ENGINEER MANUAL

EM 1110-3-136
April 1984

ENGINEERING AND DESIGN

Drainage and Erosion Control Mobilization Construction

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE CHIEF OF ENGINEERS

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, D.C. 20314

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Engineer Manual
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Engineering and Design
DRAINAGE AND EROSION CONTROL
Mobilization Construction

1. Purpose. This manual provides guidance for standards to be used for drainage and erosion control for U.S. Army mobilization construction.

2. Applicability. This manual is applicable to all field operating activities having mobilization construction responsibilities.

3. Discussion. Criteria and standards presented herein apply to construction considered crucial to a mobilization effort. These requirements may be altered when necessary to satisfy special conditions on the basis of good engineering practice consistent with the nature of the construction. Design and construction of mobilization facilities must be completed within 180 days from the date notice to proceed is given with the projected life expectancy of five years. Hence, rapid construction of a facility should be reflected in its design. Time-consuming methods and procedures, normally preferred over quicker methods for better quality, should be de-emphasized. Lesser grade materials should be substituted for higher grade materials when the lesser grade materials would provide satisfactory service and when use of higher grade materials would extend construction time. Work items not immediately necessary for the adequate functioning of the facility should be deferred until such time as they can be completed without delaying the mobilization effort.

FOR THE COMMANDER:

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DEPARTMENT OF THE ARMY
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PART ONE
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PART ONE

GENERAL

CHAPTER 1

PURPOSE AND SCOPE

1-1. Purpose. This manual prescribes the standards to be used for drainage and erosion control for mobilization construction. The maximum use of natural drainage is encouraged; however, positive surface and subsurface drainage systems will be necessary in certain cases to insure protection from saturation of subgrades, damage to slope, erosion, and loss of pavement and slab bearing capacity.

1-2. Scope. This manual provides design criteria for:

- Subsurface water collection systems.

- Surface drainage collection systems.

- Cover requirements for several types of piping with various loading conditions.

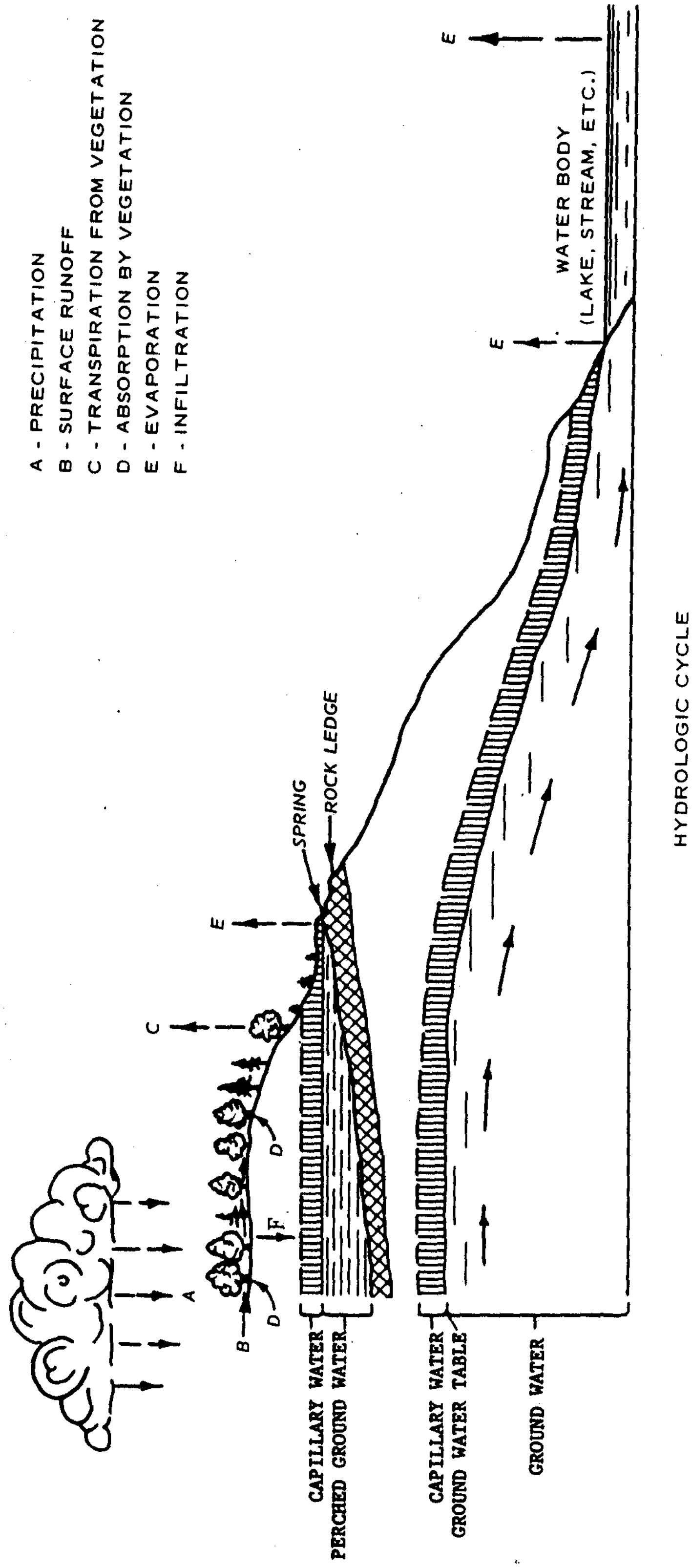
- Erosion control structures.

- Protection provisions against freezing in seasonal frost areas.

The criteria are developed for use with concrete slabs or flexible pavement systems placed on grade and exposed to atmospheric conditions. Variations are included in the criteria to allow application to the general areas of airfields, roads, and railroads. Airfield slabs and pavements include runways, overruns, shoulders, taxiways, aprons, hardstands, and wash rack areas. Roads include all classes of roadways, streets, access, and storage areas, both open or shaded which are subject to storm runoff or ground water conditions. Roadways, streets, and accesses considered may be paved or unpaved but are exclusive of bridges. Railroad considerations include utility tracks and spurs. For further information on the uses and requirements of subsurface and storm drainage see EM 1110-3-130, EM 1110-3-131, EM 1110-3-132, EM 1110-3-141, EM 1110-3-142, and EM 1110-3-152.

1-3. Hydrologic cycle.

a. Standard hydrologic cycle. A standard hydrologic cycle illustrating the flow of moisture is shown in figure 1-1. Precipitation in the form of rain or snow either runs off or infiltrates into the soil or rocks.



U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

FIGURE 1-1. OCCURRENCE, SOURCE, AND MOVEMENT OF GROUND WATER

b. Surface runoff. Surface runoff is that portion of the water resulting from rainfall, snow melt, or manmade causes which runs freely across the ground surface to lakes, streams, and rivers. Surface runoff is the major cause of ground erosion.

c. Ground water. The water which infiltrates beneath the surface of the ground is termed subsurface water or ground water. The free surface of this water is referred to as the "ground water table" or "water table." Perched water is ground water trapped in pockets by an impervious stratum, above the natural water table level. Water that saturates the soil before frost has disappeared from lower levels form a temporary perched ground water. Ground water tables rise and fall depending upon the relation between infiltration, absorption, evaporation, and ground water flow. Seasonal fluctuations are at some localities large and fluctuations, because of drought or wet periods of considerable duration, may also be very large. Land drainage ditches, railroad and highway cuts and fills, and other excavations may cause a substantial alteration of the ground water table.

d. Evaporation. Evaporation from water on the ground surface such as lakes and rivers along with transpiration from vegetation complete the hydrologic cycle.

CHAPTER 2

DESIGN CONSIDERATIONS

2-1. Design.

a. Coordination. Improper design and careless construction of various drainage structures may render facilities ineffective and dangerous to safe operation. 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 design procedures. A successful drainage system can only be obtained by coordination on the part of both the field and design engineers.

b. Surface and subsurface drainage systems. The installation of systems for drainage of surface runoff collection and subsurface water collection into one system of pipes serving this dual purpose is not recommended. Once the subsurface water has been collected and "daylighted" however, further disposal of this water can be directed to the storm drainage disposal system. The subsurface drains should enter the storm system at open channels (or gravity mains) well above the expected high water line of the storm system. Special attention must be given to preventing any backflow from the storm system to the subsurface drainage system.

c. Fuel drainage. Fuel spillage will not be collected in storm sewers. Safe disposal of fuel spillage may be facilitated by the provision of 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.

2-2. Drainage outfall considerations.

Local laws vary as to the responsibility of upstream property owners for the damage created by artificial drainage on downstream property. The construction of new runways or heliports and the modification or expansion of existing facilities, although designated as temporary, may cause permanent changes in the area's natural drainage patterns. Local laws and practices should be considered in the drainage system design.

2-3. Initial and deferred design and construction.

a. Initial design. The initial design and construction will consist of those items necessary for placing the facility in operation and maintaining uninterrupted service. These items will include all drainage and subdrainage, piping, under or immediately adjacent to runways, taxiways, aprons, helicopter pads, roadways, streets, open storage, and railroads. All inlets and box drains located in paved areas, and open channels necessary to remove water from the immediate area of the facility are also included. The initial stage will be completed within the 180-day mobilization program.

b. Storm and subdrainage systems planning. During the initial design stage the complete storm and subdrainage systems will be planned. However, the detailed design and construction of certain structures associated with these systems can be deferred. These structures including drop structures, check dams, chutes, and stilling basins serve the purpose of handling the runoff and final disposal of excess water. The primary function of proper handling of excess water is the erosion control surrounding the facility but not directly associated with the operation and safety of that facility. For instance, open channels can be constructed and operated without check dams for short periods of time. The installation of check dams however has only been deferred, for erosion of the open channel will occur in time which must be corrected and dams built. It is anticipated that the emergency mobilization program would be in its final stages before the deferred structures are needed, and the construction of these items would not be time critical. Headwalls in general should be deferred, but certain critical conditions may exist where the safety of runways, roads, railroads, and other facilities could be jeopardized by headwall exclusion. These areas would require headwalls to be included in the initial design and construction stage.

c. Deferred structures. Deferable structures are discussed in part six of this manual. It must be cautioned that these structures are only deferred for a short time, not eliminated.

PART TWO

SUBSURFACE DRAINAGE

CHAPTER 3

SUBSURFACE DRAINAGE REQUIREMENTS

3-1. General.

a. Usage requirements. Soil and ground water conditions may make necessary the installation of subsurface drainage systems in selected areas. Subsurface drain systems must be completed in their entirety within the 180-day mobilization construction period. Where subsurface drains are necessary, independent outlets or outlets free from backwater effects will be provided. The system's durability should insure a 5-year life with a minimum of maintenance. The selection of material is to consider potential deterioration from corrosion and abrasion, which could include seepage from an occasional spill of industrial wastes or petroleum products. Concrete pipe should be avoided where the pH of the soil or in the fluid carried is expected to be below 5.5 or above 9.0 for extended periods of time.

b. Criteria application. For the most part, criteria for subsurface drains apply equally to airfields, roads, and railroads. There are certain details of airfield complex systems design however, that require manholes, observation basins, and risers for access which are not necessary for simpler, smaller subsurface drainage systems associated with roads or railroads. Such structures should only be used where well justified from a maintenance standpoint for a limited 5-year duration.

3-2. Investigations to determine subsurface drainage requirements. For the satisfactory design of a subsurface drainage system, the determination of the subsurface soil and water conditions is a prerequisite. The field explorations and borings made in connection with the project design should include the following investigations pertinent to subsurface drainage.

a. Soil conditions. The soil conditions investigated for other purposes in connection with the design will supply a large amount of information that can be used for the design of the drainage system. It may be advisable to supplement these explorations at locations of subsurface drainage structures and in areas where soil information from a drainage viewpoint is incomplete. All soil investigations should be coordinated into one program. Soils investigation from former construction in the area may be used to supplement the exploration or when no other information is available.

b. Ground water. The location and depth of permanent and perched ground water can also be obtained from the soils investigation test holes. In many locations, information may be obtained from residents of the surrounding area regarding the behavior of wells and springs and other evidences of subsurface water.

c. Photogrammetry. The analysis of aerial photographs of the areas selected for construction may furnish valuable information on general soil and ground water conditions. Aerial photographs exist for almost the entire United States, taken for the Agricultural Adjustment Administration, the United States Forest Service, the Soil Conservation Service, and the Topographic Branch of the United States Geological Survey. The Map Information Office of the latter organization maintains records of available United States photographic coverage of all agencies. 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 surface features of the soil are the surficial evidence of the characteristics and behavior of subsurface materials. 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 the 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.

d. Soils tests. The major amount of information required for the subsurface drainage design can be obtained from the soils investigation with a minimum of laboratory tests. Soils information from other investigations in the area should also be used. The final selected soil properties for design purposes should be expressed as a range, one extreme representing the best average maximum value and the other the best average minimum value. The true value should lie between these two extremes but may approach or equal one or the other, depending upon variation within a defined soil stratum. The perviousness of a given soil, or its ability to conduct water, is measured by the coefficient of permeability. It is expressed as the rate of flow for a hydraulic gradient of one through a unit area of soil. The coefficient of permeability of some soils is difficult to determine and requires experienced and skilled personnel.

3-3. Criteria for determining the need for subsurface drainage. Subsurface drainage may be divided into base drainage, which consists of removing water from a base course beneath a pavement; subgrade drainage, which consists of removing water from the subgrade beneath a pavement; and intercepting drainage, which consists of collecting and removing water found flowing in pervious strata or from springs.

a. Base drainage. Base drainage is required where frost action occurs in the subgrade beneath the pavement. Frost action is assumed to occur in all frost-susceptible subgrades. Base drainage is also required where the ground water rises to the bottom of the base course as a result of either natural conditions, ponding of surface runoff, or consolidation of soil under the weight of the base course. At locations where the pavement may become inundated, and there is little possibility of water draining from the base into the subgrade, base drainage may be required. As a guide to determining where base drainage is required in cases of inundation, the following tabulation may be used.

Depth to ground water (feet):	Base drainage required if subgrade coefficient of permeability is smaller than (fpm)
Less than 8	1×10^{-5}
8 to 25	1×10^{-6}
More than 25	1×10^{-7}

Where subgrade soils vary greatly in coefficient of permeability with depth, judgment should be exercised in determining the necessity for base drainage. Base drainage is also required at the low point of longitudinal grades in excess of 2 percent except where the subgrade coefficient of permeability is 1×10^{-3} or greater.

b. Subgrade drainage. Subgrade drainage is required at locations where seasonal fluctuations of ground water may be expected to rise in the subgrade beneath a paved area to less than 1 foot below the bottom of the base course.

c. Intercepting drainage. Intercepting drainage is required where seeping water in a pervious stratum will raise the ground water table locally to a depth of less than 1 foot below the bottom of the base course. This flowing water may be found in thin pervious soil layers, in exposed rock cuts, or as seepage from springs.

CHAPTER 4

PRINCIPLES OF SOIL DRAINAGE

4-1. Flow of water through soils. The flow of water through soils is estimated by Darcy's Law which can be expressed as

$$Q = kiAt$$

where:

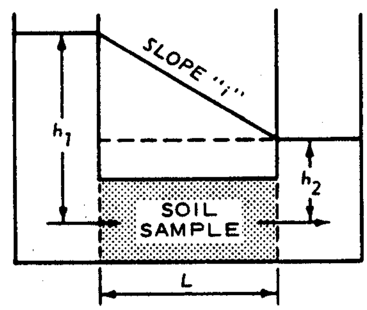
Q = Quantity of seepage
k = Coefficient of permeability
$i = \frac{h_i - h_2}{L}$ (see figure 4-1) = hydraulic gradient
($k \times i$ = discharge velocity)
A = cross-sectional area of sample
t = time

The meaning of Darcy's Law is simply illustrated in figure 4-1. 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. In most soils, the permeability varies depending on the direction in which the water is moving. The permeability in the direction parallel to the bedding plane or planes of stratification can be several times that in the direction perpendicular to the bedding. In soil deposits with erratic lenses of either coarse, pervious materials or fine, impervious materials, the permeability varies greatly from point to point and is very difficult to determine accurately. The value of permeability which has the units of velocity depends primarily on the characteristics of the permeable materials, but it is also a function of the properties of the fluid. The permeability of a soil can be estimated by the following equation:

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{(1 + e)} C \qquad \text{(eq 4-1)}$$

where:

k = the coefficient of permeability
D_s = some effective particle diameter
γ = unit weight of water
μ = viscosity of water
e = void ratio
C = shape factor



QUANTITY OF SEEPAGE	$Q = k \times i \times A \times t$
DISCHARGE VELOCITY	$v = k \times i$
HYDRAULIC GRADIENT	$i = \frac{h_1 - h_2}{L}$
CROSS SECTIONAL AREA OF SAMPLE	A
TIME	t
COEFFICIENT OF PERMEABILITY	k

U.S. Army Corps of Engineers

FIGURE 4-1. DARCY'S LAW FOR FLOW-THROUGH SOILS

This equation aids considerably in the examination of the factors affecting permeability. The following factors influence the permeability of soils.

a. Effect of pore fluid and temperature. The above equation indicates that the permeability is directly proportional to the unit weight of water γ and inversely proportional to the viscosity, μ. Values of γ are essentially constant, but values of μ vary considerably with temperature. The effect of fluid properties on the value of the permeability when other factors are constant is shown in the following equation:

$$k_1:k_2 = \mu_2:\mu_1$$

b. 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 more dense a soil, i.e., the smaller the pores, the lower the soil permeability. From the loosest to the densest condition, permeability may vary 1 to 20 times. As a general rule, the more narrow the range of particle sizes of granular materials, the less the permeability is influenced by density.

c. Effect of average grain size. Equation 4-1 suggests that permeability varies with the square of some particle diameter. It is logical that the smaller the particles, 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.

d. Effect of structure and stratification. Generally, in situ soils, water deposited soils, and windblown sand and silts present variations in structure and stratification. Therefore, an understanding of the methods of formation of soils aids in evaluating their engineering properties.

e. Effect of discontinuities. The permeability of many formations is established almost entirely by the discontinuities and in such formations tests on individual samples may be very misleading. The presence of holes, fissures, and voids due to frost action, alternate wetting and drying, and the effects of vegetation and small organisms may change even the most impervious clay into a porous material. While this does not affect most problems in the field of earthwork and foundation engineering, it is of importance in soil studies for drainage purposes.

f. Effect of entrapped air in water or voids. Small quantities of entrapped gas have a marked effect on the coefficient of permeability. Therefore, for correct test data the gas content during the tests should be equal to the gas content that occurs in the natural soil or is likely to occur in the future in the natural soil.

g. Effect of saturation. The degree of saturation of a soil has an important influence on its permeability. The higher the degree of saturation, the higher the permeability.

h. Effect of the fine soil fraction. 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 stabilized materials, depends principally upon the percentage by weight of particles passing the No. 100 sieve. The permeability is reduced more than three orders of magnitude as the percentage by weight of fine particles smaller than the No. 100 sieve is varied from 0 to 7 percent.

4-2. Drainage of water from soils. The quantity of water removed by a drain varies depending on the type of soil and location of the drain with respect to the ground water table. All of the water in soil cannot be removed by gravity drainage methods.

a. Effective porosity. The permeability of a specimen gives no indication of the total volume of water that can be drained from the material. Not all of the water contained in a given specimen can 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 (n_e), 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 and can be expressed mathematically as in the following equation:

$$n_e = 1 - \frac{\gamma_d}{G_s \, \gamma_w} (1 + G_s w_e)$$

where:

γ_d = dry density of the specimen
G_s = specific gravity of solids
γ_w = unit weight of water
w_e = effective water content (after the specimen has drained water to a constant weight) 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 soils such as medium or coarse sands may have an effective porosity of not more than 0.25.

b. Coefficient of permeability of base and subbase materials. Base and subbase materials used immediately beneath pavements generally consist of sand and gravel, sand, crushed rock, and partially crushed gravel and sand, but may consist of slag, cinders, natural subgrade, etc. In many cases, the base and subbase will consist of several layers, each of different material. The coefficient of permeability of sand and gravel courses, graded between limits usually specified for base and subbase materials, depends principally upon the percentage by weight of sizes passing the No. 200 mesh sieve. The following tabulation may be used for preliminary estimates of average coefficients of permeability for remolded samples of these materials:

Coefficient of permeability for remolded samples

Percent by weight Passing 200-mesh sieve	cm/sec	fpm
3	0.51×10^{-1}	10^{-1}
5	0.51×10^{-2}	10^{-2}
10	0.51×10^{-3}	10^{-3}
15	0.51×10^{-4}	10^{-4}
25	0.51×10^{-5}	10^{-5}

The coefficient of permeability of crushed rock and slag, each without many fines, is generally greater than 0.5 cm/seconds. The coefficient of permeability of sands and sand and gravel mixtures may be approximated from figure 4-2.

c. Horizontal permeability. The coefficient of permeability of a base course in a horizontal direction (parallel to compaction planes) may be 10 times greater than the average value tabulated above. For uniformly graded sand bases, the coefficient of permeability in a horizontal direction may be about 4 times greater than the value determined by tests on remolded samples. Very pervious base materials, such as crushed rock or slag with few fines, have essentially the same permeability in the vertical and horizontal directions. When more than one material is used for the base and subbase, the weighted coefficient of horizontal permeability, determined in accordance with the following equation:

$$k = \frac{k_1 d_1 + k_2 d_2 + k_3 d_3 + \ldots}{d_1 + d_2 + d_3 \ldots}$$

where:

k = weighted coefficient of horizontal permeability
k_1, k_2, k_3 . . . = coefficients of horizontal permeability on individual base materials
d_1, d_2, d_3 . . . = thickness of the individual layers

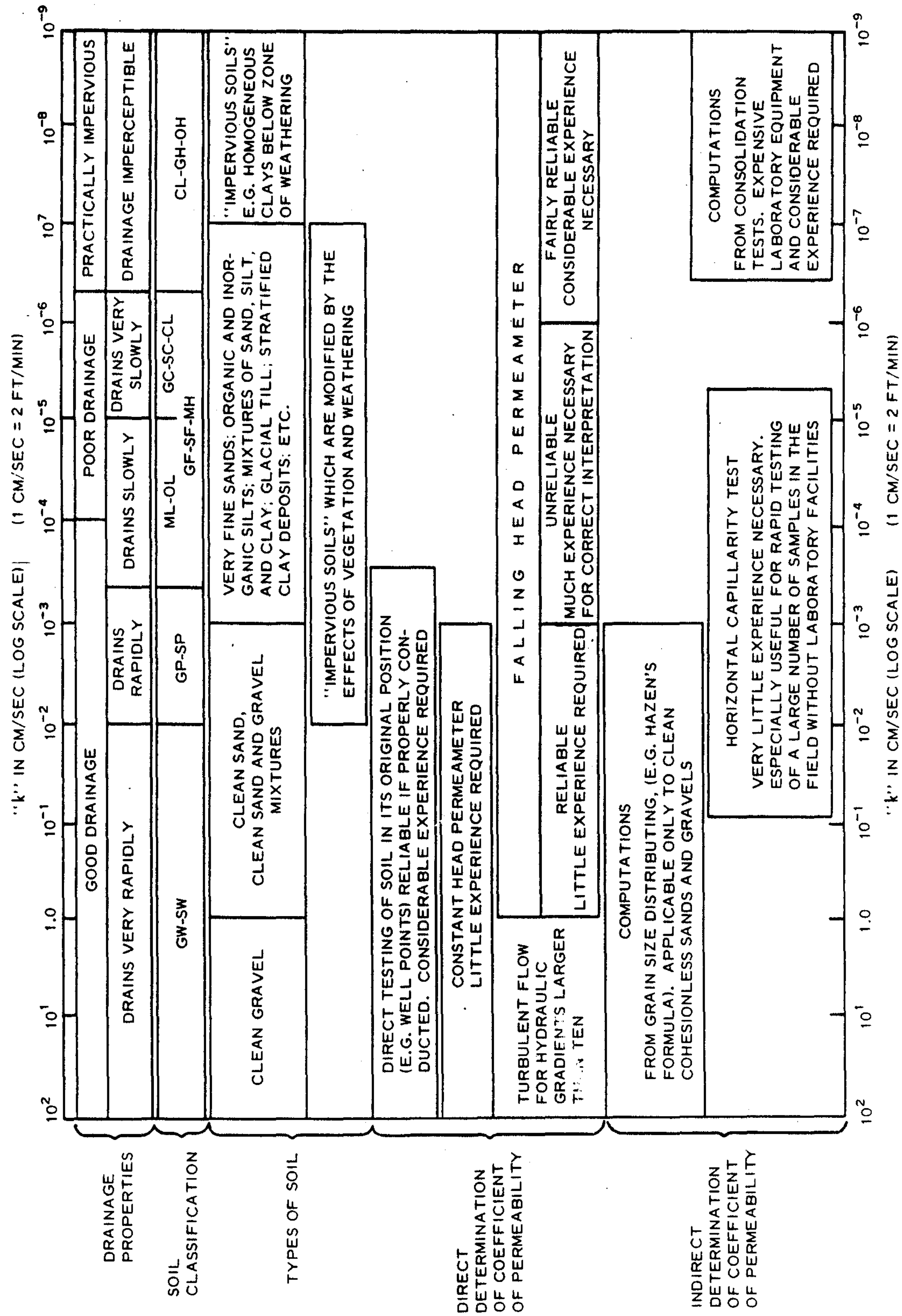


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FIGURE 4-2. PERMEABILITY CHART

d. Time for drainage. It is desirable that the moisture be drained from the base and subbase layers as rapidly as possible. Base- and subbase-course design should be based on the criterion that a degree of drainage of 50 percent in the base course should be obtained in not more than 10 days. Degree of drainage is defined as the ratio, expressed as a percent, of the amount of water drained in a given time to the total amount of water that is possible to drain from the given material. The following equation may be used to determine the time required for a saturated base course to reach a degree of drainage of 50 percent:

$$t = \frac{n_e D^2}{2880\ kH_o}$$

where:

t = time in days for 50 percent drainage
n_e = effective porosity of the soil
k = coefficient of permeability of the soil parallel to direction of seepage flow, fpm
D and H_o = base- and subbase-course geometry dimensions (illustrated in fig 4-3), feet

Since the time required to drain horizontal layers is a function of the square of the length of the flow path, it is important that the flow paths be kept as short as possible.

e. Continuity. If accumulations of water within pavement structural sections and in all parts of the drainage systems are to be prevented, the potential for drainage should increase in the direction of flow from points of entry through base drainage layers and through collector pipes and outlet pipes. This principle, which may be called a condition of continuity, insures the free discharge of water through all component parts of a subsurface drainage system.

f. Drainage boundaries. Pavements are wide flat areas with large flat expanses exposed to surface infiltration. When flow is vertical, as it is through upper surfaces of pavements, the hydraulic gradient is essentially 100 percent or 1.0, and the entire surface area is a potential source of inflow. So, inflow potentials are relatively large. But when flow is horizontal through base courses or drainage layers, the area available to drain the water is quite small and limited to the thickness of the layer, and the hydraulic gradients are limited to small values. As a consequence, to maintain continuity of drainage, basecourse layers need to be considerably more permeable than the surfaces through which water is permitted to enter.

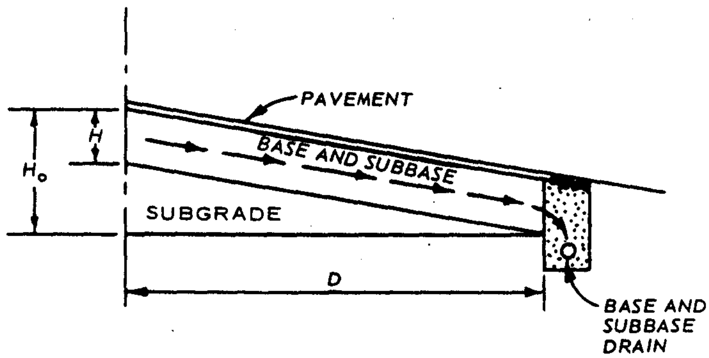

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FIGURE 4-3. DESIGN OF BASE- AND SUBBASE-COURSE DRAINAGE

CHAPTER 5

BACKFILL FOR SUBSURFACE DRAINS

5-1. General. Placing backfill in trenches around drain pipes should serve a dual purpose: it must prevent the movement of particles of the soil being drained, and it must be pervious enough to allow free water to enter the pipe without clogging it with fine particles of soil. The material selected for backfill is called filter material.

