of a 50 CBR material but to ensure sufficient permeability the permeability requirements of a filter would apply.

### 6-2.7 Use of Geotextiles

6-2.7.1 **Purpose of geotextiles.** Geotextiles (engineering fabrics) may be used to replace either the filter or the separation layer. The principal use of geotextiles is the filter around the pipe for the edge drain. Although geotextiles can be used as a replacement for the separation layer, geotextile adds no structure strength to the pavement; therefore this practice is not recommended.

6-2.7.2 **Requirements of the geotextiles for filters.** When geotextiles are to serve as a filter lining the edge drain trench, the most important function of the filter is to keep fines from entering the edge drain system. For pavement systems having drainage layers there is little requirement for water flow through the fabric; therefore for most applications, it is better to have a heavier fabric than would normally be used as a filter. Since drainage layers have a very high permeability, geotextile fabric should never be placed between the drainage layer and the edge drain. The permeability of geotextiles is governed by the size of the openings in the fabric which is specified in terms of the apparent opening size (AOS) in millimeters. For use as a filter for the trench of the edge drain the AOS of the geotextile should always be equal to or less than 0.212 mm. For geotextiles used as filters with drains installed to intercept groundwater flow in subsurface aquifers the geotextile should be selected based on criteria similar to the criteria used to design a granular filter.

6-2.7.3 **Requirements for geotextiles used for separation.** Geotextiles used as separation layers beneath drainage layers should be selected based primarily on survivability of the geotextiles with somewhat less emphasis placed on the AOS. When used as a separation layer the geotextile survivability should be rated very high by the rating scheme given by AASHTO M 28890 "Standard Specification for Geotextiles, Asphalt Retention, and Area Change of Paving Engineering Fabrics." This would ensure survival of the geotextiles under the stress of traffic during the life of the pavement. To ensure that fines will not pump into the drainage layer yet allow water flow to prevent hydrostatic pressure the AOS of the geotextile must be equal to or less than 0.212 mm and also equal to or greater than 0.125 mm.

## 6-3 DESIGN OF THE PAVEMENT SUBSURFACE DRAINAGE SYSTEM

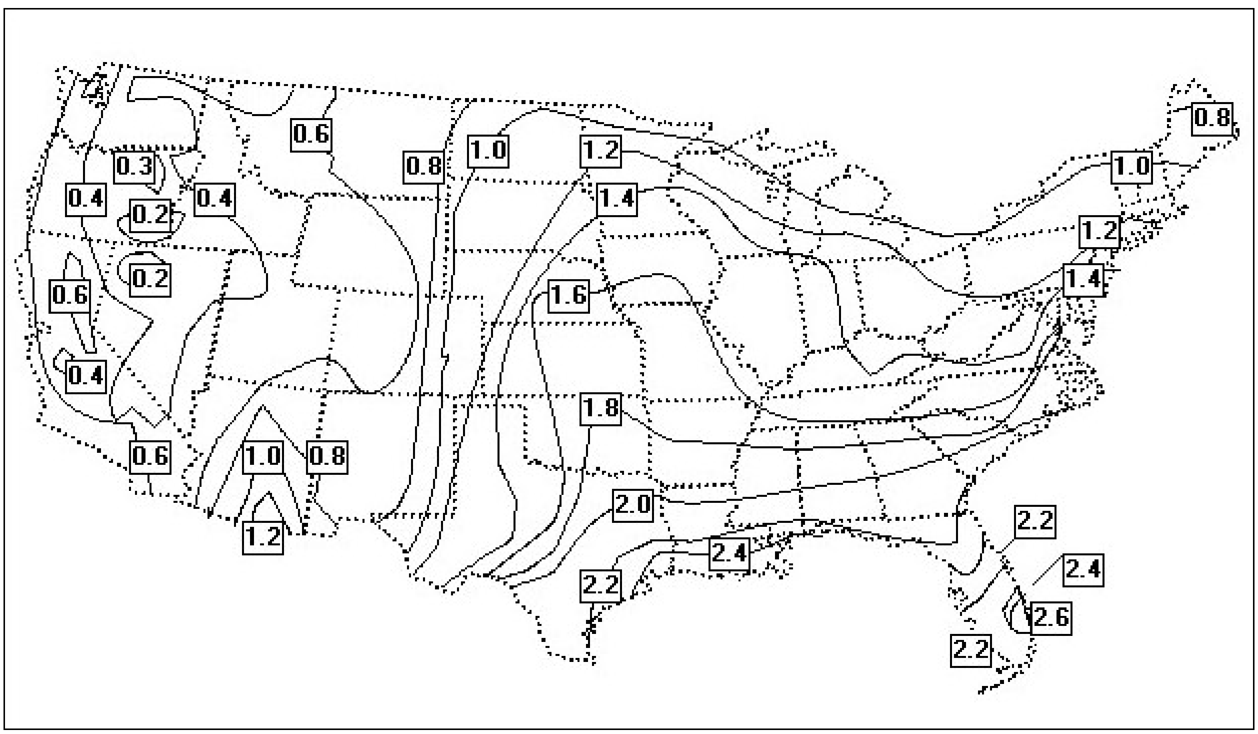
6-3.1 **General.** The design methodology contained herein is for the design of a pavement subsurface drainage system for the rapid removal of surface infiltration water and water generated by freeze-thaw action. Although the primary emphasis will be on removing water from under the pavement, there may be occasions when the system will also serve as interceptor drain for groundwater.

6-3.2 **Methods.** For most pavement structures water is to be removed by the use of a special drainage layer which allows the rapid horizontal drainage of water. The drainage layer must be designed to handle surface infiltration from a design storm and withstand the stress of traffic. A separation layer must be provided to prevent intrusion of fines from the subgrade or subbase into the drainage layer and facilitate construction of the drainage layer. The drainage layers should feed into a collection system consisting of trenches with a drain pipe, backfill, and filter. The collection system must be designed to maintain progressively greater outflow capabilities in the direction of flow. The outlet for the subsurface drains should be properly located or protected to prevent backflow from the surface drainage system. Some pavements may not require a drainage system in that the subgrade may have sufficient permeability for the water to drain vertically into the subgrade. In addition, some pavements designed for very light traffic may not justify the expense of a subsurface drain system.

6-3.3 **Design Prerequisites.** For the satisfactory design of a subsurface drainage system, the designer must have an understanding of environmental conditions, subsurface soil properties and groundwater conditions.

6-3.3.1 **Environmental conditions.** Temperature and rainfall data applicable to the local area should be obtained and studied. The depth of frost penetration is an important factor in the design of a subsurface drainage. For most areas the approximate depth of frost penetration can be determined by referring to TM 5-825-2/ AFMAN 32-8008, Vol. 2 or by using the computer program for frost analysis. Rainfall data are used to determine the volume of water to be handled by the subsurface drainage system. The data can be obtained from local weather stations or by the use of Figure 6-5.

**Figure 6-5. Design Storm Index, 1-hr Rainfall Intensity-Frequency Data for Continental United States Excluding Alaska**



6-3.3.2 **Subsurface soil properties.** In most cases the soil properties investigated for other purposes in connection with the pavement design will supply information that can be used for the design of the subsurface drainage system. The two properties of most interest are the coefficient of permeability and the frost susceptibility of the pavement materials.

6-3.3.3 **Coefficient of permeability.** The coefficient of permeability of the existing subsurface soils is needed to determine the need of special horizontal drainage layers in the pavement. For pavements having subgrades with a high coefficient of permeability the water entering the pavement will drain vertically and therefore horizontal drainage layers will not be required. For pavements having subgrades with a low coefficient of permeability the water entering the pavement must be drained horizontally to the collector system or to edge drains.

6-3.3.4 **Frost susceptible soils.** Soils susceptible to frost action are those that have the potential of ice formation occurring when that soil is subjected to freezing conditions with water available. Ice formation takes place at successive levels as freezing temperatures penetrate into the ground. Soils possessing a high capillary rate and low cohesive nature act as a wick in feeding water to ice lenses. Soils are placed into groups according to the degree of frost susceptibility as shown in Table 6-2. Because a large volume of free water is generated during thaw of ice lenses, horizontal drainage layers are required to permit the escape of the water from the pavement structure and thus facilitate the restoration of the pavement strength.

**Table 6-2. Frost Susceptible Soils**

| Typical Soil | | | |
|---|---|---|---|
| **Frost Group** | **Type of Soil** | **Percent Finer than 0.02 mm by Weight** | **Types Under Unified Soil Classification System** |
| F1 | Gravely Soils | 6-10 | GW-GM, GP-GM, GW-GC, GP-GC |
| F2 | (a) Gravely Soils<br>(b) Sands | 10-20<br>6-15 | GM, GC, GM-GC<br>SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC |
| F3 | (a) Gravely Soils<br>(b) Sands, except very fine silty sands<br>(c) Clays (PI > 12) | > 20<br>> 15<br>-- | GM, GC, GM-GC<br>SM, SC, SM-SC<br>CL, CH, ML-CL |
| F4 | (a) Silts<br>(b) Very fine sands<br>(c) Clays (PI < 12)<br>(d) Varved clays and other fine grained, with banded sediments | --<br>> 15<br>--<br>-- | ML, MH, ML-CL<br>SM, SC, SM-SC<br>CL, ML-CL<br>CL or CH layered ML, MH, SM, SC SM-SC or ML-CL |

6-3.3.5 **Sources for data.** The field explorations made in connection with the project design should include a topographic map of the proposed pavement facility and surrounding vicinity indicating all streams, ditches, wells, and natural reservoirs. An analysis of aerial photographs should be conducted for information on general soil and groundwater conditions. Borings taken during the soil exploration should provide depth to water tables and subgrade soil types. Typical values of permeability for subgrade soils can be obtained from Figure 6-3. Although the value of permeability determined from Figure 6-3 must be considered only an estimate, the value should be sufficiently accurate to determine if subsurface drainage is required for the pavement. For the permeability of granular materials, estimates of the permeability may be determined from the following equations:

$$k = \frac{217.5\,(D_{10})^{1.478}\,(n)^{6.654}}{(P_{200})^{0.597}} \text{ in mm/sec} \qquad \text{(eq 6-15)}$$

or

$$k = \frac{6.214 \times 10^{5}\,(D_{10})^{1.478}\,(n)^{6.654}}{(P_{200})^{0.597}} \text{ in ft/day} \qquad \text{(eq 6-16)}$$

where

$n$ = porosity = $1 - \dfrac{\gamma_d}{\gamma_w G}$

$G$ = specific gravity (assumed 2.7)

$\gamma_d$ = density of water, $\dfrac{gm}{mm^3}, \dfrac{lb}{ft^3}$

$\gamma_w$ = dry density of material

$D_{10}$ = effective grain size at 10 percent passing in mm

$P_{200}$ = percent passing 0.075 mm (No. 200) sieve

For the most part the permeability needed for design of the drainage layer will be assigned based on the gradation of the drainage material. In some cases, laboratory permeability tests may be necessary, but it is cautioned that the permeability of very open granular materials is very sensitive to test methods, methods of compaction and gradation of the sample. Therefore, conservative drainage layer permeability values should be used for design.

6-3.4 **Criteria for Subsurface Drain Systems**

6-3.4.1 **Criteria for requiring a subsurface drain system.** Not all pavements will require a subsurface drain system either because the subgrade is sufficiently permeable to allow vertical drainage of water into the subgrade or the pavement structure does not justify the expense of a subsurface drain system. For pavements in nonfrost areas and having a subgrade with a permeability greater than 6 m/day (20 ft/day), one can assume that the vertical drainage will be sufficient such that no drainage system is required. In addition to the above exemption for the requirement for drainage systems, flexible pavements which are in nonfrost areas and having total thickness of structure above the subgrade of 200 mm (8 in.) or less are not required to have a drainage system. All pavements not meeting the above criteria are required to have a subsurface drainage system. Even if a pavement meets the exemption requirements, a drainage analysis should be conducted for possible benefits for including the drainage system. For rigid pavements in particular, care should be taken to ensure water is drained rapidly from the bottom of the slab and that the material directly beneath the concrete slab is not susceptible to pumping.

6-3.4.2 **Design water inflow.** The subsurface drainage of the pavement is to be designed to handle infiltrated water from a design storm of 1 hr duration at an expected return frequency of 2 yr. The design storm index for different parts of the world can be obtained from Figure 6-5 or from Figure 2-2. The inflow is determined by multiplying the design storm index (R) times an infiltration coefficient (F). The infiltration coefficient will vary over the life of the pavement depending on the type of pavement, surface drainage, pavement maintenance, and structural condition of the pavement. Since the determination of a precise value of the infiltration coefficient for a particular pavement is very difficult, a value of 0.5 may be assumed for design.

6-3.4.3 **Length and slope of drainage path.** The length of drainage path is measured along the slope of the drainage layer from the crest of the slope to where the water will exit the drainage layer. In simple terms, the length of the drainage path is the maximum distance water will travel in the drainage layer. The length of drainage path (L) in meters (feet) may be computed by equation 6-13, and the slope (i) of the drainage path may be computed by equation 6-14.

6-3.4.4 **Thickness of drainage layer.** The thickness of the drainage layer is computed such that the capacity of the drainage layer will be equal to or greater than the infiltration from the design storm. When the length of the drainage path (L) is in meters (feet), the design storm index (R) is in meters/hour (feet/hour), the permeability of the drainage layer (k) is in meters/hour (feet/hour), and the length of the design storm (t) is in hours, the equation for computing the thickness (H) in meters (feet) is

$$H = 2(F)(R)(L)(t)/[1.7\, n_e\, L + k\, i\, t] \qquad \text{(eq. 6-17)}$$

The effective porosity ($n_e$), the infiltration coefficient (F) and the slope of the drainage path (i) are nondimensional. If the term (k i t) is small compared to the term 1.7 $n_e$ L, which would be the case for long drainage paths, i.e., for drainage paths longer than 6 m (20 ft), then the required thickness of the drainage layer can be estimated by deleting the term (k i t) from equation 6-17 or

$$H=(F)(R)/0.85n_e \quad \text{(eq. 6-18)}$$

where the units are the same as in equation 6-17.

6-3.4.5 **Required permeability, slope, and length.** The subsurface drainage criteria require that from the end of the design storm, the drainage layer should attain 85 percent drainage within 24 hr. The time for 85 percent drainage is computed by the equation

$$T_{85}=n_e * L/(i * k) \quad \text{(eq. 6-19)}$$

where the dimensions of $T_{85}$ will be in days when L is in meters (feet) and k is in meters/day (feet/day). The time of drainage may be adjusted by changing the drainage material, the length of the drainage path or the slope of the drainage path. Changing the drainage material will change both the effective porosity and the permeability but the effective porosity will change, at the most, by a factor of 3, whereas the permeability may change by several orders of magnitude. Thus, providing a more open drainage material would decrease the time for drainage but more open materials are less stable and more susceptible to rutting. It is therefore desirable to keep the drainage material as dense as possible. The drainage layer of a pavement is usually placed parallel to the surface; therefore the slope of the drainage path is governed by the geometry of the pavement surface. For large paved areas such as parking lots, airfield aprons, and storage areas, the time for drainage is best controlled by designing the collection system to minimize the length of the drainage path. For edge drains along roads, streets, and airfield taxiways and runways, it may be difficult to reduce the length of the drainage path without resorting to placing drains under the pavement. Pavements having long longitudinal slopes may require transverse collector drains to prevent long drainage paths. Thus, designing the subsurface drainage system to meet the criteria for time of drainage involves matching the type of drainage material with the drainage path length and slope.

### 6-3.5 Placement of Subsurface Drainage System

6-3.5.1 **Rigid pavements.** In the case of rigid pavements the drainage layer, if required, shall be placed as shown in Figure 6-6 directly beneath the concrete slab. In the structural design of the concrete slab the drainage layer along with any granular separation layer shall be considered a base layer, and structural benefit may be realized from the layers.

6-3.5.2 **Flexible pavements.** In the case of flexible pavements the drainage layer should be placed either directly beneath the surface layer as shown in Figure 6-7 or beneath a graded crushed aggregate base course as shown in Figure 6-6. If the required thickness of granular subbase is equal to or greater than the thickness of the drainage layer plus the thickness of the separation layer, the drainage layer is placed beneath the graded crushed aggregate base (Figure 6-6). Where the total thickness of pavement structure is less than 300 mm (12 in.), the drainage layer may be placed directly beneath the surface layer (Figure 6-6) and the drainage layer used as a base. When the drainage layer is placed beneath an unbound aggregate base, care must be taken to limit the material passing the 0.075 mm (No. 200) sieve in the aggregate base to 8 percent or less.

**Figure 6-6. Drainage Layer Placed Beneath Base Course**

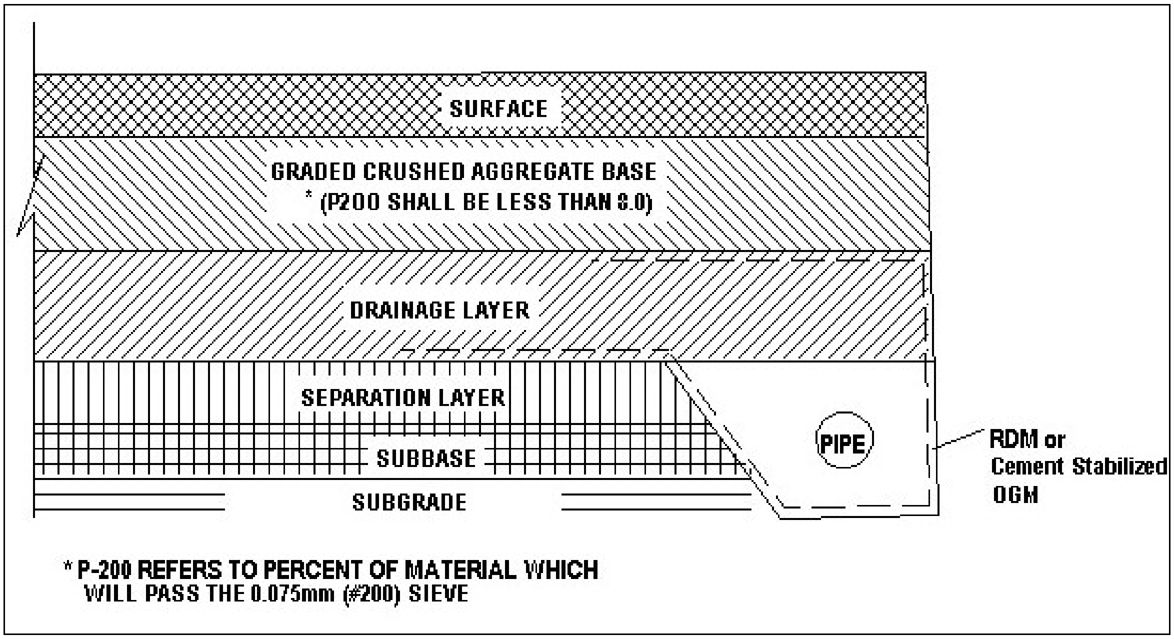
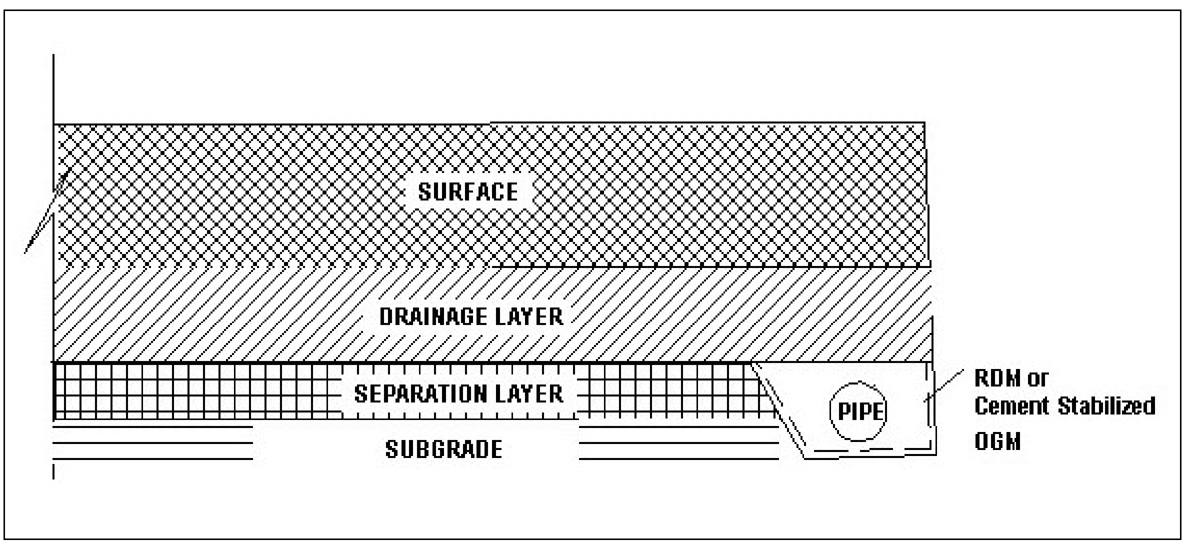



**Figure 6-7. Drainage Layer Placed Directly Below Surface Layer**



6-3.5.3 **Separation layer.** The drainage layer must be protected from contamination of fines from the underlying layers by a separation layer to be placed directly beneath the drainage layer. In most cases the separation layer should be a graded aggregate material meeting the requirements of a 50 CBR subbase and, in fact, can be considered as part of the subbase. For design situations where a firm foundation already exists and thickness of the separation layer is not needed in the structure for protection of the subgrade, a filter fabric may be substituted for the granular separation layer.

6-3.6 **Material Properties**

6-3.6.1 **For drainage layers.** The material for a drainage layer should be a hard, durable crushed aggregate to withstand degradation under construction traffic as well as in-service traffic. The gradation of the material should be such that the material has sufficient stability for the operation of construction equipment. While it is desirable for strength and stability to have the well-graded aggregate, the permeability of the material must be maintained. For most drainage layers, the drainage materials should have a minimum permeability of 300 m/day (1,000 ft/day). Two materials, a rapid draining material (RDM) and an open graded material (OGM), have been identified for use in drainage layers. The RDM is a material having a sufficiently high permeability (300 m/day (1,000 ft/day) to 1,500 m/day (5,000 ft/day)) to serve as a drainage layer and will also have the stability to support construction equipment and the structural strength to serve as a base and/or a subbase. The OGM is a material having a very high permeability (greater than 1,500 m/day (5,000 ft/day)) which can be used for a drainage layer. The OGM will normally require stabilization for construction stability and/or for structural strength to serve as a base in a flexible pavement. Gradation limits for the two materials are given in Table 6-3 and the design properties are given in Table 6-4.

**Table 6-3. Gradations of Materials for Drainage Layers and Choke Stone**

| Drainage Layer Material | | | |
|---|---|---|---|
| Sieve Designation (mm) | Rapid Draining Material | Open Graded Material | Choke Stone |
| 38.0 (1-1/2 in.) | 100 | 100 | 100 |
| 25.0 (1 in.) | 70-100 | 95-100 | 100 |
| 19.0 (3/4 in.) | 55-100 | -- | 100 |
| 12.5 (1/2 in.) | 40-80 | 25-80 | 100 |
| 9.5 (3/8 in.) | 30-65 | -- | 80-100 |
| 4.75 (No. 4) | 10-50 | 0-10 | 10-100 |
| 2.4 (No. 8) | 0-25 | 0-5 | 5-40 |
| 1.2 (No. 16) | 0-5 | -- | 0-10 |

**Table 6-4. Properties of Materials for Drainage Layers**

| Property | Rapid Draining Material | Open Graded Material |
|---|---|---|
| Permeability in m/sec (feet/day) | 300-1,500 (1,000-5,000) | > 1,500 (> 5,000) |
| Effective Porosity | 0.25 | 0.32 |
| Percent Fractured Faces (COE method) | 90% for 80 CBR<br>75% for 50 CBR | 90% for 80 CBR<br>75% for 50 CBR |
| $C_v$ | > 3.5 | -- |
| LA Abrasion | < 40 | < 40 |
| Note: $C_v$ is the uniformity coefficient = D60/D10. | | |

6-3.6.2 **Aggregate for separation layer.** The separation layer serves to prevent fines from infiltrating or pumping into the drainage layer and to provide a working platform for construction and compaction of the drainage layer. The material for the separation layer should be a graded aggregate meeting the requirements of a 50 CBR subbase as given in TM 5-825-2/AFM 88-6, Chap. 2 except that the maximum aggregate size should not be greater than 1/4 the thickness of the separation layer. The permeability of the separation layer should be greater than the permeability of the subgrade, but the material should not be so open as to permit pumping of fines into the separation layer. To prevent pumping of fines the ratio of $d_{15}$ of the separation layer to $d_{85}$ of the subgrade must be equal to or less than 5. The material property requirements for the separation layer are given in Table 6-5.

**Table 6-5. Criteria For Granular Separation Layer**

| Maximum Aggregate Size | Lesser of 50 mm (2 in.) or 1/4 of layer thickness |
|---|---|
| Maximum CBR | 50 |
| Maximum Percent Passing 2.00 mm (No. 10) | 50 |
| Maximum Percent Passing 0.075 mm (No. 200) | 15 |
| Maximum Liquid Limit | 25 |
| Maximum Plasticity Index | 5 |
| $d_{15}$ of Separation Layer to $d_{85}$ of Subgrade | ≤ 5 |

6-3.6.3 **Filter fabric for separation layer.** Filter fabric provides protection against pumping, but does not provide extra stability for compaction of the drainage layer. Therefore, fabric should be selected only when the subgrade provides adequate support for compaction of the drainage layer. The important characteristics of the fabric are strength for surviving construction and traffic loads, and apparent opening size (AOS) to prevent pumping of fines into the drainage layer. Filter fabric for separation shall be a nonwoven needle punched fabric meeting the criteria given in Table 6-6.

**Table 6-6. Criteria for Filter Fabric to be Used as a Separation Layer**

| | Criteria | ASTM Test Method |
|---|---|---|
| 50 Percent or Less Passing No. 200 Sieve | AOS (mm) < 0.6 mm Greater than No. 30 sieve | D-4751 |
| Greater Than 50 Percent Passing No. 200 Sieve | AOS (mm) < 0.297 Greater than No. 50 sieve | D-4751 |
| Minimum Grab Strength in kN(lbs) at 50% Elongation | 0.8 (180) | D-4632 |
| Minimum Puncture Strength in kN(lbs) | 0.35 (80) | D-4833 |

## 6-4 STABILIZATION OF DRAINAGE LAYER

6-4.1 **General.** Stabilization of OGM is normally required for stability and strength, and for preventing degradation of the aggregate in handling and compaction. Stabilization may also be used when high quality crushed aggregate is not available and there may even be occasions when stabilization of RDM is necessary. Stabilization may be accomplished mechanically by use of a choke stone or by the use of a binder such as asphalt or portland cement.

6-4.2 **Choke Stone Stabilization.** A choke stone is a small size stone used to stabilize the surface of an OGM. The choke stone should be a hard, durable, crushed aggregate having 90 percent fractured faces. The ratio of $d_{15}$ of the coarse aggregate to the $d_{15}$ of the choke stone must be less than 5, and the ratio of the $d_{50}$ of the coarse aggregate to $d_{50}$ of the choke stone must be greater than 2. The gradation range for acceptable choke stone is given in Table 6-3. Normally ASTM No. 8 or No. 9 stone will meet the requirements of a choke stone for the OGM.