5-2. Empirical design of filter materials. The criterion for filter and pipe perforations to keep protected soil particles from entering the filter or pipe in significant quantities is based on backfill particle sizes.

a. Filter stability criteria. The criteria for preventing movement of particles from the protected soil into or through the filter or filters are:

$$\frac{\text{15 percent size of filter material}}{\text{85 percent size of protected soil}} \leq 5$$

and

$$\frac{\text{50 percent size of filter material}}{\text{50 percent size of protected soil}} \leq 25$$

b. Exceptions to filter stability criteria. The above criteria will be used when protecting all soils except for nondispersive medium to highly plastic clays without sand or silt particles, which by the above criteria may require multiple-stage filters. For these clay soils, the D_{15} size of the filter may be as great as 0.4 millimeters, and the above D_{50} criteria will be disregarded. This relaxation in criteria for protecting medium to highly plastic clays will allow the use of a single-stage filter material; however, the filter must be well graded, and to offset the tendency of segregation of the filter material, a coefficient of uniformity not greater than 20 will be required. For dispersive clays, filter tests will be conducted to evaluate the effectiveness of the proposed filter material. Graded filters are labor intensive operations; therefore, the use of fabric filters (geotextiles) should be encouraged.

c. Filter permeability criteria. To permit free water to reach the pipe and maintain the condition of continuity, the filter material must be many times more pervious than the protected soil. It has been found that this condition is fulfilled when the following requirements are satisfied.

$$\frac{\text{15 percent size of filter material}}{\text{15 percent size of protected soil}} \leq 5$$

d. Criteria for slots and holes. When pipes are embedded in filter drains, no unplugged ends should be allowed and the filter material in contact with pipes must be coarse enough not to enter the perforations or openings. To prevent clogging of the pipe with filter material, the following limiting requirements must be satisfied:

$$\text{for slots} \qquad \frac{\text{85 percent size of filter material}}{\text{slot width}} > 1.2$$

$$\text{for circular holes} \qquad \frac{\text{85 percent size of filter material}}{\text{hole diameter}} > 1.0$$

e. Criteria for porous concrete pipe and filter cloth. At present, established criteria for the design of granular filters adjacent to porous concrete pipes or for material being drained through filter cloths are not available. In the absence of such criteria, it is suggested that the following requirements be observed: For porous concrete pipe:

$$\frac{\text{15 percent size of aggregate in porous pipe}}{\text{85 percent size of filter adjacent to porous pipe}} \leq 5$$

For filter cloth (subject to limitations stated in para 6-9):

$$\frac{\text{85 percent size of material adjacent to cloth}}{\text{equivalent size of cloth openings}} \geq 1$$

A uniform, medium to fine sand (SP), graded such that at least 15 percent will pass the filter cloth, is washed over the cloth. A sieve analysis of the sand retained on the cloth is made. The 15 percent size of the retained material is used as the equivalent size of cloth openings. The test should be performed in the following manner. A piece of filter cloth is cut to fit in a No. 10 (or coarser) US Standard Sieve, and the perimeter of the cloth is sealed to the sieve wall with paraffin or other suitable sealant. Approximately 150 grams of sand is then placed on the filter cloth and washed with a water spray for a period of at least 20 minutes.

f. Limiting statement for filter cloths. The above tentative criteria for filter cloths are based on very limited laboratory tests. Adequate field performance data on filtering capabilities, ability to resist puncture or tearing when used adjacent to angular gravel, and resistance to deterioration by incrustation or chemical attack are not yet available for the different filter cloths on the market. Desirable procedures have not yet been established for specifications for and acceptable testing of filter cloths and factory- or field-made seams. Consequently, judgment is required in preparing specifications for the particular use of filter cloth contemplated.

g. Criteria for skip- and bell-and-spigot-type pipe. The criteria recommended for slots should be used for skip- and bell-and-spigot-type pipe. To preclude infiltration of filter material into the pipe, ratios somewhat greater than those above should be used wherever practicable to allow for variation in material and segregation during placement.

h. Granular filter materials. From the standpoint of simpler construction and lower costs, the selection of a filter material should be made from one layer wherever possible. If several layers of filter material are required, each layer must be designed in accordance with the criteria stated in this section. A typical example of design is shown in figure 5-1. The sand illustrated by the band contains sufficient amounts of fine gravel sizes to be safe from infiltration into pipes with small openings.

i. Filters in frost-susceptible areas. In determining a suitable filter material, the gradation of filters within the zone of frost penetration should be examined with respect to frost susceptibility. For the design of filters in frost-susceptible areas, consult EM 1110-3-138.

5-3. Construction of filter material. The major difficulties in construction of the filter are the problem with compaction in a restricted working space and the tendency toward segregation of particles. A material with a high coefficient of uniformity will tend to segregate during placement; therefore, a coefficient of uniformity greater than 20 is usually not desirable. For the same reason, filter materials should not be skip-graded. Segregation of coarse particles will result in the formation of voids through which fine particles may wash from the subgrade material. Segregation can best be prevented by placement of the material in a moist state. Moist placement of the sand may cause bulking of the sand particles. The use of water during installation of the filter material will collapse the structure of the bulked sand, therefore aiding in compaction and forming satisfactory transition zones between the various materials. The use of polyester fabric filter has proven successful as a filter material, but acceptance when frost susceptable soils are present is questionable. When polyester fabric filter is considered where frost susceptible soils exist, subgrades should be scarified and blended to reduce differential heave.

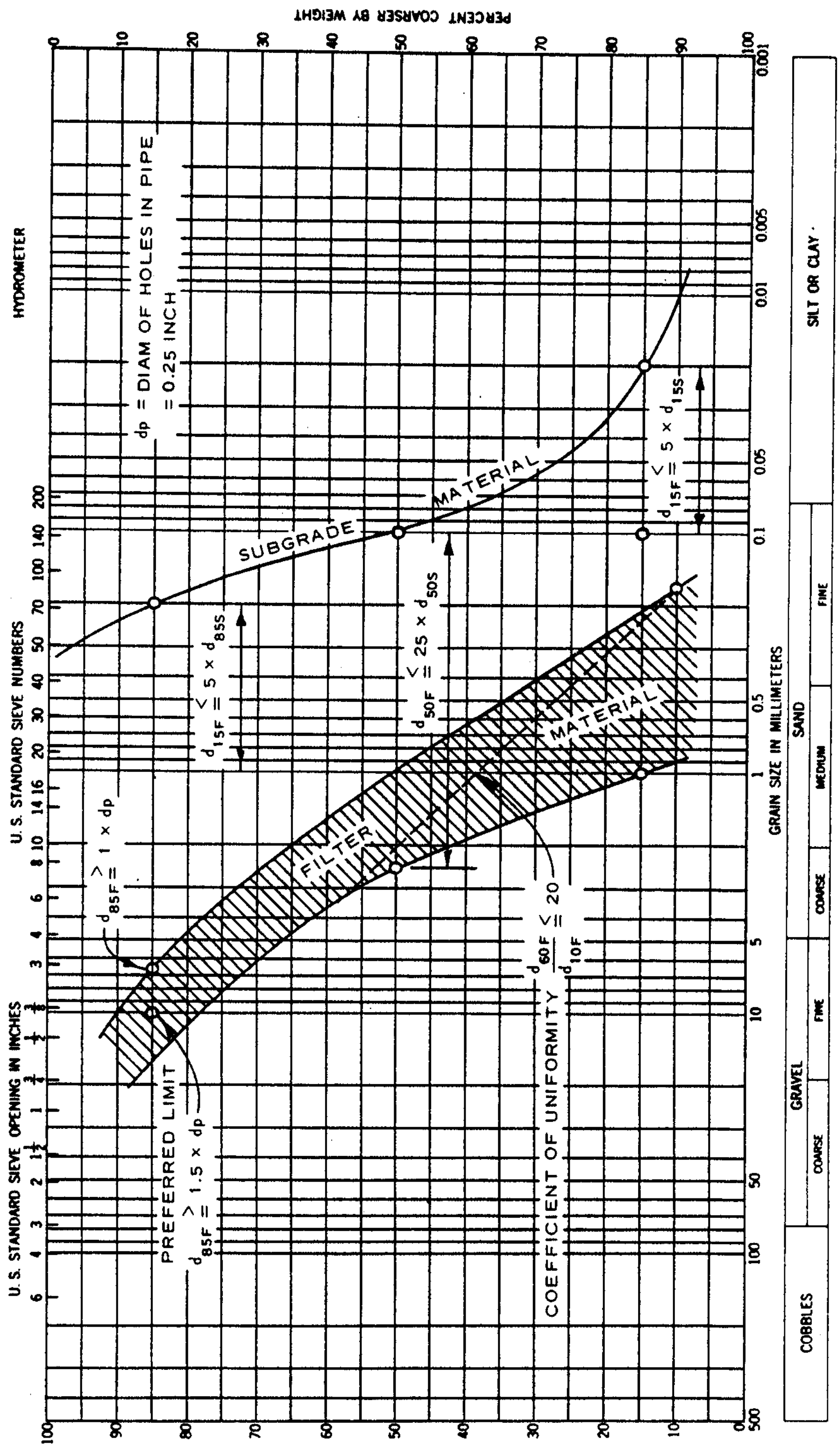

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FIGURE 5-1. DESIGN EXAMPLE FOR FILTER MATERIAL

CHAPTER 6

SUBSURFACE DRAINAGE PROCEDURES

6-1. Purpose. Subsurface drainage may be divided according to its purpose into base- and subbase-course drainage, subgrade drainage, and intercepting drainage. Base and subbase drainage consists of removing water from a base and/or subbase course beneath a pavement; subgrade drainage consists of removing water from the subgrade beneath a pavement; and intercepting drainage consists of collecting and removing water under artesian pressure found flowing in pervious foundation strata or water flowing from springs.

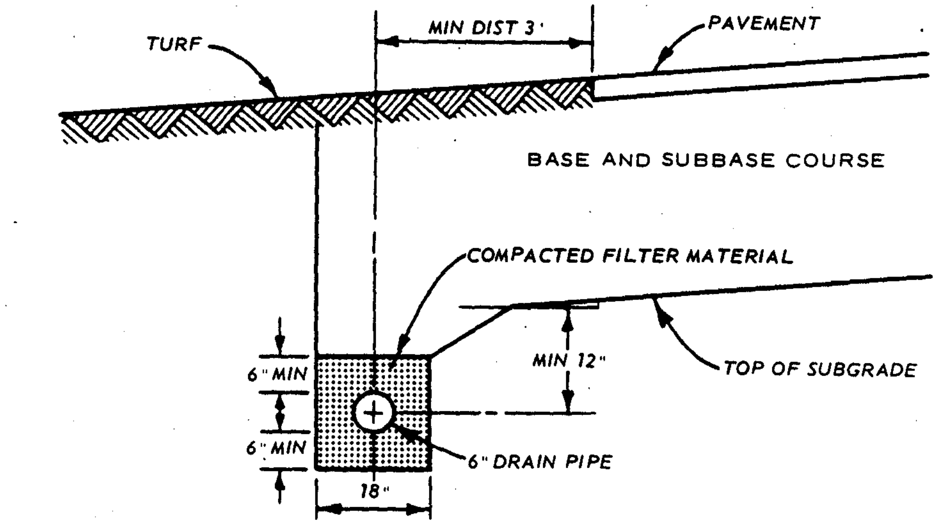
6-2. Methods. The drainage of subsurface water may be affected by a system of subsurface drainpipes placed in trenches or by circumferential or intercepting open ditches. The backfill filter for pipes in trenches should be designed to maintain progressively greater outflow capabilities in the direction of flow. The outlet for subsurface drains should be properly located or protected to prevent backwater from the surface drainage system from entering the subsurface system. In addition, there are special methods for subsurface drainage, such as dry wells that are designed to drain a perched-water table into a lower ground water reservoir.

6-3. Base- and subbase-course drainage. Base- and subbase-course drainage generally consists of subsurface drainpipes laid parallel and adjacent to pavement edges with pervious backfill material connecting the base and subbase course to the drain. The top of the subgrade beneath paved shoulder areas should be sloped to provide drainage to subsurface drainage pipelines. Additional lines of pipe may be required beneath large paved areas with relatively flat slopes to obtain adequate base- and subbase-course drainage. A sketch of a typical base and subbase drain is shown in figure 6-1.

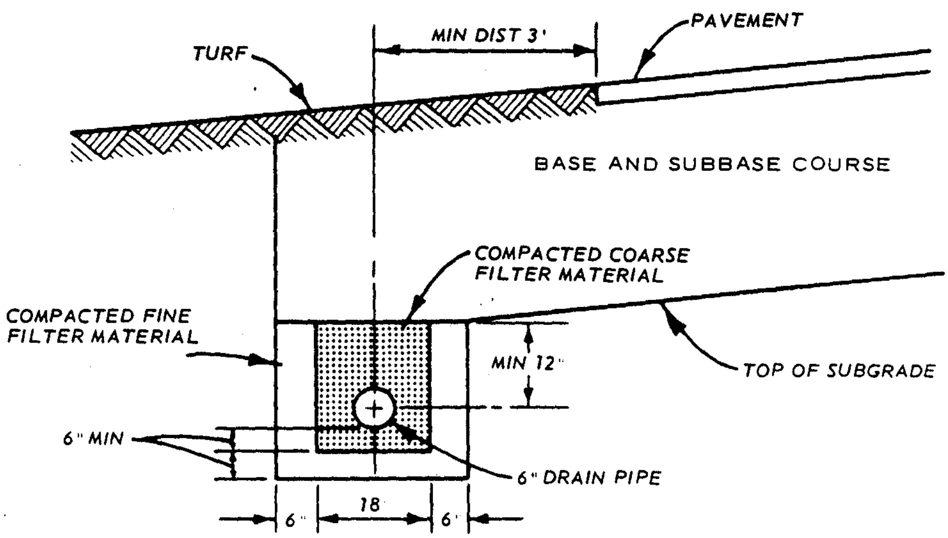
a. Ground water. Base and subbase drainage is also required where the ground water rises to the bottom of the base or subbase course as a result of either seasonal conditions, ponding of surface runoff, or consolidation of soil under the weight of the base and subbase course.

b. Design of base- and subbase-course drainage. To simplify the analysis of drainage, it is assumed that the base and subbase courses are fully saturated and no inflow occurs during drainage. It is assumed the subgrade constitutes an impervious boundary and the base and subbase courses have a free outflow into the drain trench.

(1) Maximum rate of discharge. The following equation may be used to determine the maximum rate of discharge for a saturated base and subbase course of dimensions shown in figure 4-3.



a. ONE GRADATION OF FILTER MATERIAL



b. TWO GRADATIONS OF FILTER MATERIAL

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FIGURE 6-1. TYPICAL DETAILS OF BASE- AND SUBBASE-COURSE DRAIN INSTALLATIONS

$$q = \frac{kHH_o}{60D}$$

where:

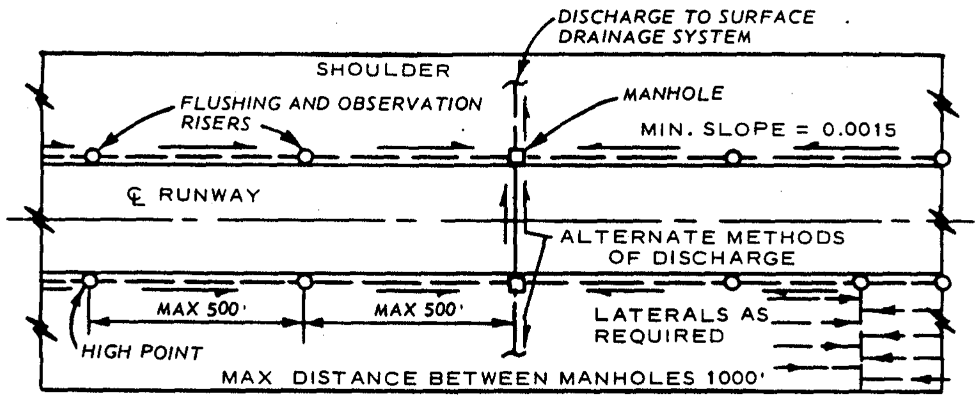
q = peak discharge quantity of drain, cfs/lin feet
k = coefficient of horizontal permeability, fpm
H, H_o, and D = dimensions as shown in figure 4-3, feet

(2) Spacing of drains. When the time in days determined using equation 4-2 is greater than 10 days, then the spacing between drains should be decreased until the time for drainage is 10 days or less, or a more pervious base and subbase material should be selected, or a greater thickness of base and subbase should be used in the design. For most runways and taxiways of widths from crown to edge of not more than 75 feet, a single line of base and subbase drains along the edges should meet the design criteria. It may be necessary on wider pavement widths, or where reasonably pervious base- and subbase-course materials are not locally available, to install intermediate lines of drains to provide satisfactory base and subbase drainage.

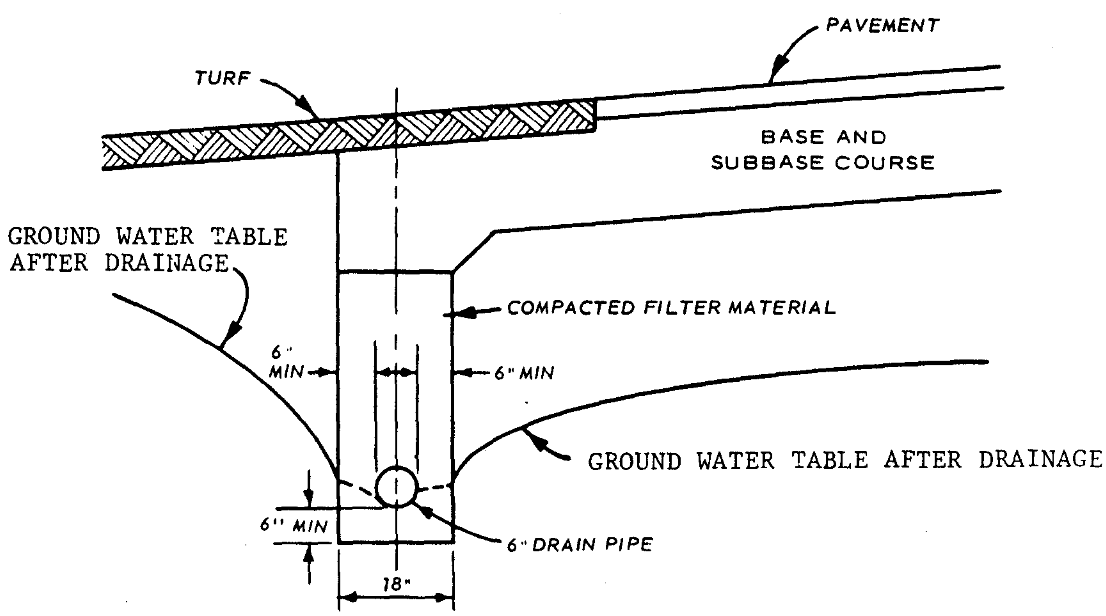
6-4. Subgrade drainage. Subgrade drainage is provided primarily to drain subsurface waters from a subgrade in which there exists a high ground water table. These drains generally consist of either a system of subsurface drainpipes or a system of open ditches. The type, location, depth, and spacing of drains depend upon the soil characteristics and depth to ground water table. Subgrade drains may also drain base and subbase courses. Sketches of a typical subgrade drainage installation and layout using pipes are shown in figure 6-2.

a. Depth and spacing of drains. Subgrade drains should be placed at a depth of not less than 1 foot below the bottom of the base and subbase course and at a depth of not less than 1 foot below the ground water table (fig 6-2). Frequently, depth of cover will be controlled by frost conditions or loading requirements in accordance with paragraph 6-10. Subgrade drains are generally required only at pavement edges. Intermediate subgrade drains on large areas are necessary only where unusual ground water conditions exist.

b. Design of subgrade drainage systems. At locations where subgrade drainage is required as indicated in the above paragraph, and to simplify the analysis of drainage of subgrade materials, the following assumptions are made: the subgrade is saturated below the ground water table, infiltration has raised the ground water table in the shoulder area adjacent to a subgrade drain as shown in figure 6-3, no appreciable quantity of flow develops from the subgrade beneath the paved area, and the drains must have a capacity sufficient to collect the peak flow from the shoulder. This peak flow occurs immediately



a. SUBSURFACE DRAINAGE SYSTEM



b. CROSS SECTION OF SUBGRADE DRAIN

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FIGURE 6-2. TYPICAL SUBGRADE DRAINAGE DETAILS

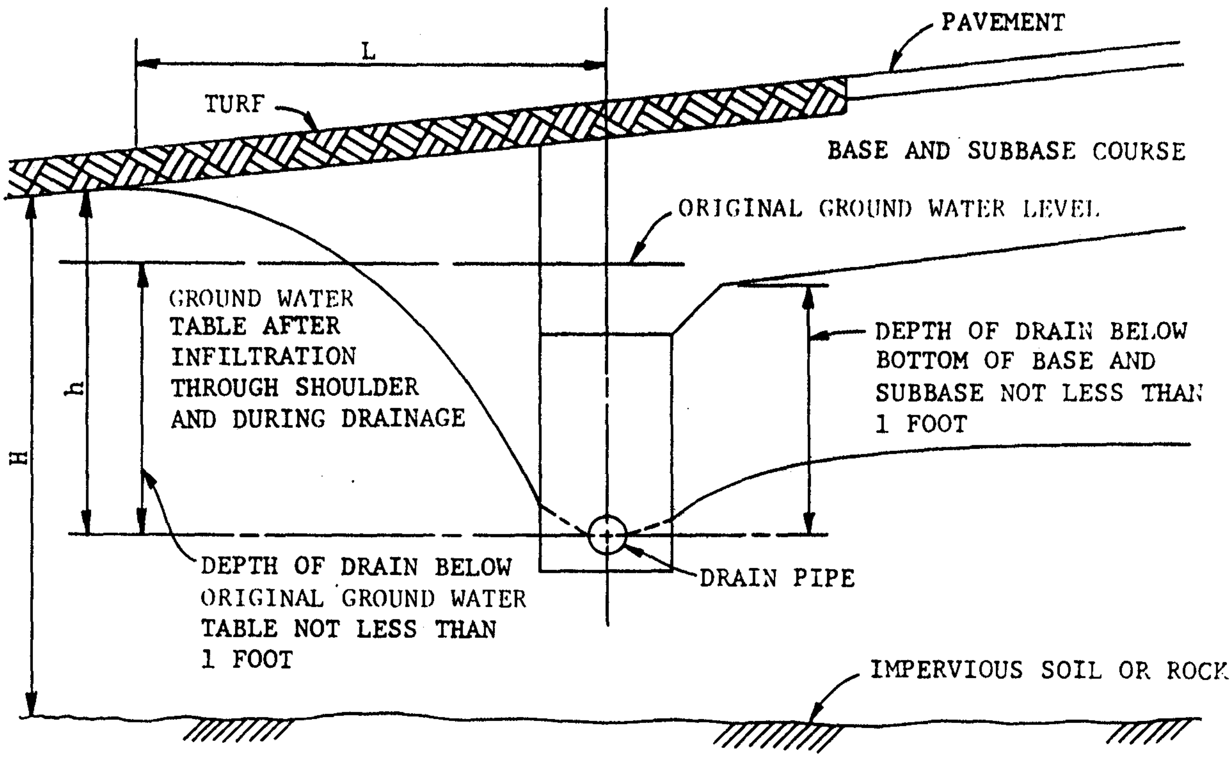

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FIGURE 6-3. GROUND WATER CONDITIONS AFTER INSTALLATION OF SUBGRADE DRAINAGE after the ground water table has risen to its maximum height, assumed to be as shown in figure 6-3.

The quantity of water discharged by the soil and collected by the drain may be determined using the following equation:

$$q = \frac{khc}{60}$$

where:

q = discharge quantity of drain, cfs/lin ft
k = coefficient of horizontal permeability of soil in the shoulder, fpm
h = difference in elevation between the midpoint of the pipe and the ground surface at L distance from drain as shown in figure 6-3, feet
c = shape factor dependent upon L and H, where H is the thickness in feet of the soil being drained as shown in figure 6-3. Determine c from figure 6-4 using L=50 for a k larger than 10^{-3}fpm

6-5. Intercepting drainage. Intercepting drainage is provided to intercept ground water flowing in a pervious shallow stratum toward a paved area or into the face of a cut. Intercepting drainage is also provided to collect water from springs in subgrade excavations. The type and depth of drains depend upon the soil and ground water conditions. These drains may consist of either subsurface drainpipe or ditches. A schematic of a typical pipe installation is shown in figure 6-5.

6-6. Type of pipe. Subsurface drainage systems may be constructed of various types of standard manufactured pipe. The type of pipe selected should satisfy local requirements such as the condition of the soil, the loading and amount of cover, the cost, and the availability of pipe. The following types of pipe are currently available: perforated, bell-and-spigot, cradle-invert (skip), porous concrete, bituminized fiber, farm tile, PVC, and corrugated polyethylene.

a. Perforated. Perforated pipes are usually laid with closed joints and with the holes down except where the holes are spaced all around the pipe. Perforated pipe is manufactured of vitrified clay, nonreinforced concrete, bituminous-coated or uncoated corrugated metal, bituminized fiber pipe, asbestos-cement, plastic, cast iron, PVC, and corrugated polyethylene.

b. Bell-and-spigot. Bell-and-spigot pipes are laid with open joints. Collars of burlap or roofing paper, or oakum in the joints are not required where the filter material is properly designed. The use

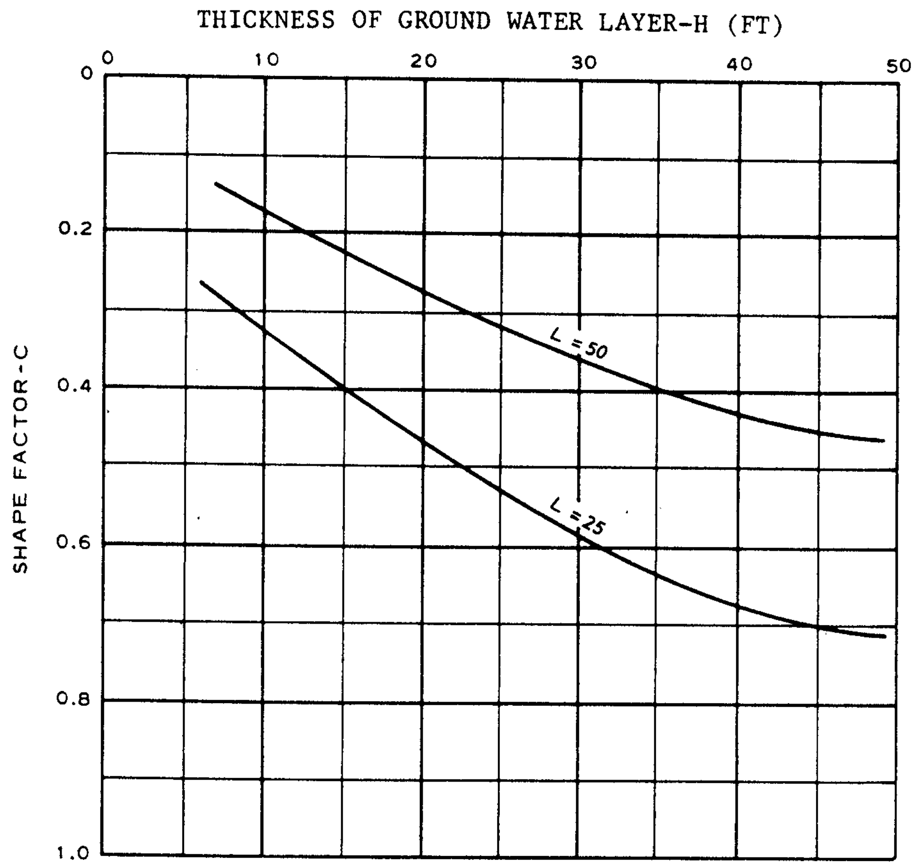

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FIGURE 6-4. THICKNESS OF GROUND WATER LAYER IN RELATION TO SHAPE FACTOR

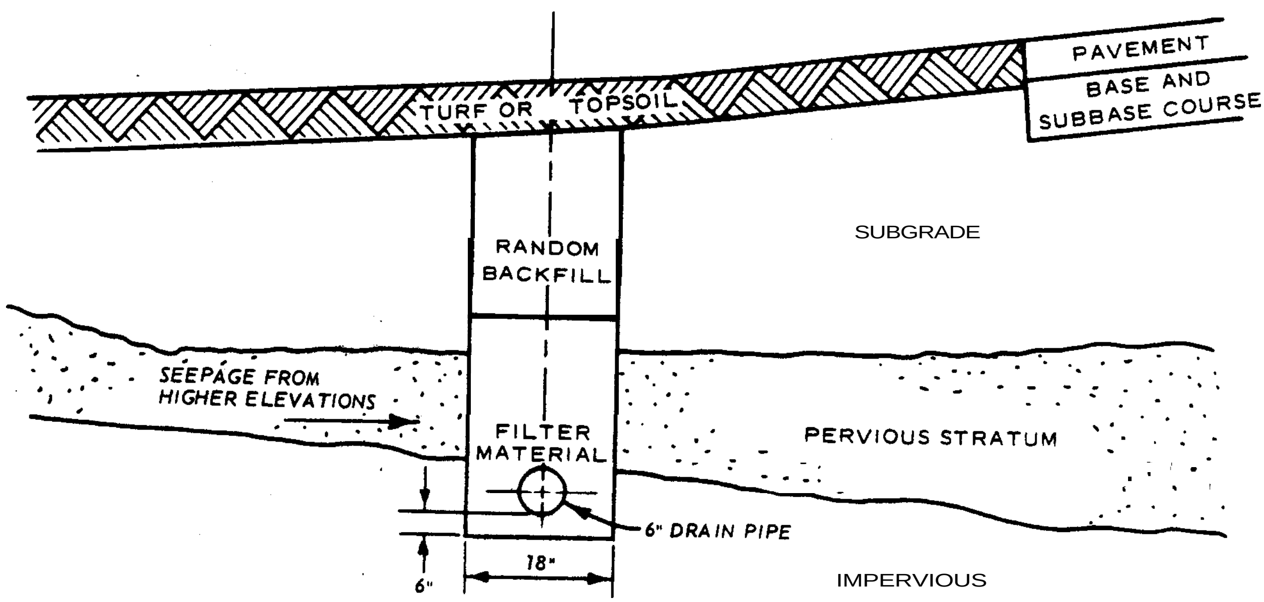

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FIGURE 6-5. TYPICAL INSTALLATION OF INTERCEPTING DRAINS of partially mortared joints is desirable to provide positive alinement and spacing of the pipe. Materials used in the manufacture of bell-and-spigot pipe are vitrified clay, nonreinforced concrete, and cast iron.

c. Cradle-invert. Cradle-invert (skip) pipe is manufactured of both vitrified clay and cast iron. The pipe is designed so that water enters the pipe at bell-and-spigot-type joints, which are designed to provide a gap around the bottom semicircumference, and through a slot across the flat top surface of the pipe. Cast-iron, cradle-invert (skip) pipe is provided with a device to hook adjoining sections together.

d. Porous concrete. Porous concrete pipe is laid with sealed joints. It collects water by seepage through the porous concrete wall of the pipe. Porous concrete pipe should not be used where chemically active waters may cause disintegration of concrete. When severe attack by chemically active ground water is anticipated, sulfate-resistant cement should be specified for other types of concrete pipe which might be considered. Criteria for granular filters and fabric filters used adjacent to porous concrete pipe are presented in paragraph 5-2e.

e. Farm tile. Farm tile pipe is manufactured of both clay and concrete. It is laid with butt joints slightly separated to permit collection of water through joints. This type of pipe is not recommended for use on airfields.

f. Plastic pipe. Criteria for the use and placement of PVC, corrugated polyethylene, and ABS pipe are to be as recommended by the manufacturer.

6-7. Manholes and observation basins. Manholes, observation basins, and risers are installed on subsurface drainage systems for points of access to the system to observe its operation and to flush or rod the pipe for cleaning. Manholes on base- and subbase-course or subgrade pipe drains should be at intervals of not over 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 on Standard Mobilization Drawing No. XEC 001.

6-8. Pipe sizes and slopes. Except for long intercepting lines and extremely severe ground water conditions, 6-inch-diameter drains are satisfactory for all subsurface drainage installations. The rate of infiltration into the drains, computed as specified in paragraph 6-4b of this manual, is used to determine whether the expected total flow in the intercepting drains may require pipe larger than 6 inches.

a. Sizes. The nomograph shown in figure 6-6 may be used for the design of drainpipes for subsurface drains. The values to be used for the coefficient of roughnesss "n" are as follows:

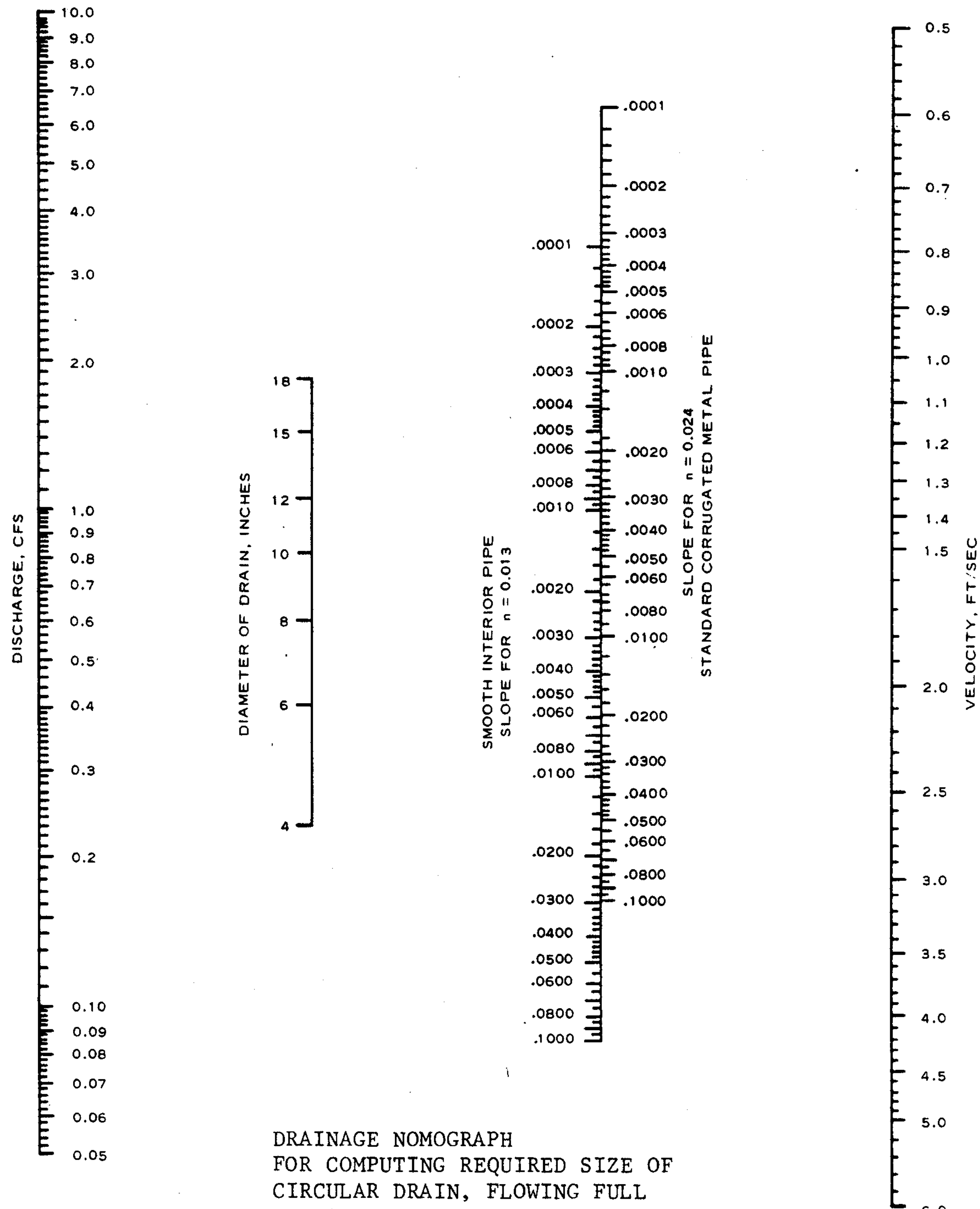
Type of pipe	Coefficient of roughness, n
Clay, concrete, bituminized fiber, plastic, and asbestos-cement pipe	0.013
Bituminous-coated or uncoated corrugated-metal pipe	0.024

The values of any two unknowns in the nomograph may be determined by connecting two known values with a straightedge.

b. Slopes. The recommended minimum slope for subdrains is 0.15 foot in 100 feet.

6-9. Filter material. A minimum thickness of 6 inches of granular filter material should be provided around all types of subsurface drains. Gradation of the granular filter material should be designed with respect to the soil being drained to conform with pertinent provisions of paragraph 5-2. When this gradation is such that it does not satisfy the provision pertaining to material adjacent to joint openings or pipe perforations, a single wrap of filter cloth around the pipe may be used in lieu of a second filter material. Fabric filters are manufactured of plastic materials in both woven and unwoven type. Size of openings vary widely from No. 30 sieve to No. 100 sieve or even finer. Normally fabric filters of sieve sizes between No. 80 and No. 100 are used. The fabric may be wrapped around open joints of unperforated pipe or around the entire length of perforated or unperforated pipe. Prefabricated fabric filter sleeves may also be used to encase the pipe. When the gradation of filter material is such that it satisfies provisions pertaining to material adjacent to joint openings or pipe perforations, but is too coarse to satisfy the filter criteria pertaining to the protected soil, a single layer fabric filter may be used adjacent to the protected soil in lieu of a second filter material. This use of fabric filters is restricted to situations where the soil to be protected is sand (SW, SP, SW-SM) with less than 85 percent passing a No. 200 sieve. When fabric filters are used, requirements stated in paragraph 5-2 pertaining to the adjacent granular material should be satisfied.

6-10. Depth of cover over drains. The cover over drains is dependent upon loading and frost requirements. Cover requirements for different design wheel loads are indicated in paragraph 8-5 of this manual. With respect to frost in areas of seasonal freezing, the depth of cover to the center line of pipe should be not less than the depth of frost penetration as determined from table 8-4 of this manual. The trench



DRAINAGE NOMOGRAPH
FOR COMPUTING REQUIRED SIZE OF
CIRCULAR DRAIN, FLOWING FULL
n = 0.013 AND n = 0.024

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FIGURE 6-6. DRAINAGE NOMOGRAPH

for subdrains should be backfilled with free draining, non-frost-susceptible material. Within the depth of frost penetration, gradual transitions should be provided between nonfrost-susceptible trench backfill and frost-susceptible subgrade materials around drains placed beneath pavements. This precaution will prevent detrimental differential heave, particularly for the case of frost condition pavement design based on reduced subgrade strength.

PART THREE

SURFACE DRAINAGE

CHAPTER 7

SURFACE DRAINAGE REQUIREMENTS

7-1. General.

a. Considerations. The primary consideration for a surface drainage system is storm drainage. Other considerations include snow melting and man-made water sources such as washracks and sprinkler systems. The governing factor for sizing the system will be storm intensity.

b. Natural drainage. The maximum use of natural drainage should be considered. Runoff from runways, roads, taxistrips, aprons, and railroads should be collected in open channels or ditches for removal from the immediate area to the greatest extent possible. The use of buried pipe should be kept to a minimum and "daylighted" to open channel drainage as soon as practical.

c. Area drains. The use of area drains and box inlets in paved areas should be minimized, and collection piping under slabs or pavement systems should be as short as possible.

d. Storm and sanitary sewer systems. Combined storm and sanitary sewer systems should not be used.

7-2. Design storm.

a. Intensity-frequency data. Studies of rainfall intensity-frequency data indicate there is a fairly consistent relation between the average intensities of rainfall for a period of 1 hour and the average intensities at the same frequency for periods less than 1 hour, regardless of the geographical location of the stations. The average rainfall for a 1-hour period at various frequencies for the continental United States, and Alaska, may be determined from figure 7-1. A 10-year frequency and a 1-hour rainfall intensity is considered the design storm index for mobilization conditions.

b. Standard rainfall intensity-duration curves. Figure 7-2 shows the standard curves which have been developed to express the rainfall intensity duration relationships which are satisfactory for the design of drainage systems. 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 required for a 40-minute design storm based on a 10-year frequency in central Kentucky is found to be 2.0 in/hr. In figure 7-2, supply curve 2.0 is used with

10-YEAR 1-HOUR RAINFALL (INCHES)

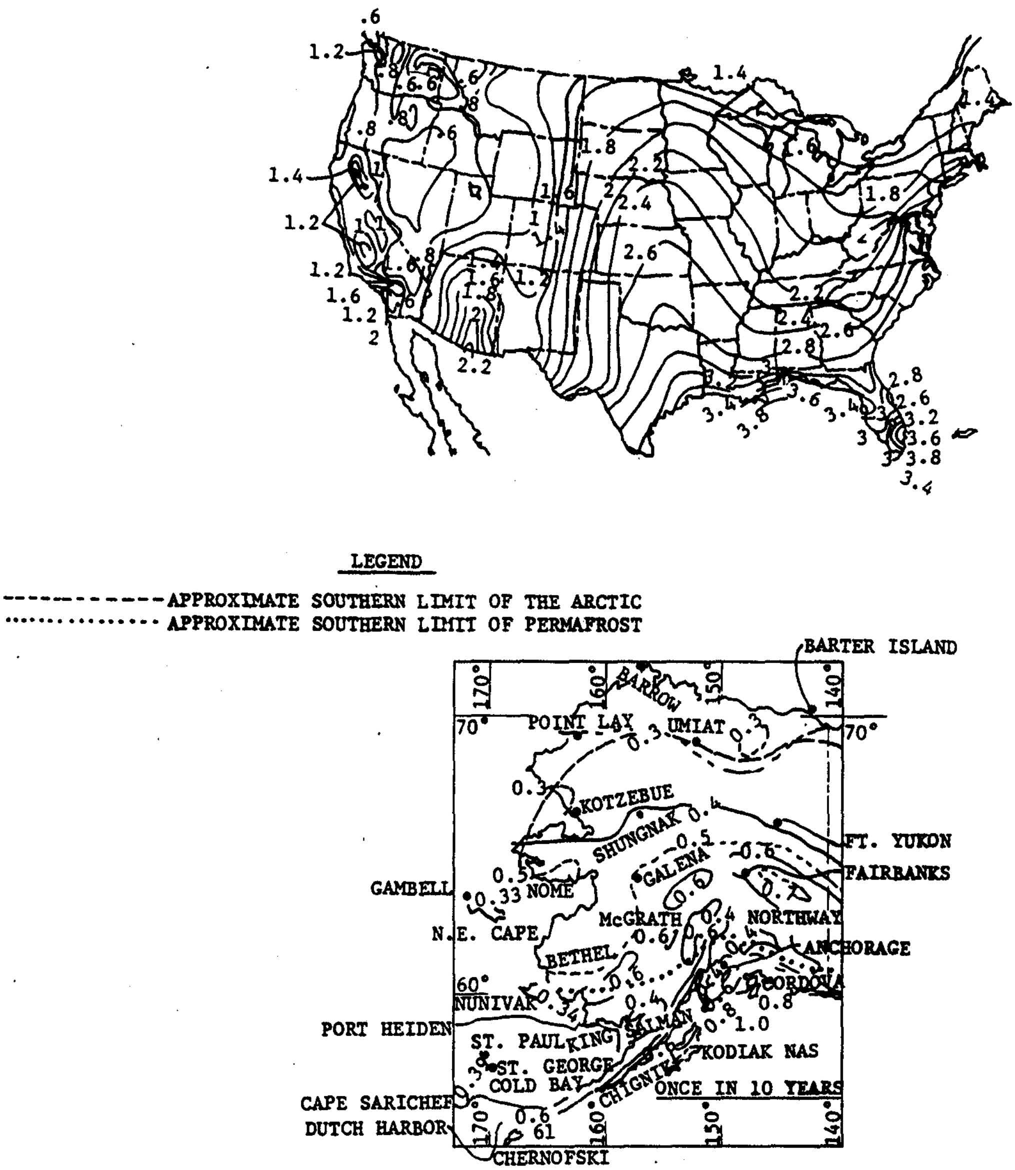
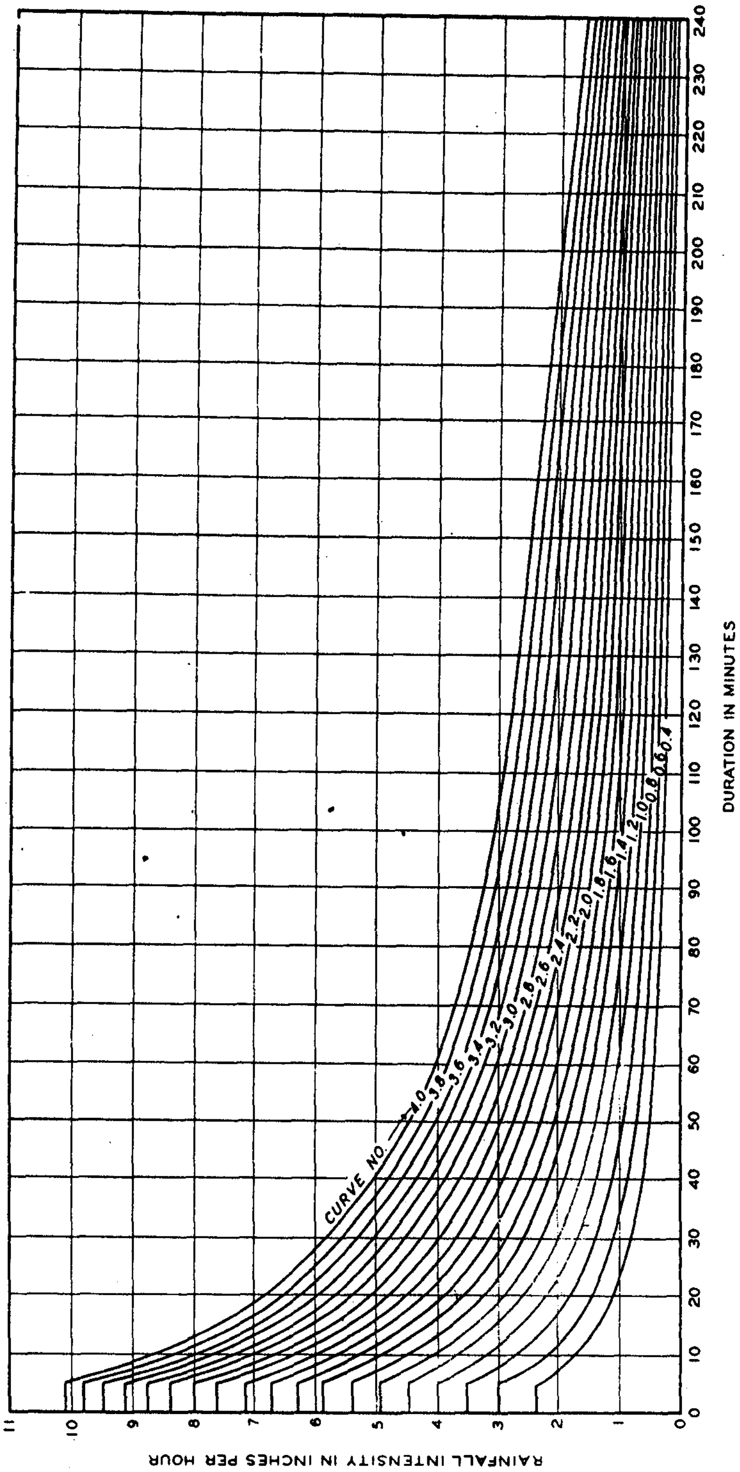


CHART REPRODUCTION FROM U.S. WEATHER BUREAU TECHNICAL PAPER NO. 40, RAINFALL FREQUENCY ATLAS OF THE UNITED STATES, WASHINGTON, D.C., MAY 1961.

FIGURE 7-1. DESIGN STORM INDEX - 1 HOUR RAINFALL INTENSITY FREQUENCY DATA FOR CONTINENTAL UNITED STATES AND ALASKA



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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FIGURE 7-2. STANDARD RAINFALL INTENSITY DURATION CURVES OR STANDARD SUPPLY CURVES the 40-minute duration of storm to determine a rainfall intensity of 2.7 in/hr.

7-3. 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-hour period of antecedent rainfall. Wherever possible determine average infiltration rates from a study of runoff records near the area in question from infiltrometer studies or from similar acceptable information. Suggested mean values of infiltration for generalized soil classifications are shown in table 7-1. The soil group symbols are those given in Unified Soil Classification System for Roads, Airfields, Embankments, and Foundations.

Table 7-1. Infiltration Rate for Generalized Soil Classifications (Uncompacted)

Description	Soil Group Symbol	Infiltration, in/hr
Sand and gravel mixture	GW, GP SW, SP	0.8-1.0
Silty gravels and silty sands to inorganic silt, and well-developed loams	GM, SM ML, MH 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

Infiltration values are for uncompacted soils. Where soils are compacted, infiltration values decrease; 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-hour of antecedent 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.

7-4. 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.

a. 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 7-2. 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 7-2. For example, if Supply Curve 2.2 in figure 7-2 were selected as the design-storm and the infiltration loss during a 1-hour storm were estimated as 0.6 inch, curve 1.6 would be adopted as the standard supply curve for the given areas.

b. 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.

7-5. Runoff.

a. Limitations. For areas of up to about 1 square mile, and where only peak discharges are required for design and extensive ponding is not involved, computation of runoff will be accomplished by the so-called rational method. For larger areas, the overland flow method should be used.

b. Rational method. In computing runoff by the rational method, use the empirical formula:

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

where:

Q = peak runoff in cubic feet per second.

C = a coefficient expressing the percentage to which the peak runoff is reduced owing to transitory storage. Its value depends primarily on the general slope and surface irregularity of the tributary area. Accurate determinations of C from available data are not readily made. For most areas, the apparent values range from 0.6 to 1.0. A value of 0.6 may be assumed applicable to areas left ungraded where meandering-flow and appreciable natural-ponding conditions exist, slopes are 1 percent or less, and vegetative cover is relatively dense. A value of 1.0 may be assumed applicable to smooth areas of substantial slope with virtually no potential for surface storage and little or no vegetative cover.

I = rainfall intensity in inches per hour for the most critical time of concentration and for the design storm frequency. Time of concentration is generally 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 feet before becoming channelized and thence moving relatively faster. 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 minutes; smaller values are rarely justifiable; and values up to 20 minutes may be used if resulting runoff excesses will not cause appreciable damage. A minimum time of 20 minutes 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 the design intensity and peak runoff would be effectively decreased. Then, that portion of the drainage area should be ignored and the peak flow computation should be for only the more efficient, downstream portion.

F = infiltration rate in inches per hour following a rainfall of 1 hour. Where F varies considerably within a given drainage area, a weighted rate may be used; it must be remembered, however, that pervious portions may require individual consideration, because a weighted overall value for F is proper for use only if rainfall intensities are equal to or greater than the highest infiltration rate within the drainage area.

A = drainage area in acres.

In Army construction drainage design, factors such as initial rainfall losses and channel percolation rarely enter into runoff computations involving the rational method.

c. 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. The peak rate is established after all parts of the drainage surface are contributing to runoff. The elapsed time for runoff to build to a peak is termed the time of concentration and it depends primarily on the surface characteristics as follows: the coefficient of roughness, the slope, and the effective length. When the supply terminates, the runoff rate begins to diminish but continues until the excess stored on the surface drains away.

(1) Effective length. The effective length 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 have indicated that by modifying the actual length, satisfactory reproduction of runoff hydrographs may be obtained regardless of channelization of flow. The values for effective length, L, is 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 7-3.

(a) 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. 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 7-3. 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 7-2.

Table 7-2. Coefficients of Roughness for Overland Flow

Surface	Value of n
Pavements and paved shoulders	0.01
Bare packed soil free of stone	0.10
Sparse grass cover, or moderately rough bare surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

(b) For example, to find the effective length of overland flow for an actual length of 900 feet on a sparse grass ground cover, n = 0.20, with an overall slope of 0.7 percent, use the following procedure. Divide the 900-foot actual length by the number 2 and enter

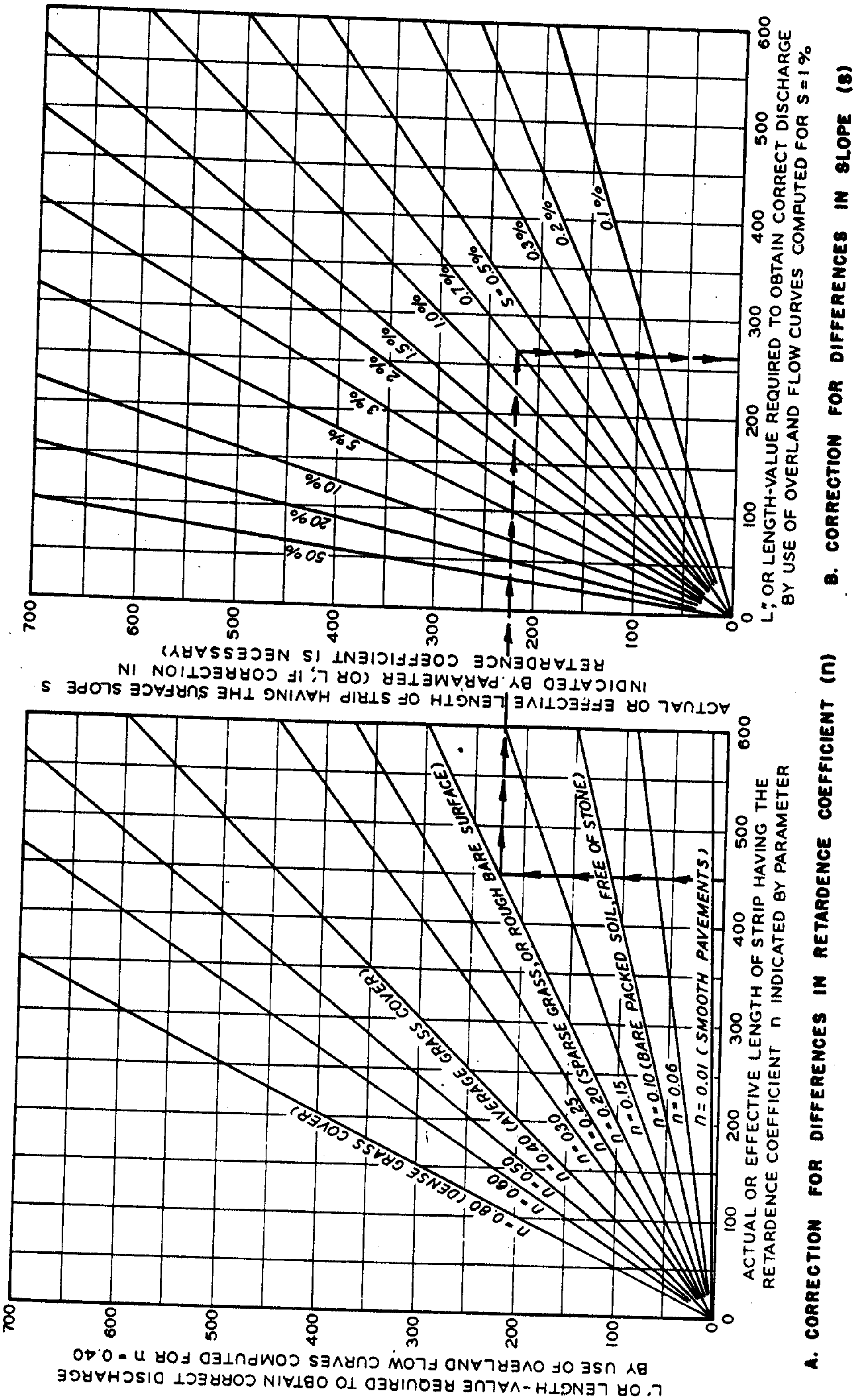

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FIGURE 7-3. MODIFICATION IN L REQUIRED TO COMPENSATE FOR DIFFERENCES IN n AND S graph A, figure 7-3 with 450 feet on the horizontal axis. Project a line vertically upward until it intersects the coefficient of roughness line; proceed horizontally to the intersection of the slope line equal to 0.7 percent on graph B, and proceed vertically down to obtain a length of 275 feet which must be multiplied by the number 2, resulting in a total effective length of overland flow of 550 feet.