6-4.3 **Asphalt Stabilization.** Stabilization of the drainage material is accomplished by using only enough asphalt required to coat the aggregate. Care should be taken so that the voids are not filled by excess asphalt. Asphalt grade used for stabilization should be AC20 or higher. For stabilization of OGM, 2 to 2-1/2 percent asphalt by weight should be sufficient to coat the aggregate. Higher rates of application may be necessary when stabilization of less open aggregate such as RDM is necessary.

6-4.4 **Cement Stabilization.** As with asphalt stabilization, portland cement stabilization is accomplished by using only enough cement paste to coat the aggregate, and care should be taken so that the voids are not filled by excess paste. The amount of portland cement required should be approximately 170 kilograms per cubic meter (2 bags/$yd^3$) depending on the gradation of the aggregate. The water-cement ratio should be just sufficient to provide a paste which will adequately coat the aggregate.

## 6-5 CONSTRUCTION OF THE DRAINAGE LAYER

6-5.1 **Experience.** Construction of drainage layers can present problems in handling, placement, and compaction. If the drainage material does not have adequate stability, major problems can develop in the placement of the surface layer above the drainage layer. Experience with highly permeable bases (drainage layers) both by the Corps of Engineers and various State Departments of Transportation indicates that pavements containing such layers can be constructed without undue difficulties provided due precautions are taken. The real key to successful construction of the drainage layers is the training and experience of the construction personnel. Prior to start of construction, the construction personnel should be indoctrinated in the handling and placing of the drainage material. The placement of test strips is recommended for training of the construction personnel.

6-5.2 **Placement of Drainage Layer.** The material for the drainage layer must be placed in a manner to prevent segregation and to obtain a layer of uniform thickness. The materials for the drainage layer will require extra care in stockpiling and handling. Placement of the RDM and OGM is best accomplished using an asphalt concrete paver. To ensure good compaction, the maximum lift thickness should be no greater than 150 mm (6 in.). If choke stone is used to stabilize the surface of OGM, the choke stone is placed after compaction of the final lift of OGM. The choke stone is spread in a thin layer no thicker than 10 mm (1/2 in.) using a spreader box or paver. The choke stone is worked into the surface of the OGM by the use of a vibratory roller and by wetting. The choke stone remaining on the surface should not migrate into the OGM by the action of water or traffic.

6-5.3 **Compaction.** Compaction is a key element in the successful construction of the drainage layer. Compaction control normally used in pavement construction is not appropriate for materials such as the RDM and OGM. It is therefore, necessary to specify compaction techniques and level of effort instead of the properties of the end product. It will be important to place the drainage material in relatively thin lifts of 150 mm (6 in.) or less and to have a good firm foundation beneath the drainage material. The recommended method of determining the required compaction effort is to construct a test section and closely monitor the aggregate during compaction to determine when crushing of the aggregate appears excessive. Experience has indicated that sufficient compaction can be obtained by six passes or less of a vibratory roller loaded at approximately 9 metric tons (10 short tons). Material not being stabilized with asphalt or cement should be kept moist during compaction. Asphalt stabilized material for drainage layers must be compacted at a somewhat lower temperature than a dense-graded asphalt material. In most cases, it will be necessary to allow an asphalt stabilized material to cool to less than 93 degrees C (200 degrees F) before beginning compaction.

6-5.4 **Protection After Compaction.** After compaction, the drainage layer should be protected from contamination by fines from construction traffic or from flow of surface water. It is recommended that the surface layer be placed as soon as possible after placement of the drainage layer. Precautions must also be taken to protect the drainage layer from disturbance by construction equipment. Only tracked asphalt pavers should be allowed for paving over any RDM or OGM that has not been stabilized. Drivers should avoid rapid acceleration, hard braking, or sharp turning on the completed drainage layer. Although curing of cement stabilized drainage layers is not critical, efforts should be made at curing until the surface layer is placed.

6-5.5 **Proof Rolling.** For Army Class IV airfield with runways over 1,524 m (5,000 ft) and Air Force heavy, modified heavy, and medium load flexible airfield pavements, proof rolling as per TM 5-825-2/AFM 88-6, Chap. 2, is required on the graded crushed aggregate base even when used over a drainage layer. Proof rolling the separation layer prior to placement of the drainage layer for other airfield pavements is recommended. For other Air Force flexible airfield pavements and Army Class IV flexible airfield pavements with runways less than 1,524 m (5,000 ft), it is recommended that the proof rolling be accomplished using a rubber-tired roller load to provide a minimum tire force of 89 kN (20,000 lb) and inflated to at least 620 kPa (90 lb/in.$^2$). A minimum of six coverages should be applied, where a coverage is the application of one tire print over each point in the surface of the designated area. For rigid pavements and flexible pavements for roads, streets, parking areas and Class I, II, and III Army airfields, proof rolling of the separation layer may be accomplished using the rubber-tired roller described above or by using a truck having tandem axles with either dual tires or super single tires. The truck should be loaded to provide 89 kN (20,000 lb) per axle. During proof rolling, action of the separation layer must be monitored for any sign of excessive movement or pumping that would indicate soft spots in the separation layer or the subgrade. Since the successful placement of the drainage layer depends on the stability of the separation layer, all weak spots must be removed and replaced with stable material. All replaced material must be proof rolled as specified above.

## 6-6 COLLECTOR DRAINS

6-6.1 **Design Flow.** Collector drains are to be provided to collect and transport water from under the pavement. For pavements having drainage layers, it is mandatory that collector drains be provided. The collector system should have the capacity to handle the water from the drainage layer plus water from other sources. The water entering the collector system from the drainage layer is computed assuming the drainage layer is flowing full. Thus, the volume of water ($Q_o$) in cubic millimeters per second per meter (cubic feet per day per foot) of length of collector pipe (assuming the drainage layer is only on one side of the collector) would be

$$Q = (H)*(i)*(k)*(1000) \text{ in cubic mm per second per meter} \qquad \text{(eq. 6-20)}$$

or

$$Q = (H)*(i)*(k) \text{ in cubic ft per day per foot} \qquad \text{(eq. 6-21)}$$

where

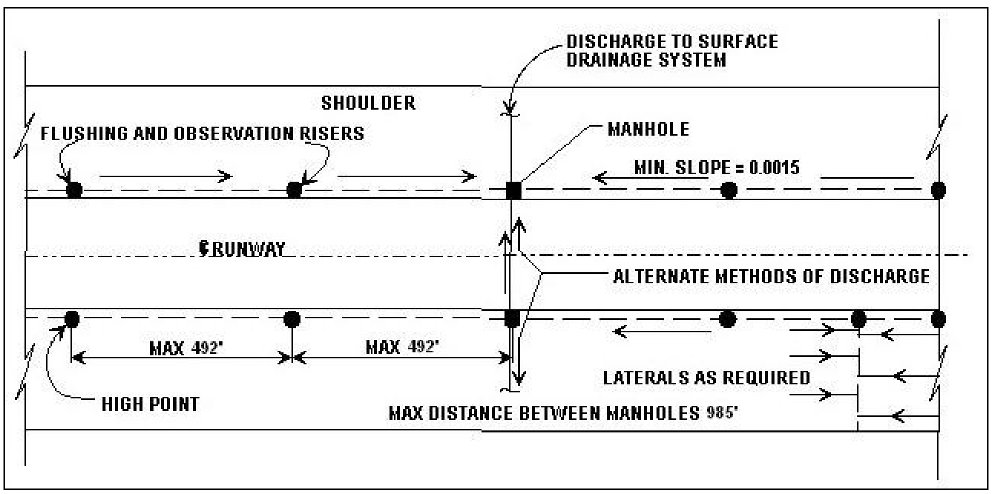
H = thickness of the drainage layer, mm (ft)
i = slope of the drainage layer
k = permeability of the material in the drainage layer, mm/sec (ft/day)

If the collector system has water entering from both sides, the volume of water entering the collector would be double that given by equation 6-20.

6-6.2 **Design of Collector Drains**

6-6.2.1 **Drain system layout**. The collector drains are normally placed along the shoulder of the pavement as illustrated in Figure 6-8. The system will consist of the drain pipe, flushing and observation risers, manholes, discharge laterals, filter fabric, and trench backfill. The drainage system for large areas of pavement may require placement of subsurface drains under the pavement. Typical designs for the collector drains are given in Figures 6-9, 6-10, 6-11, and 6-12.

**Figure 6-8. Plan View of Subsurface Drainage System**




6-6.2.2 **Collector pipe**. The collector pipe may be perforated flexible, ABS, corrugated polyethylene (CPE) or smooth rigid polyvinyl chloride pipe (PVC). Pipe should conform to the appropriate AASHTO Specification. Most State Highway Agencies use either CPE or PVC. For CPE pipe, AASHTO specification M 252 “Corrugated Polyethylene Drainage Tubing” is suggested, while for PVC pipe, AASHTO Specification M 278, “Class PC 50 Polyvinyl Chloride (PVC) Pipe,” is recommended. It is recommended that asphalt stabilized material not be used as backfill around pipe, but, if it is to be used, then the pipe should be PVC 90 degrees C electric plastic conduct, EPC40 or EPC80 conforming to the requirements of National Electrical Manufacturers Association Specification TC2. Geocomposite edge drains (strip drains) may be used in special situations but only with the approval of HQUSACE (CEMPET) or the appropriate Air Force major command. Geocomposite edge drains should only be considered for pavements not having a drainage layer.

**Figure 6-9. Typical Concrete Pavement Interior Subdrain Detail**

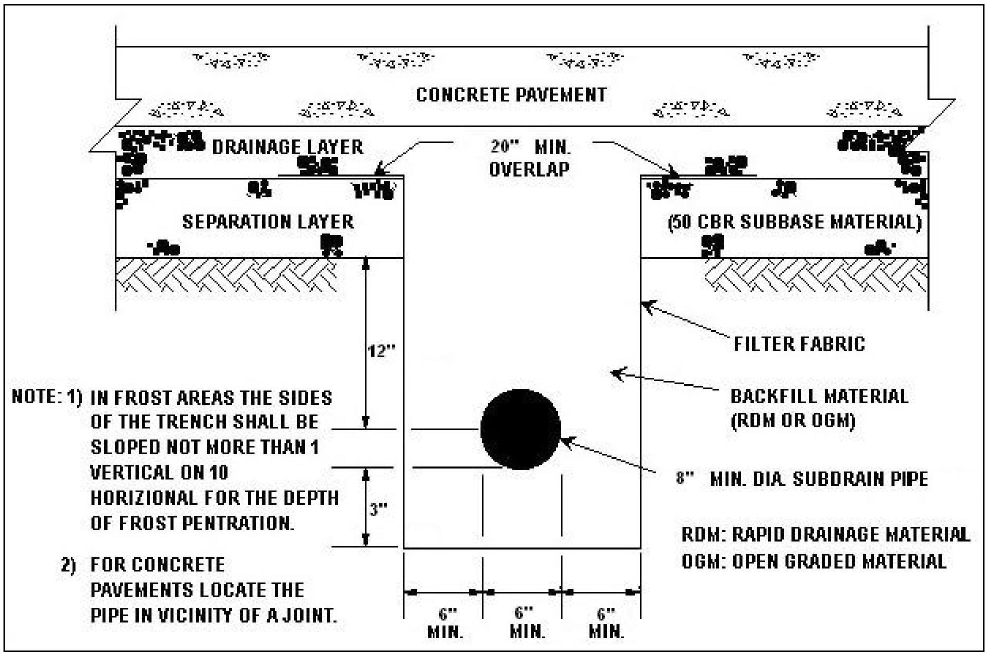



**Figure 6-10. Typical Edge Subdrain Detail for Flexible Pavements**

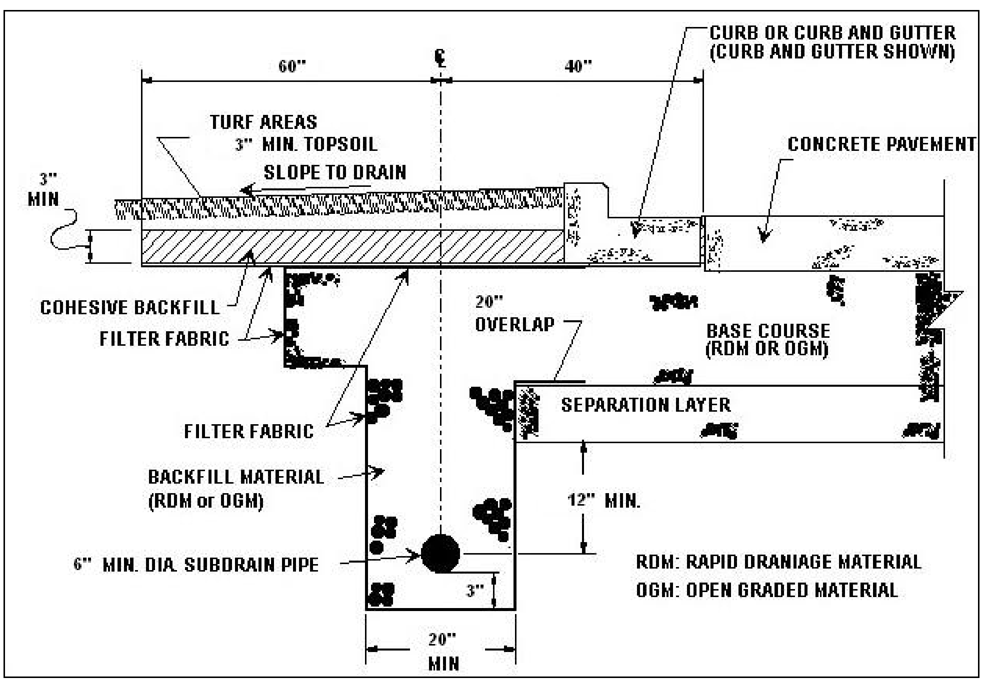



**Figure 6-11. Typical Flexible Pavement Interior Subdrain Detail**

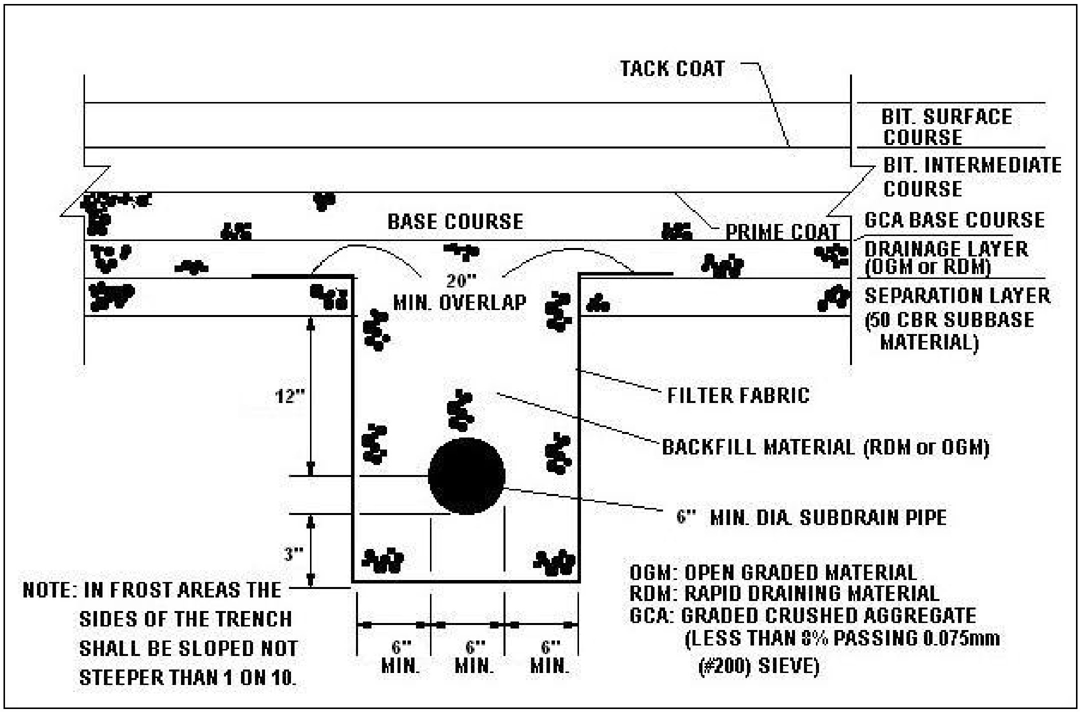
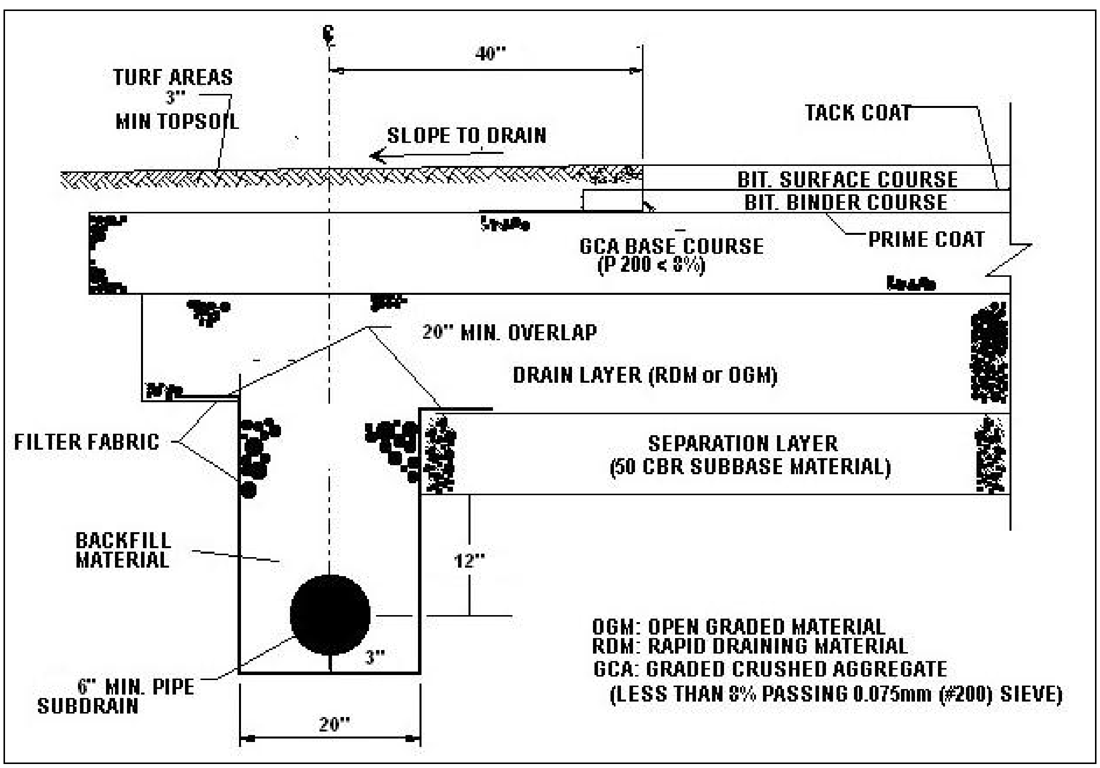



**Figure 6-12. Typical Edge Subdrain Detail for Flexible Pavements**



6-6.2.3 **Pipe size and slopes**. The pipe must be sized, according to equations 6-22 or 6-23, to have a capacity sufficient to collect the peak flow from under the pavement. Equations 6-22 and 6-23 are Manning equations for computing the capacity of a full flowing circular drain. The equation for flow (Q) in cubic feet per second is:

$$Q = \frac{1.486}{n}(A)\left(\frac{d}{4}\right)^{2/3}(s^{1/2}) \qquad \text{(eq. 6-22)}$$

where

n = coefficient of roughness for the pipe
A = area of the pipe, $ft^2$
d = pipe diameter, ft
S = slope of the pipe invert

For metric units the equation for flow in cubic meters per second is:

$$Q = \frac{1.0}{n}(A)\left(\frac{d}{4}\right)^{2/3}(s^{1/2}) \qquad \text{(eq. 6-23)}$$

where

n and s are as defined in equation 6-22
A = pipe area, $m^2$
d = pipe diameter, m

The coefficient of roughness for different pipe types can be obtained from Table 6-7. Except for long intercepting lines and extremely severe groundwater conditions, 150 mm (6 in.) diameter drains should be satisfactory for most subsurface drainage installations. The minimum size pipe recommended for all collector drains is a 150 mm (6 in.) diameter pipe. The recommended minimum slope for subdrains is 0.15 percent.

**Table 6-7. Coefficient of Roughness for Different Types of Pipe**

| Type of Pipe | Coefficient of Roughness, n |
|---|---|
| Clay, concrete, smooth-wall plastic, and Asbestos-cement | 0.013 |
| Bituminous-coated, non-coated corrugated metal pipe or corrugated metal pipe | 0.024 |

### 6-6.3 Trench Construction

6-6.3.1 **Design.** The trench for the collector drains should be constructed of sufficient width to provide 150 mm (6 in.) clearance on each side of the pipe. The depth of the trench must be sufficient to provide a minimum 300 mm (12 in.) from the top of the pavement subgrade to the center of the pipe plus 80 mm (3 in.) clearance beneath the pipe. The minimum cover requirements for pipe is dependent upon loading and frost requirements. Cover requirements for different design wheel loads are indicated in Appendix D. In frost areas the center of the pipe should be placed below the depth of frost penetration. In areas where the depth of frost penetration is greater than 1.2 m (4 ft) below the bottom of the drainage layer, the pipe need not be located deeper than 1.2 m (4 ft) from the bottom of the drainage layer. Also in frost areas and when differential heave will cause pavement problems, the sides of the trench shall be sloped not steeper than 1 vertical on 10 horizontal for the depth of frost penetration. The sloping of the trench sides is not required for the parts of the trench in nonfrost susceptible materials nor for F1 or S1 soils unless the pavement over the trench is subjected to high speed traffic.

6-6.3.2 **Backfill.** The trench should be backfilled with a permeable material to rapidly convey water to the drainage pipe. The backfill material may be either a OGM, RDM, or other uniform graded aggregate. A minimum of 80 mm (3 in.) of aggregate should be placed beneath the drainage pipe. Proper compaction or chemical stabilization of the backfill is necessary to prevent settlement of the fill. In placing the backfill, the backfill should be compacted in lifts not exceeding 300 mm (12 in.). When geocomposites are used in place of pipe, the geocomposites are placed against the material to be drained and thus the backfill is not expected to convey water. For this reason the backfill for the geocomposites will not require the high permeability required for the backfill around the pipe drains. However, since the backfill for the geocomposites will be against the side of the trench, the backfill should meet the requirements of a granular filter.

6-6.3.3 **Geotextiles in the trench.** The trench should be provided with a geotextile filter fabric as shown in Figures 6-9 through 6-12 for the typical details. The filter fabric should be placed to separate the permeable backfill of the trench from the subgrade or subbase materials. The filter fabric must not be placed so as to impede the flow of water from the drainage layer to the drain pipe. The filter fabric must also protect from the infiltration of fines from any surface layers. This is particularly important for drains placed outside the pavement area where surface water can enter the drain through a soil surface. The filter fabric for the trench shall be a nonwoven needle punched fabric meeting the criteria given in Table 6-8.

**Table 6-8. Criteria for Fabrics Used in Trench Construction**

| | ASTM Test Method | Criteria |
|---|---|---|
| Soil With 50 Percent or Less Passing No. 200 Sieve | D 4751 | AOS < 0.6 mm (Sieve No. 30) |
| Soil With Greater Than 50 Percent Passing No. 200 Sieve | D 4751 | AOS < 0.297 mm (Sieve No. 50) |
| Minimum Grab Strength in kN (lb) at 50% Elongation | D 4632 | 0.6 (130) |
| Minimum Puncture Strength in kN (lb) | D 4833 | 0.25 (55) |

6-6.3.4 **Trench cap.** Edge drains placed outside of a paved area should be capped with a layer of low permeability material to reduce the infiltration of surface water into subsurface drainage system.

6-6.4 **Lateral Outlet Pipe**

6-6.4.1 **Design.** The lateral outlet pipe provides both a means of getting water out of the edge drains, and for cleaning and inspecting the system. Edge drains should be provided with lateral outlet pipes spaced at intervals (90 to 150 m) (300 to 500 ft) along the edge drains and at the low point of all vertical curves. To facilitate drain cleanout, the outlet pipes should be placed at about a 45 degree angle from the direction of flow in the collector drain. The lateral pipe should be a metal or rigid solid-walled pipe and should be equipped with an outlet structure. A 3 percent slope from the edge drain to the outlet structure is recommended. To reduce outlet maintenance, outlet pipes should, where possible, be connected to existing storm drains or inlets. For lateral pipe flowing to a ditch, the invert of the outlet pipe should be a minimum of 150 mm (6 in.) above the 2-yr design flow in the ditch. To prevent piping, the trench for the outlet pipes must be backfilled with a material of low permeability, or provided with a cutoff wall or diaphragm. Dual outlets are recommended for maintenance considerations, as shown in Figure 6-13. The dual outlet system allows sections of collector drains to be flushed out to clear any debris material blocking the free flow of water. Other recommended design details for drainage outlets are as follows:

6-6.4.1.1 Provide dual outlet with large radius bend, as shown in Figure 6-14.

6-6.4.1.2 Use rigid walls, not perforated pipes. For pipe drains use the same diameter pipe as the collector drains. For prefabricated geocomposite drains, 102-mm to 152-mm (4-in. to 6-in.) diameter pipe should provide adequate hydraulic capacity. The flow capacity of the outlets must be greater than that of the collector drains. In general, because of the greater slope provided for outlet pipes, the hydraulic capacity is not a problem.

6-6.4.1.3 The discharge end of the outlet pipe should be placed at least 152 mm (6 in.) above the 10-yr design flow in the drainage ditch (Figure 6-15). The same requirement applies even if the outlet is discharging into storm drain inlets.

6-6.4.2 **Outfall for outlet pipe.** The outfall for the outlet pipe should be provided with a headwall to protect the outlet pipe from damage, prevent slope erosion, and facilitate the location of outlet pipes. Headwalls should be placed flush with the slope so that mowing operations are not impaired. Easily removed rodent screens should be installed at the pipe outlet. The headwall may be precast or cast-in-place. An example for a design for a headwall is given in Figure 6-16.

6-6.4.3 **Reference markers.** Although not a requirement, reference markers are recommended for the outlets to facilitate maintenance and/or observation. A simple flexible marker post or marking on the shoulder will suffice to mark the outlet.