(c) Distances across paved areas may be neglected when calculating effective length.

(2) Runoff rates. The peak runoff rates and critical duration of supply can be obtained from figure 7-4.

(a) The following example is provided to illustrate the use of figure 7-4. Assume an effective length of overland flow of 315 feet 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 line and proceed horizontally to the right to the critical duration of supply which, in this example, is 23 minutes. 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 per acre of drainage area.

(b) The total drainage flow is determined by multiplying the maximum rate from figure 7-4 by the contributing area (in acres).

7-6. Investigations to determine surface drainage requirements.

a. On-site investigation. 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 affected drains will be obtained. Topography, size, and shape of drainage area, and extent and type of areal 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 or perviousness, vegetative cover, depth to and movement of subsurface water, and depth of frost will be secured.

b. Maps, charts, photographs, and surveys. Maps and charts showing necessary detailed topography and other essential features of the areas to be drained and outlining the watershed area and subareas for determining runoff quantities will be prepared for layout and design. Aerial photographs including stereoscopic pairs will be used to the maximum practicable extent.

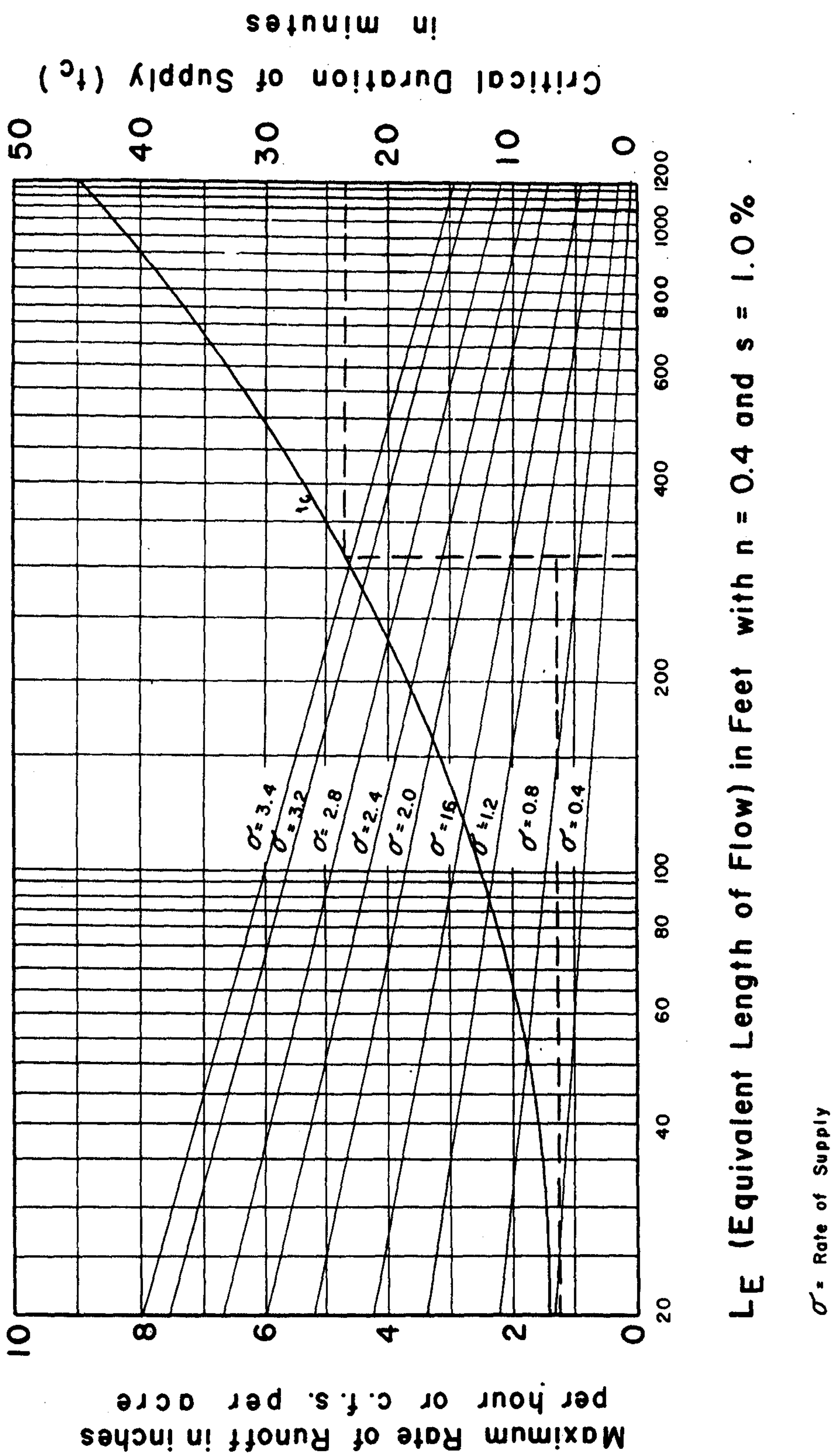

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FIGURE 7-4. RATE OF OVERLAND FLOW

c. Existing utilities. The location, type, size, elevations, and condition of existing utilities in addition to drains that may affect or be affected by the new drainage system will be determined.

d. Snowfall records. Snow cover depths and convertibility factors to inches of rainfall along with rainfall records must be obtained.

e. Runoff records. Runoff records for drainage areas in the same locality having similar characteristics and soil conditions should be utilized.

f. 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.

g. Soils investigation. Information gathering for soil investigations should be coordinated with subdrainage and other apsects of the overall design.

CHAPTER 8

DRAINAGE PIPE

8-1. General. A drainage pipe is defined as a structure (other than a bridge) to convey water through or under a runway, roadway, or some other obstruction. Materials for these installations include plain or nonreinforced concrete, reinforced concrete, clay, asbestos-cement, PVC, corrugated steel, corrugated aluminum, and corrugated polyethylene.

8-2. Selection of type of pipe.

a. Material selection. 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. Scarcity of materials under mobilization condition may require last minute substitution. The design should be flexible enough to readily accept substitutions. It is desirable to permit alternatives so that bids can be received with Contractor's options for the different types of pipe suitable for a specific installation. Where field conditions dictate the use of one pipe material in preference to others, the reasons will be clearly presented in the design analysis.

b. Design considerations. 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 effects on the pipe structure or on the joints or overlying materials, and cost of maintenance. It may be necessary to obtain an acceptable pipe installation to meet design requirements by establishing special provisions for several possible materials.

8-3. Selection of n values. Because of the temporary nature (5-year life expectancy) of the installation, "n" should tend toward new pipe values. Sedimentation or paved pipe can affect the coefficient of roughness. Table 8-1 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.

8-4. Restricted use of bituminous-coated pipe. Corrugated-metal pipe with any percentage of bituminous coating will not be installed where fuel spillage, wash rack waste, or solvents can be expected to enter the pipe.

Table 8-1. Roughness Coefficients for Various Pipes

Type of Pipe	n Valve Unpaved	n Valve 25% Paved
Smooth interior*	0.013	0.013
Annular Corrugated Metal		
Corrugation size		
2 + 2/3 by 1/2 inch	0.024	0.021
3 by 1 inch	0.027	0.023
6 by 2 inch	0.028 - 0.033	0.024 - 0.028
9 by 2 + 1/2 inch	0.033	0.028
Helical Corrugated Metal (2 + 2/3 by 1/2 inch corrugations)		
Pipe diameter		
12 - 18 inches	0.011 - 0.014	x
24 - 30 inches	0.016 - 0.018	0.015 - 0.016
36 - 96 inches	0.019 - 0.024	0.017 - 0.021

*Pipes of any size, shape or type including asbestos cement, bituminized fiber, cast iron, clay, PVC, concrete (precast or cast-in-place) or fully paved corrugated pipe.

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8-5. Minimum cover.

a. Conduits under pavement. 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 roads, 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. Live loads assume increasing importance with decreasing fill height. This section refers to minimum cover considerations only.

b. Capacity. Drainage systems should be designed in order to provide an ultimate capacity sufficient to serve the planned pavement configuration. Additions to, or replacement of, drainage lines following initial construction is both costly and disrupting to traffic.

c. Construction cover. 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.

d. Anticipated loads. For minimum cover design, the maximum anticipated loads (H20-44, Cooper E60, 15-kip and 25-kip single wheel, 100-kip twin wheel, 265-kip twin-twin [B-52] and 360-kip 12-wheel [C-5A] assembly loads referred to as single, dual, dual-tandem and multiple wheel) have been considered. 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.

e. Recommended cover. Tables 8-2 and 8-3 identify the recommended minimum cover requirements for storm drains and culverts. Minimum cover requirements have been formulated for: asbestos-cement pipe, corrugated-steel pipe, reinforced concrete culverts and storm drains, standard strength clay and nonreinforced concrete pipe, extra strength clay and nonreinforced concrete pipe and PVC pipe. The cover depths recommended are valid for average bedding and backfill conditions. Deviations from these conditions may result in significant changes in the minimum cover requirements.

Table 8-2. Minimum Pipe Cover Requirements for Airfields and Heliports
(Cover in Feet for Indicated Wall Thickness or Pipe Class)

Pipe diameter in.	For Corrugated Steel (2 + 2/3-in. Corrugations) 0.052 in.	0.064 in.	0.079 in.	0.109 in.	0.138 in.	0.168 in.	Reinforced-Concrete Culverts and Storm Drains Class I 1200 D	Class II 1500 D	Class III 2000 D	Class IV 3000 D	Class V 3750 D	Asbestos-Cement Pipe Class 1500	Class 2500	Class 3300	Class 4000	Class 5000	Clay and Non-Reinforced Concrete Pipe Standard Strength	Extra Strength	PVC Pipe Sch. 40	Pipe diameter in.
											15,000-lb SINGLE-WHEEL LOAD									
6	1.0	1.0	-	-	-	-	-	2.5	2.0	1.5	1.0	1.5	1.0	1.0	-	-	2.0	1.5	2.5	6
12	1.0	1.0	1.0	-	-	-	-	2.5	2.0	1.5	1.0	2.5	2.0	1.5	1.0	1.0	2.5	2.0	3.5	12
24	1.0	1.0	1.0	1.0	1.0	-	-	2.5	2.0	1.5	1.0	-	3.5	2.0	2.0	1.5	3.5	2.0	*3.5	24
36	1.0	1.0	1.0	1.0	1.0	1.0	-	2.5	2.0	1.5	1.0	-	-	-	2.5	2.0	2.5	2.0	(* 15.inch	36
48	-	1.0	1.0	1.0	1.0	1.0	2.5	2.0	2.0	1.5	1.0								diameter	48
60	-	-	1.0	1.0	1.0	1.0	2.5	2.0	2.0	1.5	1.0								pipe)	60
72	-	-	-	1.0	1.0	1.0	2.5	2.0	2.0	1.5	-									72
84	-	-	-	-	1.0	1.0	2.5	2.0	2.0	-	-									84
96	-	-	-	-	-	1.0	2.5	2.0	2.0	-	-									96
											25,000-lb SINGLE-WHEEL LOAD									
6	1.0	1.0	-	-	-	-	-	3.5	3.0	2.0	1.5	2.0	1.5	1.0	-	-	2.5	1.5	3.5	6
12	1.0	1.0	1.0	-	-	-	-	3.5	2.5	2.0	1.5	3.5	2.5	2.0	1.5	1.5	3.5	2.5	5.0	12
24	1.0	1.0	1.0	1.0	1.0	-	-	3.0	2.5	2.0	1.5	-	4.5	3.0	2.5	2.0	4.5	2.5	*5.0	24
36	1.0	1.0	1.0	1.0	1.0	1.0	-	3.0	2.5	2.0	1.5	-	-	-	3.0	2.5	3.0	2.5		36
48	-	1.0	1.0	1.0	1.0	1.0	3.5	3.0	2.5	2.0	1.5									48
60	-	-	1.0	1.0	1.0	1.0	3.5	2.5	2.0	2.0	1.5									60
72	-	-	-	1.0	1.0	1.0	3.0	2.5	2.0	2.0	-									72
84	-	-	-	-	1.0	1.0	3.0	2.5	2.0	-	-									84
96	-	-	-	-	-	1.0	3.0	2.5	2.0	-	-									96
											C-5A									
6	1.0	1.0	-	-	-	-	-	-	-	6.5	2.0									6
12	1.0	1.0	1.0	-	-	-	-	-	-	6.0	2.0									12
24	1.0	1.0	1.0	1.0	-	-	-	-	-	5.0	2.0									24
36	1.0	1.0	1.0	1.0	1.0	1.0	-	-	-	4.5	2.0									36
48	-	1.0	1.0	1.0	1.0	1.0	-	-	-	4.5	2.0			Not Available			Not Available		Not Available	48
60	-	-	1.0	1.5	1.0	1.0	-	-	-	4.5	2.0									60
72	-	-	-	-	1.0	1.0	-	-	-	4.0	2.0									72
84	-	-	-	-	1.0	1.0	-	-	8.0	4.0	2.0									84
96	-	-	-	-	-	1.5														96
											100,000-lb TWIN-WHEEL ASSEMBLY LOAD									
6	1.0	1.0	-	-	-	-	-	-	-	5.0	4.0	5.0	3.0	2.5	-	-	8.0	3.5	6.5	6
12	1.0	1.0	1.0	-	-	-	-	-	7.5	4.5	3.5	-	6.0	4.5	3.5	3.0	-	6.0	7.0	12
24	1.0	1.0	1.0	1.0	1.0	-	-	-	7.0	4.0	3.0	-	-	-	7.5	5.5	-	6.5	*7.0	24
36	1.5	1.0	1.0	1.0	1.0	1.0	-	-	6.0	4.0	3.0	-	-	-	-	-	-	7.0		36
48	-	1.5	1.0	1.0	1.0	1.0	-	-	6.0	4.0	3.0									48
60	-	-	1.5	1.0	1.0	1.0	-	-	5.5	3.5	3.0									60
72	-	-	-	1.5	1.5	1.0	-	-	5.5	3.5	-									72
84	-	-	-	-	2.0	1.5	-	-	5.5	-	-									84
96	-	-	-	-	-	2.5	-	-	5.0	-	-									96
											265,000-lb TWIN-WHEEL ASSEMBLY LOAD									
6	1.0	1.0	-	-	-	-	-	-	-	9.5	6.5	8.0	5.0	3.5	-	-	-	6.0	9.0	6
12	1.0	1.0	1.0	-	-	-	-	-	-	8.0	6.0	-	-	8.0	6.0	5.0	-	-	10.0	12
24	1.0	1.0	1.0	1.0	1.0	-	-	-	-	7.5	6.0	-	-	-	-	-	-	-	*10.0	24
36	1.5	1.5	1.0	1.0	1.0	1.0	-	-	-	7.0	5.5	-	-	-	-	-	-	-		36
48	-	2.0	1.5	1.0	1.0	1.0	-	-	-	6.5	5.0									
60	-	-	-	1.5	1.0	1.0	-	-	-	6.0	5.0									
72	-	-	-	2.0	1.5	1.0	-	-	-	6.0	-									
84	-	-	-	-	2.5	1.5														
96	-	-	-	-	-	3.5														

NOTES:

(1) Except where individual pipe installation designs are made, cover for pipe beneath runways, taxiways, aprons, or similar traffic areas will be provided in accordance with this table for flexible pavement or unpaved surfaces. See note 8 for pipe underlying rigid pavements.

(2) Cover for pipe in airfield non-traffic areas will be designed for 15,000-lb. single-wheel load.

(3) Cover depths are measured from top of flexible pavement or unsurfaced areas to top of pipe, except top of pipe is not to be above bottom subbase material.

(4) Pipe produced by certain manufacturers exceeds strength requirements established by indicated standards. When additional strength is proved, the minimum cover may be reduced accordingly.

(5) At present, minimum cover for aluminum alloy and polyethylene corrugated pipe installed beneath flexible pavements is not available. In the absence of other criteria, the following minimum requirements will be followed:

15,000-lb. single wheel	Use values shown for corrugated steel pipe
25,00-lb. single wheel	Increase cover depths shown for corrugated steel pipe by 0.5 feet
C-5A	Same as above, except increase cover depth 1.0 feet
265,000-lb. twin-twin assembly	Same as above, except increase cover depth 1.5 feet

(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 internal pipe diameter.

(7) The class designation number for asbestos-cement pipe is the minimum required 3-edge test load to produce ultimate 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.

(8) Pipe placed under airfield rigid pavements will have a minimum cover, measured from the bottom of the slab, as follows:

Pipe Sizes, in.	15,000-lb. Single-Wheel	25,000-lb. Single-Wheel	C-5A	100,000-lb. Twin Assembly	265,000-lb. Twin-Twin Assembly
6-60	0.5	0.5	1.0	1.0	1.0
66-180	1.0	1.0	1.5	1.5	1.5

Table 8-3. Minimum Pipe Cover Requirements for Roads and Railroads (Feet)

Pipe Material	Diameter of Pipe (inches) 8	10	12	15	18	21	24	30	36	48	60	72	84
Corrugated Steel Pipe (2-2/3 x 1/2)													
Flexible Pavement	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.5	1.5
Rigid Pavement	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	1.0	1.0
Railroad	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.5	1.5
Reinforced Concrete													
Flexible Pavement	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.5	3.0	3.5
Rigid Pavement	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	2.0	2.0	2.5
Railroad	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.5	3.0	3.5
Asbestos-Cement													
Flexible Pavement	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0			
Rigid Pavement	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0			
Railroad	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0			
Non-Reinforced Concrete													
Flexible Pavement	2.5	2.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0				
Rigid Pavement	2.0	2.0	2.5	2.5	2.5	2.5	2.5	2.5	2.5				
Railroad	2.5	2.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0				
Clay (Std. Str.)													
Flexible Pavement	2.5	2.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0				
Rigid Pavement	2.0	2.0	2.5	2.5	2.5	2.5	2.5	2.5	2.5				
Railroad	2.5	2.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0				
PVC and ASB Sch 40													
Flexible Pavement		As Recommended											
Rigid Pavement		by the											
Railroad		Manufacturer											
Corrugated Polyethylene													
Flexible Pavement		As Recommended											
Rigid Pavement		by the											
Railroad		Manufacturer											

NOTES:

(1) Table will be used for typical installations with dead load plus either Cooper E-60 railway or H20-44 highway loading.

(2) Minimum cover for pipe will be measured from the bottom of the railway tie to the top of pipe. Minimum cover for pipe placed under rigid pavements will be measured from the bottom of the slab to top of pipe. When pipe is to be placed beneath flexible pavements, minimum cover for pipe will be measured from the top of pavement surface to top of pipe; however, in no case is the top of pipe to be above the bottom of subbase.

(3) Minimum wall thickness for corrugated steel pipe under rigid or flexible pavements for pipe diameters less than 36 inches, 16 gage; 36 and 48 inches, 12 gage; over 48 inches, 10 gage, for pipe under railroads, less than 36 inches, 14 gage; 36 and 48 inches, 10 gage; over 48 inches, 8 gage.

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f. Embedment. Figures 8-1, 8-2, 8-3, and 8-4 indicate the three main classes of acceptable rigid conduit bedding, the free-body conduit diagrams, load factors and class of bedding, and beddings for positive projecting conduits, respectively. Figure 8-5 is a schematic representation of the subdivision of classes of conduit installation which influences loads on underground conduits.

8-6. Frost condition considerations.

a. Frostheave. The detrimental effects of heaving of frost-susceptible soils around and under storm drains and culverts is a principal consideration 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. 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 due to 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 alinement, joint failures and, in extreme cases, pipe breakage. Placing drains and culverts beneath pavements should be avoided whenever possible. When this is unavoidable, the pipes should be installed before the base course is placed in order to obtain maximum uniformity.

b. Base-course excavation. The practice of excavating through base courses to lay drains, pipes, and other conduits is unsatisfactory since it is almost impossible to attain uniformity between the compacted trench backfill and the adjacent material. Special design considerations for frost conditions and recommended minimum depth of cover for protection of storm drains and culverts in seasonal frost areas are given in table 8-4.

c. Design considerations. The following design criteria should be considered for installations located in seasonal frost areas.

(1) Cover requirement for traffic loads will govern when such depth exceeds that necessary for frost protection.

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

(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.

(4) Depth of frost penetration in well drained, granular, non-frost-susceptible soil beneath pavements kept free of snow and ice can be determined from figures 8-6 or 8-7. For other soils and/or

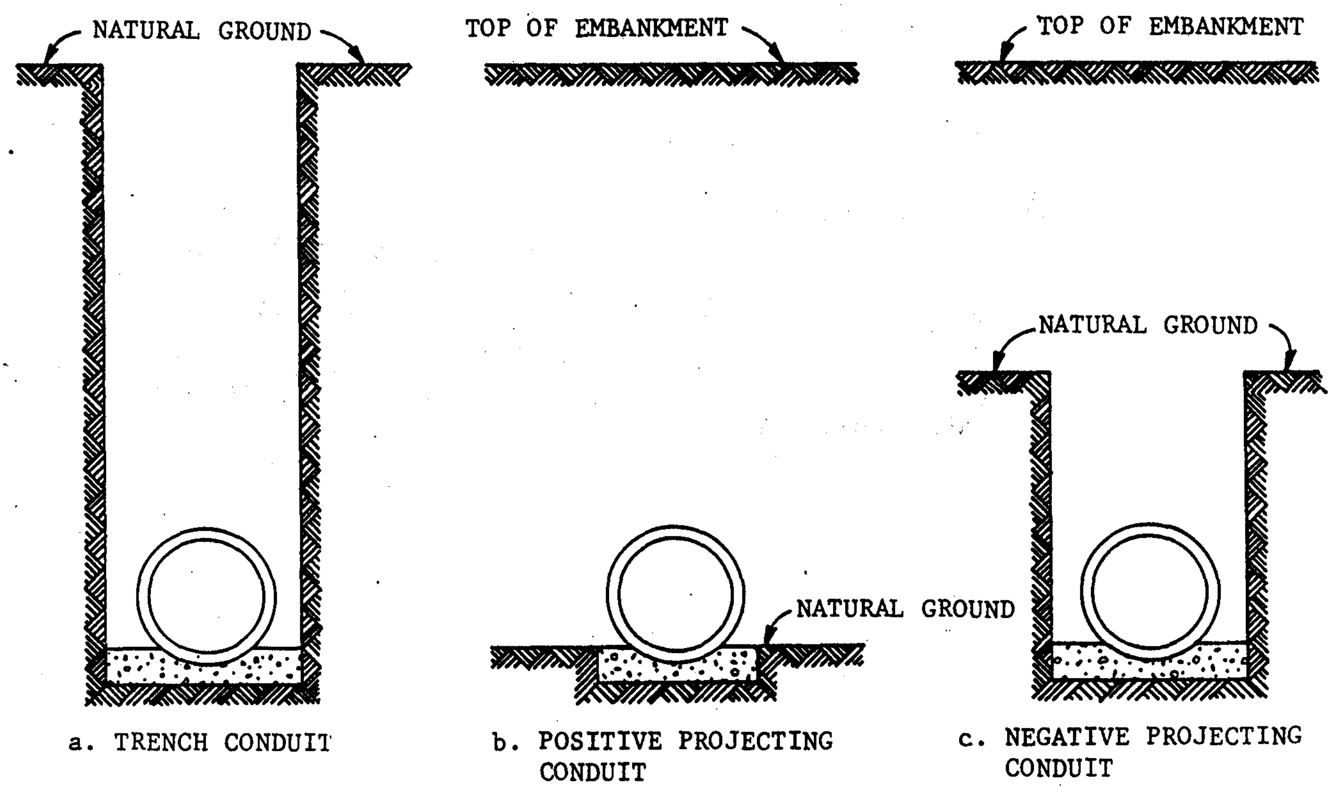

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FIGURE 8-1. THREE MAIN CLASSES OF CONDUITS

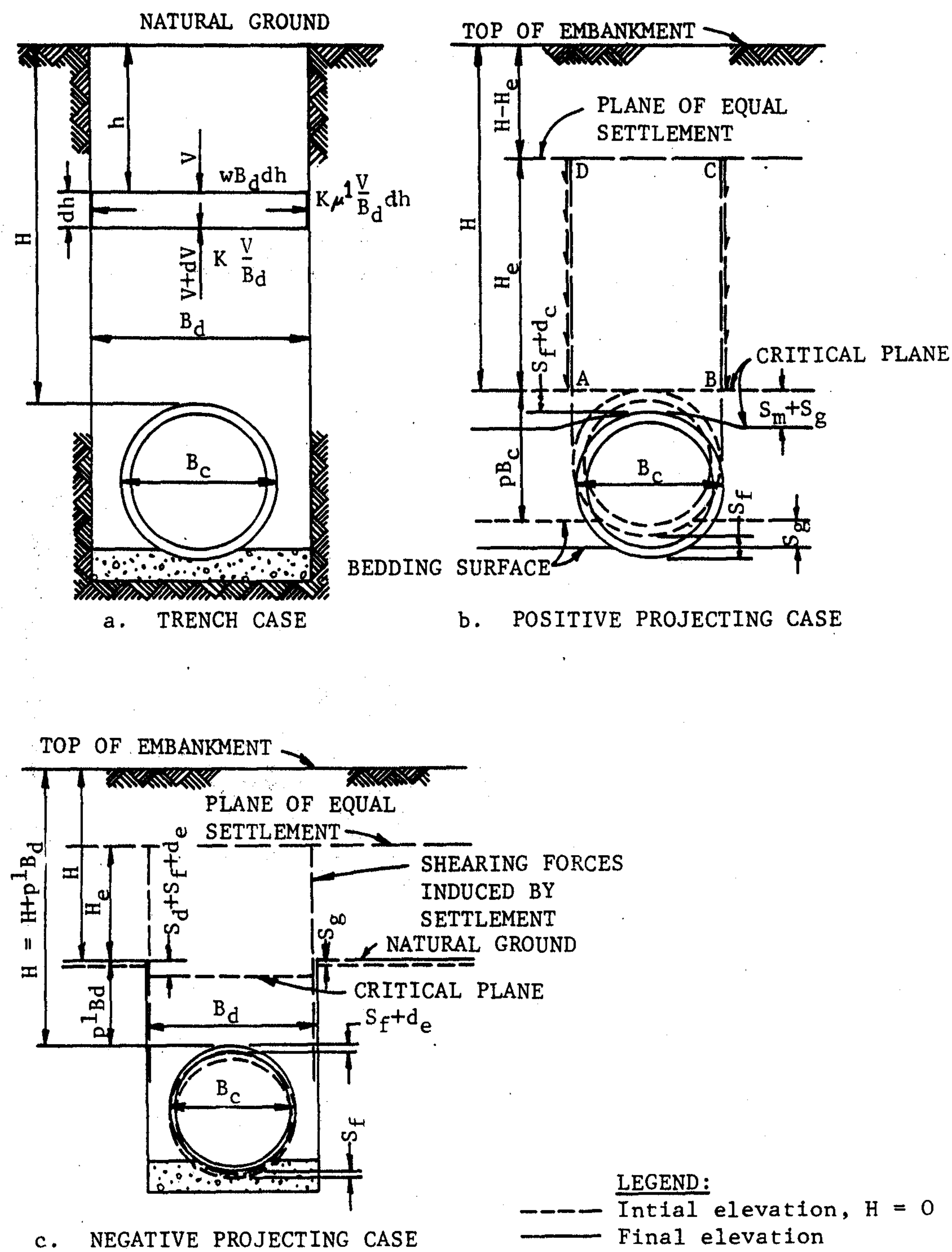

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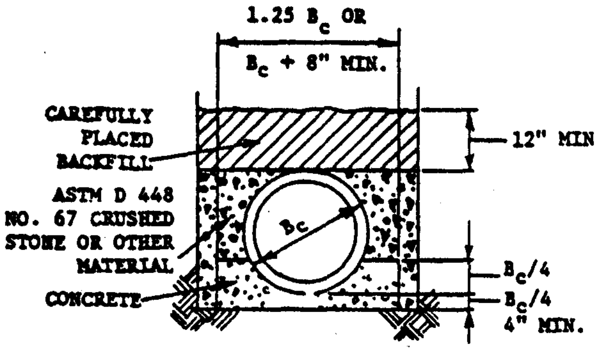
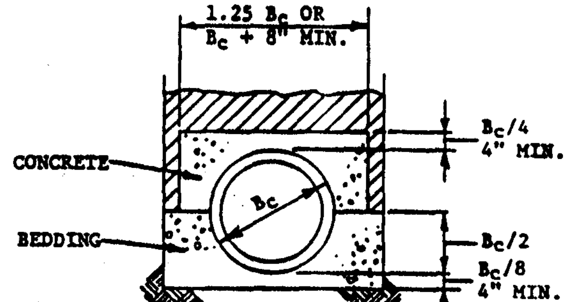
FIGURE 8-2. FREE-BODY CONDUIT DIAGRAMS