**Figure 6-13. Schematic of Dual Outlet System Layout (Baumgardner 1998)**

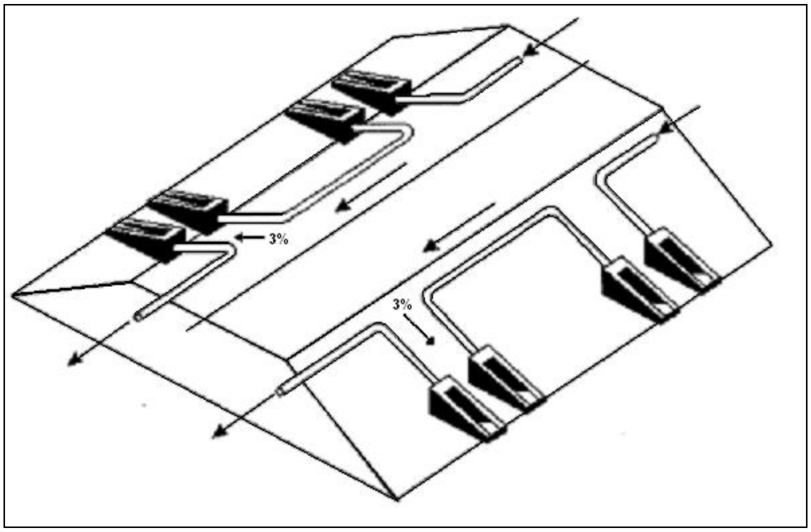



**Figure 6-14. Illustration of Large-Radius Bends Recommended for Drainage Outlet**

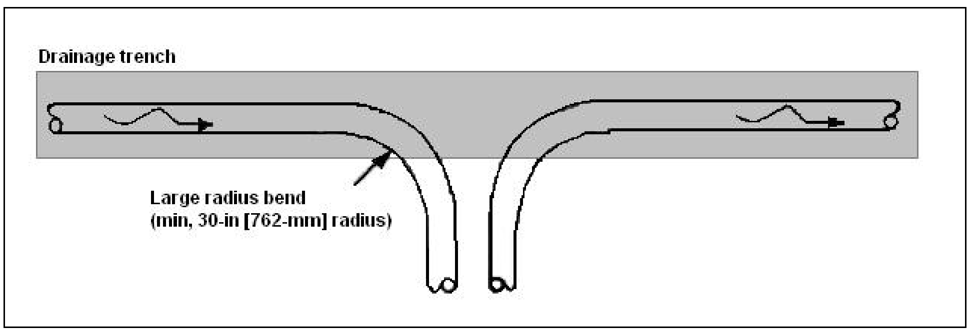



**Figure 6-15. Recommended Outlet Design Detail**

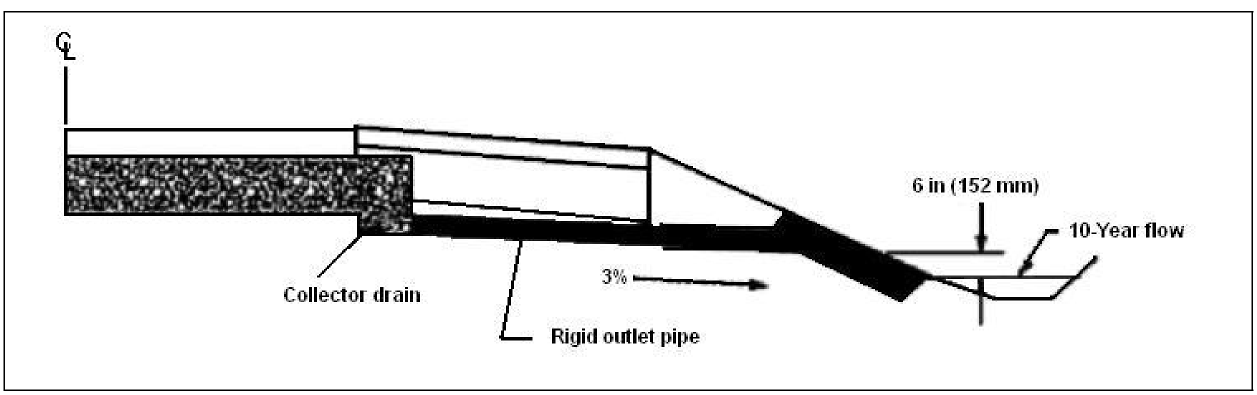
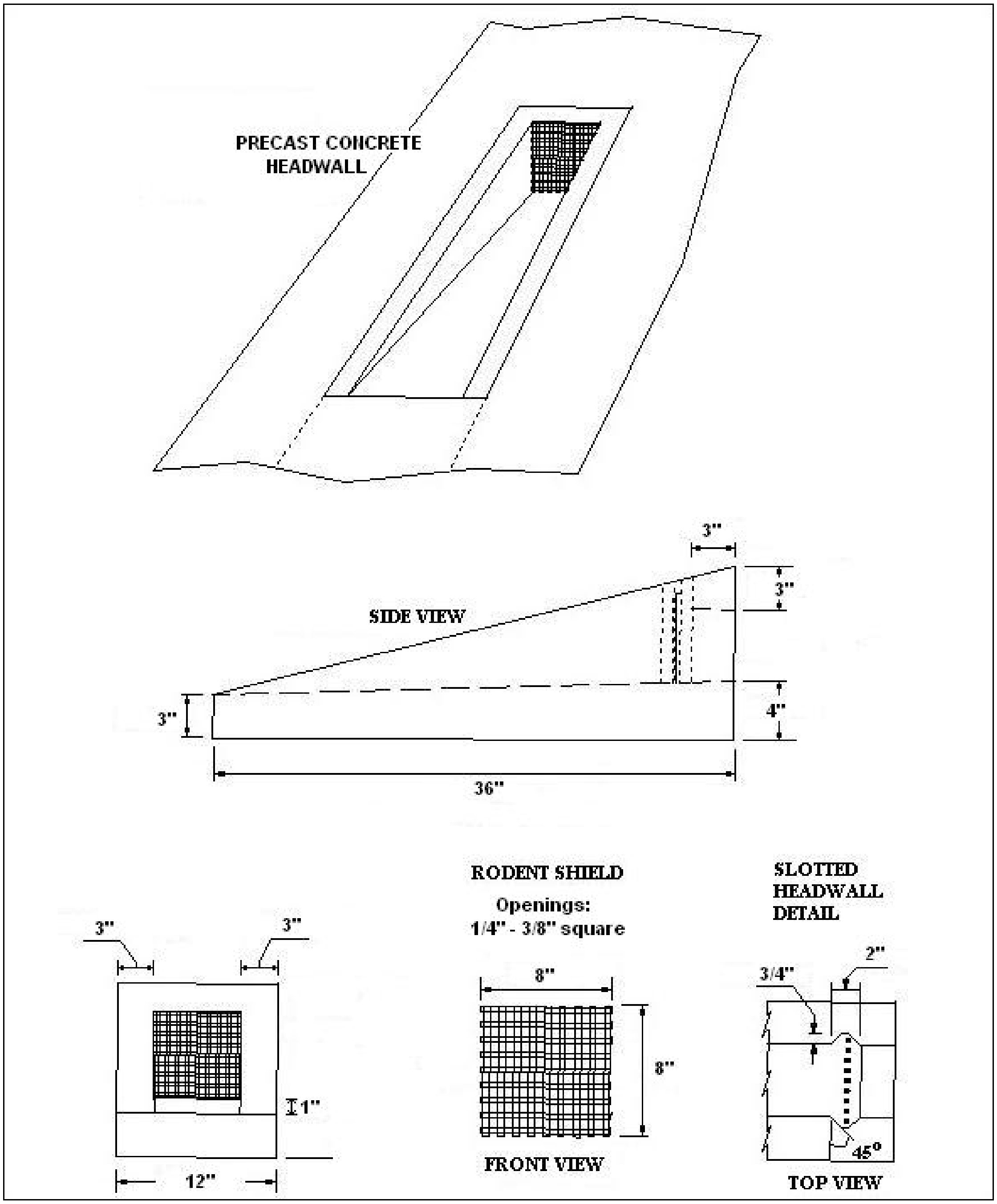


**Figure 6-16. Example Design for a Headwall**




6-6.5 **Cross Drains.** Cross drains may be required at locations where flow in the drainage layer is blocked, for steep longitudinal grades, or at the bottom of vertical curves. For example, cross drains may be required where pavements abut building foundations, at bridge approach slabs, or where drainage layers abut impermeable bases.

6-6.6 **Manholes and Observation.** Manholes, observation basins, and risers are installed on subsurface drainage systems for access to the system to observe its operation and to flush or rod the pipe for cleaning. When required, manholes on subgrade pipe drains should be located at intervals of not over 300 meters (1,000 feet) with one flushing riser located between manholes and at dead ends. Manholes should be provided at principal junction points of several drains. Typical details of construction are given in Chapter 4.

## 6-7 MAINTENANCE OF SUBSURFACE DRAINAGE SYSTEMS

6-7.1 **Monitoring Program**. Commitment to maintenance is as important as providing subsurface drainage systems. In fact, an improperly maintained drainage system can cause more damage to the pavement structure than if no drainage were provided at all. Poor maintenance leads to clogged or silted outlets and edgedrain pipes, missing rodent screens, excessive growth of vegetation blocking outlet pipes and openings on daylighted bases, and growth of vegetation in side ditches. These problems can potentially cause backing up of water within the pavement system, thereby defeating the purpose of providing the drainage system. Therefore, inspections and maintenance of subsurface drainage systems should be made an integral part of the policy of any agency installing these systems. The inspection process comprises of two parts: (a) visual inspection and (b) video inspection.

6-7.1.1 **Visual inspection**. The visual inspection process includes the following items:

6-7.1.1.1 Evaluation of external drainage-related features, including measurement of ditch depths and checking for crushed outlets, excessive vegetative growth, clogged and debris-filled daylighted openings, condition of headwalls, presence of erosion, and missing rodent screens. This operation should be performed at least once a year.

6-7.1.1.2 Pavement condition evaluation to check for moisture-related pavement distresses such as pumping, faulting, and D-cracking in PCC pavements and fatigue cracking and AC stripping in AC pavements. This operation could be either a full-scale PCI survey or a brief overview survey, depending on agency needs. The recommended frequency for this activity is once every 2 years.

6-7.1.2 **Video inspection**. Video inspections play a vital role in monitoring in-service drainage systems. The video inspection process can be used to check for clogged drains due to silting and intrusion of surrounding soil, as well as any problems with the drainage system, such as ruptured pipes and broken connections. Video inspections should be carried out on an as-needed basis whenever there is evidence of drainage-related problems. A detailed list of equipment used in an FHWA Study (Daleiden 1998) is given in Table 6-9. A video inspection system typically consists of a camera head, long flexible probe mounted on a frame for inserting the camera head into the pipe, and a data acquisition unit fitted with a video screen and a video recorder. This system can be used to detect and correct any construction problems before a project is accepted. The construction-related problems that are easily detected using the video equipment include crushed or ruptured drainage pipes and improper connections between drainage pipes, as well as the connection between the outlet pipe and headwall.

**Table 6-9. Equipment Description or FHWA Video Inspection Study (Daleiden 1998)**

| |
|---|
| **Camera:** The camera is a Pearpoint flexiprobe high-resolution, high-sensitivity, waterproof color video camera engineered to inspect pipes 76 to 152 mm (3 to 6 in.) in diameter. The flexiprobe lighthead and camera has a physical size of 71 mm (2.8 in.) and is capable of negotiating 102 mm × 102 mm (4 in. × 4 in.) plastic tees. The lighthead incorporates six high-intensity lights. This lighting provides the ability to obtain a “true” color picture of the entire surface periphery of a pipe. The camera includes a detachable hard plastic ball that centers the camera during pipe inspections. |
| **Camera Control Unit** The portable color control unit includes a built-in 203-mm (8-in.) color monitor and controls including remote iris, focus, video input/output, audio in with built-in speaker, and light level intensity control. Two VCR input/output jacks are provided for video recording as well as tape playback verification through the built-in monitor. |
| **Metal Coiler and Push Rod With Counter:** The portable coiler contains 150 m (6 in.) of integrated semi-rigid push rod, gold and rhodium slip rings, electro-mechanical cable counter, and electrical cable. The integrated push rod/electrical cable consists of a special epoxy glass reinforced rod with polypropylene sheathing material, which will allow for lengthy inspections due to the semi-rigid nature of this system. |
| **Video Cassette Recorder:** The video cassette recorder is a high-quality four-head industrial grade VHS recorder with audio dubbing, still frame, and slow speed capabilities. |
| **Generator:** A compact portable generator capable of providing 650 watts at 115 V to power the inspection equipment. |
| **Molded Transportation Case:** A molded transportation case, specifically built for air transportation, encases the control unit, camera, and videocassette recorder. |
| **Color Video Printer:** A video printer is incorporated into the system to allow the technician to obtain color prints of pipe anomalies or areas of interest. |

### 6-7.2 Maintenance Guidelines

6-7.2.1 **Collector drains and outlets**. The collector drains and outlets should be flushed periodically with high-pressure water jets to loosen and remove any sediment that has built up within the system. The key to this operation is having the appropriate outlet details that facilitate the process, such as the dual headwall system shown in Figure 6-13. The area around the outlet pipes should be kept mowed to prevent any buildup of water. Missing rodent screens and outlet markers, damaged pipes and headwalls need to be either repaired or replaced.

6-7.2.2 **Daylighted systems**. Routine removal of roadside debris and vegetation clogging the daylighted openings of a permeable or dense-graded base is very important for maintaining the functionality of these systems.

6-7.2.3 **Drainage ditches**. The drainage ditches should be kept mowed to prevent excessive vegetative growth. Debris and silt deposited at the bottom of the ditch should be cleaned periodically to maintain the ditch line and to prevent water from backing up into the pavement system.

# CHAPTER 7

## FROST PROTECTION DESIGN FOR AIRFIELD PAVEMENTS

7-1 **SCOPE.** This chapter presents criteria for the design of frost protection for airfield pavements. Included in this chapter are criteria for subsurface exploration as it relates to frost and drainage, and frost protection.

7-2 **RELATED CRITERIA**.

| **Subject** | **Source** |
|---|---|
| Pavements | NAVFAC DM-5.04 |
| Soil Mechanics | NAVFAC DM-7.01 |
| Foundations and Earth Structures | NAVFAC DM-7.02 |
| Pavement Design for Airfields | NAVFAC DM-21.10 |
| Airfield Pavement Design | MIL-HDBK-1021 (Series) |
| Airfield and Heliport Planning and Design | NAVFAC P-971 |

7-3 **DEFINITIONS**. The following specialized terms are used in this chapter.

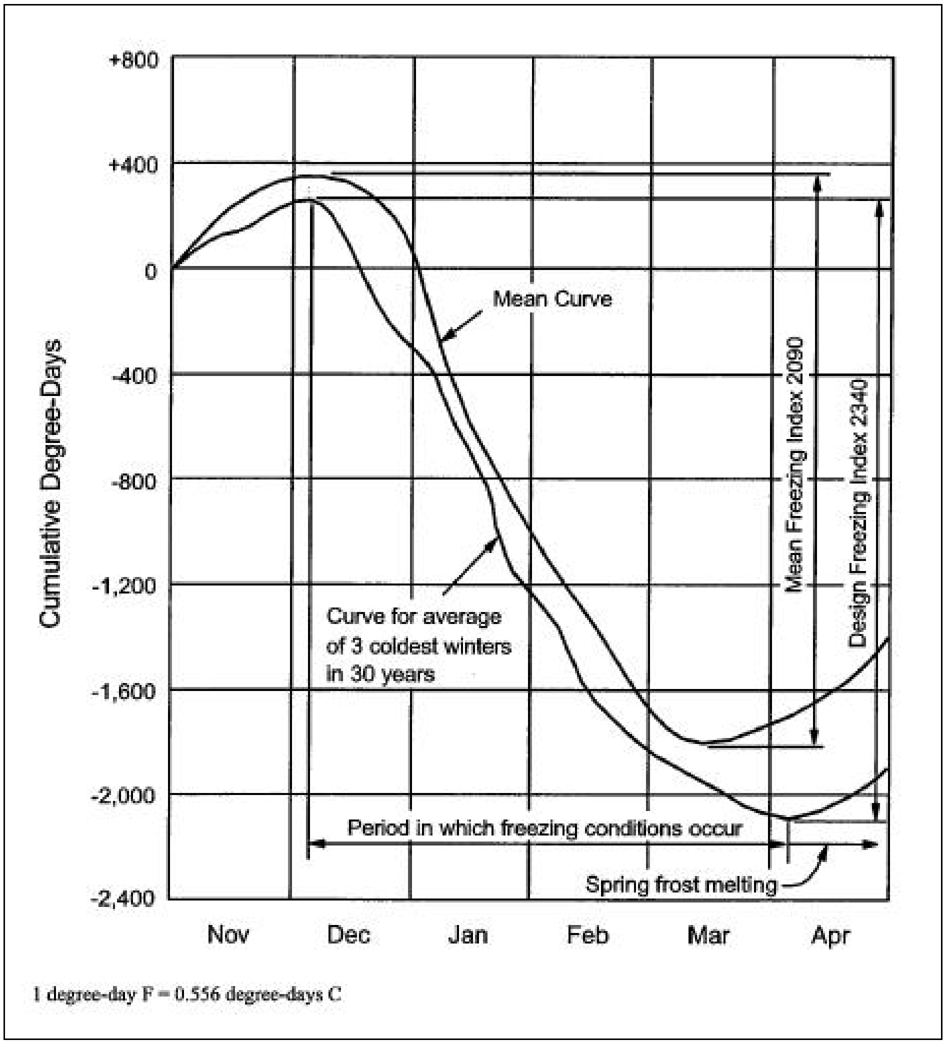
7-3.1 **Average Daily Temperature**. The average of the maximum and minimum temperatures for one day, or the average of several temperature readings taken at equal time intervals (typically on an hourly basis) during one day.

7-3.2 **Degree Days**. The degree-days for any one day is the difference between the average daily air temperature and 32 degrees F (0 degrees C). The degree days are negative when the average daily temperature is below 32 degrees F (freezing degree-days) and positive when it is above 32 degrees F (thawing degree-days). Figure 7-1 shows curves obtained by plotting cumulative degree-days against time.

7-3.3 **Design Freezing Index**. The average air freezing index of the three coldest winters in the latest 30 years of record. If 30 years of record are not available, the index for the latest 10-yr period may be used. The design freezing index at a site with continuing construction need not be changed more often than once in 5 years unless recent temperature records indicate a significant change in thickness design requirements for frost. Design freezing index is illustrated in Figure 7-1.

7-3.4 **Freezing Index**. The number of degree-days between the highest and lowest points on a cumulative degree-days versus time curve for one freezing season. Freezing Index is a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season. The index determined for air temperatures at 1.35 m (4.5 ft) above the ground is commonly designated as the air freezing index, while that determined for temperatures immediately below the surface is known as the surface freezing index.

**Figure 7-1. Example Determination of Freezing Index**




7-3.5 **Frost**. As it related to pavements, frost is the condition of free water freezing within the pavement structure or in the subgrade. The action of frost includes expansion or heaving, as well as the loss of support during the melt period. The frost action may result in the formation of ice crystals in any frost-susceptible material within or below the pavement structure to which freezing temperatures penetrate.

7-3.6 **Mean Daily Temperature**. The average of the average daily temperatures for a given day for several years.

7-3.7 **Mean Freezing Index**. The freezing index determined based on mean temperatures. The period of record over which temperatures are averaged is usually a minimum of 10 years, the preferred being 30 years. The latest available data should be used. Mean freezing index is illustrated in Figure 7-1.

7-4 **EFFECTS OF FROST ACTION**. Frost action can cause differential heaving, cracking, surface roughness, blocked drainage, and a reduction in bearing capacity during thaw periods. The extent of these problems ranges from slight to severe, depending on the type and uniformity of the subgrade soil and availability of water. The most effective method of addressing the effects of frost action is taking measures to avoid this problem. This is typically accomplished by either removing and replacing all frost-susceptible material within frost penetration depth, or providing sufficient cover over the susceptible material with non-frost susceptible material.

7-4.1 **Frost Heaving**. Upon freezing, the volume of water expands by about 9 percent; however, this volume expansion alone is not sufficient to account for the heaving of several inches or more that occurs in some pavements. Frost heaving results from the growth of ice lenses in susceptible subgrade or unbound materials in the pavement structure. Uniform heave is generally not troublesome, but nonuniform heave can result in serious surface irregularities in flexible pavements and cracking in rigid pavements. Differential heave is usually the result of variations in subgrade soils, soil moisture, and transitions from cut to fill with high groundwater level.

7-4.2 **Formation of Ice Lenses**. Ice lenses form in soils that are highly susceptible to capillary action. As the soil is slowly cooled, the water in the voids begins to freeze to form ice crystals. If the soil is susceptible to capillary action, water is drawn to these ice crystals, which grow to form ice lenses. The ice lenses continue to grow as long as the freezing conditions remain and supply of water is present. To have serious formation of ice lenses, three conditions must exist:

a. Presence of frost-susceptible materials.

b. Penetration of freezing temperatures into the susceptible material.

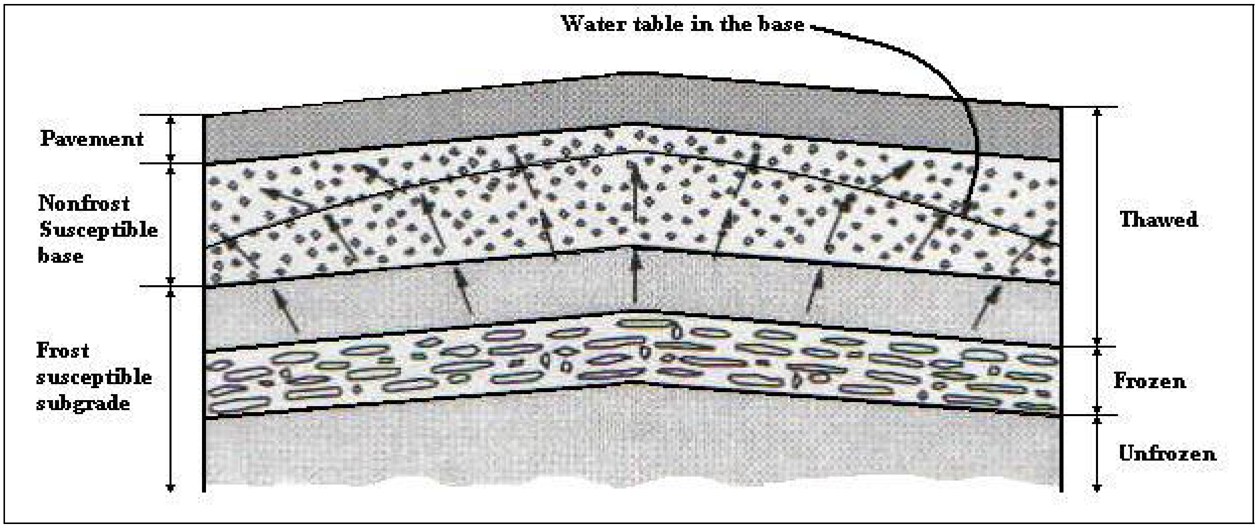
c. Available supply of water.

The potential for significant frost heaving is the greatest when the groundwater table is relatively close to the surface and just below the freezing zone. Surface infiltration and lateral flow are other potential sources of water; however, when freezing starts and a layer of ice develops, the water supply from above will be cut off by the ice layer itself.

7-4.3 **Thawing and Reduction in Bearing Capacity**. During thawing periods, the upper ice lenses melt, releasing water into the base course (see Figure 7-2). If the pavement structure is inadequately drained, or if the drains are blocked with ice, the base course becomes saturated and weakened. Traffic during this period causes large pavement deflections and the development of high pore pressures. The resulting problems are the same as those associated with excess free water in the pavement structure discussed in Section 6-1.5.

7-5 **GENERAL PRELIMINARY DESIGN DATA**. The need for frost protection must be identified during the design stage to enable incorporation of appropriate features into the pavement design. Verification of design assumptions is important to obtain reliable designs. If during construction any of the site conditions were found different than those assumed in the design, the design may have to be modified. Various site-related factors affect the need for frost protection and the need for subsurface drainage.

**Figure 7-2. Upward Movement of Moisture into Base Course During Thaw Period**




7-6 **INVESTIGATION FOR FROST DESIGN**. The key factors that determine the need for frost protection include type and gradation of subgrade, climate, and depth of groundwater table. Frost heaving will occur only if the following three conditions exist:

a. Presence of frost-susceptible material.

b. Penetration of freezing temperatures into the susceptible material.

c. Available supply of water.

The investigation for frost design involves evaluating site conditions for the determination of the presence of these conditions.

7-6.1 **Subsoil Investigations**. Frost action is detrimental if it results in differential heaving, which is caused by variations in subsurface conditions. Variability of subsurface conditions, therefore, is an important consideration for frost design. Subsoil investigation should include assessment of horizontal and vertical variations in subgrade soil type, natural moisture content, and water table elevations. In some situations, variable pavement sections may be needed for different parts of the project to accommodate the differences in subsurface conditions along the project. These conditions must be identified during the subsoil investigation. Consider removing isolated pockets or sections of frost-susceptible soil to eliminate abrupt changes in subgrade conditions.

7-6.2 **Classification of Soils for Frost Susceptibility**. Frost susceptibility of a soil is the potential for the formation of ice lenses in the soil under freezing conditions. Because the water needed for formation and growth of ice lenses is supplied through capillary action, severe frost heave occurs in soils with a high capillary rate. As the freezing temperatures penetrate deeper into the ground, a heavy formation of ice lenses takes place at each successive level, resulting in severe frost heave. All inorganic soils that contain more than 3 percent by weight of particles finer than 0.02 mm in diameter are generally frost-susceptible. Some uniform sandy soils that contain as much as 10 percent finer than 0.02 mm may remain non-susceptible. These sands are usually interbedded with other soils and, in general, cannot be considered separately. Frost-susceptible soils have been classified into four groups (F1, F2, F3, and F4) according to the degree of susceptibility, as shown in Table 6-2. The following are additional comments on the frost susceptibility of various types of soils:

7-6.2.1 **Sands and gravels**. Little or no frost action is likely to occur under normal freezing conditions in sands, gravels, crushed rock, cinders, and similar granular materials when they are clean and free draining. The large voids permit water to freeze in place without segregation into ice lenses.

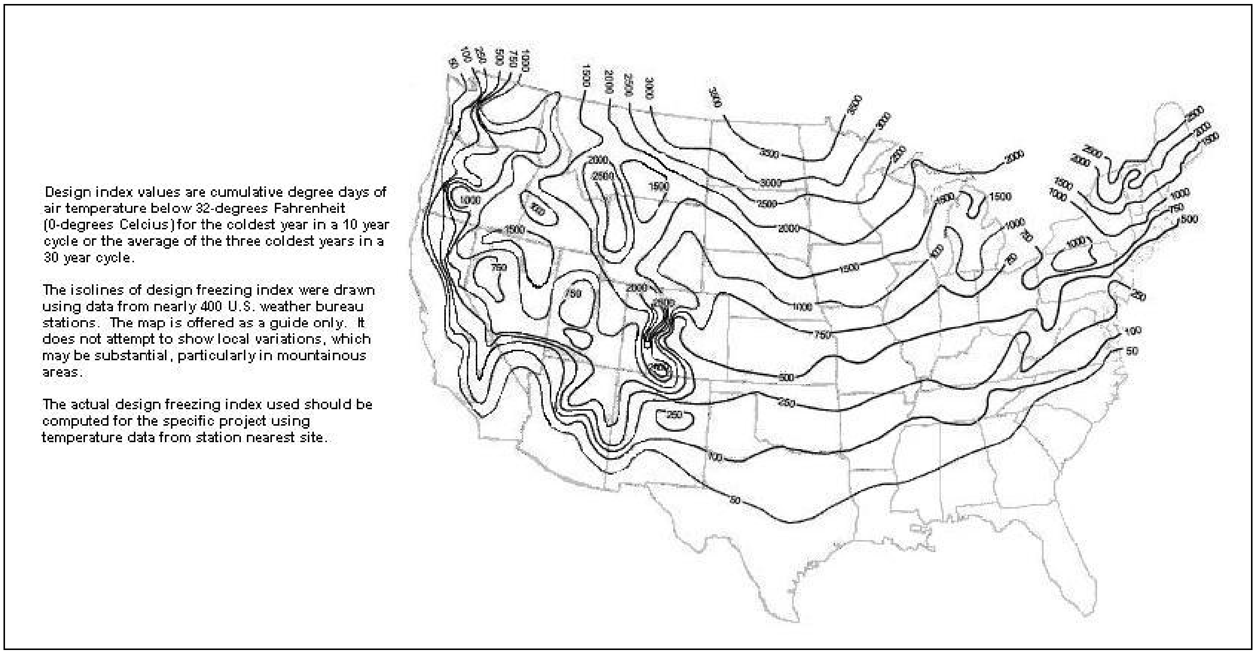
7-6.2.2 **Silts**. Typical silts, such as rock flour, are highly frost-susceptible because of the combination of relatively small voids, high capillary, and relatively good permeability of these soils.