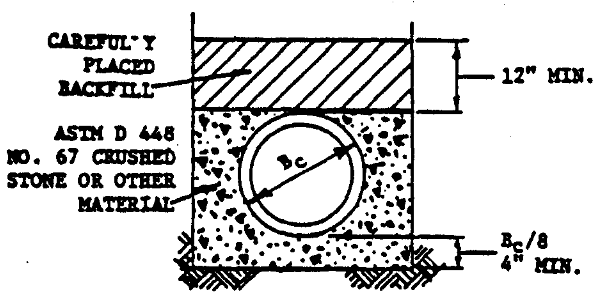
LOAD FACTORS 2.8 PLAIN CONCRETE
3.4 REINFORCED CONCRETE P=0.4%
(TRANSVERSE STEEL)

CLASS A-I



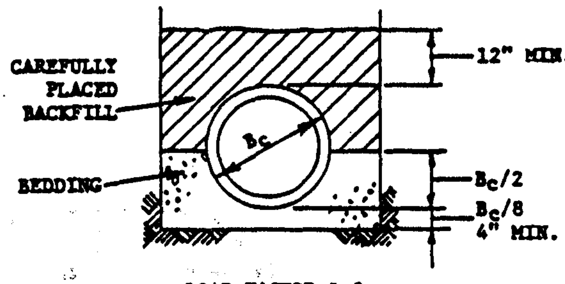
LOAD FACTORS 2.8 PLAIN CONCRETE
3.4 REINFORCED CONCRETE P=0.4%
(TRANSVERSE STEEL)

CLASS A-II



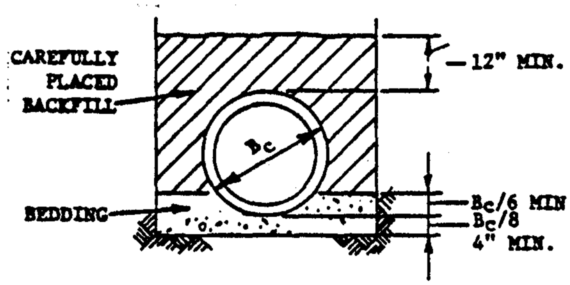
LOAD FACTOR 2.2

CRUSHED STONE ENCASEMENT



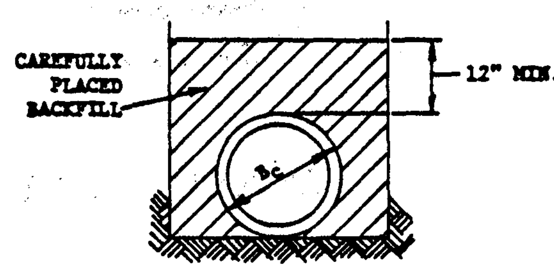
LOAD FACTOR 1.9

CLASS B



LOAD FACTOR 1.5

CLASS C



FLAT OR RESTORED TRENCH BOTTOM
LOAD FACTOR 1.1

CLASS D

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 = Area of transverse steel in the cradle of arch expressed as 3 percent of the area of concrete at the invert or crown

NOTE: For rock or other incompressible material, the trench should be overexcavated a minimum of 6 inches and refilled with granular material.

CLAY PIPE ENGINEERING MANUAL BY NATIONAL CLAY PIPE INSTITUTE, 1982, P. 52-53.

FIGURE 8-3. LOAD FACTORS AND CLASS OF BEDDING

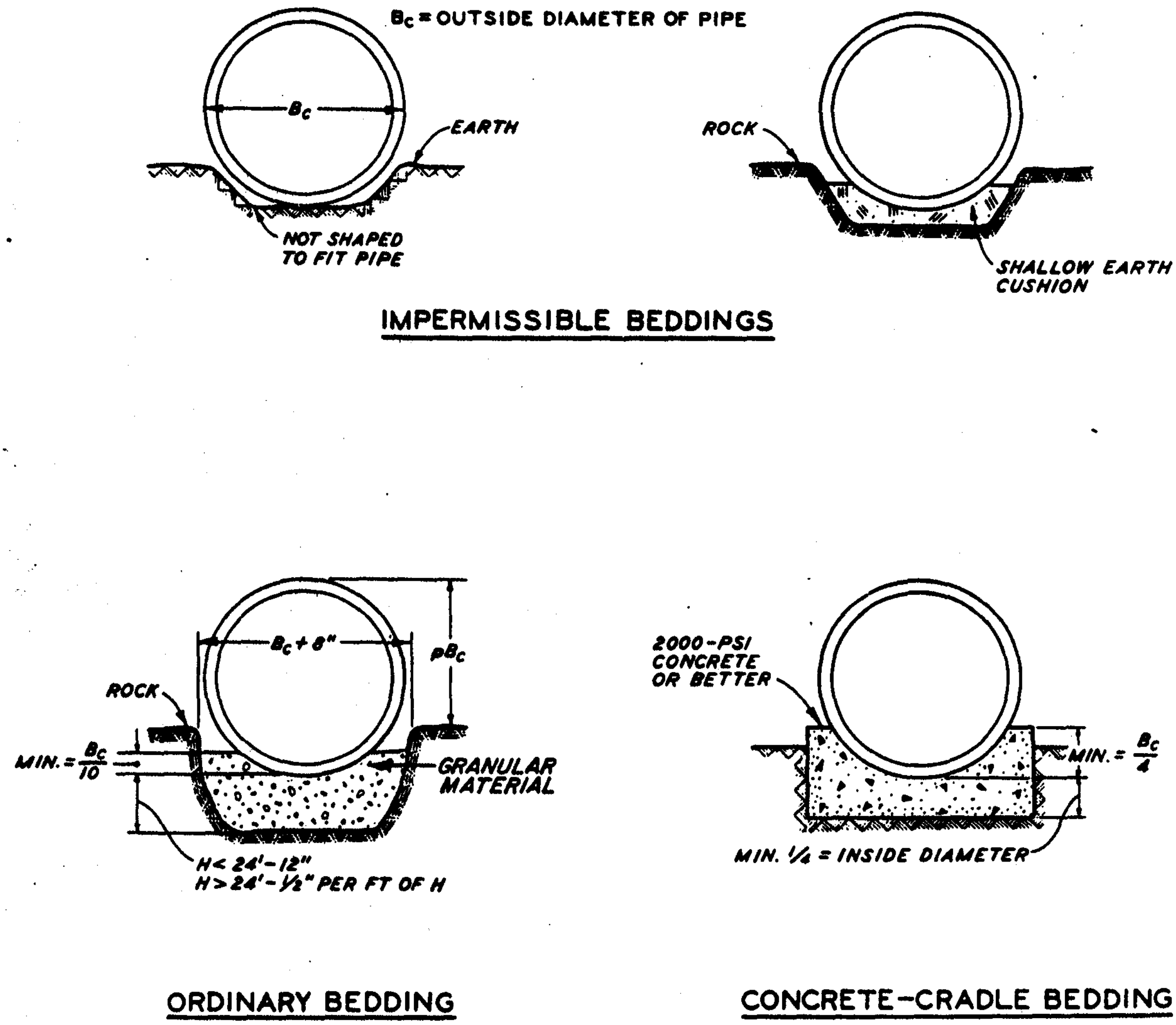


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FIGURE 8-4. BEDDING FOR POSITIVE PROJECTING CONDUITS

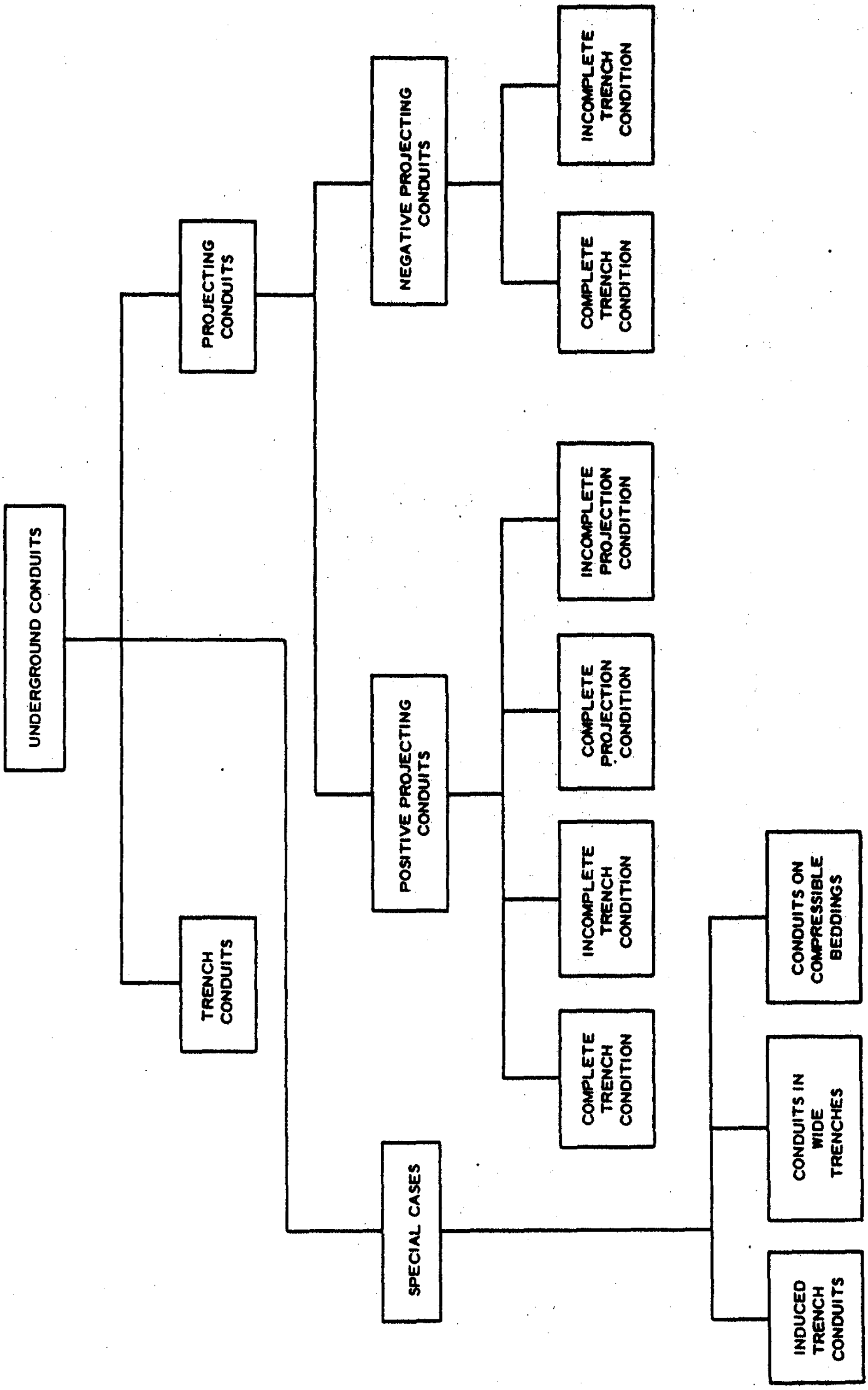

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FIGURE 8-5. INSTALLATION CONDITIONS WHICH INFLUENCE LOADS ON UNDERGROUND CONDUITS

Table 8-4 Minimum Required Depth of Cover Protection of Storm Drains and Culverts in Seasonal Frost Areas

Position of highest ground water table	Nonfrost susceptible subgrade		Frost susceptible subgrade	
	To prevent heave	To prevent freezing of water in pipe	To prevent heave	To prevent freezing of water in pipe
Less than 5 ft. below maximum depth of frost penetration	No special measures required	Place invert of pipe at or below depth of maximum frost penetration	For pipe diameters smaller than 18 in. place centerline of pipe at or below depth of maximum frost penetration. For pipe diameters 18 in. or larger, place centerline of pipe 1/3 diameter below depth of maximum frost penetration or place centerline of pipe at a depth of maximum frost penetration and backfill around pipe with highly free draining, non-frost-susceptible material.	Place invert of pipe at or below depth of maximum frost penetration
5 ft. or more below maximum depth of frost penetration	No special measures required	Place invert of pipe at or below depth of maximum frost penetration	Place centerline of pipe at at or below depth of maximum frost penetration	Place invert of pipe at or below depth of maximum frost penetration

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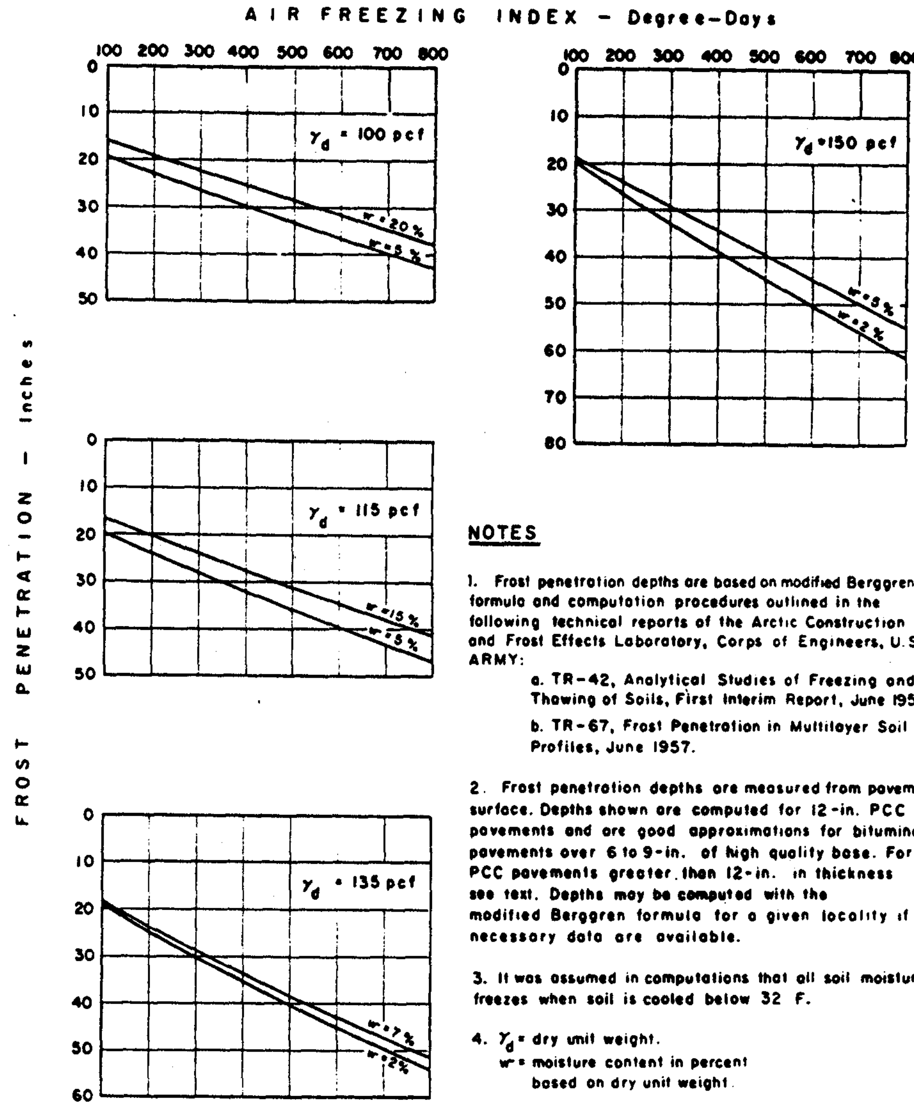

NOTES

1. Frost penetration depths are based on modified Berggren formula and computation procedures outlined in the following technical reports of the Arctic Construction and Frost Effects Laboratory, Corps of Engineers, U.S. ARMY:
   a. TR-42, Analytical Studies of Freezing and Thawing of Soils, First Interim Report, June 1953.
   b. TR-67, Frost Penetration in Multilayer Soil Profiles, June 1957.
2. Frost penetration depths are measured from pavement surface. Depths shown are computed for 12-in. PCC pavements and are good approximations for bituminous pavements over 6 to 9-in. of high quality base. For PCC pavements greater than 12-in. in thickness see text. Depths may be computed with the modified Berggren formula for a given locality if necessary data are available.
3. It was assumed in computations that all soil moisture freezes when soil is cooled below 32 F.
4. γ_d = dry unit weight.
   w = moisture content in percent based on dry unit weight.

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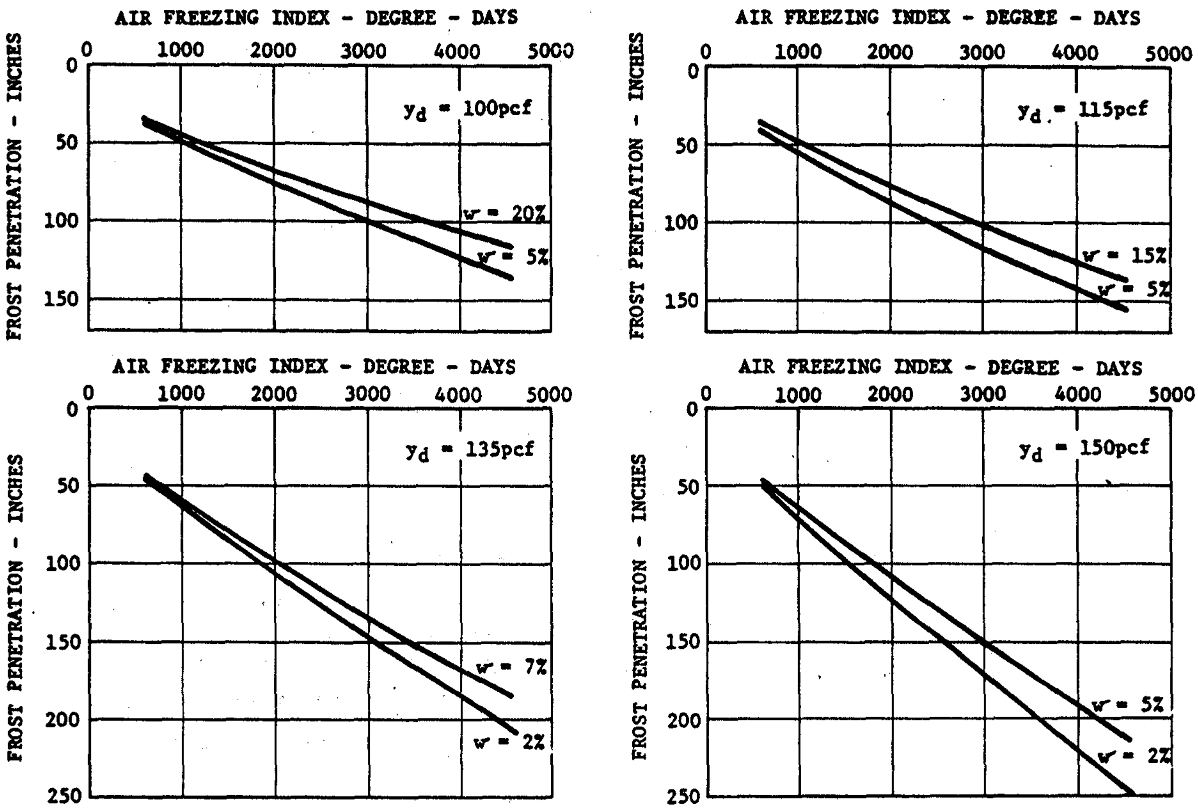
FIGURE 8-6. RELATIONSHIPS BETWEEN AIR FREEZING INDEX AND FROST PENETRATION INTO GRANULAR, NONFROST-SUSCEPTIBLE SOIL BENEATH PAVEMENTS KEPT FREE OF SNOW AND ICE FOR FREEZING INDEXES BELOW 800



SEE NOTES ON FIGURE 8-6

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FIGURE 8-7. RELATIONSHIPS BETWEEN AIR FREEZING INDEX AND FROST PENETRATION INTO GRANULAR, NONFROST-SUSCEPTIBLE SOIL BENEATH PAVEMENTS KEPT FREE OF SNOW AND ICE subsurface conditions, frost penetrations should be determined using conservative surface condition assumption and standard methods. Table 8-5 lists frost penetration for various locations and figure 8-8 indicates areas of varying frost penetrations in the continental United States. In all cases, estimates of frost penetrations should be confirmed by the U.S. Weather Bureau from data gathered at the nearest station to the proposed site. Frost penetrations are to be based on the design freezing index for the coldest winter in the past 10 years.

(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.

8-7. Infiltration of fine soils through drainage pipe joints.

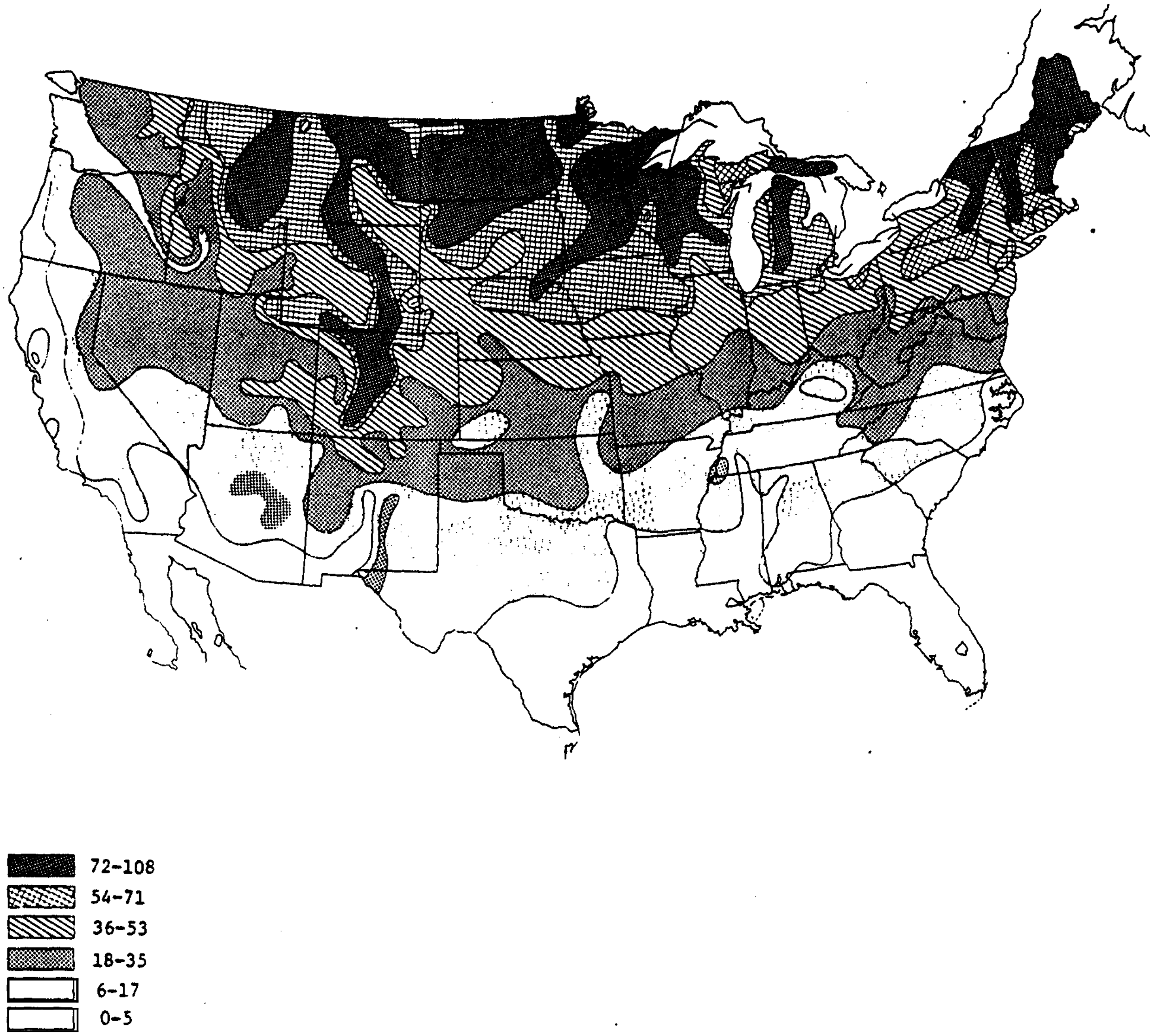
a. Broken back culverts. Infiltration of fine-grained soils into drainage pipelines through joint openings is one of the major causes of ineffective drainage facilities. This is particularly a problem along pipes on relatively steep slopes such as those encountered with broken back culverts or stilling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

b. High water table. 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.

c. Steep slopes. Culverts and storm drains placed on steep slopes should be sufficiently large and be 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 and thereby reduce the tendency for infiltration of soil and water through joints. Pipes on steep slopes may tend to prime and flow full periodically due to entrance or outlet condition effects until the hydraulic or pressure gradient is lowered sufficiently to cause venting or loss of prime at either the inlet or outlet. The alternate increase and reduction of pressure relative to atmospheric pressure is considered to be a primary cause of severe piping and infiltration. It is recommended that a vertical riser 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

Table 8-5. Estimated Frost Penetration for Selected Locations

Location	Depth (Inches)
ALABAMA	
Brookley AFB	6
Maxwell AFB	9
Montgomery	6
ARIZONA	
Davis Monthan AFB	5
Luke AFB	7
Phoenix	7
ARKANSAS	
Little Rock AFB	12
CALIFORNIA	
Castle AFB	5
Hamilton AFB	5
March AFB	5
Travis AFB	5
Vandenberg AFB	5
San Diego	0
San Francisco	5
Oakland	5
Mare Island	5
Sacramento	5
COLORADO	
Lowry AFB	60
Denver	60
CONNECTICUT	
New London	35
New Haven	35
DELAWARE	
Dover AFB	20
FLORIDA	
Eglin AFB	5
Homestead AFB	0
MacDill AFB	2
Patrick AFB	2
Jacksonville	2
Key West	0
Pensacola	2
GEORGIA	
Hunter AFB	5
Robins AFB	5
Turner AFB	5
Atlanta	7
Savannah	3
Macon	5
IDAHO	
Mountain Home AFB	40
ILLINOIS	
Chanute AFB	35
Scott AFB	35
Chicago	40
INDIANA	
Fort Wayne	40
Indianapolis	30
IOWA	
Sioux City	54
KANSAS	
Forbes AFB	30
Schilling AFB	24
KENTUCKY	
Lexington	18
Louisville	18
LOUISIANA	
Barksdale AFB	5
Chennault AFB	4
New Orleans	2
MAINE	
Dow AFB	75
Loring AFB	75
Portland	65
Bangor	72
MARYLAND	
Andrews AFB	25
Baltimore	22
MASSACHUSETTS	
L.G. Hanscom Field	50
Otis AFB	50
Westover AFB	70
Boston	50
Springfield	70
MICHIGAN	
Kinchelow AFB	65
Selfridge AFB	50
Detroit	50
MINNESOTA	
Minn-St. Paul IAP	75
Minneapolis	75
Duluth	75
MISSISSIPPI	
Jackson	3
MISSOURI	
Kansas City	28
St. Louis	27
MONTANA	
Malmstrom AFB	75
NEBRASKA	
Offutt AFB	55
Omaha	55
NEVADA	
Nellis AFB	8
Stead AFB	23
Fallon	12
Hawthorne	30
Reno	23
NEW HAMPSHIRE	
Pease AFB	60
Portsmouth	60
NEW JERSEY	
McGuire AFB	30
Atlantic City	20
Bayonne	30
NEW MEXICO	
Cannon AFB	15
Holloman AFB	20
Walker AFB	15
Albuquerque	17
NEW YORK	
Griffis AFB	50
Plattsburg AFB	70
Stewart AFB	45
Buffalo	35
Albany	54
New York	40
Syracuse	56
NORTH CAROLINA	
Pope AFB	9
Charlotte	8
Wilmington	5
NORTH DAKOTA	
Grand Forks AFB	85
Minot	80
OHIO	
Wright-Patterson AFB	40
Columbus	40
Cincinnati	20
OKLAHOMA	
Tinker AFB	20
OREGON	
Portland Int. Apt.	6
Portland	6
PENNSYLVANIA	
Olmsted AFB	35
Harrisburg	30
Pittsburgh	38
Philadelphia	30
RHODE ISLAND	
Providence	45
SOUTH CAROLINA	
Shaw AFB	6
Charleston	3
SOUTH DAKOTA	
Ellsworth AFB	55
TENNESSEE	
Sewart AFB	10
Memphis	10
TEXAS	
Amarillo AFB	20
Bergstrom AFB	4
Biggs AFB	6
Carswell AFB	12
Dyess AFB	10
Ellington AFB	3
Kelly AFB	4
TEXAS (Cont.)	
Reese AFB	15
Sheppard AFB	15
Corpus Christi	2
El Paso	6
Fort Worth	10
Galveston	3
Houston	3
San Antonio	4
Amarillo	20
UTAH	
Hill AFB	35
Salt Lake City	35
VERMONT	
Burlington	72
VIRGINIA	
Langley AFB	6
Newport News	10
Norfolk	10
Richmond	14
WASHINGTON	
Fairchild AFB	65
Larson AFB	35
McChord AFB	10
Bremerton	9
Seattle	8
Spokane	30
Pasco	25
Tacoma	8
WEST VIRGINIA	
Charleston	30
WISCONSIN	
Truax Field	50
Milwaukee	54
Green Bay	54
WYOMING	
Francis E. Warren AFB	70
WASHINGTON, D.C.	20



MAXIMUM DEPTH OF FROST PENETRATION IN INCHES

THE DATA FOR THIS CHART WERE PREPARED BY THE U. S. WEATHER BUREAU IN 1938 AND PUBLISHED BY "HEATING AND VENTILATING." IT IS THE BEST INFORMATION ON THE SUBJECT AVAILABLE AT PRESENT. BOTH VARIABLE-RECORD OBSERVATIONS AND ESTIMATIONS OF MAXIMUM FROST PENETRATIONS WERE USED, RESULTING IN A HIGHLY-DETAILED PRESENTATION THAT MAY IMPLY A RELIABILITY BEYOND THAT ORIGINALLY INTENDED.