7-6.2.3 **Clays**. Clays are usually cohesive and have high potential capillary, but their capillary rate is low. Frost heaving may occur in clays, but not as severely as in silts because of the impervious nature of the clays, which makes passage of water slow. Although significant heaving does not occur in clays, clayey soils are not necessarily free of the adverse effects of frost action. Moisture introduced into the soil during thaw periods because of melting ice can cause a drastic reduction in stiffness of clayey soils. Thawing usually takes place from the top down, leaving very high moisture content in the upper strata. Upon saturation, the stiffness of clayey soils can drop by a factor of two or more, compared to that under dry conditions.

7-6.2.4 **Varved Clays**. Varved clays consist of alternating layers of medium gray inorganic silt and darker silty clay. The thickness of the layers rarely exceeds 0.5 in. (13 mm). Where subgrade conditions are uniform and there is local evidence that the degree of heave is not exceptional, the varved clay may be assigned to Group F3 for frost susceptibility. Nonuniform varved clays are considered to have very high frost susceptibility.

7-6.3 **Temperature Design Values**. For frost considerations, the design freezing index is the basic value for measuring temperature effects. Freezing index is proportional to the magnitude and duration of subfreezing temperatures during the winter season. For airfield pavement design, the design freezing index is the freezing index for the coldest year in a 10-yr cycle or the average of the three coldest winters in the latest 30 years on record. Figure 7-3 shows design freezing index values for the continental United States. Values for locations not shown in Figure 7-3 should be determined using the terms from Section 7-3 and the procedure illustrated in Figure 7-1.

**Figure 7-3. Distribution of Design Freezing Index Values in the Continental United States**




7-6.4 **Local Frost Data**. Local history of frost heaving may be a strong indication that careful evaluation of site conditions for frost activities is needed. Study all locally available records of maximum and differential frost heaving of airfield and highway pavement in the area. Local public utility companies may be a good source of information for depth of soil freezing.

7-6.5 **Water Source for Ice Formation**. A groundwater level within 1.5 m (5 ft) of the proposed subgrade elevation is an indication that sufficient water is available for ice lens formation, if the subgrade is frost-susceptible. Other conditions that warrant special attention include the following:

a. Homogeneous clay subgrade soils contain sufficient moisture for ice formation, even with the depth to ground water in excess of 3.0 m (10 ft).

b. Unsealed joints and cracks in pavement surface, poorly drained pavements, and shoulder surfaces are common sources of trapped water.

Identification of all potential sources of water for frost activity is an important aspect of site investigations. The pavement design should incorporate appropriate joint details and grades to minimize surface infiltration water.

## 7-7 FROST PROTECTION DESIGN

7-7.1 **Need for Frost Protection**. Differential frost heaving can cause pavement cracking, significant roughness, and a drastic reduction in pavement service life. If prevented from free movement, frost heaving can exert enormous forces on pavements, structures, or utilities. The forces involved are so great that any attempt to accommodate frost heaving by providing a more substantial pavement structure is not practical. The only practical solution is prevention. Even if frost action does not result in significant heaving, the excess free water during thaw periods, and consequent softening of the subgrade and base material, can also be detrimental to pavement performance. If the investigation for frost design reveals that frost action is possible at the project site, frost protection design must be considered. In general, the following combination of conditions denotes a potential for frost action and the need for frost protection:

a. Presence of frost-susceptible soil.

b. Groundwater level within 1.5 m (5 ft) of the proposed subgrade elevation.

c. Frost penetration depth greater than the planned overall thickness of the pavement structure (typically, design freezing index greater than 83.3 degrees C [150 degrees F]).

7-7.2 **Design Approach**. There are two basic approaches to frost protection: (a) complete prevention of subgrade freezing and (b) limiting frost penetration into the subgrade. The first method involves providing a sufficient cover over the frost-susceptible material to prevent penetration of freezing temperatures into the subgrade. This may require removing and replacing a certain thickness of frost-susceptible material or providing a layer of non-susceptible fill, if the combined thickness of the pavement structure and any fills needed for geometric requirements are not sufficient to provide adequate cover. The second approach allows limited frost penetration into the subgrade. The applicability and details of each of these design approaches are discussed in the following.

7-7.3 **Design to Prevent Subgrade Freezing**. In this method, the adverse effects of frost action are eliminated by preventing the freezing temperatures from reaching the frost-susceptible subgrade. This is accomplished by providing a cover of sufficient thickness of nonfrost-susceptible material over the susceptible subgrade.

7-7.3.1 **Criteria for Application**. This is the only acceptable method of frost protection in all areas where freezing of the subgrade beneath the pavement structure is possible, if accompanied by any of the following conditions:

a. Subgrade soil and moisture conditions are extremely variable.

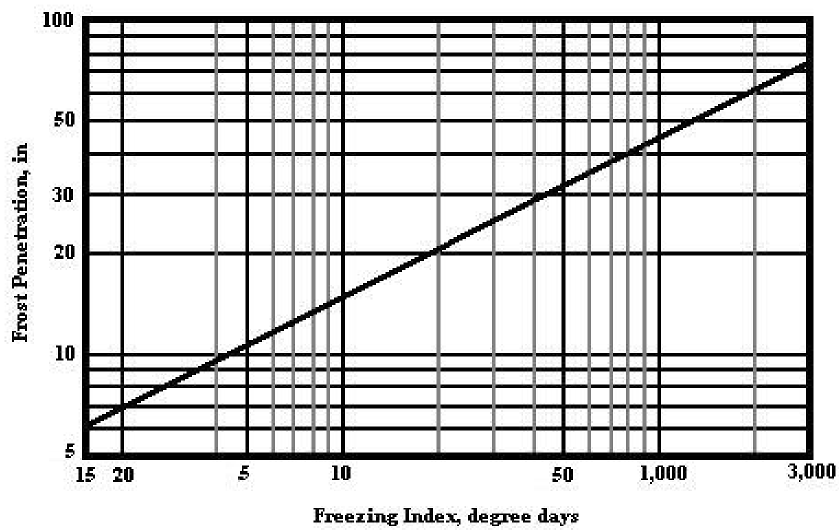
b. The subgrade soil belongs to the frost group F3 or F4.

c. Limited differential heave can present severe operational problems.

7-7.3.2 **Design Procedure**.

7-7.3.2.1 Determine the design freezing index and depth of frost penetration from Figures 7-3 and 7-4, respectively. Adjust these values based on local experience, if reliable information is available.

**Figure 7-4. Empirical Relationship Between Freezing Index and Frost**




1 degree-day F = 0.556 degree-days C
1 in = 25.4 mm

7-7.3.2.2 The frost penetration depth determined in the step above (7-7.3.2.1) is the required overall pavement thickness, which includes asphalt or concrete surface, base, subbase, and any additional nonfrost-susceptible material courses. The additional depth of material required for frost protection must consist of nonfrost-susceptible material. Refer to MIL-HDBK-1021 Series and NAVFAC P-971 to determine the minimum required base and subbase thicknesses.

7-7.4 **Design to Limit Frost Penetration in Subgrade**

7-7.4.1 **Criteria for Application**. Use this method for all but the situations described in 7-7.3.1.a above.

7-7.4.2 **Design Procedure**

7-7.4.2.1 Determine the design freezing index and depth of frost penetration from Figures 7-3 and 7-4, respectively. Adjust these values based on local experience, if reliable information is available.

7-7.4.2.2 From the frost penetration depth determined in 7-7.3.2.1 above, subtract the proposed thickness of asphalt or concrete surface course, and multiply the remaining thickness by 2/3. This value is the thickness of limited frost penetration into the subgrade. Provide the required base, subbase, and any additional fill to equal the thickness of limited frost penetration into the subgrade. The material in each of these courses must be nonfrost susceptible.

# CHAPTER 8

## GUIDELINES FOR DESIGN OF STORM DRAINS IN THE ARCTIC AND SUBARCTIC

8-1 **GENERAL.** Chapter 4 provides general design criteria for drainage and erosion control structures commonly used for airfields and heliports. Certain of the principles used in design are particularly applicable to drainage facilities in arctic and subarctic regions. These and others which are most important for arctic and subarctic drainage are discussed in this chapter. Although this manual is directed primarily to the subject of storm drain design, it is also applicable to design of culverts and open ditches, and the other conventional but important types of drainage structures. The type and capacity of storm drain facilities required to accomplish economically the general objectives outlined in Section 2-2.2 are determined primarily by the promptness with which design storm runoff must be removed to avoid serious interruption of traffic or hazardous conditions on important operational areas, and to prevent serious damage to pavement subgrades. It is presumed that all phases of site reconnaissance have been carefully completed and that information is available that shows topography and natural drainage patterns, groundwater conditions, seasonal frost levels, and permafrost levels, as discussed in TM 5-852-2/AFM 88-19, Chapter 2. Regions not adequately mapped and about which little, if any factual information is available can be evaluated by application of airphoto techniques as described in TM 5-852-9/AFM 88-19, Chapter 8. Even though rainfall is light in arctic and subarctic regions, drainage is an important factor in the selection of an airfield or heliport site and subsequent planning and development. The planner should be cognizant of several features related to drainage to assure a successful design. Some of these are as follows:

8-1.1 Sites should be selected in areas where cuts, or the placement or base course fills, will not intercept or block existing natural drainageways or subsurface drainageways. Adequate provision should be made for the changed drainage conditions.

8-1.2 Areas with fine-grained, frost-susceptible soils should be avoided if possible. In arctic and subarctic regions most soils are of single grain structure with only a very small percentage of clay. Since the cohesive forces between grain particles are very small, the material erodes easily. Fine-grained soil profiles may also contain large amounts of ice lenses and wedges when frozen.

8-1.3 If the upper surface of the permafrost layer is deep, design features of a drainage system can be similar to those used in frost regions of the continental United States if due provisions are made for lower temperatures.

8-1.4 The avoidance, control, and prevention of icing are discussed in Section 2-8.

8-1.5 The flow of water in a drainage channel accelerates the thawing of frozen soil and bedrock. This may cause the surface of the permafrost to dip considerably beneath streams or channels that convey water, and may result in thaw of ice such as that contained in rock fissures and cracks. The latter could develop subsurface drainage channels in bedrock. Bank sloughing and significant changes in channel become prominent. Sloughing is often manifested by wide cracks paralleling the ditches. For this reason, drainage ditches should be located as far as practicable from runway and road shoulders and critical structures.

8-1.6 In many subarctic regions, freezing drainage channels of drifted snow and ice becomes a significant task before breakup each spring. In these areas it is advantageous to have ditch shapes and slopes sufficiently wide and flat to accommodate heavy snow-moving equipment. In other locations where flow continues year-round, narrow deep ditches are preferable to lessen exposed water surface and avoid icing.

8-1.7 Large cut sections should be avoided in planning the drainage layout. Thawed zones or water-bearing strata may be encountered and later cause serious icing. Vegetative cover in permafrost areas should be preserved to the maximum degree practicable; where disturbed, it should be restored as soon as construction permits.

8-1.8 Fine-grained soils immediately above a receding frost zone are very unstable; consequently much sliding and caving is to be expected on unprotected ditch side slopes in such soils.

8-1.9 Locations of ditches over areas where permafrost lies on a steep slope should be avoided if possible. Slides may occur because of thawing and consequent wetting of the soil at the interface between frozen and unfrozen ground.

8-1.10 Provisions should be made for removal and disposal or storage of snow and ice with due consideration to control of snowmelt water. Drainage maintenance facilities should include heavy snow-removal equipment and electric cables with energy sources or a steam boiler with accessories for thawing structures that become clogged with ice. Pipes or cables for this purpose are often fastened inside the upper portions of culverts prior to their placement.

8-1.11 Usually inlets to closed conduits should be sealed before freezeup and opened prior to breakup each spring.

8-2 **GRADING**. Proper grading is a very important factor contributing to the success of any drainage system. The development of grading and drainage plans must be most carefully coordinated. In arctic and subarctic regions, the need for elimination of soft, soggy areas cannot be overemphasized.

8-3 **TEMPORARY STORAGE.** Trunk drains and laterals should have sufficient capacity to accommodate the project design runoff. Supplementary detention ponds upslope from drain inlets should not be considered in drainage designs for airfield or heliports in the Arctic and Subarctic. Plans and schedules should be formulated in sufficient detail to avoid flooding even during the time of actual construction.

8-4 **COMPUTATION OF STORM DRAIN CAPACITIES.** Appendix C includes a design example for drainage facilities to serve a typical portion of an airfield in a *subarctic* region. A separate design example for a typical airfield drainage system in an *arctic* region is not included in this manual as it would follow identical methodology but with two simplifications, as follows: (1) layout would be relatively more austere, usually limited to an aircraft parking apron and a single runway with no parallel taxiway, and (2) as infiltration would be zero, the rate of supply would be the design rainfall rate plus snowmelt. In the subarctic design, the main procedures and steps followed in the determination of storm drain or culvert capacities are given in a step-by-step outline with tables as the design example. It is assumed that the airfield in the Subarctic has a 1-hr rainfall of 0.6 in. plus 0.1 in. runoff from snowmelt, or a total of 0.7 in., a mean annual temperature of about 25 degrees F, the design storm frequency as for most airfields is 2 years, and the infiltration rate for unpaved areas is 0.2 in. per hour. Standard supply curve numbers to be used are therefore 0.7 and 0.5 for paved and turfed areas, respectively. Details are outlined in Appendix C.

8-5 **MATERIALS.** Selection of suitable types of drainage materials for specific projects will be based on design requirements—hydraulic, structural, and durability—and economics for the specific drainage installation. In the Arctic and Subarctic, the flexible thin-walled pipe materials—corrugated metal (galvanized steel or clad aluminum alloy)—have been most widely used for drainage applications because of their availability, weight and transportability considerations, relative ease of installation, and dependability of jointing. Heavier rigid type pipe, reinforced and nonreinforced concrete, particularly with recently developed flexible gasketed joints, and the newer types of plastic pipe are used under certain conditions in the Subarctic.

8-6 **STRUCTURAL DESIGN.** Airfield and heliport culvert and storm drain structural requirements—pipe wall minimum thickness or gages—are usually determined based on minimum amounts of protective earth or pavement cover above the pipe and the maximum aircraft gear loadings to be accommodated. These structural design criteria are given in Chapter 4. Appendix D lists the minimum cover requirements to protect culverts and storm drains in seasonal frost areas from frost heave or from water freezing in the pipe.

8-7 **SERVICE LIFE AND DURABILITY.** These factors will influence drainage material selection. Although the commonly used drainage materials are acceptable in most soil and water environments, there are environmental conditions which limit their service life. Principal among these detrimental factors are corrosion, abrasion, and freezing and thawing action. Protective or periodic maintenance measures to prolong service life where conditions are adverse are difficult, costly, and limited in effectiveness. Often the most practical measure is periodic removal and replacement of damaged or failed drainage components. While this can be readily accomplished under nontraffic shoulders or other less important airfield areas, designs should be based on avoidance of replacement under primary runways, important pavement intersections of high fills. Report "Durability of Drainage Pipe," prepared by the Transportation Research Board, National Research Council, gives guidelines for the selection of durable materials and protective treatments for various adverse environments. The main adverse situations are briefly cited below. This is a complex subject, addressed only in generalities in this manual.

8-7.1 **Corrosion.** Common types of corrugated metal pipe generally corrode when the soil or water is highly acid or alkaline (pH below 5 or above 9) and high electrical conductivity (low soil resistivity) conditions prevail. Mining operations, storage or use of chlorides for snow- and ice-melting, peat or cinder deposits, and salt water in coastal environments are common causes of metal pipe deterioration. Concrete is also vulnerable to acids and certain chemicals (sulfates, chlorides, carbonates) in soils. Plastic, stainless steel or clay pipe or special newly developed protective coatings available for the various pipe materials may be required for use in particularly aggressive environments.

8-7.2 **Abrasion.** This process, more common in culverts than in storm drains, is the wearing down or grinding away of metal, concrete, plastics, clay and other pipe materials and their protective coatings. It occurs when water laden with sand, gravel, stones, ice or other debris flows through, particularly if the flow has a high velocity and if heavy runoff events occur frequently and with long duration. Where severe abrasion is anticipated, extra thickness of pipe materials can be provided, especially along the bottom where wear from bedload movement is concentrated. In some places, abrasive sediment can be removed by providing upstream debris control structures.

8-8 **SHAPE OF DRAINAGE STRUCTURES.** The required hydraulic capacity of a storm drain or culvert can be provided by any of several configurations. While they are usually circular, other factors such as limited headroom, debris accumulation, icing formation, fish passage, fill height, and hydraulic performance may dictate selection of another shape of hydraulically equal capacity—rectangular, oval, arch or multiples. Similarly, options are available in the shape of lined or unlined open drainage channels, ditches or swales with adherence to airfield or heliport lateral safety clearance criteria.

8-9 **MAINTENANCE.** Access for maintenance equipment and personnel is necessary for major drainage channels, debris control barriers and icing control installations. Structures should be periodically inspected, particularly before fall freezeup and after annual spring thaw-breakup periods.

8-10 **JOINTING.** Disjointing, leakage or failure in pipe joints can occur, especially where drainage lines are subject to movement caused by backfill settlement, live loads, or frost action. Flexible watertight joint pipe is available for use in such situations. Most watertight joints rely on use of close tolerance pipe ends connected over a closely fitting gasket.

8-11 **END PROTECTION.** End structures, factory-made or constructed in the field, are attached to the ends of storm drains or culverts to provide structural stability, hold the fill, reduce erosion and improve hydraulic characteristics. A drain projecting beyond the slope of an airfield or roadway embankment is a hazard and subject to damage or failure caused by ice, drift or the current. Drain ends can be mitered to fit embankment slopes or provided with prefabricated flared end sections. Headwalls and wingwalls to contain pipe ends are often constructed, usually of concrete, to meet the several design requirements including provision of weight to offset uplift or buoyancy and to inhibit piping (Section 8-13). Headwalls or wingwalls should be oriented or skewed to fit the drain line for maximum hydraulic efficiency and to lessen icing formation and drift or debris accumulation. The effect of pipeline entrance design on hydraulic efficiency of drainage systems is discussed in Chapter 4. A properly shaped culvert entrance can be an important factor in reducing ponding at an inlet which can wash out an airfield or roadway embankment.

8-12 **ANCHORAGE AND BUOYANCY.** Forces on a drain line inlet during high flows, especially during spring breakup, are variable and unpredictable. Currents and vortexes cause scour which can undermine a drainage structure and erode or fail embankments. These conditions are accentuated in the Arctic and Subarctic by accumulated ice and debris. Corrugated metal pipe sections, being thin-walled and flexible, are particularly vulnerable to entrance distortion or failure. Ends can be protected by providing secure heavy anchorage. This could be a concrete or grouted rock endwall or slope pavement. Rigid type pipe with its shorter sections is subject to disjointing if undermined by scour unless provided with steel tiebars to prevent movement and separation. Buoyant forces must be determined for possible conditions such as blockage of a drainage line end by ice or debris, flow around the outside of a pipe or, in coastal locations, tidal effects. These forces must be counteracted by adequately weighting the line, tying it down, or providing vents. Catastrophic drainage failures have resulted from failure to safeguard against such occurrences, even in temporary situations during construction.

8-13 **PIPING.** Piping is the result of seepage along the exterior of a drain line or culvert which removes backfill material, forming a pipe-like void the full length of the line. Provision of watertight joints (and, if warranted, locked or welded seams in metal pipe) will also reduce exfiltration, a source of seepage flow. The washout of fine-grained soils along the pipeline can ultimately cause its collapse and loss of the overlying embankment. Measures taken to prevent piping include provision of impervious backfill or a large headwall at the upstream end of the line or installation of seepage-preventive metal or concrete bulkheads or collars circling the entire periphery of the drain. The availability of plastic filter cloth which will permit controlled seepage without migration of fine-grained soils provides another useful design expedient to limit piping.

8-14 **DEBRIS AND ICING CONTROL.** It is essential to control debris and icing to achieve desired hydraulic and structural performance and to avoid damages and operational interruption from flooding and uncontrolled icing (see Section 2-8). The debris problem can be solved by providing a structure large enough to pass the material or by retaining it at a convenient adequate storage and removal location upstream from the drainage structure.

8-15 **TIDAL AND FLOOD EFFECTS.** Airfields, with their requirements for large level areas, are often sited on coastal or alluvial floodplains where their drainage systems are subject to tidal and stream flood effects. In arctic and subarctic regions, ice jam and spring break-up dynamic forces and flood heights create major problems, including stream migration, which can adversely affect airfield embankments and protective levees, degrade permafrost, and shift or block drainage outlets. Stream meander control is difficult and costly, especially in the Arctic. Flap gates may be required to prevent backflow into drainage systems, a situation particularly undesirable in tidal or brackish water locations due to corrosive action on drainage pipelines. These gates require a high level of maintenance to assure their operation despite ice, debris, sand or silt accumulation.

8-16 **FISH PASSAGE.** The need to accommodate seasonal fish migration along certain streams should be determined through early coordination with Federal and state fish and wildlife agencies. In some locations fish barriers may be required to prevent migration of undesirable fish species into upstream water bodies. See Section 1-7.11.

8-17 **EROSION CONTROL.** Drainage and erosion control are discussed in Chapter 4. Erosion is important, not just in the design and maintenance of airfields, heliports, and other facilities, but also during construction, when special care must be taken to minimize erosion and siltation from denuded and excavated areas. Temporary siltation basins, check dams, and straw-bale sediment traps should be considered for use in drainage ditches and above drain inlets. Vegetative cover should be reestablished as soon as practicable.

8-18 **INSTALLATION.** Pipe construction in the Arctic and Subarctic, as in other regions, requires shaped bedding and systematic, layer-by-layer backfilling and compaction, and maintaining equal heights of fill along both sides of the pipe. Many culvert and storm drain failures during construction are caused by operating equipment too close to pipe, or failure to remove large projecting stones from backfill near the pipe, or inadequate caution in handling frozen backfill material.

8-19 **SAFETY REQUIREMENTS.** Fuel spillage must not drain into storm sewers or other underground conduits. Safe disposal of spilled fuel can be facilitated by providing ponding areas for drainage so that any spilled fuel can be removed from the surface. Curbs, gutters, and storm drains will not be provided for drainage around tank-car or tank-truck loading or unloading areas, or tanks in bulk fuel storage areas.

# APPENDIX A

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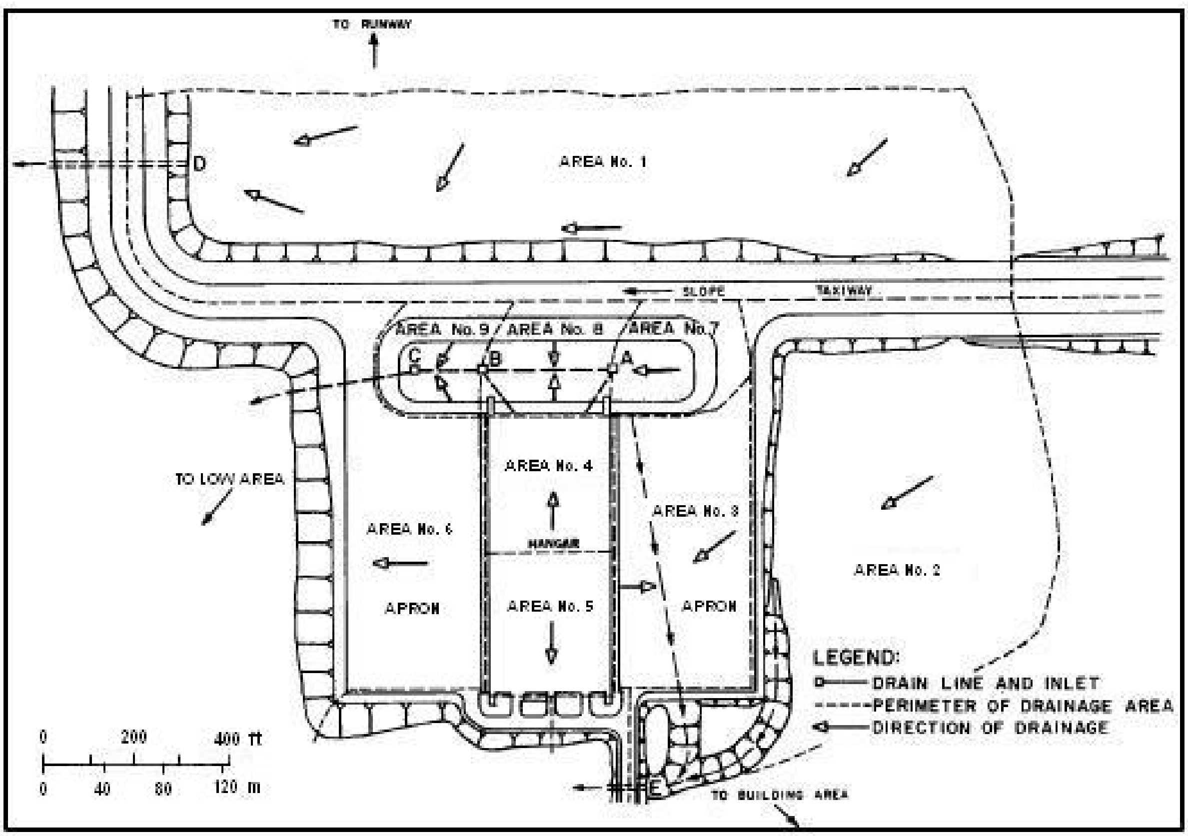
# APPENDIX C

# DESIGN EXAMPLES

## C-1 ARCTIC AND SUBARCTIC DRAINAGE

C-1.1 **Preliminary Layout**. Prepare a map (scale 1 in. = 200 ft or larger) showing the outline of runways, taxiways, parking aprons, paved shoulders, facility areas, and roads. Superimpose on this network 1-ft-interval contours that will show the finished airfield or heliport. Insure that grades conform with current safety criteria as set forth in TM 5-803-4 for Army facilities of AFM 88-6, Chapter 1 for Air Force facilities unless waiver approvals are secured. If the airfield is also to be used for civil aviation, coordinate the site selection with the District Airport Engineer of the Federal Aviation Administration and the state aviation agency. Indicate locations of test pits, soil borings, and probings, and designate significant items clearly.

**Figure C-1. Drainage Problem: Airfield in Subarctic Region-Hangar, Taxiway and Apron**




By Corps of Engineers

C-1.2 **Profiles**. Profiles of all runways, taxiways, helipads and parking areas, so that elevations and controlling grades can be ascertained for any point.

C-1.3 **Drain Outlets**. With general consideration of the limiting grade elevations and feasible channels for the disposal of storm runoff and snow melt, select locations that are considered most suitable for outlets of drains serving various portions of the field. With this information, select a tentative layout for primary storm drains. In general, the most economical and efficient design is obtained by maximum use of open ditches in preference to underground drains and by maintaining the steepest hydraulic gradient feasible in the main trunk drain, while making laterals on each side approximately equal in length, insofar as practicable.

C-1.4 **Cross-sectional Profiles of Intermediate Areas**. Assume lines for cross-sectional profiles of intermediate areas, plot data showing controlling elevation, and indicate the tentatively selected locations for inlets by means of vertical lines. In some cases, the projection of runways, taxiways, helipads, or aprons should be shown on the profiles, to facilitate a comparison of elevations of intermediate areas with those of paved areas. Generally, one cross-sectional profile should follow each line of the underground storm drain system and others should pass through each of the inlets at approximately right angles to paved runways, taxiways, helipads or aprons.

C-1.5 **Correlation of Controlling Elevations and Limiting Grades**. Beginning at points corresponding to controlling elevations, such as the crown or edges of a runway, sketch in the ground profile from the given points to the respective drain inlets, making the grades conform to limiting slopes for the areas involved. Review the tentative grading and inlet elevations and adjust the locations of drain inlets and grading details as necessary to obtain the most satisfactory general plan.

C-1.6 **Determination of Drainage Area**. Using the completed grading plan, sketch the boundaries of drainage areas tributary to the respective drain inlets and compute the area of paved and unpaved areas tributary to the respective inlets.

C-1.7 **Ponding Basins**. Avoid the use of ponding basins in arctic and subarctic areas.

C-1.8 **Average Retardance Coefficient**. Assign values of n to various turfed, bare, frozen ground, or paved subareas as explained in Section 2-7, and compute average roughness factors for overland and channel flow. See columns 6 and 20, and note 2 in Table C-1.

C-1.9 **Average Slope**. Estimate the average slope of overland and channel flow conditions for each inlet drainage area using the data indicated on the grading plan.

C-1.10 **Effective Length**. From the grading plan determine the effective length of flow, giving due consideration to the occurrence of overland and channelized flow. By use of Figure 2-5, convert the measured lengths of flow to equivalent lengths of flow in 10-ft increments which correspond to S = 1.0 percent and n = 0.40. For actual lengths

Table C-1

Table C-2

exceeding 600 ft, divide by any convenient factor and determine corrected length therefore, then multiply by this factor to find the corrected length for the full distance. For example, if actual length is 700 ft, determine corrected length for 350 ft and multiply by 2. See also columns 8-10 of Table C-1.

C-1.11 **Project Design Storm**. By use of Figure 2-1 and the known geographic location of the airfield or heliport, select a project design storm of the specified frequency of occurrence.

C-1.12 **Snowmelt**. Add an amount of 0.05 to 0.1 in 1 hour for snowmelt to the project design storm (see C-1.11 above).

C-1.13 **Infiltration**. If the airfield or heliport site is located in the Arctic, assume that the infiltration rate is zero. If in the Subarctic, determine average infiltration rates from local studies but not higher than 0.3 in./hr.

C-1.14 **Standard Supply Curves**. Standard supply curves for areas with zero infiltration loss will be the same as the standard rainfall plus snowmelt curves (Figure 2-3). Where infiltration losses occur, the standard supply curve number corresponding to a given standard rainfall plus snowmelt curve number is computed by subtracting the estimated 1-hour infiltration value from the 1-hour rainfall plus snowmelt quantity. See columns 11-14 of Table C-1.

C-1.15 **Weighted Standard Supply Curve**. Determine a weighted standard supply curve for the composite drainage area proportional to the standard supply curves for the various subareas. See column 15 of Table C-1.

C-1.16 **Determination of Drain-Inlet Capacities**. With reference to Figures 2-7 through 2-12, select the two graphs with supply curve numbers closest to the weighted standard supply curve determined above. The following procedure is carried through on both graphs and interpolated for the weighted standard supply curve. The critical duration of supply $t_c$ (col. 16, Table C-1) and the maximum rate of runoff $q_d$ (col. 17) for the individual inlet drainage area can be read directly from the graph for the given value of effective length. Value of $t_c$ should not be less than the minimum values of 10 minutes for paved for bare areas and 20 minutes for turfed areas (Section 2-7). In order for the maximum rate of flow to be attained at a given point in a drainage system during a supply of uniform intensity, the storm must last long enough to produce a maximum rate of inflow into each upstream drain inlet and to permit the inflow to travel through the drain from the "critical inlet" to the given point. The duration of supply necessary for this purpose is referred to herein as $t'_c$ and is given approximately by the equation

$$t'_c - t_c + t_d \qquad \text{(eq. C-1)}$$

in which $t_c$ is the duration of supply that would provide the maximum design storm runoff from the area tributary to the critical drain inlet and $t_d$ is the time required for water to flow from the critical drain inlet to the point under consideration. The critical drain inlet to the point under consideration. The critical inlet can normally be assumed to be the inlet located the greatest distance upstream from the given point. To simplify the determination of drain-inlet capacities, the computed values of $t'_c$ can be rounded off to the nearest 5 minutes as shown in column 19 of Table C-1. The procedure for computing values of $t'_c$ is described in Chapter 2. Inspection of Figures 2-7 through 2-12 will show that for large values of effective length and low values of supply curve, the maximum rate of runoff is approximately constant after a duration of supply equal to $t_c$. Under these conditions, it will facilitate the design computations to use the constant value $q_d$ for $t_c$ duration of supply for all durations of supply in excess of $t_c$.

C-1.17 **Computation of Pipe Sizes and Cover**. The size and gradient of storm drain required to discharge storm runoff may be determined by using Mannings' formula or the charts provided in Chapter 3. In any case, calculated capacities should be liberal to provide a safety factor against high flows during spring thaw and possible clogging due to icing (Section 2-8). It is recommended that minimum pipe diameter be at least 18 in. and preferably larger, even where the calculated runoff may require a smaller size. In selecting proposed inlet elevations and slope of pipelines, minimum cover required for the various pipe materials and strengths should be in accord with Chapter 4. At each site, prior to design, the suitability of embedment depths should be confirmed by field investigations.

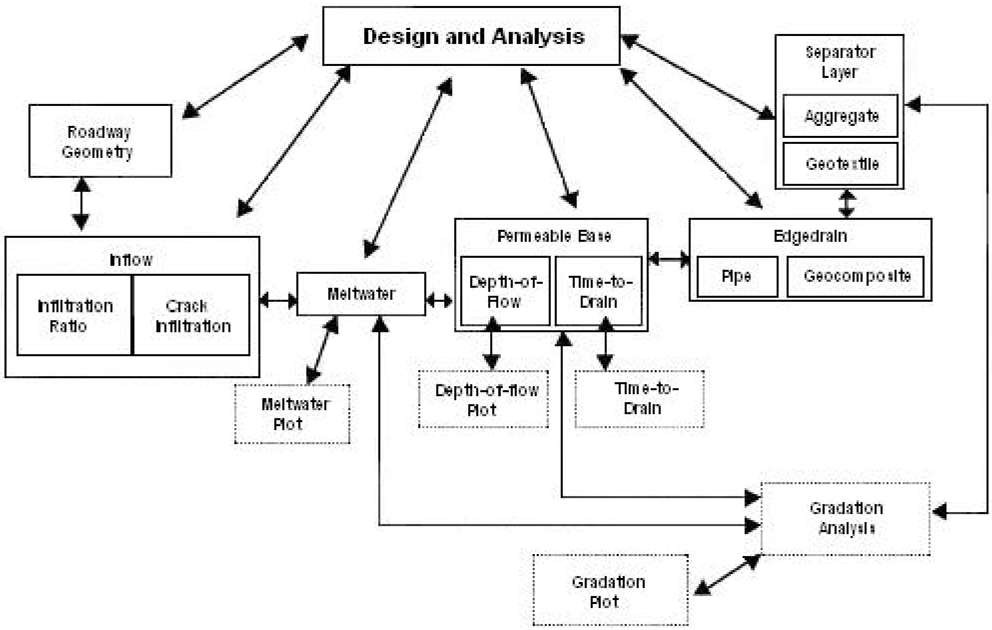
C-1.18 **Determination of Ditch Sizes**. The ditch should be large enough to accommodate the storm runoff with liberal allowances for blockage or flow retardation due to formation of icing or accumulation of debris. The shape of ditches depends on airfield or heliport lateral clearance safety criteria, snow removal and storage practices, susceptibility to icing, erosion and debris control, and local environmental conditions.

## C-2 SUBSURFACE DRAINAGE DESIGN USING DRIP

C-2.1 **Introduction**. The microcomputer program *Drainage Requirements in Pavements (DRIP)*, developed under an FHWA contract (Wyatt et al. 1998a), is designed to assist engineers in designing subsurface drainage systems for highway pavements. The modular framework of DRIP is illustrated in Figure C-2. Each of these modules can be accessed either individually to perform a specific design task or sequentially as part of an overall design process. The Design and Analysis node is central to the program and controls the flow of information between modules. Not all of the modules presented in Figure C-2 is required to perform the design of the drainage systems recommended in this manual. Therefore, only the relevant modules and their design windows are presented in this example.

C-2.2 **System Requirements**. DRIP was developed to run under Windows 3.1. The program has been fully tested and verified to run error-free under Windows 95 and NT. Other than the Windows operating system, DRIP does not have any special requirements. However, a 16-color display with small fonts and at least 800×600 resolution is recommended because of the graphical nature of the program.

**Figure C-2. Modular Framework of the DRIP Program**




C-2.3 **Getting Started**. The opening screen of DRIP is shown in Figure C-3. From this screen you can either start a DRIP session by clicking on the *Begin* button or quit the program by clicking on the *Close* button.

C-2.4 **Design and Analysis Window**. The *Design and Analysis* window is shown in Figure C-4. This window is the central node of the program. The items listed on the left side of the window—*Roadway Geometry, Inflow, Permeable Base, Separator,* and *Edgedrain*—each correspond to a specific design module. The DRIP design modules may be accessed either by clicking on the respective icons or using the *Go To* list box. Prior to accessing the design modules, however, you need to suitably configure the design options by clicking on the check boxes located on the left side of the window.

C-2.4.1 **Permeable base**: Select *Time-to-Drain Method* for the design of permeable base. This is the analysis method used in the guide.

C-2.4.2 **Separator**: Check *Use Separator Layer* to evaluate separator layer materials.

C-2.4.3 **Edgedrain**: Select *Pipe* edgedrain. For airfield applications, the guide recommends pipe edgedrains.

**Figure C-3. The Opening Screen of DRIP**

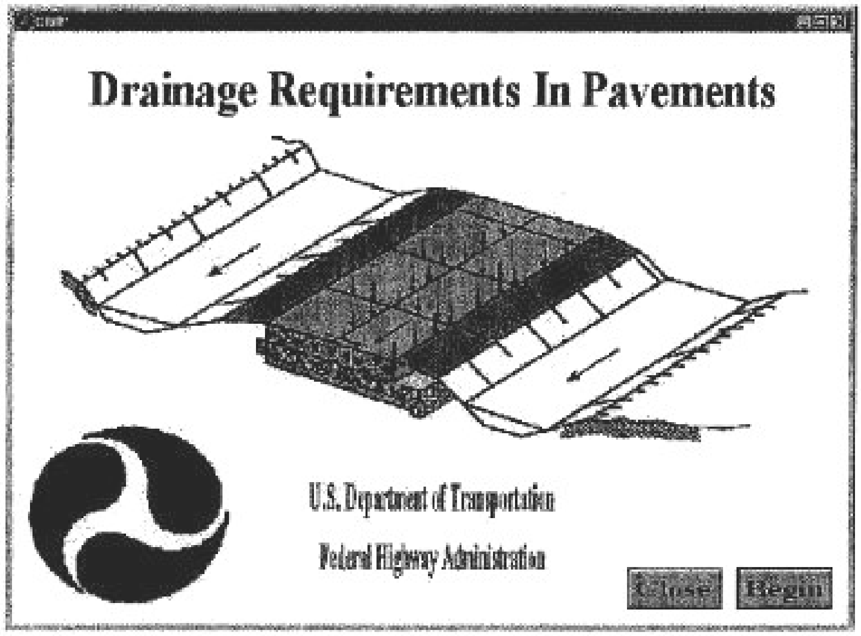
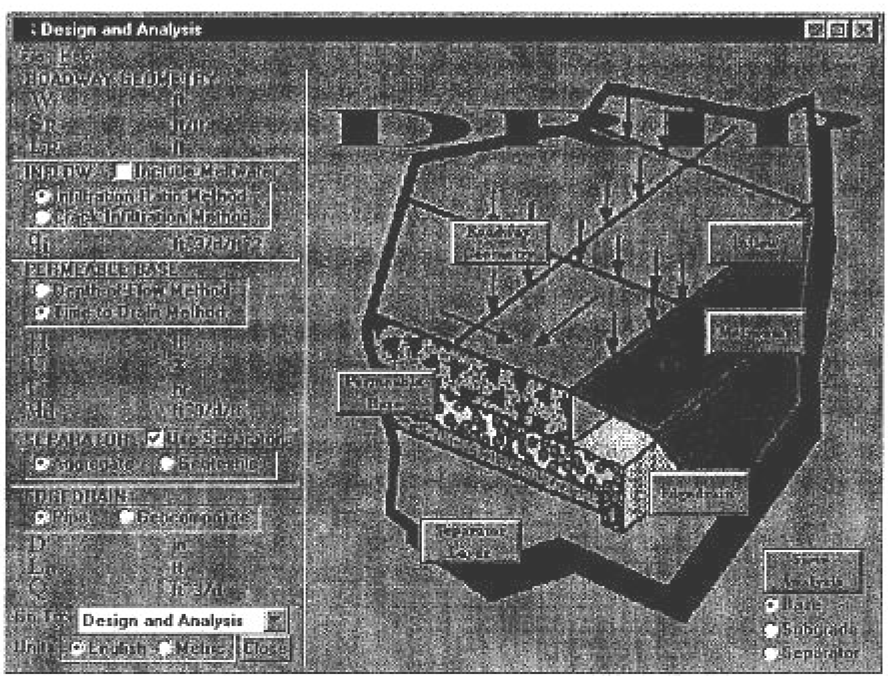



**Figure C-4. The Design and Analysis Window**



C-2.4.4 **Units**: Select the desired unit system. You have the option to set the unit system for each module, but the unit system selected on the *Design and Analysis* window will be the default.

C-2.5 **Drip Modules**. In this section, the DRIP modules that are relevant to hydraulic design of airfield pavements are explained in detail. Example problems are included to demonstrate the usage of DRIP. DRIP uses the following general convention:

C-2.5.1 When several design modules are executed under the same DRIP session, relevant data are automatically shared between modules.

C-2.5.2 Any window can be closed using the *Close* button at the bottom of the window or by selecting *Exit* from the File menu.

C-2.5.3 Every design window displays a number of inputs and outputs. Also displayed are the equations that related the inputs to the respective outputs. Once all the input data values are for a given equation are entered, a calculator icon next to the output is activated, indicating that the particular output is ready to be computed. Click on the calculator icon to process the input data.

C-2.5.4 If any of the DRIP-calculated fields are entered manually, DRIP issues a warning message. For example, the resultant slope and drainage path is needed for time-to-drain calculation in the *Permeable Base* module. DRIP includes *Roadway Geometry* module for calculating these values. Therefore, DRIP will issue a warning message if these values are entered manually.

C-2.5.1 **Sequence of operation**. DRIP is modular and the sequence of execution of the modules need not follow any particular order. However, the following sequence is recommended:

C-2.5.1.1 **Roadway geometry**: Use the module to determine the resultant slope and drainage path. To access *Roadway Geometry* module click on the *Roadway Geometry* button or select *Roadway Geometry* from the *Go To* drop-down menu.

C-2.5.1.2 **Sieve analysis**: This module is used to calculate the gradation parameters required in various modules. To access this module, click on the *Sieve Analysis* button or select *Sieve Analysis* from the *Go To* drop-down menu.

C-2.5.1.3 **Permeable base**: Perform hydraulic design of permeable base using the time-to-drain method. Choose *Time-to-Drain Method* of analysis under *Permeable Base*, and click on the *Permeable Base* button on the *Design and Analysis* window to access this module. This window requires inputs from the *Sieve Analysis* module for permeable base gradation.

C-2.5.1.4 **Edgedrain**: Perform pipe edgedrain design using the *Edgedrain* module.

C-2-5.1.5 **Separator layer**: Use this module to perform separator layer design. There are two selections for separator layers. Based on the project requirements, the appropriate layer type must be chosen. This module also requires inputs from the *Sieve Analysis* module for subgrade and separator layer gradations (in the case of aggregate separators).

As the design progresses from one step to another, the inputs and outputs of a given module are made available to all modules that are subsequently invoked. However, if a step is inadvertently missed, you need to go back to the module in question and perform the necessary calculations.

C-2.5.2 **Roadway geometry calculations**: The resultant slope, $S_R$, and the resultant length, $L_R$, of the flowpath are needed for time-to-drain calculations. The resultant slope is the resultant of the longitudinal slope, S, and cross-slope, $S_x$, of the pavement; the resultant length is the distance over which water flows within the pavement structure in the direction of the resultant slope. These quantities can be computed using the *Roadway Geometry* module in DRIP.

C-2.5.2.1 **Roadway geometry inputs**

a. Roadway cross-section (crowned or superelevated).

b. Lane and shoulder widths.

c. Longitudinal grade of roadway (S).

d. Cross-slope of roadway ($S_x$).

C-2.5.2.2 **Roadway geometry outputs**

a. Resultant slope ($S_R$).

b. Resultant drainage path ($L_R$).

<u>Example C-2A: Roadway Geometry Design</u>

Determine the resultant slope, $S_R$, and the resultant length, $L_R$, for the following crowned runway section:

| | |
|---|---|
| Cross-slope, $S_x$: | 0.015 ft/ft |
| Longitudinal slope, S: | 0.0015 ft/ft |
| Pavement width: | 150 ft |
| Shoulder width: | 0 ft |

*Solution*

1. Click on *Roadway Geometry* button from the *Design and Analysis* window to access *Roadway Geometry* module.

2. Enter the lane width, b, and the shoulder width, c. The shoulder width, c, is the distance from the pavement edge to the edgedrain. Typically, edgedrain is located at least or 1 or 2 ft away from the pavement edge. Assume c = 2 ft.

3. Choose *Geometry A*.

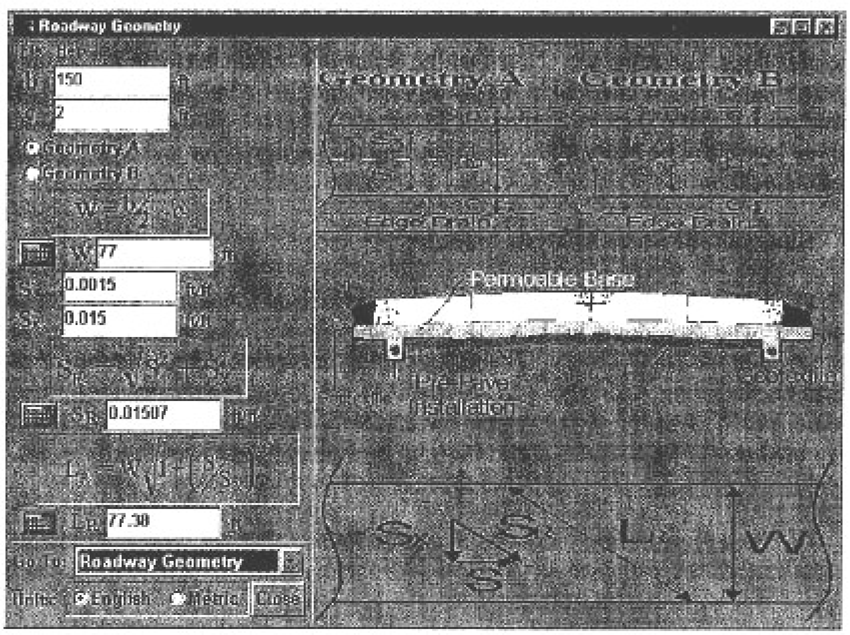
4. The calculator icon next to "W" should now turn blue. Click on the calculator icon to compute the width of the drainage path, "W."

5. Enter values of the slopes S and $S_x$.

6. The calculator icons next to the quantities $S_R$ and $L_R$ should now turn blue, indicating that the solutions are ready to be computed. Compute $L_R$ and $S_R$ by clicking on the respective icons.

Figure C-5 shows the *Roadway Geometry* design window with the inputs and outputs for this example. The resultant slope is 0.01507 ft/ft, and the drainage path is 77.38 ft.

**Figure C-5. Roadway Geometry Design Window**




C-2.5.3 **Sieve analysis**. The *Sieve Analysis* module is used to determine gradation parameters for base, separator layer, and subgrade. Three selection buttons are provided under the *Sieve Analysis* button on the *Design and Analysis* window for the selection of the analysis for base, separator layer, and subgrade. Note that the *Separator* button becomes active only if the *Use Separator* check box is checked in the *Design and Analysis* window. The VASDAM (Visual Analysis of Sieve Data for Aggregate Materials) program window corresponding to each of these three layers can be accessed by first selecting the desired layer and then clicking on the *Sieve Analysis* button.

C-2.5.3.1 **Input to the sieve analysis module**

a. Material Name: The name supplied here is used to identify the gradation data being analyzed. The drop-down list box attached to this input can be used to retrieve any gradations saved in the DRIP library. The default DRIP library includes a number of permeable base gradations, including AASHTO # 57, AASHTO # 67, Iowa, Minnesota, New Jersey, Pennsylvania, and Wisconsin. You can save the gradation data that you entered from a DRIP session by clicking on *File* from the *Sieve Analysis* module and then selecting *Save As*. To retrieve previously saved gradation data, click on *File*, then select *Open*.

b. Sieve Data: Select either the *Range* or *Value* selection button. When the *Range* is specified, the gradation parameters are computed for the midpoint of the gradation band.

c. Sieve Number: A sieve size can be entered with the help of the drop-down menu attached to this input. The drop-down menu is activated by clicking on the *Sieve Number* input field. Click on the desired sieve to make the selection.

d. %-Passing: A numeric value indicating the percent of material passing the current sieve number. Enter the appropriate values and click on *Add to Table* button to add the information to the table. To modify the previously entered %-Passing data, select the row to be modified, enter the appropriate values, and click on *Add to Table* button to update the table.

e. Unit Wt: Laboratory determined unit weight of the base material. Guidance for determining unit weight can be accessed by clicking on the *?* button located to the left of this input.

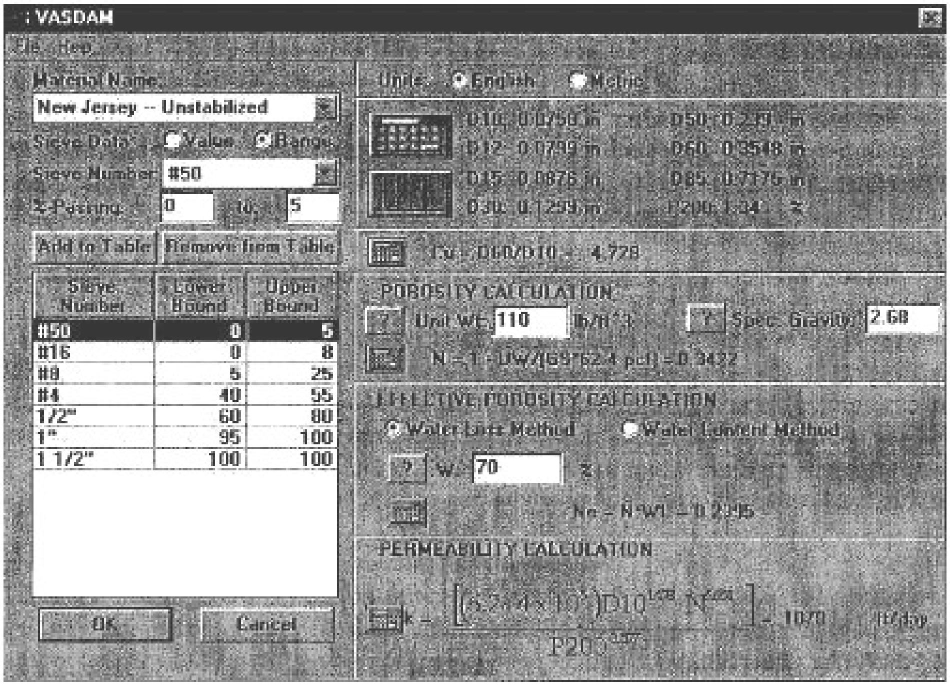
f. Spec. Gravity: Laboratory-determined specific gravity of the base material. Guidance for determining specific gravity can be accessed by clicking the *?* button located to the left of this input.

g. Effective Porosity Calculation: Effective porosity can be calculated using either the *Water Loss Method* or the *Water Content Method*. Select the desired method by clicking on the appropriate selection button.

h. W: The water loss coefficient, W. DRIP provides a table of recommended water loss values based on the type and amount of fines (material passing No. 200 Sieve (0.075-mm) material) present in the material. This table is accessed by clicking on the *?* button located next to the symbol *W*.

The sieve analysis window for permeable bases is shown in Figure C-6. As with other DRIP modules, the calculator icon becomes enabled as the required data are provided. Click on the calculator icon to perform the required calculation.

**Figure C-6. Sieve Analysis Window for Permeable Bases**




C-2.5.3.2 **Outputs of the sieve analysis module**. The sieve analysis module provides the following output:

a. $D_{10}$, $D_{12}$, $D_{15}$, $D_{30}$, $D_{50}$, $D_{60}$, and $D_{85}$. These values are needed for checking filter criteria for the separator layer.

b. $P_{200}$ (percent passing the 0.075-mm sieve).

c. Coefficient of uniformity, $C_U$.

d. Porosity, N.

e. Effective porosity, $N_e$.

f. Permeability, k. The permeability estimated in this module is based on empirical correlation for fine-grained soils. The permeability of aggregate materials can deviate significantly from this value. Therefore, this value is not recommended for use; a laboratory-estimated value should be used.