FIGURE 8-8. MAXIMUM DEPTH OF FROST PENETRATION steep slope. The riser may also be equipped with an inlet and used simultaneously to collect runoff from a berm or adjacent area.

d. Flexible joint material. 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 calked with oakum or other similar rope-type calking materials and sealed with a 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. The results of laboratory research concerning soil infiltration through pipe joints and the effectiveness of gasketing tapes for waterproofing joints and seams are available.

e. Flexible joint installation. Installation of flexible watertight joints will conform closely to manufacturer's recommendations, and a conclusive infiltration test will be required for each section of pipeline involving watertight joints. 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 inches 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.

CHAPTER 9

INLETS AND BOX DRAINS

9-1. General

a. Materials. Inlet structures to collect storm runoff may be constructed of any suitable construction material. The structures must insure efficient removal 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 employed although brick, concrete block, precast concrete, or rubble masonry have been used. The use of precast or preformed structures should be encouraged to reduce excavation and backfill for construction and overall expedition of installation. The material, including the slotted drain corrugated metal pipe to handle surface flow if employed, should be of sufficient strength to withstand the loads to which it will be subjected.

b. Locations. 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. Natural drainage of paved areas should be utilized to the greatest extent possible thereby reducing or eliminating the need for paved area inlets.

c. Influences. Local practices and requirements greatly influence design and construction details. Experience has indicated the following features should be considered by the designer.

9-2. Inlets versus catch basins.

Catch basins are not considered necessary where storm drainage lines are laid on self-cleaning grades. Proper maintenance of catch basins is difficult; without frequent cleaning, the sediment basin may quickly be rendered ineffective. Proper selection of storm drain gradients may largely eliminate the need for catch basins. When catch basins are required to prevent solids and debris from entering the drainage system, they must be cleaned frequently. Catch basin installation should be avoided whenever practicable.

9-3. Design features.

a. Elevations. 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 insure 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 feet around the inlet is effective in reducing erosion if suitable turf is established and maintained on the adjacent drainage area. Paved aprons around the perimeter of a graded inlet should only be considered where extreme erosion or silting conditions exist.

b. Elevation in paved areas. Drainage structures located in the usable areas on airports should be so designed that they do 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.

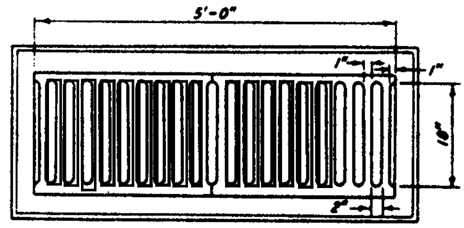
c. Low heads. Under low head situations, a grating in a ponded area operates as a weir. At higher heads the grating acts as an orifice, and model tests of a grating shown in the typical plan of a double inlet grating (fig 9-1) indicates that vortex action influences the discharge characteristics when the head exceeds 0.4 foot. 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 foot.

d. Discharge characteristics. 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.

e. Grate materials. Grates may be 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 designs of typical metal grates and inlets are shown in figures 9-2 and 9-3.

f. Field fabricated grates. Material shortages and construction schedules may require that nonstandard or field fabricated gratings and frames be used in turfed areas. Care should be taken to insure proper loading and tie down is provided in paved areas.

g. Loading. Commercially manufactured grates and frames for airport loadings have been specifically designed for airport loadings from 50 to 250 psi. 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.



TYPICAL PLAN OF DOUBLE INLET GRATING

WATERWAY OPENING = 5.0 SQ. FT. (DOUBLE GRATING)

ASSUME GRATING IS PLACED SO THAT FLOW WILL OCCUR FROM ALL SIDES OF INLET

FOR LOW HEADS DISCHARGE WILL CONFORM WITH GENERAL WEIR EQUATION.

$Q = CLH^{3/2}$

WHERE

$C = 3.0$
$L = 13.0$ FT. GROSS PERIMETER OF GRATE OPENING (OMITTING BARS)
H = HEAD IN FEET

FOR HIGH HEADS DISCHARGE WILL CONFORM WITH ORIFICE FORMULA:

$Q = CA\sqrt{2gH}$

WHERE

$C = 0.6$
$A = 5.0$ SQ. FT.
g = ACCELERATION OF GRAVITY IN FEET PER SECOND^2
H = HEAD IN FEET

THEORETICAL DISCHARGE RELATION TO BE MODIFIED BY 1.25 TO 2.0 SAFETY FACTOR

COEFFICIENTS BASED ON MODEL TESTS OF SIMILAR GRATES WITH RATIO:

NET WIDTH OF GRATE OPENING TO GROSS WIDTH = 2:3

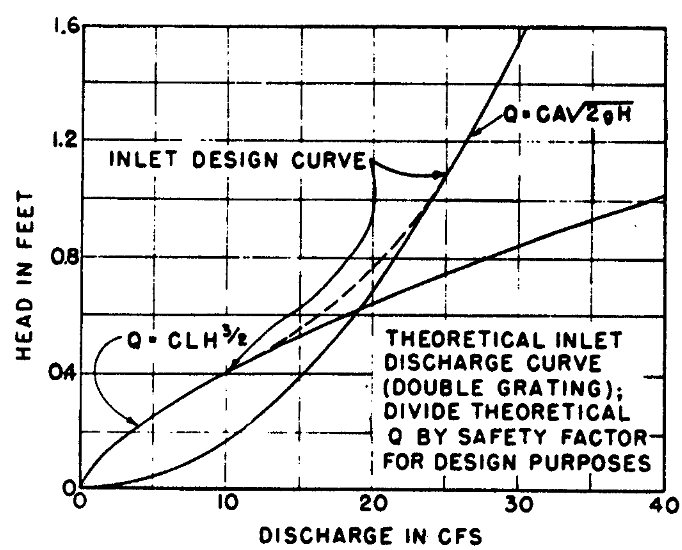


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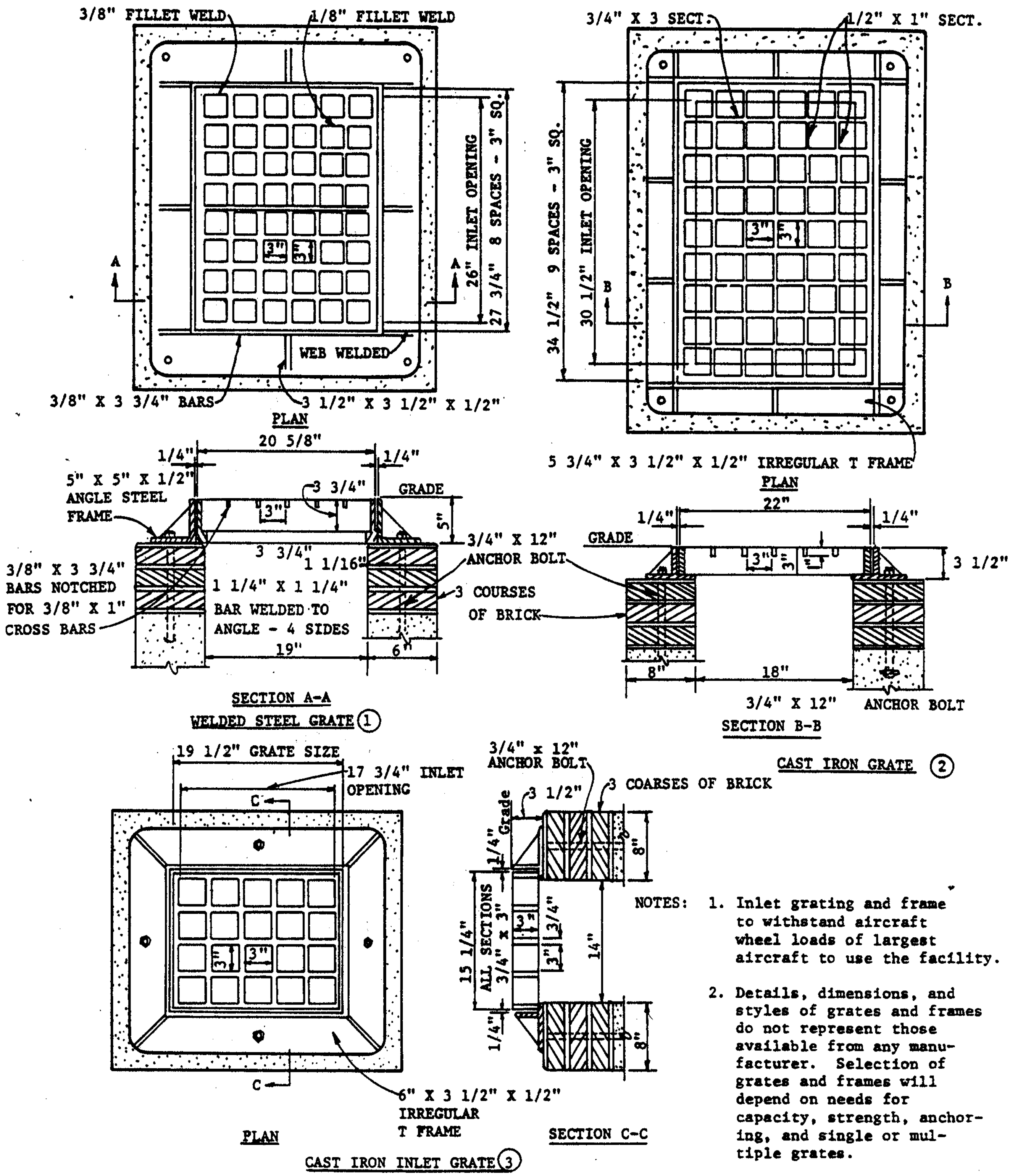
FIGURE 9-1. DETERMINATION OF TYPICAL INLET GRATING DISCHARGE CURVE



NOTES:
1. Inlet grating and frame to withstand aircraft wheel loads of largest aircraft to use the facility.
2. Details, dimensions, and styles of grates and frames do not represent those available from any manufacturer. Selection of grates and frames will depend on needs for capacity, strength, anchoring, and single or multiple grates.

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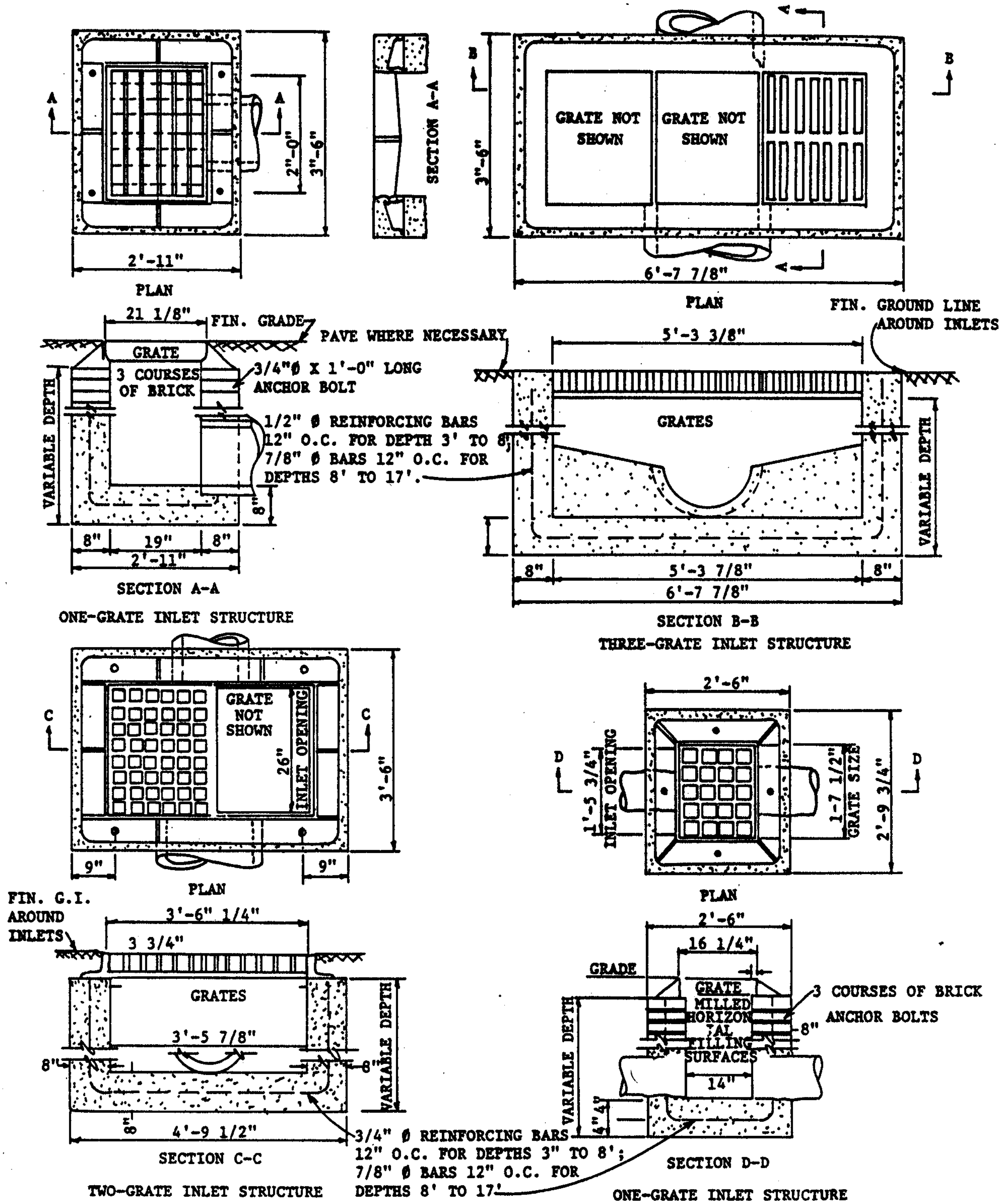
FIGURE 9-2. EXAMPLES OF TYPICAL INLET GRATES



NOTES: Details dimensions, and materials for inlets as well as for grates and frames are illustrative only.

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FIGURE 9-3. EXAMPLES OF INLET DESIGN

h. Configuration. 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.

i. Construction joints. For rigid concrete pavements, the drainage structure 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.

9-4. Box drains.

a. Design and construction. Where box drains are necessary 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 normally more costly and time consuming to construct than conventional inlets. Accordingly, their use will be restricted to unusual drainage and grading 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 9-4. The walls of the box drain will extend to the surface of the pavement. The pavement will have a free thickened edge at the drain. An 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-inch thickness of filler is usually sufficient, but thicker fillers may be required. Grating for box drains can be fabricated 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.

b. Settlement. Inlet drainage structures, particularly box drains, have been known to settle at rates different from the adjacent pavement causing depressions which permit pavement failure should the subgrade deteriorate. Construction specifications requiring careful backfilling around inlets will aid in preventing the differential settling rates.

9-5. 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,

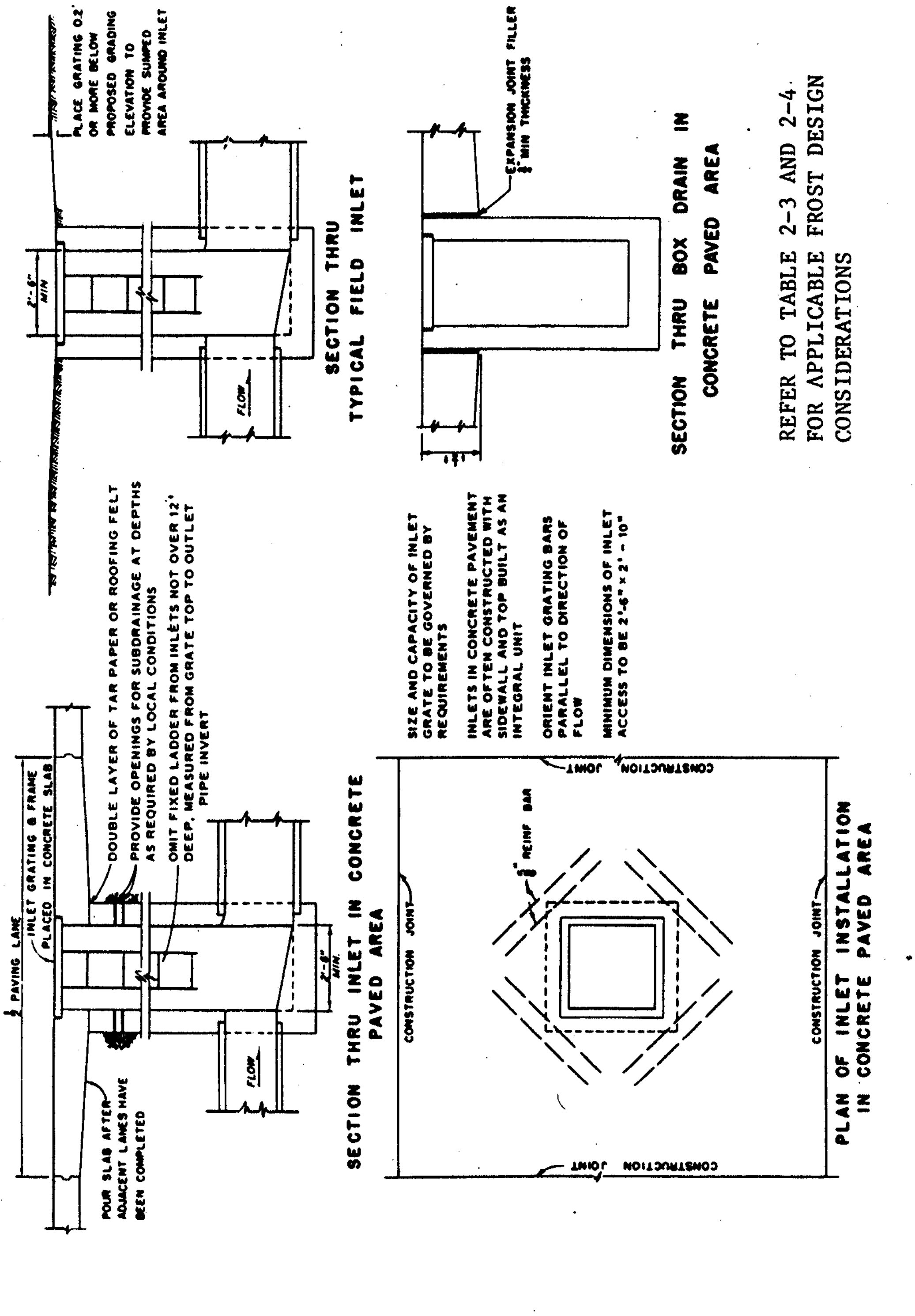

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FIGURE 9-4. TYPICAL INLET AND BOX DRAIN DESIGNS FOR AIRFIELD AND HELIPORT STORM DRAINAGE SYSTEMS approximately three to five joints along all pipe entering or leaving the manhole.

CHAPTER 10

OPEN CHANNELS

10-1. General. Maximum use will be made of open channels for drainage consistent with safety and operational requirements since greater flexibility and a higher safety factor can be obtained at lower cost.

10-2. Channel design. The following items merit special consideration in designing channels.

a. Hydraulics. The hydraulic characteristics of the channel may be studied by use of an open channel formula such as the Manning equation:

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

where:

V = velocity of flow in feet per second
n = a coefficient of roughness
R = hydraulic radius in feet
S = slope of the energy gradient

Suggested retardance coefficients and maximum permissible velocities for non-vegetated channels are given in table 10-1. 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 10-1. It is suggested that maximum velocity in turf-lined channels not exceed 6 fps. In regions where runoff has appreciable silt load, particular care will be given to securing generally nonsilting velocities.

b. Cross section. 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. Figure 10-2 indicates typical airfield slope and clearance criteria. Proposed maintenance methods affect the selection of side slopes for turfed channels since gang mowers cannot be used on slopes steeper than 1 to 3, and hand cutting is normally required on steeper slopes. In addition, 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 should be addressed.

c. Linings. 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. Several grass species are discussed in

Table 10-1.

Suggested Coefficients of Roughness and Maximum Permissible Mean Velocities for Open Channels

Material	n	Maximum Permissible Mean Velocity Feet per Second
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
Silty 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 10-1

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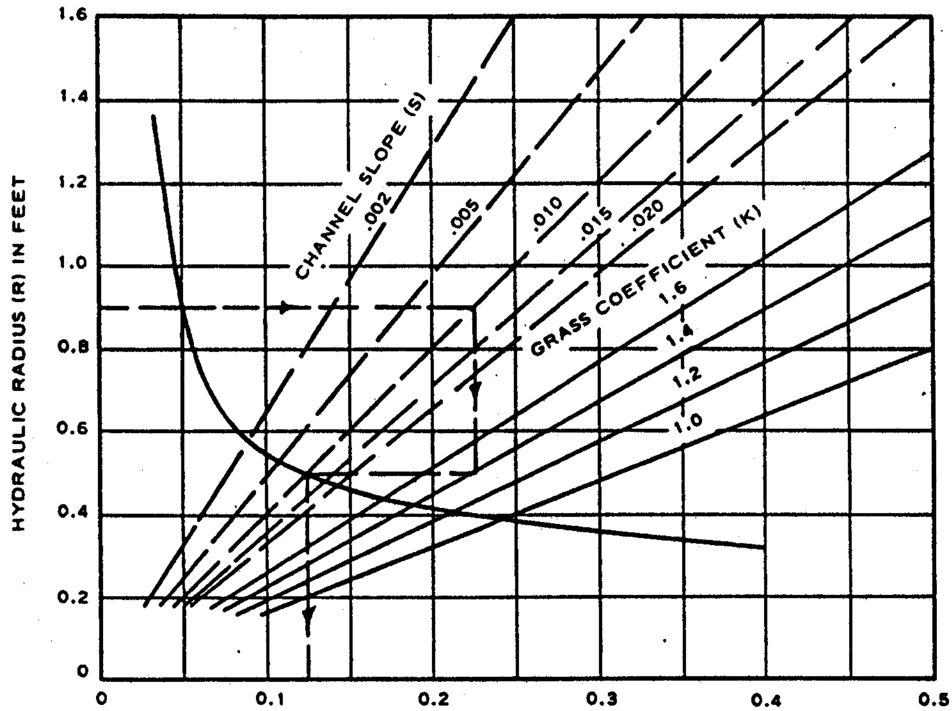

GRASS COEFFICIENTS (K) FOR DENSE AIRFIELD TURF

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 = .010.