C-2.5.4 **Permeable base design**. The *Permeable Base* module can be accessed from the *Design and Analysis* window by clicking the *Permeable Base* button. Ensure that *Time-to-Drain Method* is selected under *Permeable Base on the Design and Analysis* window before entering this module. The design inputs and outputs for this module are as follows:

C-2.5.4.1 **Inputs for permeable base designs based on the time-to-drain method**.

a. $n_e$: The effective porosity of the base material. The effective porosity can be determined using the *Sieve Analysis* module. If you completed the sieve analysis using DRIP, the value determined from the sieve analysis module should already be shown on the time-to-drain analysis window. Clicking on the calculator icon next to the edit box for $n_e$ will take you to the *Sieve Analysis* module where $n_e$ for the selected gradation can be calculated. Alternatively, $n_e$ determined from laboratory testing can be entered manually.

b. k: The coefficient of permeability of the base material. The value determined by laboratory testing should be used, although the *Sieve Analysis* module can also be used to determine a rough estimate. As with $n_e$, clicking on the calculator icon next to the edit box for *k* will take you to the *Sieve Analysis* module for estimating *k* using the formula shown on that window.

d. $S_R$: The resultant slope of the permeable base. This parameter is an output of the *Roadway Geometry* module and automatically appears on this window if that module was previously executed. Otherwise, $S_R$ can be entered manually.

e. $L_R$: The resultant length of the drainage path. This parameter is also an output of the *Roadway Geometry* module and automatically appears on this window if that module was previously executed in the same DRIP session. Otherwise, $L_R$ can be entered manually.

f. H: Thickness of the permeable base. A fixed value of 6 in. (150 mm) is recommended for airfield pavements.

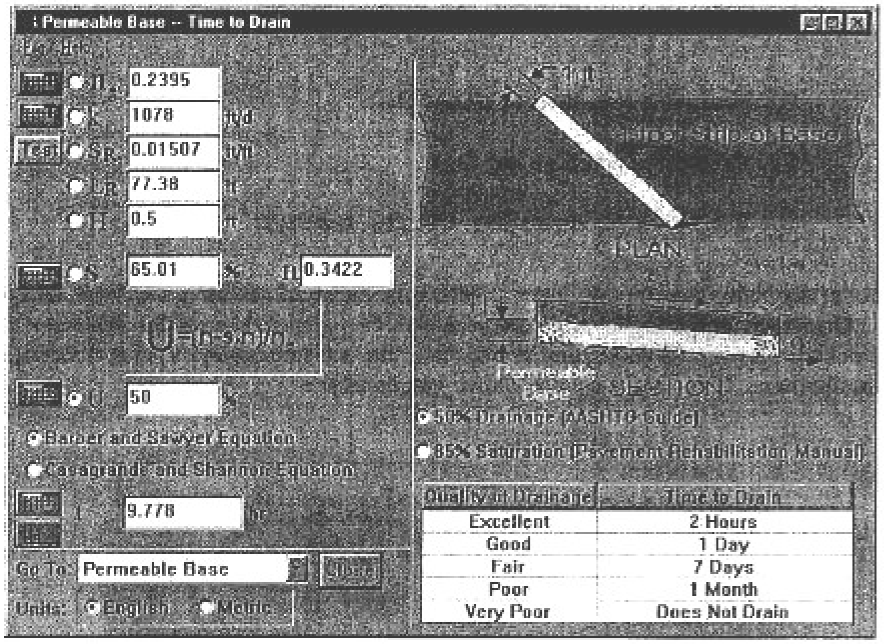
g. Either the target percent saturation, S, or percent drained, U is needed to determine time-to-drain. The drainage criteria used in DM 21.06 is based on the time to 50 percent drainage (i.e., U = 50). The relationship between S and U are shown on *Permeable Base — Time to Drain* window. Once either S or U is entered, the other value can be determined by clicking on the calculator icon next to the input parameter.

C-2.5.4.2 **Outputs of the time-to-drain method for permeable base design**

a. The time required to drain the base to the target percent saturation or percent drained.

b. The drainage history plot. A plot of the percent-drained or percent-saturation of the base with time can be viewed by clicking on the plot icon located immediately below the calculator icon for the time-to-drain calculation (see Figure C-7).

**Figure C-7. Time-to-Drain Design Window**




Located on the lower right of the *Permeable Base — Time to Drain* window is the quality of drainage assessment table for highway pavements. Note that the time-to-drain requirements for airfield pavements, as specified in this handbook, are less stringent than those for highways. See Table C-3 for the assessment of the quality of drainage for airfield pavements.

**Table C-3. Quality of Drainage Rating for Highways and Airfield Pavements**

| Quality of Drainage | Time to Drain | |
|---|---|---|
| | Highways | Airfields |
| Excellent | 2 hr | 1 day |
| Good | 1 day | 7 days |
| Fair | 7 days | 15 days |
| Poor | 30 days | 30 days |

Example C-2B: Time-to-Drain Determination and Permeable Base Design

Determine the time required for 50 percent drainage for the pavement section given in Example C-2A. The permeable base should satisfy the requirements for an *Excellent* quality of drainage as defined in Table C-3 (50 percent drainage in 12 hours or less ). New Jersey permeable base gradation with a laboratory coefficient of permeability (*k*) of 1,000 ft/day is proposed as the base material. Assume a unit weight of 110 pcf, specific gravity of 2.68, and a water loss coefficient of 70 percent. Assume a permeable base thickness of 6 in.

*Solution*

1. Click on *Permeable Base* button from *Design and Analysis* window to access *Permeable Base* module. Be sure that the *Time-to-Drain Method* is selected under *Permeable Base* on the *Design and Analysis* window. If you completed Example C-2A, the *Permeable Base—Time-to-Drain* window should already display the values of the resultant slope ($S_R$) and resultant length ($L_R$) calculated from the *Roadway Geometry* window.

2. Click on the calculator icon next to the $n_e$ input box. This opens the VASDAM window (Figure C-6). From the *Material Name* drop-down box, select "New Jersey—Unstabilized." The gradation for this parameter appears and the $D_x$ calculator icon is activated. Click on this icon to compute $D_x$. Enter the given unit weight, specific gravity, and water loss coefficient in the respective boxes of the VASDAM window. Click on appropriate calculator buttons to calculate the coefficient of uniformity ($C_u$), porosity ($N$), and effective porosity ($N_e$). Click the *OK* button to close the VASDAM window and return to the *Permeable Base — Time-to-Drain* window.

3. Enter the base permeability (k) and base thickness (0.5 ft).

4. Enter the target percentage drained value, U(%) = 50 percent. Click on the calculator icon next to percent saturation, *S*, to see what degree of saturation 50 percent drainage represents.

5. Click on the calculator icon next to *t* (time-to-drain) to determine the time required to drain 50 percent of the drainable water. The plot icon below *t* should also become active when all inputs are entered. Click on this button to view the drainage history plot.

6. Check to see if the chosen gradation meets the design standard.

Figure C-7 shows the DRIP window with all inputs and outputs for this example. The calculated time-to-drain for this example is 9.778 hours. Therefore, the selected permeable base material meets the design standard.

C-2.5.5 **Separator layer design**. The DRIP *Separator Layer* module performs the automated checking of the filter criteria for aggregate and geotextile separator layers. However, the filter criteria for geotextile separator layer incorporated in DRIP is slightly different than the recommendations given in this manual. Therefore, DRIP should be used for checking the filter criteria for aggregate separator layer only.

C-2.5.5.1 **Aggregate separator layer design**. The DRIP window for aggregate separator layer design is shown in Figure C-8. The criteria that need to be satisfied for the design are listed on the right side of the window. The inputs required to compute these criteria are listed to the left of the window.

a. Inputs for Aggregate Separator Layer Design

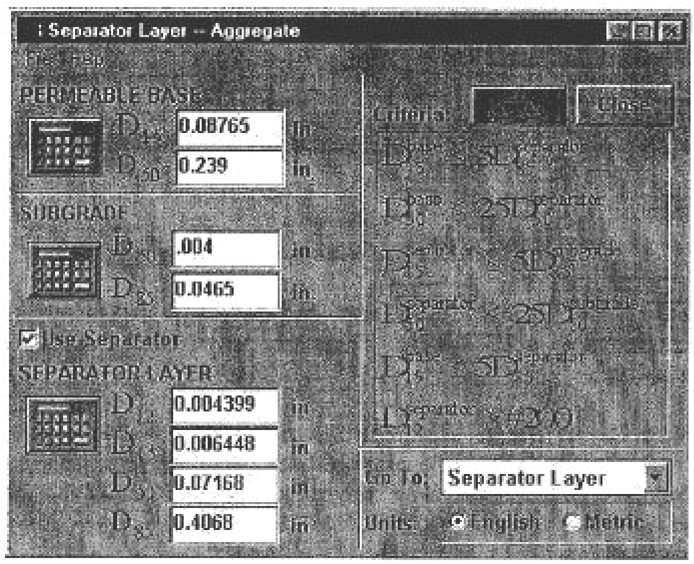
1. Permeable base inputs ($D_{15}$ and $D_{50}$).

2. Subgrade inputs ($D_{50}$ and $D_{85}$).

3. Separator layer inputs ($D_{12}$, $D_{15}$, $D_{50}$, and $D_{85}$).

Click on the calculator icon for each layer to determine these values using the *Sieve Analysis* module. Once the required input values are provided, the balance icon on the *Separator Layer* window becomes active. Click on this icon to see if the selected separator layer material satisfies the required criteria. The results are also shown graphically.

**Figure C-8. DRIP Window for Aggregate Separator Layer Design**




C-2.5.6 **Edgedrain design**. Pipe edge drains are recommended for use in this handbook. Ensure that *Pipe* radio button is selected under *Edgegrain* on *Design and Analysis* window and click on the *Edgedrain* button to access the *Pipe Edgedrain* window.

Pipe edgedrain design is a two-step process involving the calculation of the pipe capacity, Q, and the outlet spacing, $L_o$. The output of the first step is an input to the second. Three different options are available for determining the pavement discharge rate: *Pavement Infiltration, Permeable Base*, and *Time-to-Drain*. As explained in this handbook, the permeable base discharge option provides the maximum possible discharge from the base layer, but if the base material is extremely highly permeable, the results may be overly conservative. For very highly permeable base, the *Time-to-Drain* method should be used, with the time-to-drain manually entered to achieve the desired quality of drainage (e.g., enter 12 hr for *Excellent* or 168 hr for *Good* drainage). The inputs and outputs for this module are as follows:

C-2.5.6.1 **Input**. The pipe edgedrain design inputs are the following:

Longtudinal grade, S
Pipe diameter, D
Manning's roughness coefficient (= 0.012 for smooth pipes or 0.024 for rough pipes)

For permeable base discharge calculation, the following are required:

Base thickness, H
Transverse slope, $S_T$
Base permeability, *k*

For time-to-drain discharge calculation, the following are required:

Base thickness, H
Base width, W
Time-to-drain
Effective porosity, $n_e$
Percent drained, U (50 percent)

If the *Roadway Geometry* module was used to determine resultant slope and drainage path, the values from that module will automatically be copied to the appropriate input boxes in this module. Similarly, if *Sieve Analysis* module was used to determine gradation parameters, the effective porosity calculated from that module will be automatically imported to this module.

<u>Example C-2C. Pipe Edgedrain Design</u>

Design a pipe edgedrain for the permeable base in Example C-2B. Assume corrugated pipe drain with 6-in. diameter.

*Solution*

1. From the *Design and Analysis* window, ensure that the *Pipe* radio button is selected and click on the *Edgedrain* button to open the *Pipe Edgedrain* window.

2. Enter the values for the longitudinal slope, S, and the pipe diameter, D. Click the *Corrugated Pipe* checkbox to enter the appropriate Manning's roughness coefficient, n. The longitudinal slope, S, will automatically be imported into this window if the *Roadway Geometry* module was previously used in the same session.

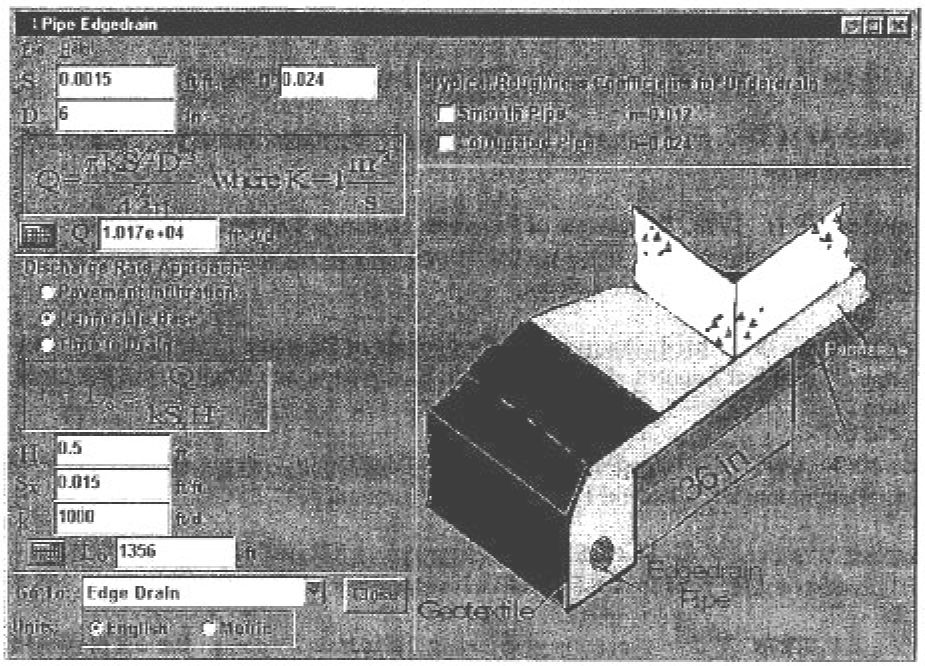
3. Click on the calculator button next to pipe capacity, *Q*, to calculate the flow capacity of the edgedrains.

4. Select the *Permeable Base* discharge rate approach and enter the base thickness ($H$), transverse slope ($S_T$), and base permeability ($k$). If you completed Example C-2B, the values from the *Permeable Base* module will be automatically imported into the appropriate input boxes.

5. Click on the calculator icon next to the outlet spacing, $L_o$, to determine the maximum outlet spacing based on hydraulic considerations.

The inputs and outputs for this example are illustrated in Figure C-9. The maximum outlet spacing determined based on hydraulic consideration for this example is 1,356 ft. However, this value far exceeds the recommended maximum outlet spacing of 250 ft (500 ft for smooth pipes), based on maintenance consideration.

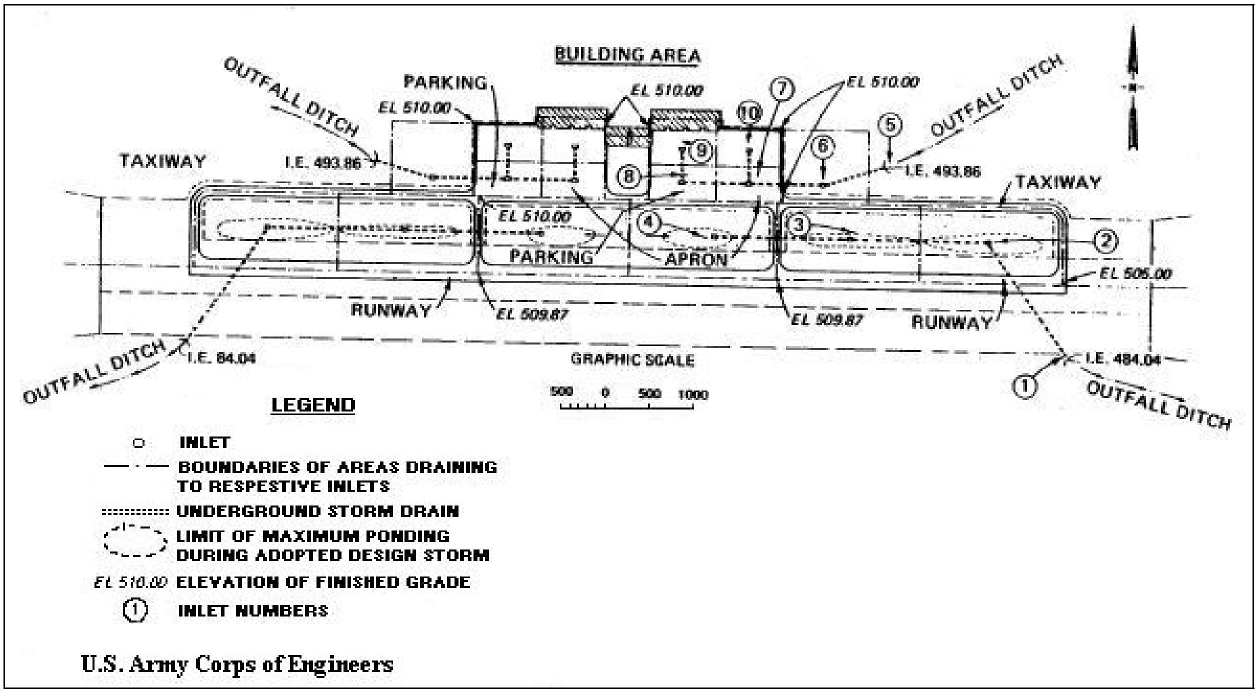
**Figure C-9. Pipe Edgedrain Design Window**




## C-3 EFFECT OF PONDING ON PIPE SIZE REQUIREMENTS

C-3.1 The proposed layout for the primary storm drainage system for an airfield is depicted in Figure C-10. This airfield is to be located in central Mississippi where the design storm index for a 2-year 1-hour rainfall intensity, according to Figure 2-2, is 2.0 inches per hour. The duration of storm being considered is 60 minutes; thus, Figure 3-1 need not initially be used. Infiltration values for the paved and turfed area are considered to be 0.0 and 0.5 inches per hour, respectively, according to Section 3-6. The supply curves applicable to this airfield are No. 2.0 for paved areas (2.0-0.0) and No. 1.5 for turfed areas (2.0-0.5). These supply curves are provided in Figure 3-1. Coefficients of roughness have been selected for the paved and turfed areas as 0.01 and 0.40, respectively, as suggested in Table C-5.

**Figure C-10. Sample Computations of Layout of Primary Storm Drainage System**




C-3.2 In this example, two conditions are considered: where ponding is permissible at Inlets 4, 3, and 2, and where no ponding is allowed at these inlets. The purpose of these examples is to portray the difference in pipe size requirements under these two imposing conditions. Tables C-4, C-5, and C-6 reflect the design where ponding is permissible, and Tables C-7, C-8, and C-9 reflect the design where ponding is not acceptable.

## C-4 OUTLET PROTECTION DESIGN

C-4.1 This section contains examples of recommended application to estimate the extent of scour in a cohesionless soil and alternative schemes of protection required to prevent local scour.

C-4.2 Circular and rectangular outlets with equivalent cross-sectional areas that will be subjected to a range of discharges for a duration of 1 hr are used with the following parameters:

Dimensions of rectangular outlet = $W_o$ = 10 ft, $D_o$ = 5 ft

Diameter of circular outlet, $D_o$ = 8 feet

Range of discharge, Q = 362 to 1,086 cubic feet per second

Discharge parameter for rectangular culvert, $q/D_o^{3/2}$ = 3.2 to 9.7

Table C-4

Table C-4 (cont)

Table C-4 (cont)

Table C-5

Table C-5 (cont)

Table C-5 (cont)

Table C-6

Table C-6 (cont)

Table C-6 (cont)

Table C-7

Table C-7 (cont)

Table C-8

Table C-9

Discharge parameter for circular culvert, $Q/D_o^{5/2}$ = 2 to 6

Duration of runoff event, t = 60 minutes

Maximum tailwater el = 6.4 feet above outlet invert (>0.5 $D_o$)

Minimum tailwater el = 2.0 feet above outlet invert (<0.5 $D_o$)

Example C-4A. Determine maximum depth of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see Figure 4-15)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80 \left( \frac{q}{D_o^{3/2}} \right)^{0.375} t^{0.10} \qquad \text{(eq. C-2)}$$

$$D_{sm} = 0.80\,(3.2 \text{ to } 9.7)^{0.375}\ (60)^{0.1}\ (5) = 9.3\ \mathit{ft}\ \mathit{to}\ 14.0\ \mathit{ft} \qquad \text{(eq. C-3)}$$

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left( \frac{q}{D_o^{3/2}} \right)^{0.375} t^{0.10} \qquad \text{(eq. C-4)}$$

$$D_{sm} = 0.74\,(3.2 \text{ to } 9.7)^{0.375}\ (60)^{0.1}\ (5) = 8.6\ \mathit{ft}\ \mathit{to}\ 13.0\ \mathit{ft} \qquad \text{(eq. C-5)}$$

CIRCULAR CULVERT (see Figure 4-15)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80 \left( \frac{Q}{D_o^{5/2}} \right)^{0.375} t^{0.10} \qquad \text{(eq. C-6)}$$

$$D_{sm} = 0.80\,(2 \text{ to } 6)^{0.375}\ (60)^{0.1}\ (8) = 12.5\ \mathit{ft}\ \mathit{to}\ 18.9\ \mathit{ft} \qquad \text{(eq. C-7)}$$

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left( \frac{q}{D_o^{5/2}} \right)^{0.375} \quad t^{0.1} \qquad \text{(eq. C-8)}$$

$$D_{sm} = 0.74\ (2 \text{ to } 6)^{0.375}\ (60)^{0.1}\ (8) = 11.6\ ft\ to\ 17.5\ ft \qquad \text{(eq. C-9)}$$

<u>Example C-4B. Determine maximum width of scour for minimum and maximum flow conditions:</u>

RECTANGULAR CULVERT (see Figure 4-16)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left( \frac{q}{D_o^{3/2}} \right)^{0.915} \quad t^{0.15} \qquad \text{(eq. C-10)}$$

$$W_{sm} = 1.00\ (3.2 \text{ to } 9.7)^{0.915}\ (60)^{0.15}\ (5) = 27\ ft\ to\ 74\ ft \qquad \text{(eq. C-11)}$$

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (27 \text{ to } 74) + \frac{10}{2} - \frac{5}{2} = 29.5\ ft\ to\ 76.5\ ft \qquad \text{(eq. C-12)}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72 \left( \frac{q}{D_o^{3/2}} \right)^{0.915} \quad t^{0.15} \qquad \text{(eq. C-13)}$$

$$W_{sm} = 0.72\ (3.2 \text{ to } 9.7)^{0.915}\ (60)^{0.015} = 19\ ft\ to\ 53\ ft \qquad \text{(eq. C-14)}$$

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (19 \text{ to } 53) + \frac{10}{2} - \frac{5}{2} = 21.5\ ft\ to\ 55.5\ ft \qquad \text{(eq. C-15)}$$

CIRCULAR CULVERT (see Figure 4-16)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left( \frac{Q}{D_o^{5/2}} \right)^{0.915} \quad t^{0.15} \qquad \text{(eq. C-16)}$$

$$W_{sm} = 1.00\ (2 \text{ to } 6)^{0.915}\ (60)^{0.15}\ (8) = 28\ ft\ to\ 76\ ft \qquad \text{(eq. C-17)}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72\left(\frac{Q}{D_o^{5/2}}\right)^{0.915} t^{0.15} \qquad \text{(eq. C-18)}$$

$$W_{sm} = 0.72\,(2 \text{ to } 6)^{0.915}\ (60)^{0.15}\ (8) = 20\ ft\ to\ 55\ ft \qquad \text{(eq. C-19)}$$

Example C-4C – Determine maximum length of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see Figure 4-17)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40\left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq. C-20)}$$

$$L_{sm} = 2.4\,(3.2 \text{ to } 9.7)^{0.71}\ (60)^{0.125}\ (5) = 46\ ft\ to\ 101\ ft \qquad \text{(eq. C-21)}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10\left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq. C-22)}$$

$$L_{sm} = 4.10\,(3.2 \text{ to } 9.7)^{0.71}\ (60)^{0.125}\ (5) = 78\ ft\ to\ 171\ ft \qquad \text{(eq. C-23)}$$

CIRCULAR CULVERT (see Figure 4-17)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40\left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq. C-24)}$$

$$L_{sm} = 2.4\,(2 \text{ to } 6)^{0.71}\ (60)^{0.125}\ (8) = 52\ ft\ to\ 114\ ft \qquad \text{(eq. C-25)}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10\left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq. C-26)}$$

$$L_{sm} = 4.10\,(2 \text{ to } 6)^{0.71}\ (60)^{0.125}\ (8) = 90\ ft\ to\ 195\ ft \qquad \text{(eq. C-27)}$$

Example C-4D. Determine profile and cross section of scour for maximum discharge and minimum tailwater conditions (see Figure 4-19):

CIRCULAR CULVERT

| For $L_{sm}$ = 114 ft and $D_{sm}$ = 18.9 ft | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $L_s/L_{sm}$ | 0.0 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 0.9 | 1.0 |
| L | 0.0 | 11.4 | 22.8 | 34.2 | 45.6 | 57.0 | 68.4 | 79.8 | 91.2 | 102.6 | 114.0 |
| $D_s/D_{sm}$ | 0.7 | 0.75 | 0.85 | 0.95 | 1.0 | 0.95 | 0.75 | 0.55 | 0.33 | 0.15 | 0.0 |
| $D_s$ | 13.2 | 14.2 | 16.1 | 18.0 | 18.9 | 18.0 | 14.2 | 10.4 | 6.3 | 2.9 | 0.0 |
| For $W_{sm}$ = 76 ft and $D_{sm}$ = 18.9 ft | | | | | | | | | | | |
| $W_s/W_{sm}$ | 0.0 | | 0.2 | | 0.4 | | 0.6 | | 0.8 | | 1.0 |
| $W_s$ | 0.0 | | 15.2 | | 30.4 | | 45.6 | | 60.8 | | 76.0 |
| $D_s/D_{sm}$ | 1.0 | | 0.67 | | 0.27 | | 0.15 | | 0.05 | | 0.0 |
| $D_s$ | 18.9 | | 12.6 | | 5.1 | | 2.8 | | 0.95 | | 0.0 |

RECTANGULAR CULVERT

| For $L_{sm}$ = 101 ft and $D_{sm}$ = 14.0 ft | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $L_s/L_{sm}$ | 0.0 | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 0.9 | 1.0 |
| L | 0.0 | 10.1 | 20.2 | 30.3 | 40.4 | 50.5 | 60.6 | 70.7 | 80.8 | 90.9 | 101.0 |
| $D_s/D_{sm}$ | 0.7 | 0.75 | 0.85 | 0.95 | 1.0 | 0.95 | 0.75 | 0.55 | 0.33 | 0.15 | 0.0 |
| $D_s$ | 9.8 | 10.5 | 11.9 | 13.3 | 14.0 | 13.3 | 10.5 | 7.7 | 4.6 | 2.1 | 0.0 |
| For $W_{sm}$ = 74 ft and $D_{sm}$ = 14.0 ft | | | | | | | | | | | |
| $W_s/W_{sm}$ | 0.0 | | 0.2 | | 0.4 | | 0.6 | | 0.8 | | 1.0 |
| $W_s$ | 0.0 | | 14.8 | | 29.6 | | 44.4 | | 59.2 | | 74.0 |
| $D_s/D_{sm}$ | 1.0 | | 0.67 | | 0.27 | | 0.15 | | 0.05 | | 0.0 |
| $D_s$ | 14.0 | | 9.38 | | 3.78 | | 2.10 | | 0.70 | | 0.0 |
| $W_{sr} = W_s$ $W_s + \frac{W_o}{2} - \frac{D_o}{2}$ | 0-2.5 | | 17.3 | | 32.1 | | 46.9 | | 61.7 | | 76.5 |

Example C-4E. Determine depth and width of cutoff wall:

RECTANGULAR CULVERT, Maximum depth and width of scour = 14 ft and 76.5 ft

From Figure 4-19, depth of cutoff wall = 0.7 ($D_{sm}$) = 0.7 (14) = 9.8 ft

From Figure 4-19, width of cutoff wall = 2 ($W_{smr}$) = 2 (76.5) = 153 ft

CIRCULAR CULVERT, Maximum depth and width of scour = 18.9 ft and 76.0 ft

From Figure 4-19, depth of cutoff wall = 0.7 ($D_{sm}$) = 0.7 (18.9) = 13.2 ft

From Figure 4-19, width of cutoff wall = 2 ($W_{sm}$) = 2 (76) = 152 ft

Note: The depth of cutoff wall may be varied with width in accordance with the cross section of the scour hole at the location of the maximum depth of scour. See Figures 4-19 and 4-20.