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

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FIGURE 10-1. RETARDANCE COEFFICIENTS FOR FLOW IN TURFED CHANNELS

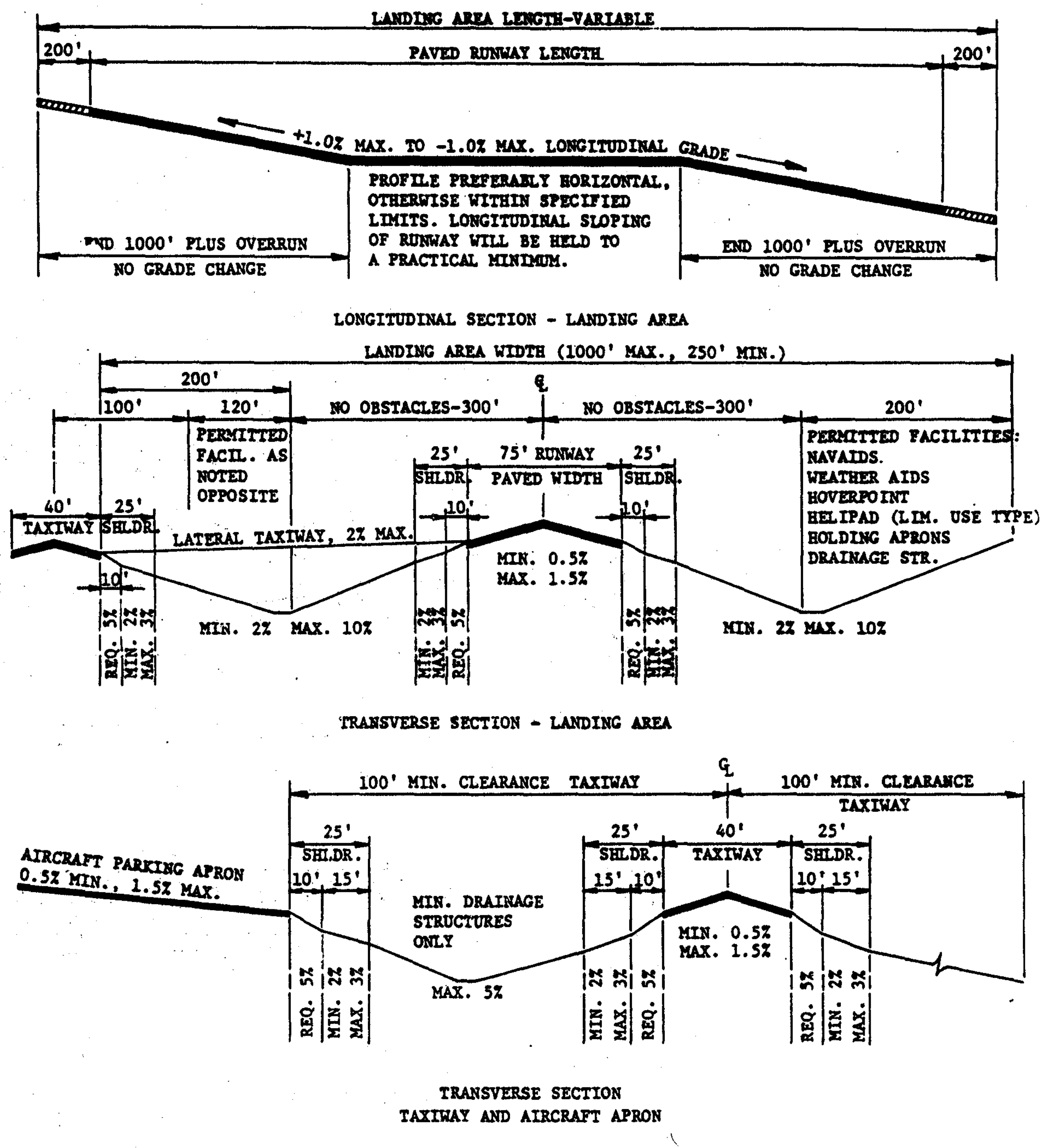

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FIGURE 10-2. AIRFIELD RUNWAY, TAXIWAY, APRON, AND OVERRUN GRADES paragraph 15-2. In particularly erosive soils, special methods will be necessary to establish the turf quickly or to provide supplemental protection by mulching or similar means. Where excessive velocities are to be encountered or where satisfactory turf cannot be established and maintained, it may be necessary to provide a paved channel.

d. Turbulence. An abrupt change in the normal flow pattern induces turbulence and results in 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.

e. Inflow conditions. Uncontrolled inflow from drainage areas adjacent to open channels has been a source of numerous failures and requires special consideration in the design of a surface drainage system. This local inflow 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. Experience indicates the desirability of constructing a berm at the top edge of the channel to prevent inflow except at designated points where an inlet properly protected against erosion is 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. See part four chapter 15 for more details on turfs.

f. Froude number of flow. 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 equation 10-1:

$$F = 1.88\ (d_{50}/D)^{1/3} \qquad \text{(eq 10-1)}$$

where:

F = Froude number of flow
d_{50} = diameter of average size stone, feet
D = maximum desired depth of flow, feet

10-3. Design problem.

a. Design procedure. 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. The following steps will permit the design of a channel that will satisfy the conditions desired for the design discharge and one that will insure no deposition or erosion under these conditions.

(1) Determine gradation of material common to drainage basin from representative samples and sieve analyses.

(2) Determine maximum discharges to be experienced annually and during the design storm.

(3) Assume maximum desirable depth of flow, D, to be experienced with the design discharge.

(4) Based on the gradation of the local material (sizes and percentages of the total by weight) exercise judgment as to the sizes of material that should and should not be transported. 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. Compute ratios of the diameter of the materials that should and should not be transported at the maximum depth of flow, d_{50}/D.

(5) Compute the Froude numbers of flow required to initiate transport of the selected sizes of cohesionless materials based on equation 10-1 to determine the range of F desired in the channel.

b. Channel design. Design the desired channel as indicated in the following steps.

(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 cfs. Also assume that a channel depth of 6 feet is the maximum that can be tolerated from the standpoint of the existing ground water level, minimum freeboard of 1 foot, and other considerations such as ease of excavation, maintenance, and aesthetics.

(2) From figure 10-3 or equation 10-1, 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 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

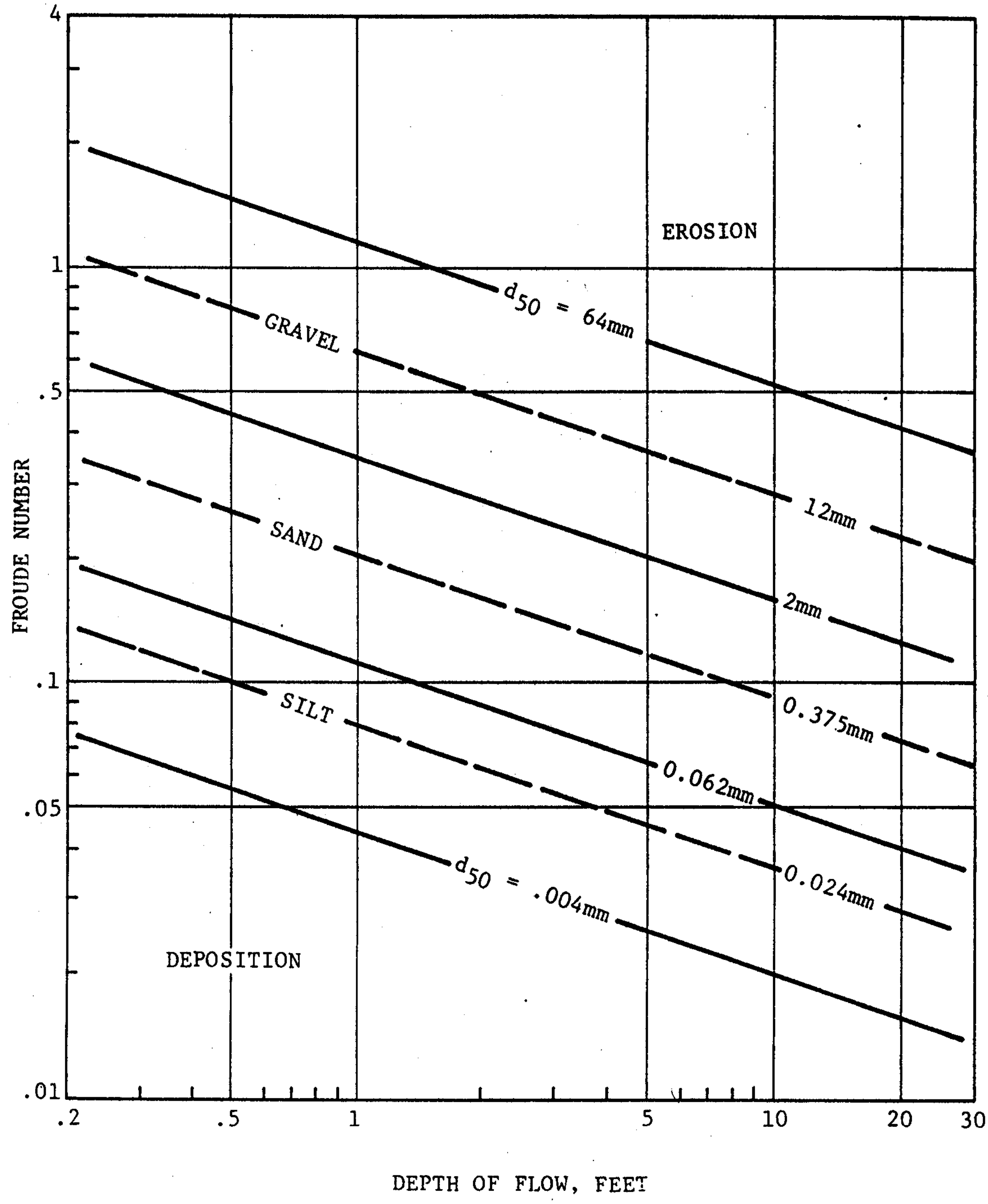

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FIGURE 10-3. FROUDE NUMBER AND DEPTH OF FLOW REQUIRED FOR INCIPIENT TRANSPORT OF COHESIONLESS MATERIAL should not cause severe erosion or deposition of the medium sand common to the basin with a flow depth of 5-foot in the desired channel.

(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 cfs/foot of width from the following equation:

$$q = 10.66\ d_{50}{}^{1/3} D^{7/6}$$

where:

q = discharge per foot of width, cfs/foot
d_{50} = diameter of average size stove, feet
D = depth of flow in channel, feet

or figure 10-4. In addition, it is indicated that a unit discharge of about 13 cfs/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 cfs/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.

(4) The width of a rectangular channel (B) and the average width of a trapezoidal channel required to convey the maximum rate of runoff of 400 cfs can be determined by dividing the design 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 (B) with side slopes of 1V-on-3H required to convey the design discharge with a 5-foot depth of flow would be 25 feet.

(5) The values of the parameters D/B and $Q/(gB^5)^{1/2}$ can now be calculated as 0.2 and 0.0225, respectively. Entering figure 10-5 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_{ch}, respectively. Assuming a Manning's n of 0.025, a slope of 0.000203 ft/ft 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.

(6) The Froude number of flow in the channel is 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

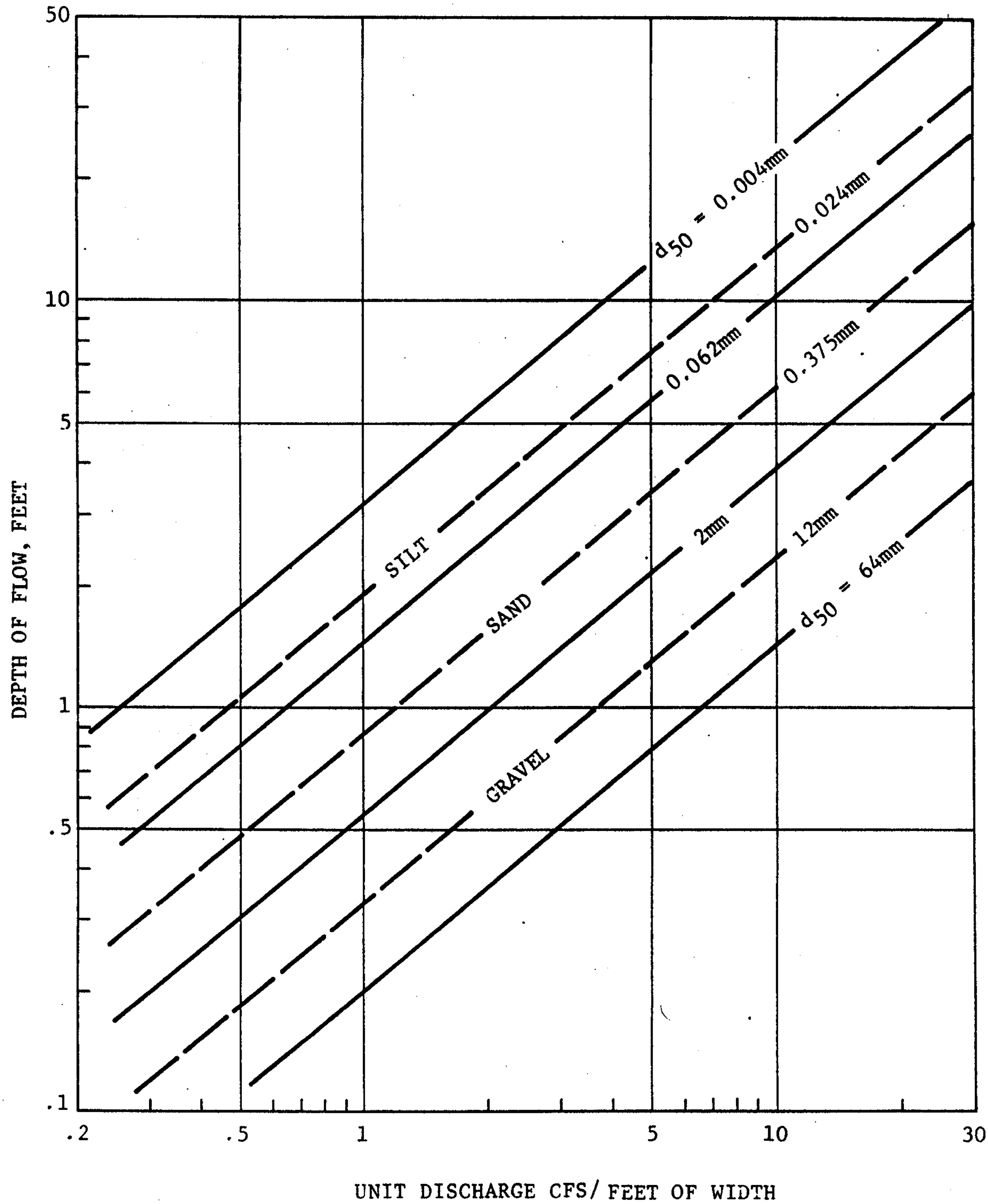

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FIGURE 10-4. DEPTH OF FLOW AND UNIT DISCHARGE FOR INCIPIENT TRANSPORT OF COHESIONLESS MATERIAL

DEFINITION OF TERMS:

- D = DEPTH OF FLOW IN CHANNEL, FEET
- B = BASE WIDTH OF CHANNEL, FEET
- Q = DISCHARGE, CFS
- G = ACCELERATION DUE TO GRAVITY, FEET/SECOND
- S = SLOPE OF CHANNEL BOTTOM
- N = MANNING'S COEFFICIENT OF ROUGHNESS

F_{ch} = FROUDE NUMBER OF FLOW IN CHANNEL

$$F_{ch} = \frac{Q}{\sqrt{g\frac{A^3}{T}}}$$

WHERE:

A = CROSS SECTIONAL AREA OF FIGW IN CHANNEL, SQ. FT.

T = TOP WIDTH OF FLOW IN CHANNEL, FEET

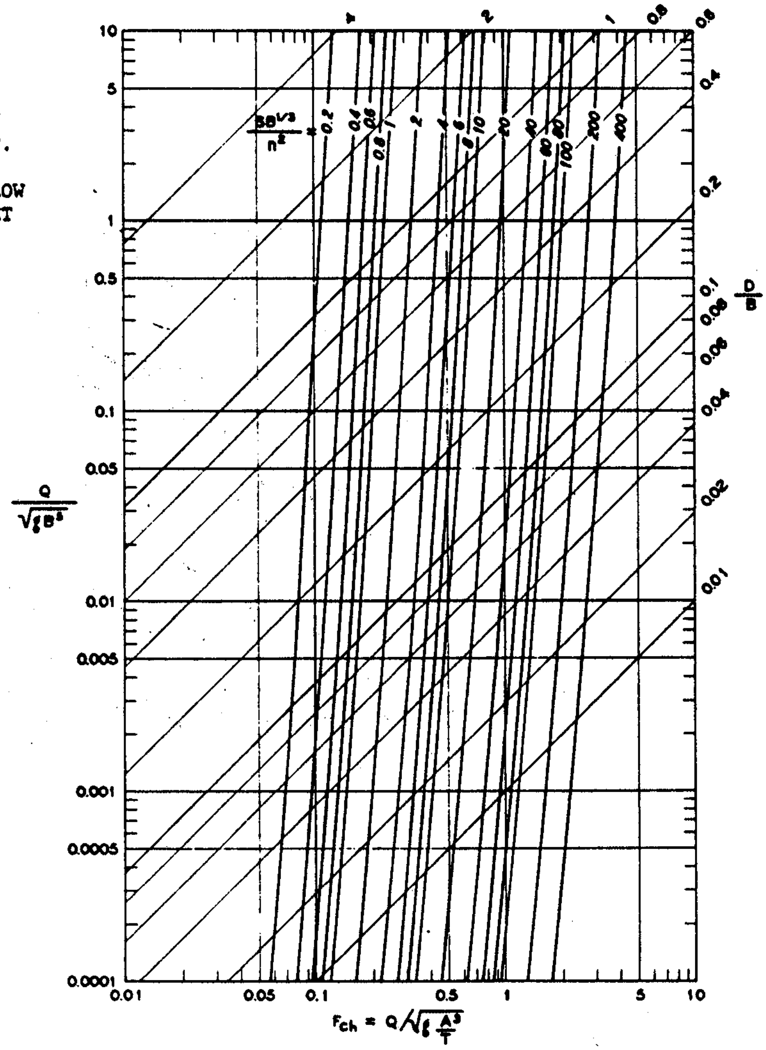

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FIGURE 10-5. FLOW CHARACTERISTICS OF TRAPEZOIDAL CHANNELS WITH 1-ON-3 SIDE SLOPES desired to convey the design discharge of 400 cfs with a Froude number of 0.16 in a trapezoidal channel of 25-foot base width and 1V-on-3H side slopes, the values of 0.0225 and 0.16 for $Q/(gB^5)^{1/2}$ and F_{ch}, respectively, can be used in conjunction with figure 10-5 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/foot would be required for the channel to convey the flow with a Froude number of 0.16.

(7) The slopes required for either the rectangular or the trapezoidal channels are extremely mild. In the event that 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.

(8) Assume that a source of stone exists 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 cfs with depths of flow up to 5 feet can be investigated as follows. The equation, $F = 1.42\ (d_{50}/D)^{1/3}$, or figure 10-6 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. Similarily, the maximum unit discharge permissible (33 cfs/foot of width) can be determined by the following equation:

$$q = 8.05\ {d_{50}}^{1/3} D^{7/6}$$

or figure 10-7. For conservative design, it is recommended that the maximum unit discharge be limited to about 2/3 of this value or say 22 cfs/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 cfs 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.

(9) The values of D/B and $Q/(gB^5)^{1/2}$ can be calculated as 1.67 and 4.52, respectively. Entering figure 10-5 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_{ch}, respectively. Assuming 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/foot would be required for the 5-foot-deep riprap lined trapezoidal channel with base

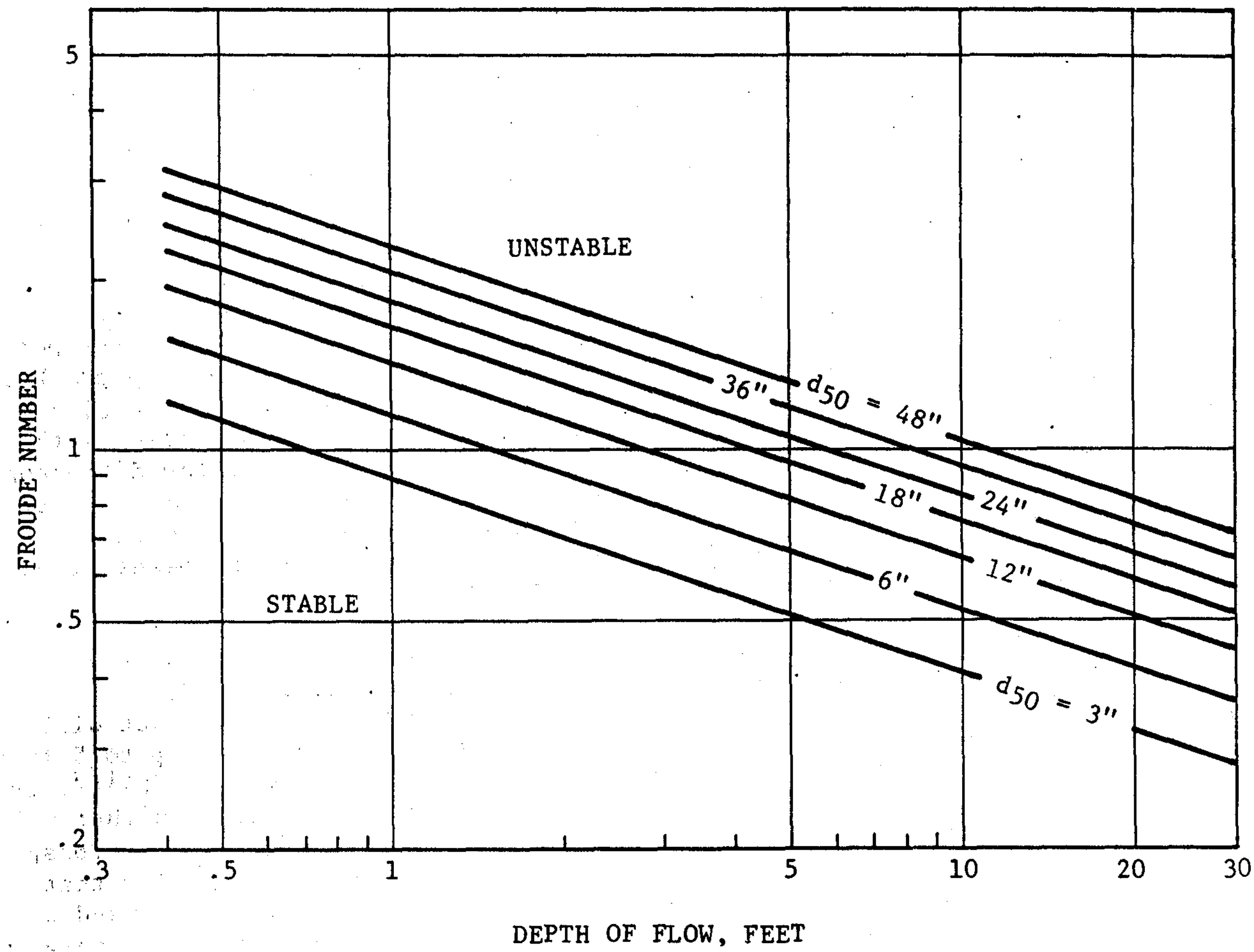

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FIGURE 10-6. FROUDE NUMBER AND DEPTH OF FLOW FOR INCIPIENT FAILURE OF RIPRAP LINED CHANNEL

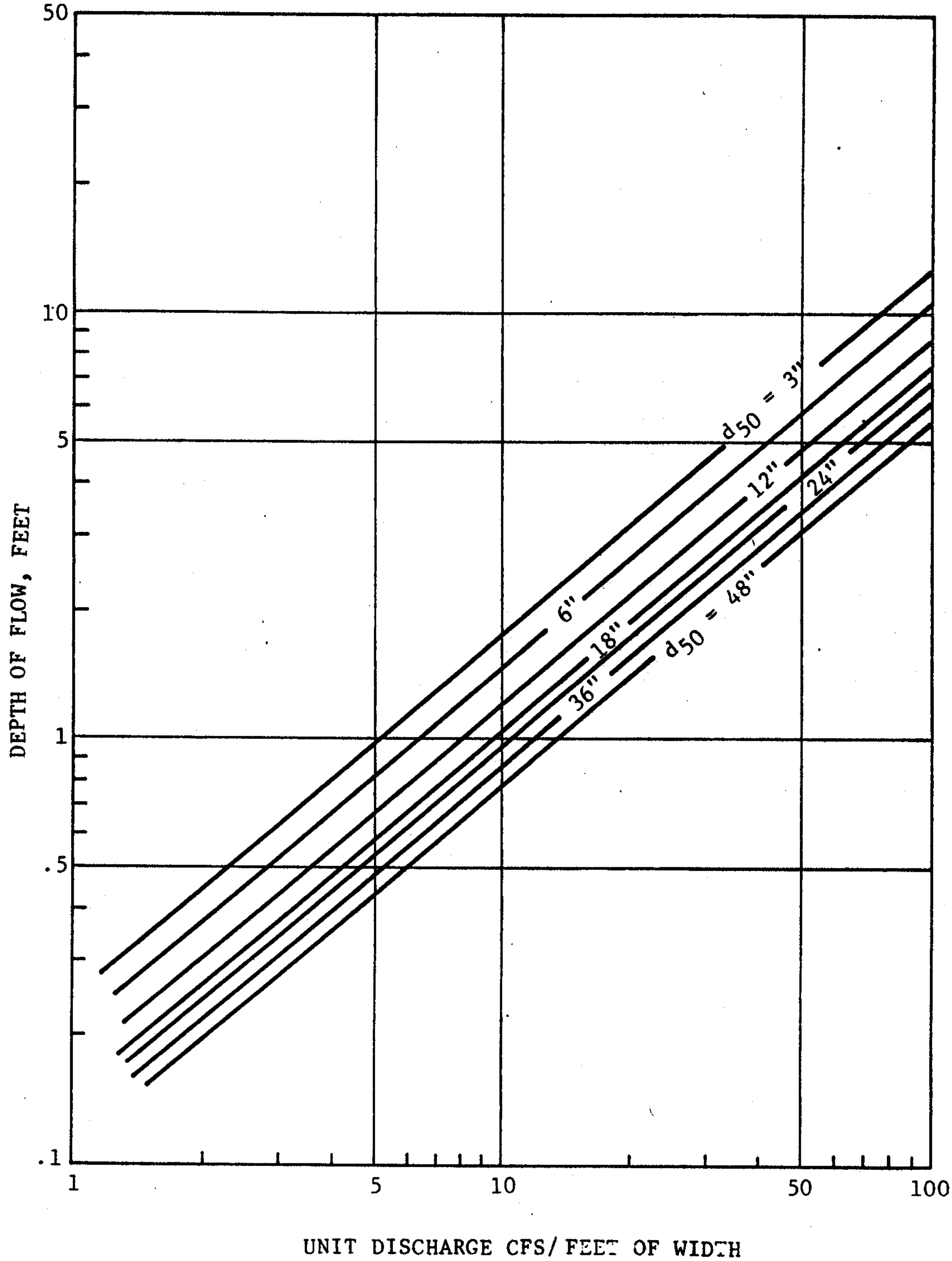

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FIGURE 10-7. DEPTH OF FLOW AND UNIT DISCHARGE FOR INCIPIENT FAILURE OF RIPRAP LINED CHANNEL width of 3 feet and 1V-on-3H side slopes. The Froude number of flow in the channel would be satisfactory relative to the stability of the 3-inch-diameter average size of riprap as well as the maximum recommended value of 0.8 to prevent instabilities of flow and excessive wave heights in subcritical open channel flow.

(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.

(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 a slope commensurate with that of the local terrain.

(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/foot (average slope of local terrain) for conveyance of a design discharge of 400 cfs 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 10-8. Corresponding values of the parameter $Q/(gB^5)^{1/2}$ (0.44-0.68) can also be determined from figure 10-8 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 10-9 to determine the respective relations for supercritical rectangular channels with a depth-to-width ratio of 0.5, a slope of 0.01 foot/foot, and a Manning's n of 0.015. Figure 10-9 may then be used to select a channel width of 7.5 feet for conveyance of the design discharge of 400 cfs. 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 10-8 to obtain corresponding values of the remaining constraining parameters, $Q/(gB^5)^{1/2} = 0.48$ and $F = 1.4$, required to satisfy all of the dimensionless relations shown in figure 10-8. 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

DEFINITION OF TERMS:

D = DEPTH OF FLOW IN CHANNEL, FEET
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Q = DISCHARGE, CFS
G = ACCELERATION DUE TO GRAVITY, FEET/SECOND
S = SLOPE OF CHANNEL BOTTOM
N = MANNING'S COEFFICIENT OF ROUGHNESS

$$F_{ch} = \frac{Q}{\sqrt{g\,\frac{A^3}{T}}}$$

WHERE:

A = CROSS SECTIONAL AREA OF FIGW IN CHANNEL, SQ. FT.