Example C-4F. Determine size and extent of horizontal blanket of riprap:

RECTANGULAR CULVERT

MINIMUM TAILWATER

$$\text{From Figure } 4-21,\ \frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3} \tag{eq. C-28}$$

$$d_{50} = 0.020\,(5/2)\,(3.2\ \ to\ \ 9.7)^{4/3}\ \ (5) = 1.2\ ft\ to\ 5.2\ ft \tag{eq. C-29}$$

$$\text{From Figure } 4\text{ - }22,\ \frac{L_{sp}}{D_o} = 1.8\left(\frac{q}{D_o^{3/2}}\right) + 7 \tag{eq. C-30}$$

$$L_{sp} = [1.8\,(3.2\ \text{ to }\ 9.7) + 7]\ \ 5 = 64\ ft\ to\ 122\ ft \tag{eq. C-31}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3} \tag{eq. C-32}$$

$$d_{50} = 0.020\ \ (5/6.4)\,(3.2\ \ to\ \ 9.7)^{4/3}\,(5) = 0.37\ ft\ to\ 0.76\ ft \tag{eq. C-33}$$

$$\frac{L_{sp}}{D_o} = 3 \left( \frac{q}{D_o^{3/2}} \right) \qquad \text{(eq. C-34)}$$

$$L_{sp} = 3\,(3.2 \text{ to } 9.7)\;\; 5 = 48\ ft\ to\ 145\ ft \qquad \text{(eq. C-35)}$$

CIRCULAR CULVERT

MINIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\;\; \frac{D_o}{TW} \left( \frac{Q}{D_o^{5/2}} \right)^{4/3} \qquad \text{(eq. C-36)}$$

$$d_{50} = 0.020\;\; (8/2)\,(2\ \ to\ \ 6)^{4/3}\;\; (8) = 1.6\ ft\ to\ 7.0\ ft \qquad \text{(eq. C-37)}$$

$$\frac{L_{sp}}{D_o} = 1.8 \left( \frac{Q}{D_o^{5/2}} \right) + 7 \qquad \text{(eq. C-38)}$$

$$L_{sp} = 1.8\,(2 \text{ to } 6) + 7 \quad\; 8 = 85\ ft\ to\ 142\ ft \qquad \text{(eq. C-39)}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\;\; \frac{D_o}{TW} \left( \frac{Q}{D_o^{5/2}} \right)^{4/3} \qquad \text{(eq. C-40)}$$

$$d_{50} = 0.020\;\; (8/6.4)\,(2\ \ to\ \ 6)^{4/3}\,(8) = 0.50\ ft\ to\ 2.18\ ft \qquad \text{(eq. C-41)}$$

$$\frac{L_{sp}}{D_o} = 3 \left( \frac{Q}{D_o^{5/2}} \right) \qquad \text{(eq. C-42)}$$

$$L_{sp} = 3\,(2 \text{ to } 6)\;\; 8 = 48\ ft\ to\ 144\ ft \qquad \text{(eq. C-43)}$$

Use Figure 4-23 to determine recommended configuration of horizontal blanket of riprap subject to minimum and maximum tailwaters.

Example C-4G – Determine size and geometry of riprap-lined preformed scour holes 0.5- and 1.0-$D_o$ deep for minimum tailwater conditions:

RECTANGULAR CULVERT (see Figure 4-21)

0.5-$D_o$-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125 \ \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq. C-44)}$$

$$d_{50} = 0.0125 \ (5/2)(3.2 \ \ to \ \ 9.7)^{4/3} \ (5) = 0.73 \ ft \ to \ 3.2 \ ft \qquad \text{(eq. C-45)}$$

1.0-$D_o$-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082 \ \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq. C-46)}$$

$$d_{50} = 0.0082 \ (5/2)(3.2 \ \ to \ \ 9.7)^{4/3} \ (5) = 0.48 \ ft \ to \ 2.1 \ ft \qquad \text{(eq. C-47)}$$

CIRCULAR CULVERT

0.5-$D_o$-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125 \ \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq. C-48)}$$

$$d_{50} = 0.0125 \ (8/2)(2 \ \ to \ \ 6)^{4/3} \ (8) = 1.0 \ ft \ to \ 4.4 \ ft \qquad \text{(eq. C-49)}$$

1.0-$D_o$-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082 \ \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq. C-50)}$$

$$d_{50} = 0.0082 \ (8/2)(2 \ \ to \ \ 6)^{4/3} \ (8) = 0.66 \ ft \ to \ 2.9 \ ft \qquad \text{(eq. C-51)}$$

See Figure 4-24 for geometry.

Example 4-CH. Determine size and geometry of riprap-lined-channel expansion for minimum tailwaters (see Figure 4-26):

RECTANGULAR CULVERT

$$\frac{d_{50}}{D_o} = 0.016 \ \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq. C-52)}$$

$$d_{50} = 0.016 \ (5/2)(3.2 \ \ to \ \ 9.7)^{4/3} \ (5) = 0.94 \ ft \ to \ 4.1 \ ft \qquad \text{(eq. C-53)}$$

CIRCULAR CULVERT

$$\frac{d_{50}}{D_o} = 0.016 \ \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq. C-54)}$$

$$d_{50} = 0.016 \ (8/2)(2 \ \ to \ \ 6)^{4/3} \ (8) = 1.29 \ ft \ to \ 5.6 \ ft \qquad \text{(eq. C-55)}$$

See Figure 4-25 for geometry.

Example 4-CI. Determine length and geometry of a flared outlet transition for minimum tailwaters:

RECTANGULAR CULVERT

$$\frac{L}{D_o} = 0.30 \ \left(\frac{D_o}{TW}\right)^2 \ \left(\frac{q}{D_o^{3/2}}\right)^{2.5(TW/D_o)1/3} \qquad \text{(eq. C-56)}$$

$$L = 0.3\,(5/2)^2\ (3.2 \ \ to \ \ 9.7)^{2.5(2/5)^{1/3}} \ \ 5 = 80 \ ft \ to \ 616 \ ft \qquad \text{(eq. C-57)}$$

CIRCULAR CULVERT

$$\frac{L}{D_o} = \left[0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_o^{5/2}}\right)^{2.5(TW/D_o)^{1/3}}\right] \qquad \text{(eq. C-58)}$$

$$L = \left[0.3\,(8/2)^2\ (2 \ \ to \ \ 6)^{2.5(2/8)^{1/3}}\right] 8 = 114 \ ft \ to \ 645 \ ft \qquad \text{(eq. C-59)}$$

See Figure 4-27 for geometric details; above equations developed for H = 0 or horizontal apron at outlet invert elevation without an end sill.

Example 4-CJ. Determine diameter of stilling well required downstream of the 8-ft-diam outlet:

From Figure 4-28

$$\frac{D_W}{D_o} = 0.53 \ \left(\frac{Q}{D_o^{5/2}}\right)^{1.0} \qquad \text{(eq. C-60)}$$

$$D_W = 0.53\,(2 \ \ to \ \ 6)\ \ 8 = 8.5\ ft\ to\ 25.4\ ft \qquad \text{(eq. C-61)}$$

See Figure 4-28 for additional dimensions.

<u>Example 4-CK. Determine width of U.S. Bureau of Reclamation type VI basin required downstream of the 8-ft-diam outlet:</u>

From Figure 4-29

$$\frac{W_{VI}}{D_o} = 1.30\ \left(\frac{Q}{D_o^{5/2}}\right)^{0.55} \qquad \text{(eq. C-62)}$$

$$W_{VI} = \left[1.3\,(2 \ \ to \ \ 6)^{0.55}\right]\ 8 = 15.2\ ft\ to\ 27.9\ ft \qquad \text{(eq. C-63)}$$

See Figure 4-29 for additional dimensions.

<u>Example 4-CL. Determine width of SAF basin required downstream of the 8-ft-diam outlet:</u>

From Figure 4-30

$$\frac{W_{SAF}}{D_o} = 0.30\ \left(\frac{Q}{D_o^{5/2}}\right)^{1.0} \qquad \text{(eq. C-64)}$$

$$W_{SAF} = 0.30\,(2 \ \ to \ \ 6)\ \ 8 = 4.8\ ft\ to\ 14.4\ ft \qquad \text{(eq. C-65)}$$

See Figure 4-30 for additional dimensions.

<u>Example 4-CM. Determine size of riprap required downstream of 8-ft-diam culvert and 14.4-ft-wide SAF basin with discharge of 1,086 cfs:</u>

$$q = \frac{Q}{W_{SAF}} = \frac{1086}{14.4} = 75\ cfs/ft \qquad \text{(eq. C-66)}$$

$$V_1 = \frac{Q}{A} = \frac{1086}{0.785(8)^2} = 21.6\ fps \qquad \text{(eq. C-67)}$$

$$d_1 = \frac{q}{V_1} = \frac{75}{21.6} = 3.5\ ft \qquad \text{(eq. C-68)}$$

$d_2$ = 8.4 ft (from conjugate depth relations)

MINIMUM TAILWATER REQUIRED FOR A HYDRAULIC JUMP = 0.90 (8.4) = 7.6 ft

$$d_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{(eq. C-69)}$$

$$V = \frac{q}{D} = \frac{75}{7.6} = 9.9\ fps \quad \text{(eq. C-70)}$$

$$d_{50} = 1.0\left[\frac{9.9}{\sqrt{32.2\,(7.6)}}\right]^3\ 7.6 \quad \text{(eq. C-71)}$$

$$d_{50} = 1.9\ ft \quad \text{(eq. C-72)}$$

## C-5 CHANNEL DESIGN

C-5.1 **Design Procedure**. The following steps will permit the design of a channel that will satisfy the conditions desired for the design discharge and one that will ensure no deposition or erosion under these conditions.

C-5.1.1 Determine gradation of material common to drainage basin from representative samples and sieve analyses.

C-5.1.2 Determine maximum discharges to be experienced annually and during the design storm.

C-5.1.3 Assume maximum desirable depth of flow, D, to be experienced with the design discharge.

C-5.1.4 Determine the sizes of material to be transported by examining the gradation of the local material (sizes and percentages of the total by weight). Particular attention should be given to the possibility of the transport of material from upper portions of the basin or drainage system and the need to prevent deposition of this material within the channel of interest.

C-5.1.5 Compute ratios of the diameter of the materials that should and should not be transported at the maximum depth of flow, ($d_{50}$/D).

C-5.1.6 Compute the Froude numbers of flow required to initiate transport of the selected sizes of cohesionless materials based on the equation, $F = 1.88\,(d_{50}/D)^{1/3}$, to determine the range of F desired in the channel.

C-5.2 **Channel Design**.

C-5.2.1 Design the desired channel as indicated in the following steps.

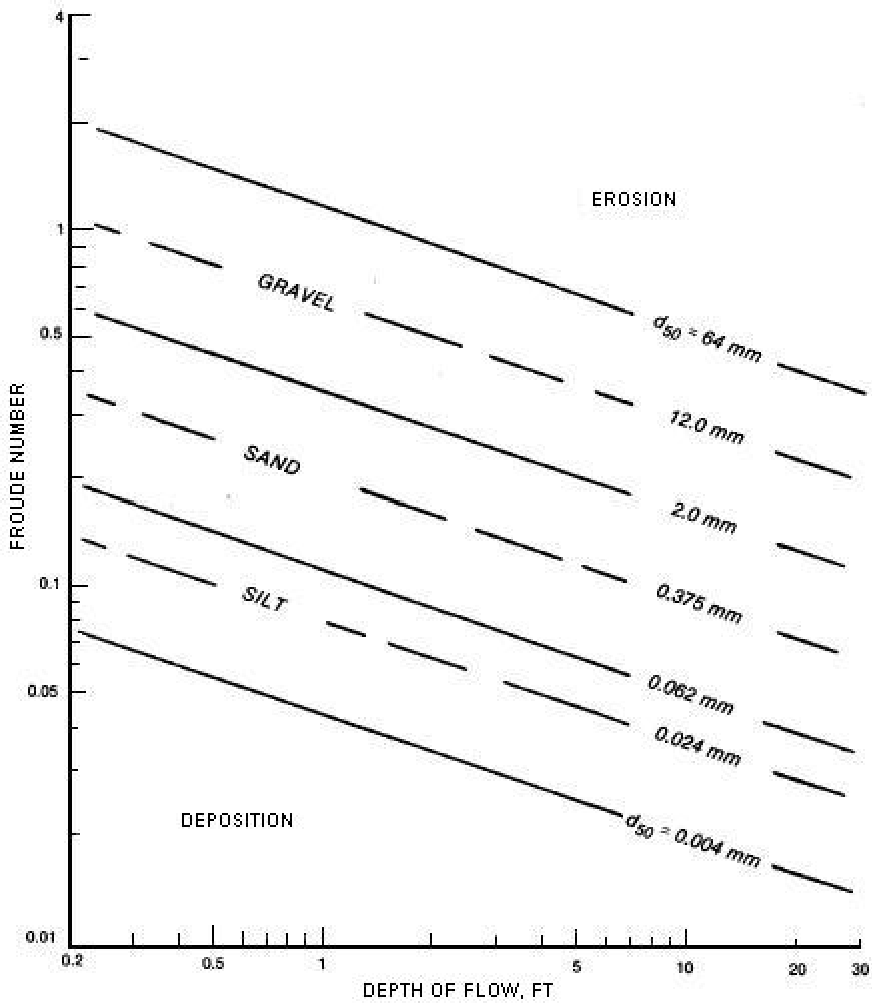
C-5.2.1.1 Assume that a channel is to be provided within and for drainage of an area composed of medium sand (grain diameter of 0.375 mm) for conveyance of a maximum rate of runoff of 400 cubic feet per second. Also assume that a channel depth of 6 feet is the maximum that can be tolerated from the standpoint of the existing groundwater level, minimum freeboard of 1 foot, and other considerations such as ease of excavation, maintenance, and aesthetics.

C-5.2.1.2 From Figure C-11 or the equation

$$F = 1.88(d_{50} / D)^{1/3} \quad \text{(eq. C-73)}$$

the Froude number of flow required for incipient transport and prevention of deposition of medium sand in a channel with a 5-foot depth of flow can be estimated to be about

**Figure C-11. Froude Number and Depth of Flow Required for Incipient Transport of Cohesionless Material**



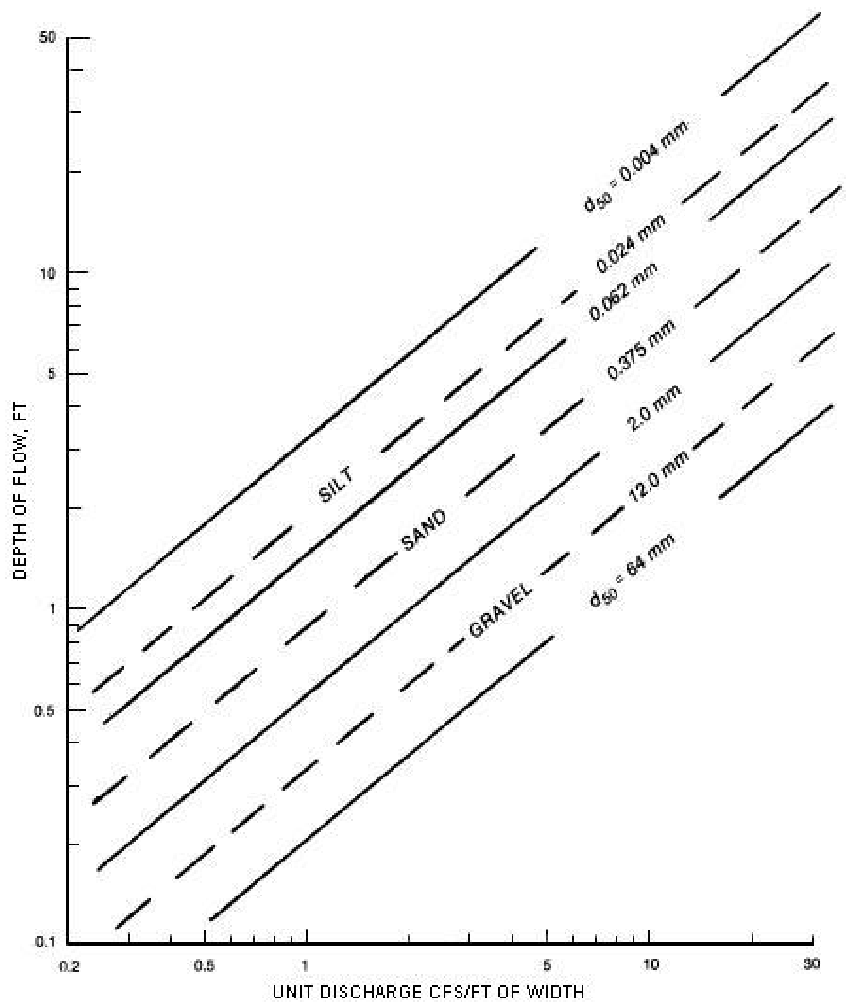
0.12. Further, it is indicated that a Froude number of about 0.20 would be required to prevent deposition of very coarse sand or very fine gravel. Therefore, an average Froude number of about 0.16 should not cause severe erosion or deposition of the medium sand common to the basin with a flow depth of 5 feet in the desired channel.

C-5.2.1.3 The unit discharge required for incipient transport and prevention of deposition of medium sand in a channel with a 5-foot depth of flow can be estimated to be about 7.4 cubic feet per second per foot of width from the equation

$$q = 10.66\, d_{50}^{\ 1/3} D^{7/6} \qquad \text{(eq. C-74)}$$

or Figure C-12. In addition, it is indicated that a unit discharge of about 13 cubic feet per second per foot of width would be required to prevent deposition of very coarse sand or very fine gravel. Thus, an average unit discharge of about 10 cubic feet per

**Figure C-12. Depth of Flow and Unit Discharge for Incipient Transport of Cohesionless Material**



second per foot of width should not cause severe erosion or deposition of the medium sand common to the basin and a 5-foot depth of flow in the desired channel.

C-5.2.1.4 The width of a rectangular channel and the average width of a trapezoidal channel required to convey the maximum rate of runoff of 400 cubic feet per second can be determined by dividing the discharge by the permissible unit discharge. For the example problem an average channel width of 40 feet is required. The base width of a trapezoidal channel can be determined by subtracting the product of the horizontal component of the side slope corresponding to a vertical displacement of 1 foot and the depth of flow from the previously estimated average width. The base width of a trapezoidal channel with side slopes of 1V on 3H required to convey the design discharge with a 5-foot depth of flow would be 25 feet.

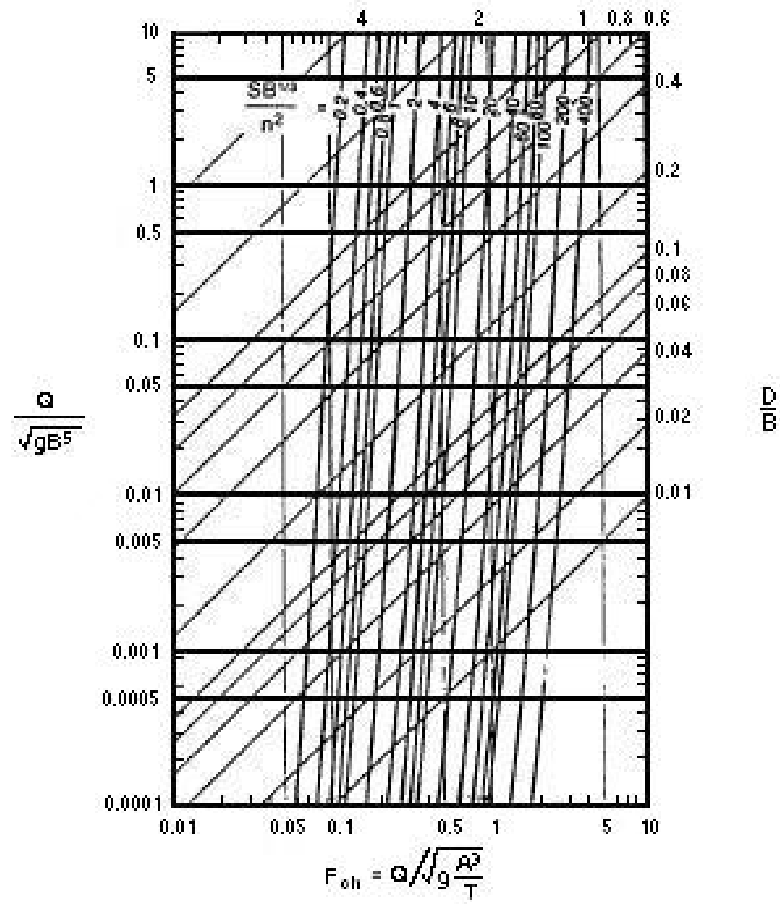
C-5.2.1.5 The values of the parameters D/B and $Q/\sqrt{gB^5}$ can now be calculated as 0.2 and 0.0225, respectively. Entering Figure C-13 with these values, it is apparent that corresponding values of 0.95 and 0.185 are required for the parameters of $SB^{1/3}/n^2$ and F, respectively. Assuming a Manning's n of 0.025, a slope of 0.000203 foot per foot would be required to satisfy the $SB^{1/3}/n^2$ relation for the 5-foot deep trapezoidal channel with base width of 25 feet and 1V-on-3H side slopes.

C-5.2.1.6 The Froude number of flow in the channel slightly in excess of the value of 0.16 previously estimated to be satisfactory with a depth of flow of 5 feet, but it is within the range of 0.12 and 0.20 considered to be satisfactory for preventing either severe erosion or deposition of medium to very coarse sand. However, should it be desired to convey the design discharge of 400 cubic feet per second with a Froude number of 0.16 in a trapezoidal channel of 25-foot base width and 1V-on-3H side slopes, the values of 0.0225 and 0.16 for $Q/\sqrt{gB^5}$ and F, respectively, can be used in conjunction with the Figure C-13 to determine corresponding values of $SB^{1/3}/n^2$ (0.72) and D/B (0.21) required for such a channel. Thus, a depth of flow equal to 5.25 feet, and a slope of 0.000154 foot per foot would be required for the channel to convey the flow with a Froude number of 0.16.

C-5.2.1.7 The slopes required for either the rectangular or the trapezoidal channels are extremely moderate. If a steeper slope of channel is desired for correlation with the local topography, the feasibility of a lined channel should be investigated as well as the alternative of check dams or drop structures in conjunction with the channel previously considered. For the latter case, the difference between the total drop in elevation desired due to the local topography and that permissible with the slope of an alluvial channel most adaptable to the terrain would have to be accomplished by means of one or more check dams and/or drop structures.

C-5.2.1.8 Assume that there is a source of stone for supply of riprap with an average dimension of 3 inches. The feasibility of a riprap-lined trapezoidal channel with 1V-on-3H side slopes that will convey the design discharge of 400 cubic feet per second with

**Figure C-13. Flow Characteristics of Trapezoidal Channels with 1V-on-3H Side Slopes**

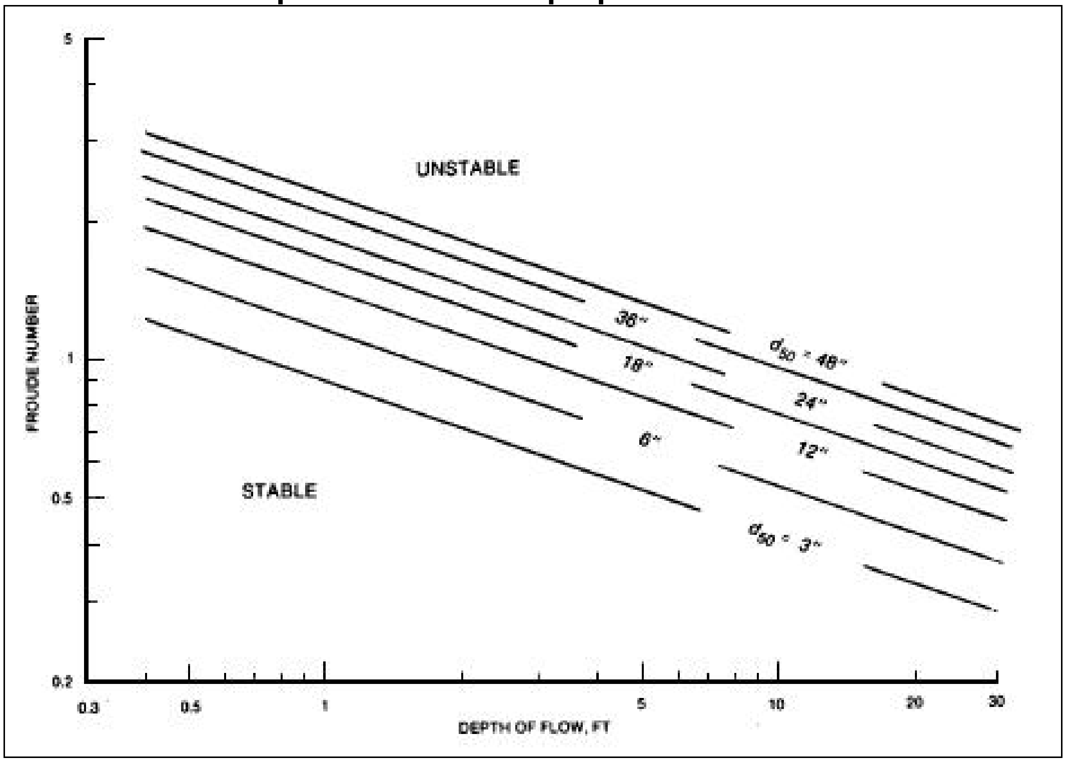


depths of flow up to 5 feet can be investigated as follows. The equation, $F = 1.42(d_{50}/D)^{1/3}$, or Figure C-14 can be used to estimate the Froude number of flow that will result in failure of various sizes of natural or crushed stone riprap with various depths of flow. The maximum Froude number of flow that can be permitted with average size stone of 0.25-foot-diameter and a flow depth of 5 feet is 0.52. Similarly, the maximum unit discharge permissible (33 cubic feet per second per foot of width) can be determined by the equation,

$$q = 8.05\, d_{50}^{\;1/3}\, D^{7/6} \qquad \text{(eq. C-75)}$$

or Figure C-15. For conservative design, it is recommended that the maximum unit discharge be limited to about two thirds of this value or say 22 cubic feet per second per foot of width for this example. Thus, an average channel width of about 18.2 feet is required to convey the design discharge of 400 cubic feet per second with a depth of 5 feet. The base width required of the riprap-lined trapezoidal channel with side slopes of 1V on 3H would be about 3 feet.

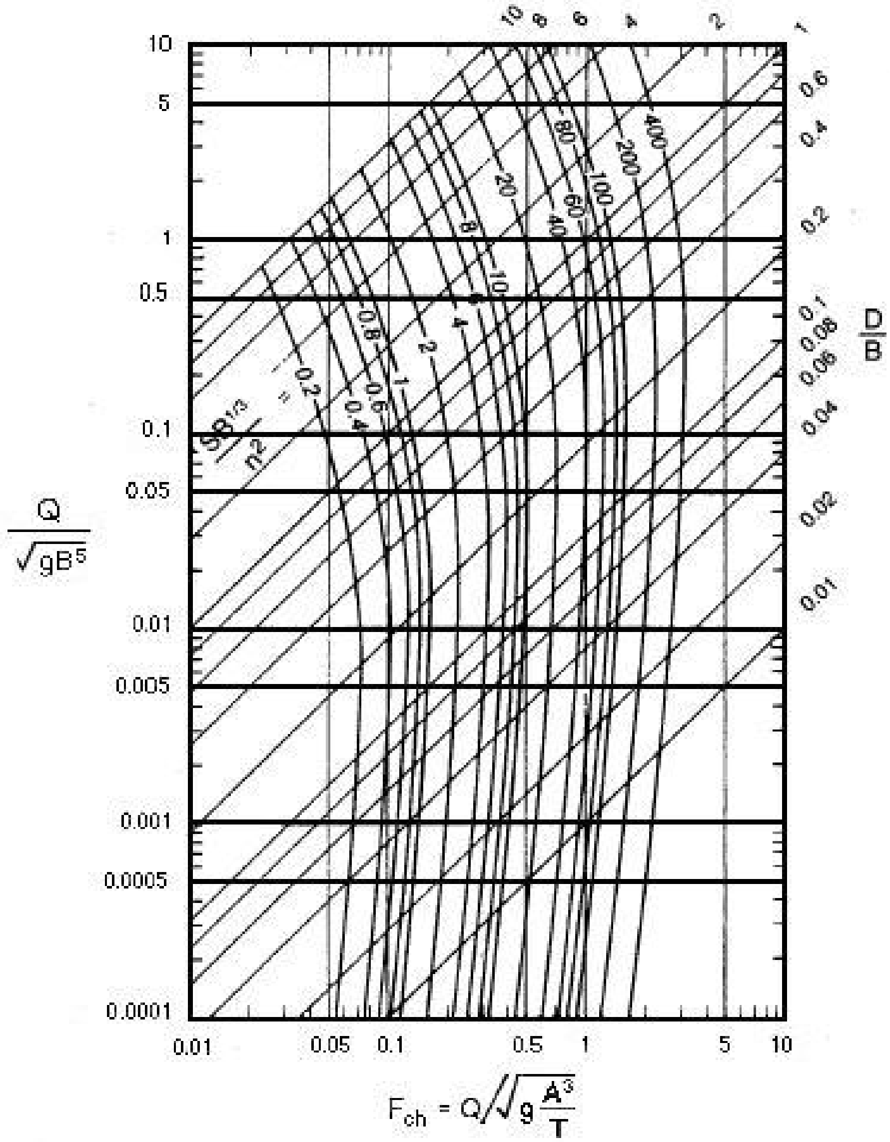
**Figure C-14. Froude Number and Depth of Flow for Incipient Failure of Riprap-Lines Channel**




C-5.2.1.9 The values of D/B and $Q/\sqrt{gB^5}$ can be calculated as 1.67 and 4.52, respectively. Entering Figure C-13 with these values, it is apparent that corresponding values of 4.5 and 0.52 are required for the parameters of $SB^{1/3}/n^2$ and F, respectively. Assume n = 0.035 $(d_{50})^{1/6}$ and calculate Manning's roughness coefficient of 0.25-foot-stone to be 0.028. A slope of 0.00245 foot per foot would be required for the 5-foot-deep riprap-lined trapezoidal channel with base width of 3 feet and 1V-on-3H side slopes. The Froude number of flow in the channel would meet the 3-inch-diameter average size requirement for riprap as well as the maximum recommended value of 0.8 needed to prevent instabilities of flow and excessive wave heights in subcritical open channel flow.

C-5.2.1.10 Similar analyses could be made for design of stable channels with different sizes of riprap protection should other sizes be available and steeper slopes be desired. This could reduce the number of drop structures required to provide the necessary grade change equal to the difference in elevation between that of the local terrain and the drop provided by the slope and length of the selected channel design.

**Figure C-15. Depth of Flow and Unit Discharge of Incipient Failure of Riprap-Lined Channel**




C-5.2.1.11 The feasibility of a paved rectangular channel on a slope commensurate with that of the local terrain for conveyance of the design discharge at either subcritical or supercritical velocities should also be investigated. Such a channel should be designed to convey the flow with a Froude number less than 0.8 if subcritical, or greater than 1.2 and less than 2.0 if supercritical to prevent flow instabilities and excessive wave heights. It should also be designed to have a depth-to-width ratio as near 0.5 (the most efficient hydraulic rectangular cross section) as practical depending upon the local conditions of design discharge, maximum depth of flow permissible, and commensuration of a slope with that of the local terrain.

C-5.2.1.12 For example, assume that a paved rectangular channel is to be provided with a Manning's n = 0.015 and a slope of 0.01 foot per foot (average slope of local terrain) for conveyance of a design discharge of 400 cubic feet per second at supercritical conditions. A depth-to-width ratio of 0.5 is desired for hydraulic efficiency and a Froude number of flow between 1.2 and 2.0 is desired for stable supercritical flow. The range of values of the parameter $SB^{1/3}/n^2$ (70-180) required to satisfy the desired D/B and range of Froude number of supercritical flow can be determined from Figure C-16. Corresponding values of the parameter $\sqrt{gB^5}$ (0.44-0.68) can also be determined from Figure C-16 for calculation of the discharge capacities of channels that will satisfy the desired conditions. The calculated values of discharge and channel widths can be plotted on log-log paper as shown in Figure C-17 to determine the respective relations for supercritical rectangular channels with a depth-to-width ratio of 0.5, a slope of 0.01 foot per foot, and a Manning's n of 0.015. Figure C-17 may then be used to select a channel width of 7.5 feet for conveyance of the design discharge of 400 cubic feet per second. The exact value of the constraining parameter $SB^{1/3}/n^2$ can be calculated to be 87 and used in conjunction with a D/B ratio of 0.5 and Figure C-16 to obtain corresponding values of the remaining constraining parameters, $Q\sqrt{gB^5}$ = 0.48 and F = 1.4, required to satisfy all of the dimensionless relations shown in Figure C-16. The actual discharge capacity of the selected 7.5-foot-wide channel with a depth of flow equal to 3.75 feet can be calculated based on these relations to ensure the adequacy of the selected design. For example, based on the magnitude of a discharge parameter equal to 0.48, the channel should convey 419 cubic feet per second:

$$Q = 0.48\sqrt{g(7.5)^{5/2}} = 419 \text{ cubic feet per second} \qquad \text{(eq. C-76)}$$

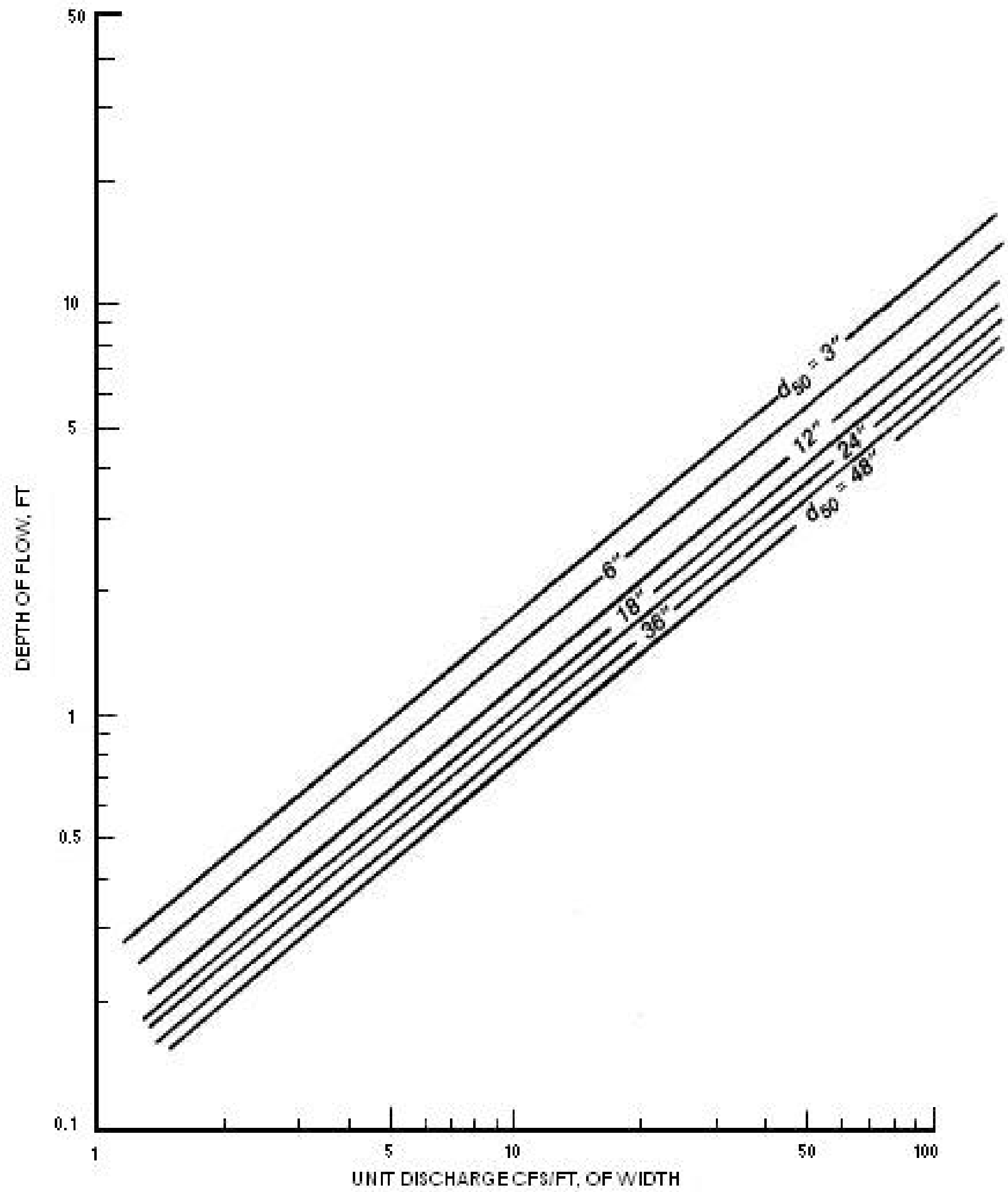
Similarly, based on the magnitude of a Froude number of flow equal to 1.4, the channel should convey a discharge of 432 cubic feet per second:

$$Q = 1.4\frac{\sqrt{g(7.5\times 3.75)^3}}{7.5} = 432 \text{ cubic feet per second} \qquad \text{(eq. C-77)}$$

Obviously, the capacity of the 7.5-foot-wide channel is adequate for the design discharge of 400 cubic feet per second.

C-5.2.1.13 The feasibility of a paved channel with a slope compatible with that of the local for conveyance of the design discharge at subcritical conditions should be investigated. However, it may not be feasible with slopes of 1 percent or greater. Paved channels for subcritical conveyance of flows should be designed to provide Froude numbers of flow ranging from about 0.25 to 0.8 to prevent excessive deposition and flow instabilities, respectively. If rectangular, paved channels should be designed to have a depth of width radio as near 0.5 as practical for hydraulic efficiency; if

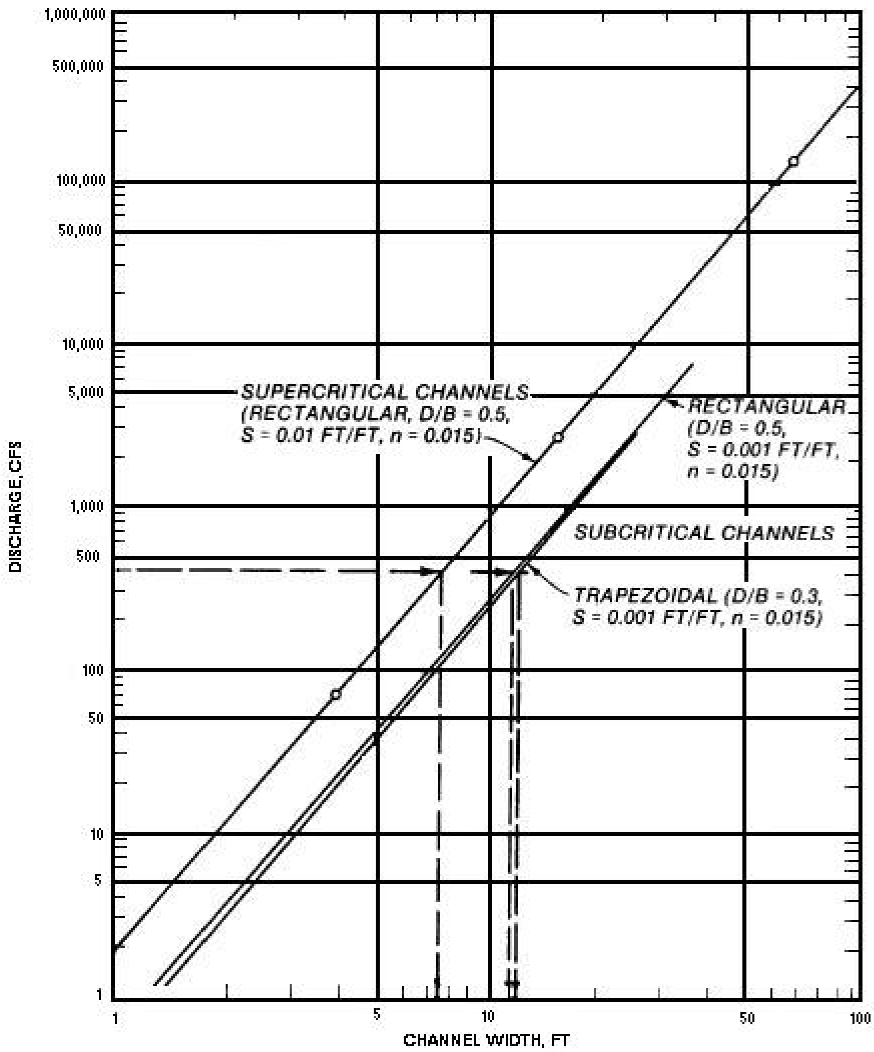
**Figure C-16. Flow Characteristics of Rectangular Channels**




trapezoidal, they should be designed to have side slopes of 1V on 3H and a depth-to-width ratio of 0.3.

C-5.2.1.14 For example, assume a subcritical paved channel with a Manning's n of 0.015 and slope of 0.01 foot per foot is to be provided for a design discharge of 400 cubic feet per second. The maximum slope and discharge permissible for conveying flow with a Froude number less than 0.8 in a hydraulically efficient rectangular channel with a minimum practical width of 1.0 foot can be determined from Figure C-16. For a D/B = 0.5 and Froude number of flow of 0.8, the corresponding

**Figure C-17. Discharge Characteristics of Various Channels**




values of $SB^{1/3}/n^2$ and $Q\sqrt{gB^5}$ are determined as 30 and 0.275, respectively. Solving these regulations for S and Q based on n = 0.015 and B = 1 foot yields

$$S = 30\ n^2 / B^{1/3} = 0.00675 \text{ foot per foot} \qquad \text{(eq. C-78)}$$

$$Q = 0.275\sqrt{gB^{5/2}} = 1.56 \text{ cubic feet per second} \qquad \text{(eq. C-79)}$$

Greater widths of hydraulically efficient rectangular channels would convey greater discharges, but slopes flatter than 0.00675 foot per foot would be required to prevent the Froude number of flow from exceeding 0.8. Therefore, a rectangular channel of the most efficient cross section and a slope as steep as 0.01 foot per foot are not practical for subcritical conveyance of the design discharge and the example problem. A similar analysis for any shape of channel would result in the same conclusion; stable subcritical conveyance of the design discharge on a slope of 0.01 foot per foot is not feasible.

C-5.2.1.15 Assuming that the average slope of the local terrain was about 0.001 foot per foot for the example problem, practical subcritical paved channels could be designed as discussed in paragraphs (16) through (19) below.

C-5.2.1.16 Based on the desired range of Froude numbers of flow (0.25 to 0.8) in a rectangular channel of efficient cross section (D/B = 0.5), Figure C-16 indicates the corresponding range of values of the restraining parameters $SB^{1/3}/n^2$ and $Q\sqrt{gB^5}$ to be from 3 to 30 and 0.085 to 0.275, respectively. The relations between discharge and channel width for subcritical rectangular channels with a depth-to-width ratio of 0.5, a slope of 0.001 foot per foot, and a Manning's n of 0.015 can be plotted as shown in Figure C-17 to select the 11.5-foot-width of channel required to convey the design discharge of 400 cubic feet per second.

C-5.2.1.17 As a check, the exact value of $SB^{1/3}/n^2$ can be calculated to be 10.1 and used in conjunction with a D/B ratio of 0.5 and Figure C-16 to obtain corresponding values of the remaining constraining parameters, $Q\sqrt{gB^5}$ = 0.16 and F = 0.47, required to satisfy all of the dimensionless relations for rectangular channels. The actual discharge capacity of the selected 11.5-foot-wide channel with a depth of 5.75 feet can be calculated based on these relations to ensure the adequacy of the selected design. For example, based on the magnitude of the discharge parameter (0.16), the channel should convey 407 cubit feet per second:

$$Q = 0.16\sqrt{g(11.5)^{5/2}} = 407 \text{ cubic feet per second} \qquad \text{(eq. C-80)}$$

Similarly, based on the Froude number of flow to 0.47, the channel should convey a discharge of 422 cubic feet per second:

$$Q = 0.47 \quad \frac{\sqrt{g(11.5 \times 5.75)^3}}{11.5} = 422 \text{ cubic feet per second} \qquad \text{(eq. C-81)}$$

Therefore, the 11.5-foot-wide channel is sufficient for subcritical conveyance of the design discharge of 400 cubic feet per second and, based on Figure C-11, is sufficient for transporting materials as large as average size gravel.

C-5.2.1.18 A similar procedure would be followed to design a trapezoidal channel with a depth-to-width ratio of 0.3, a slope of 0.001 foot per foot, and a Manning's n of 0.015 utilizing Figure C-13. For example, in order to maintain a Froude number of flow between 0.25 and 0.75 in a trapezoidal channel with side slopes 1V on 3H and a depth-to-width ratio of 0.3, the constraining parameter of $SB^{1/3}/n^2$ would have to have a value between 2 and 15 (Figure C-13). The relations between discharge and base width for these subcritical trapezoidal channels were plotted as shown in Figure C-17 to select the 12-foot-base width required to convey the design discharge of 400 cubic feet per second.

C-5.2.1.19 As a check, the exact value of $SB^{1/3}/n^2$ was calculated to be 10.2 and used in conjunction with D/B of 0.3 and Figure C-13 to obtain corresponding values of the remaining constraining parameters, $Q/\sqrt{gB^5}$ = 0.15 and F = 0.63, required to satisfy the dimensionless relations of trapezoidal channels. The actual discharge capacity of the selected trapezoidal channel with a base width of 12 feet and a flow depth of 3.6 feet based on these relations would be 425 and 458 cubic feet per second, respectively.

$$Q = 0.15\sqrt{g(12)^{5/2}} = 425 \text{ cubic feet per second} \qquad \text{(eq. C-82)}$$

$$Q = 0.63\sqrt{\frac{\dfrac{g\ 45.6 \times 3.6^3}{2}}{33.6}} = 458 \text{ cubic feet per second} \qquad \text{(eq. C-83)}$$

Therefore, the selected trapezoidal channel is sufficient for subcritical conveyance of the design discharge of 400 cubic feet per second and based on Figure C-11 is sufficient for transporting materials as large as coarse gravel.

C-5.2.2 Having determined a channel that will satisfy the conditions desired for the design discharge, determine the relations that will occur with the anticipated maximum annual discharge and ensure that deposition and/or erosion will not occur under these conditions. It may be necessary to compromise and permit some erosion during design discharge conditions in order to prevent deposition under annual discharge conditions. Lime stabilization can be effectively used to confine clay soils, and soil-cement stabilization may be effective in areas subject to sparse vegetative cover. Sand-cement and rubble protection of channels may be extremely valuable in areas where rock protection is unavailable or costly. Appropriate filters should be provided to prevent leaching of the natural soil through the protective material. Facilities for subsurface drainage or relief of hydrostatic pressures beneath channel linings should be provided to prevent structural failure.

## C-6 CONCRETE CHUTE DESIGN

C-6.1 Design a concrete chute to carry 25 cubic feet per second down a slope with a 25 percent grade. The allowable head is 1 foot and Manning's n is 0.014.

C-6.2 Solution one. Using equation 4-21 with no drop at the entrance, $Q=3.1W(H)^{1.5}$, with Q=25 cubic feet per second and H = 1 foot.

$$25 = 3.1W\ (1)^{1.5} \text{ or } W = 8.06\ \textit{feet} \qquad \text{(eq. C-84)}$$

Use W = 8 feet

Now

$$A = Wd = 8d \quad \text{(eq. C-85)}$$

and

$$R = \frac{\text{area}}{\text{wetted perimeter}} = \frac{8d}{W + 2d} = \frac{8d}{8 + 2d} \quad \text{(eq. C-86)}$$

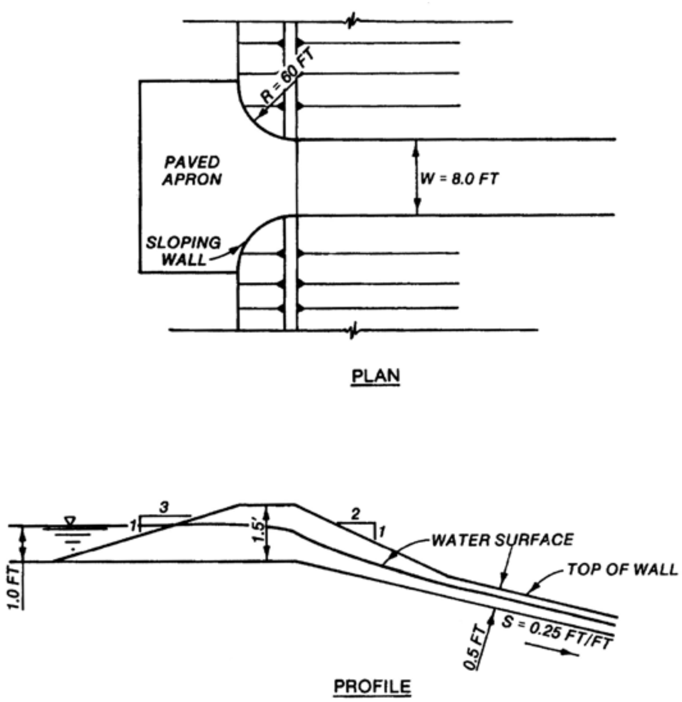
Use Manning's equation (4-22) to determine depth of water:

$$Q = \frac{1.486}{n} A\, S^{1/2} R^{2/3} = \frac{1.486}{0.014} A (0.25)^{1/2} R^{2/3} = 25 \quad \text{(eq. C-87)}$$

$$25 = \frac{1.486}{0.014} \times 8d \times (0.25)^{1/2} \times = \left( \frac{8d}{8 + 2d} \right)^{2/3} \quad \text{(eq. C-88)}$$

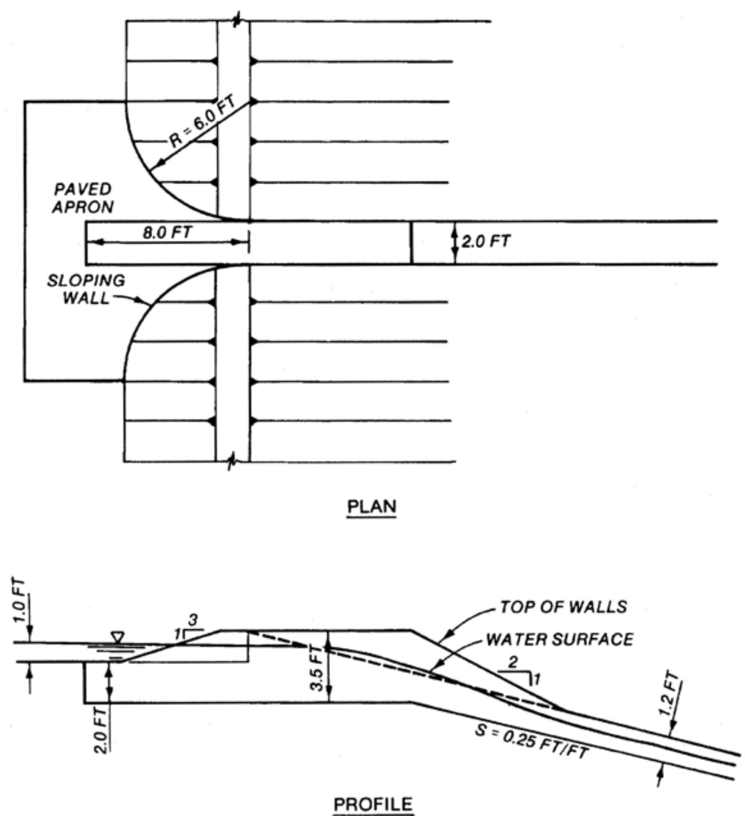
Solving for d by trial and error, the depth of water is d=0.186 foot. For use in Figure 4-39, the size of the angle of the chute is equal to 0.243 and q=Q/W=25/8=3.125. Thus, $S/q^{1/5}$ equals 0.1935, which corresponds to a design air concentration T = $d_{air}$/ ($d_{air}$ +d) = 0.471. Solving for $d_{air}$ gives 0.166 foot. Then, the total depth of flow is depth of water plus depth of air, 0.352 foot. Wall height should be 1.5 times the total depth of flow or 0.528 foot. One should use 0.5 foot. This design is shown in Figure C-18.

**Figure C-18. Design Problem – Solution One**



C-6.3 A drop will be provided at the entrance. Therefore, a width of chute can be selected and the appropriate length and depth of drop determined from the curves in Figure 4-38. For this design select a width of 2 feet. Then H/W = 1/2 = 0.5 and $Q/W^{5/2} = 25/(2)^{5/2} = 4.42$. From Figure 4-38, find a curve that matches these values. This is found on the curve for D/w 1.0, on the chart for B/W = 4. Therefore, B = 8 feet and D =2.0 feet. Using Manning's equation (4-22) to determine depth of water as in the first solution, find $d_w$ = 0.493 foot. From Figure 4-39, with q equals 12.5, sine of angle of slope equals 0.243 and $d_w$ equals 0.493 foot, determine the depth of air to be 0.311 foot. Thus, total depth is 0.804 foot. Use 0.80 foot. Wall height is 1.5 times 0.80 foot, or 1.20 feet. This design is shown in Figure C-19.

**Figure C-19. Design Problem – Solution Two**



# APPENDIX D

## COVER TABLES

To be discussed