T = TOP WIDTH OF FLOW IN CHANNEL, FEET

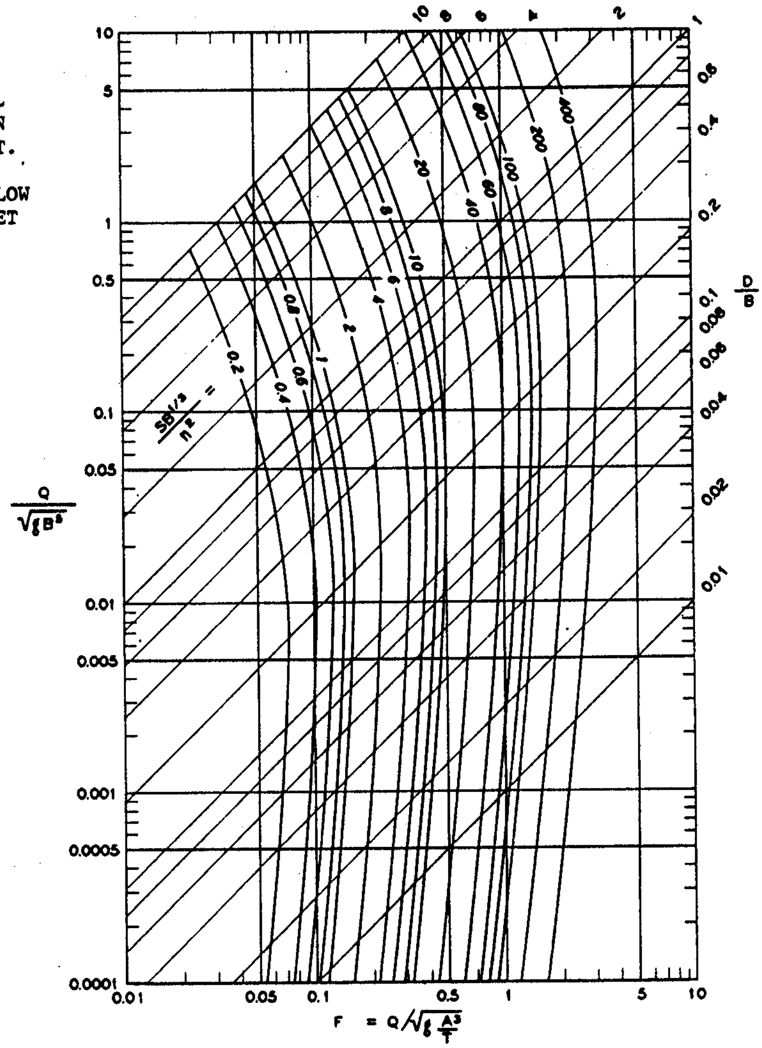

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FIGURE 10-8. FLOW CHARACTERISTICS OF RECTANGULAR CHANNELS

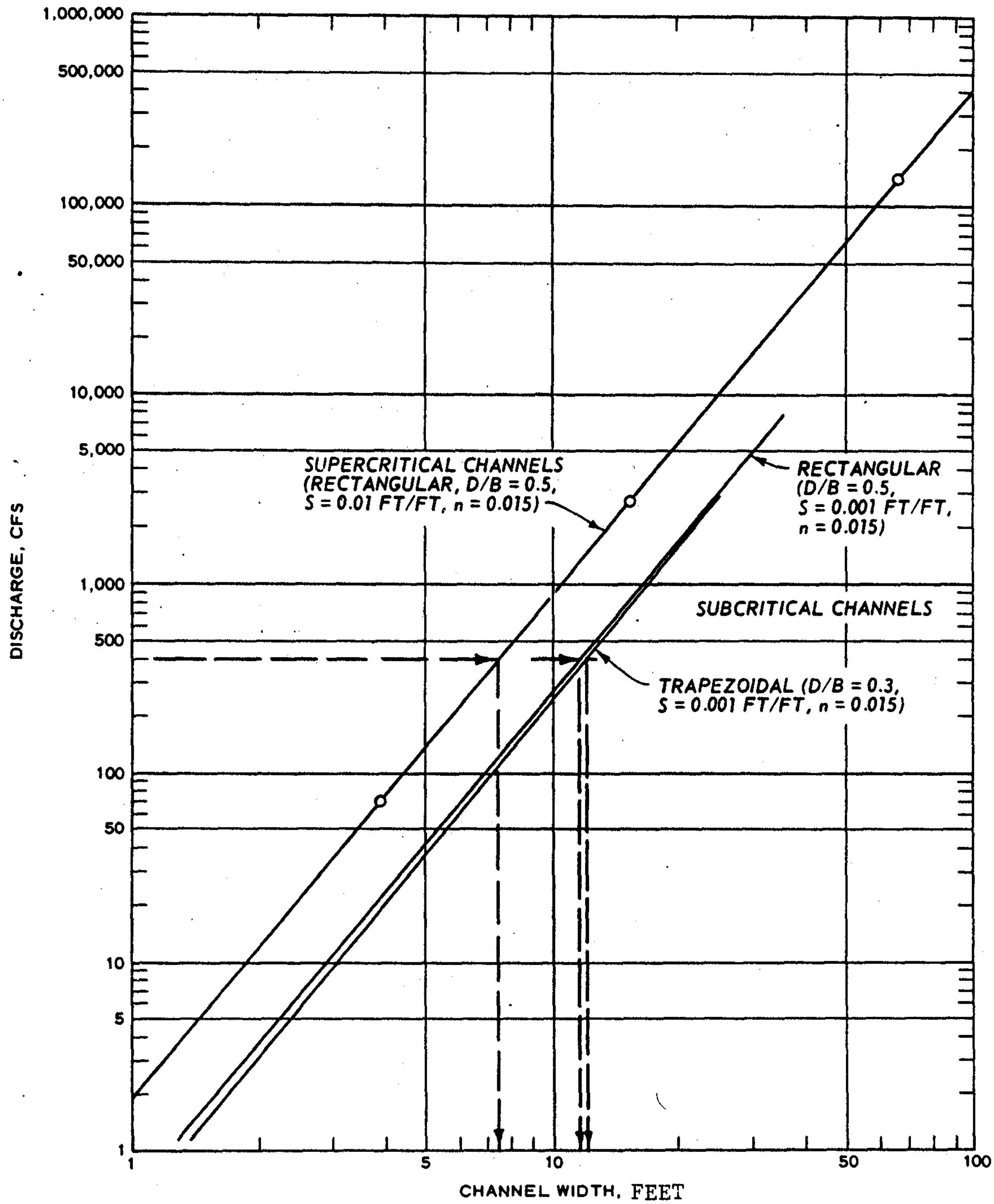

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FIGURE 10-9. DISCHARGE CHARACTERISTICS OF VARIOUS CHANNELS relations to insure the adequacy of the selected design. For example, based on the magnitude of a discharge parameter equal to 0.84, the channel should convey 419 cfs:

$$Q = 0.48\ (g(7.5)^5)^{1/2} = 419 \text{ cfs}$$

Similarly, based on the magnitude of a Froude number of flow equal to 1.4 the channel should convey a discharge of 432 cfs:

$$Q = 1.4\left(\frac{g(7.5 \times 3.75)^3}{7.5}\right)^{1/2} = 432 \text{ cfs}$$

Obviously, the capacity of the 7.5-foot-wide channel is adequate for the design discharge of 400 cfs.

(13) The feasibility of paved channel with a slope compatible with that of the local terrain 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 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 ratio as near 0.5 as practical for hydraulic efficiency; if trapezoidal, they should be designed to have side slopes of 1V-on-3H and a depth-to-width ratio of 0.3.

(14) For example, assume a subcritical paved channel with a Manning's n of 0.015 and slope of 0.01 foot/foot is to be provided for a design discharge of 400 cfs. 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 10-8. For a $D/B = 0.5$ and Froude number of flow of 0.8, the corresponding values of $SB^{1/3}/n^2$ and $Q/\ gB^5$ are determined as 30 and 0.275, respectively. Solving these relations for S and Q based on $n = 0.015$ and $B = 1$ foot yields.

$$S = 30\ n^2/B^{1/3} = 0.00675 \text{ ft/ft}$$
$$Q = 0.275\ g^{1/2}\ B^{5/2} = 1.56 \text{ cfs}$$

Greater widths of hydraulically efficient rectangular channels would convey greater discharges, but slopes flatter than 0.00675 foot/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/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/foot is not feasible.

(15) Assuming that the average slope of the local terrain was about 0.001 foot/foot for the example problem, practical subcritical paved channels could be designed as discussed in (16) through (19) below.

(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 10-8 indicates the corresponding range of values of the restraining parameters $SB^{1/3}/n^2$ and $Q/\ gB^5$ 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 0.01 foot/foot, and a Manning's n of 0.015 can be plotted as shown in figure 10-9 to select the 11.5-foot-width of channel required to convey the design discharge of 400 cfs.

(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 10-8 to obtain corresponding values of the remaining constraining parameters, $Q/(gB^5)^{1/2} = 0.16$ 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 insure the adequacy of the selected design. For example, based on the magnitude of the discharge parameter (0.16), the channel should convey 407 cfs;

$$Q = 0.16\ (g(11.5)^5)^{1/2} = 407 \text{ cfs}$$

Similarly, based on the Froude number of flow equal to 0.47, the channel should convey a discharge of 422 cfs:

$$Q = 0.47 \left(\frac{g\ (11.5 \times 5.75)^3}{11.5} \right)^{1/2} = 422 \text{ cfs}$$

Therefore, the 11.5-foot-wide channel is sufficient for subcritical conveyance of the design discharge of 400 cfs and, based on figure 10-3 is sufficient for transporting materials as large as average size gravel.

(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/foot, and a Manning's n of 0.015 utilizing figure 10-5. 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 (fig 10-5). The relations

between discharge and base width for these subcritical trapezoidal channels were plotted as shown in figure 10-4 to select the 12-foot-base width required to convey the design discharge of 400 cfs.

(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 10-6 to obtain corresponding values of the remaining constraining parameters, $Q/(gB^5)^{1/2} = 0.15$ 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 cfs, respectively.

$$Q = 0.15\,(g(12)^5)^{1/2} = 425 \text{ cfs}$$

$$Q = 0.63\left(\frac{g\,(45.6/2 \times 3.6)^3}{33.6}\right)^{1/2} = 458 \text{ cfs}$$

Therefore, the selected trapezoidal channel is sufficient for subcritical conveyance of the design discharge of 400 cfs and based on figure 10-3 is sufficient for transporting materials as large as coarse gravel.

c. Channel analysis. 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 insure that deposition and/or erosion will not be experienced 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 have considerable merit 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.

CHAPTER 11

CULVERTS

11-1. General.

a. Configuration. Culverts are generally of circular, oval, elliptical, arch, or box cross section and may be of either single or multiple construction, the choice depending on available headroom and economy. Culvert materials include plain concrete, reinforced concrete, asbestos cement, clay, and plastic. These pipes are considered smooth for flow characteristics; additional materials include corrugated metal and plastics (considered corrugated for flow characteristics). 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 requirements determined, the design requires detailed consideration of such hydraulic factors as shape and slope of approach and exit channels, tailwater levels, hydraulic and energy gradelines, 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.

b. Capacity. The capacity of a culvert is the ability of a culvert 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.

c. Culvert shapes. The majority of cases of culvert uses will involve round pipe or box culverts; therefore the following discussion will center around circular and square configuration culverts. Additional information on other configurations can be found in appendix A. Ponding at inlets to culverts immediately adjacent to runways and roads is discouraged. However, where large variances are present between the design storm and yearly peak storms indicating only rare occurrences of design storm magnitude, ponding for these rare occurrences will be permitted.

11-2. Culvert flow. Laboratory tests and field observations show two major types of culvert flow: flow with inlet control and flow with outlet control. In some instances the flow control changes with change in discharge, 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. The two types of flow are discussed briefly in subsequent paragraphs.

a. Inlet control. The discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the area, slope, and type of inlet edge. Inlet flow for unsubmerged entrance is shown in figures 11-1a and submerged entrance in figure 11-1b. A mitered entrance produces little if any improvement in efficiency over that of the straight, sharp-edged, projecting inlet for inlet control. Both types of inlets tend to inhibit the culvert from flowing full. 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. Nomographs (figures 11-2, 11-3, and 11-4) give headwater-discharge relations for most conventional culverts flowing with inlet control. See appendix A for nomographs with modified inlets (flared, tapered, beveled, etc.) or of noncircular shapes.

b. Outlet control. Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it. Two common types of outlet-control flow are shown in figures 11-1c and 11-1d. The procedure given in this chapter for outlet-control flow does not give an exact solution for a free-water-surface condition throughout the barrel length shown in figure 11-1a. However, 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. These three parts are usually expressed in feet of water and include a velocity head, an entrance loss, and a friction loss.

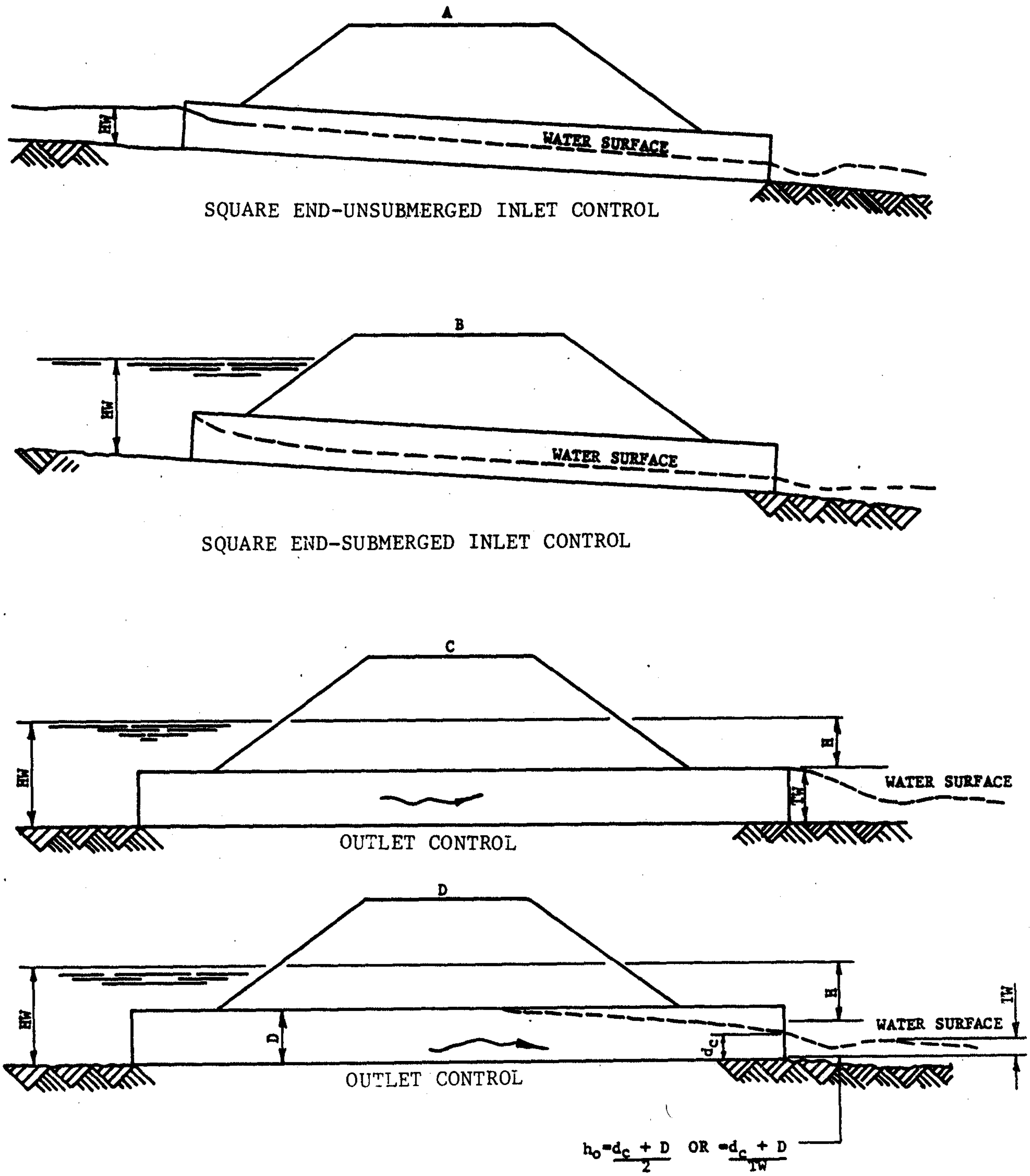

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FIGURE 11-1. INLET AND OUTLET CONTROL

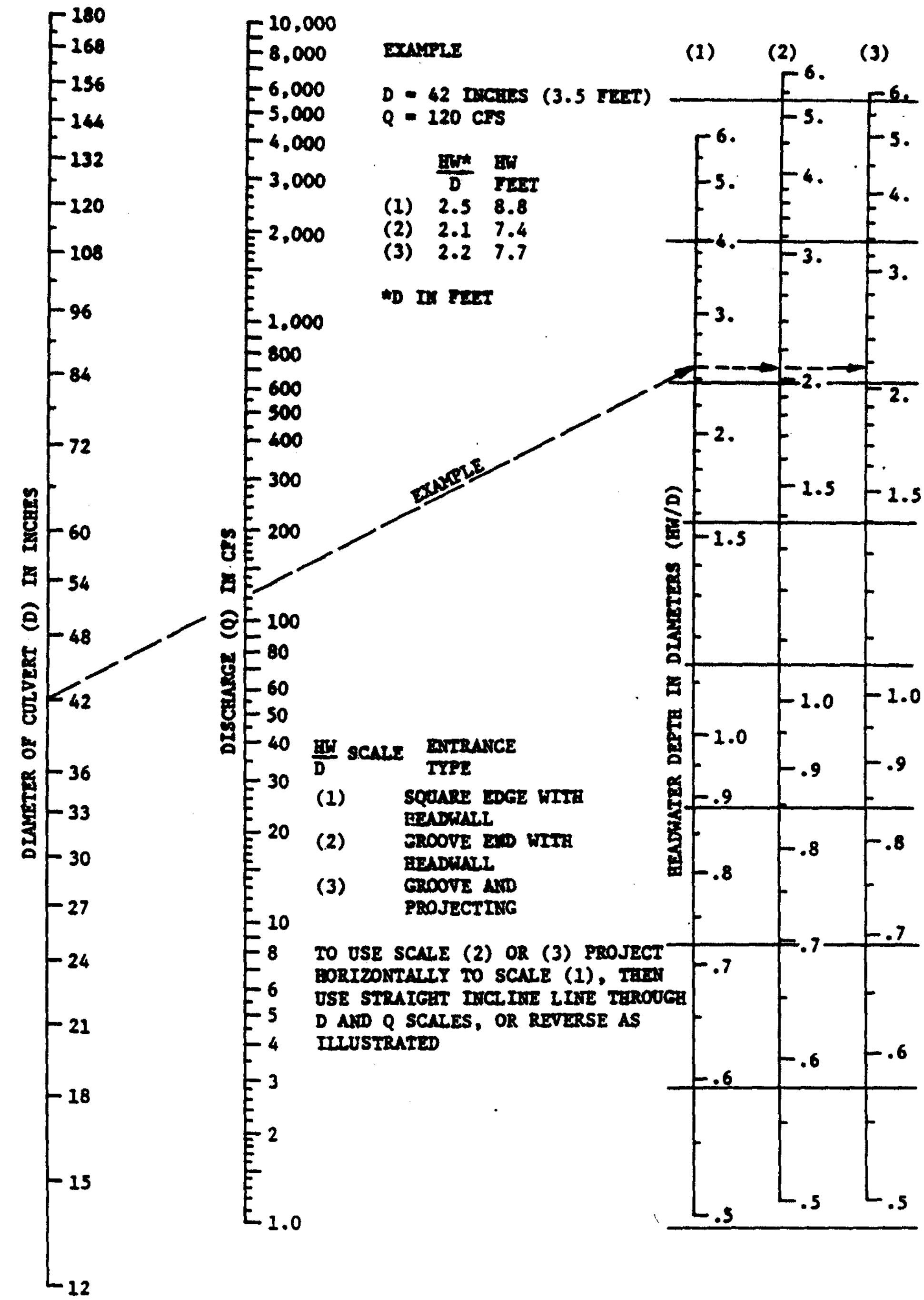

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FIGURE 11-2. HEADWATER DEPTH FOR SMOOTH PIPE CULVERTS WITH INLET CONTROL

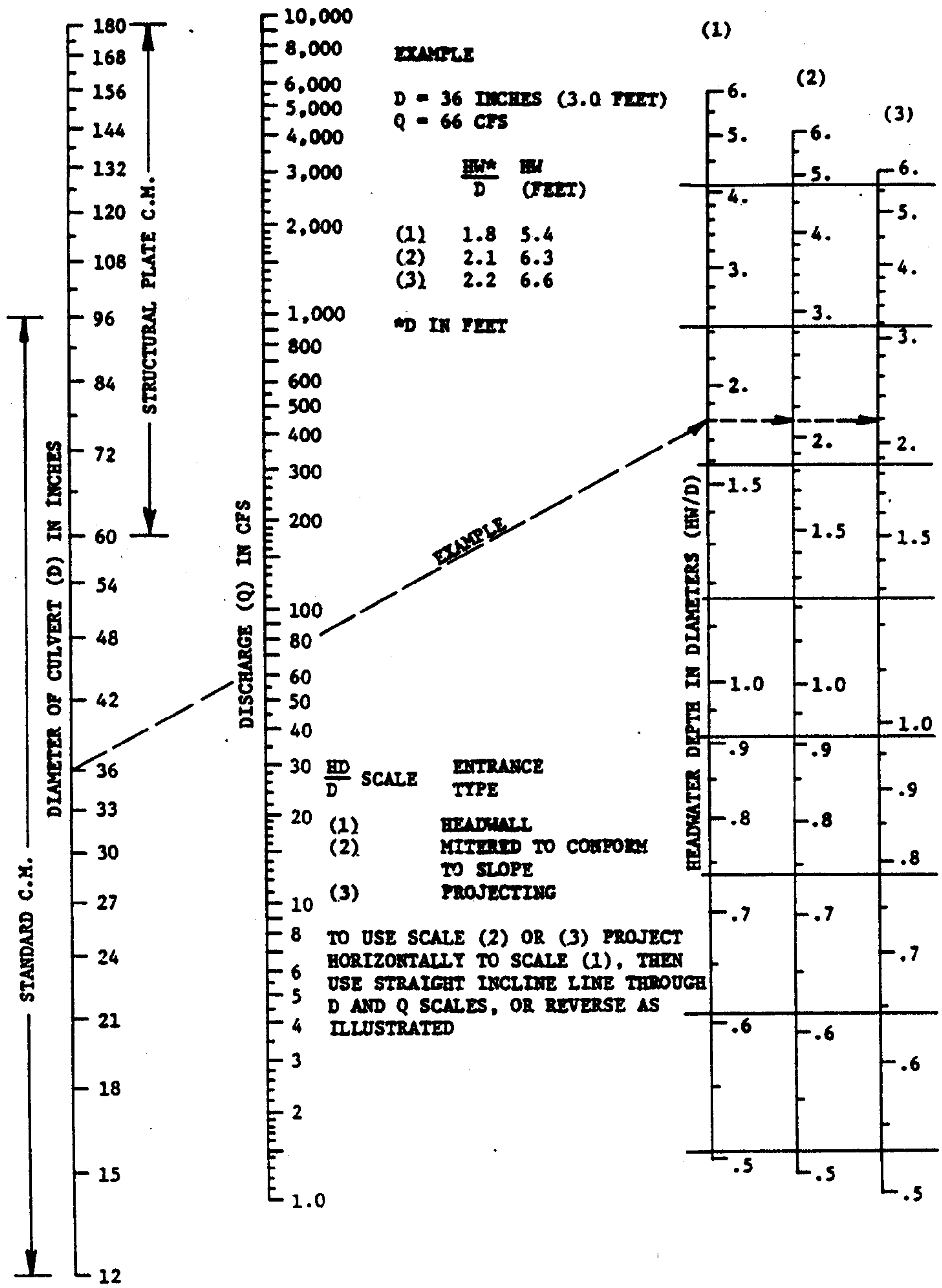

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FIGURE 11-3. HEADWATER DEPTH FOR CORRUGATED PIPE CULVERTS WITH INLET CONTROL

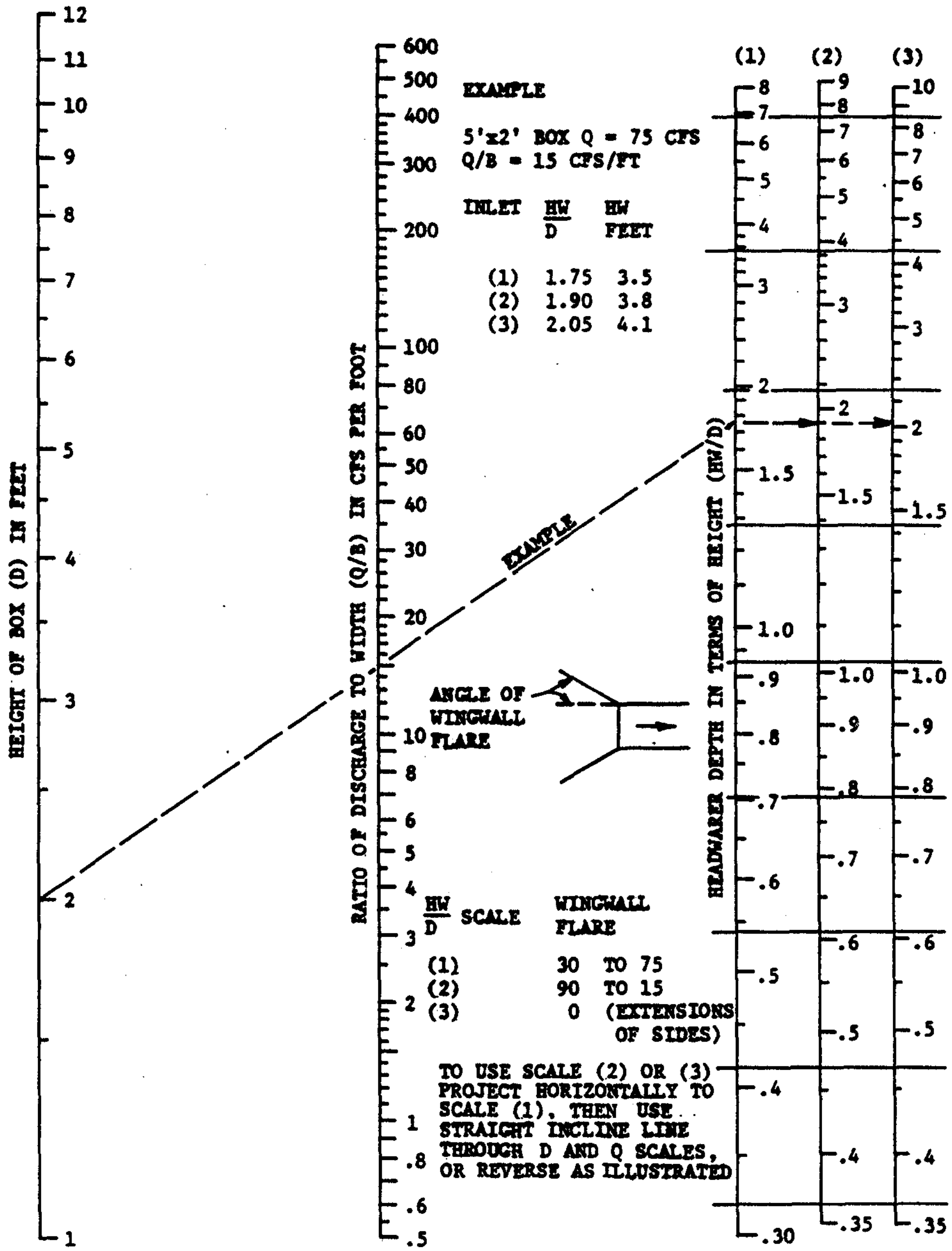

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FIGURE 11-4. HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

(1) The following equation represents the three parts when simplified for full pipe flow:

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

where:

H = head in feet (see figure 11-1b)
k_e = entrance loss coefficient (See table 11-1)
n = Manning's friction factor
L = length of culvert barrel in feet
R = hydraulic radius in feet, or A/WP
A = area of flow for full cross section in square feet
WP = wetted perimeter in feet
V = mean velocity of flow in culvert barrel in feet per second
g = acceleration of gravity in feet per second^2 (32.16)

This equation can be solved readily by the use of full flow nomographs, figures 11-5, 11-6, and 11-7. Each nomograph is drawn for a single value of n as noted in the respective plate. These nomographs may be used for other values of n by modifying the culvert length as:

$$\text{Modified Length (to be entered in nomograph)} = \left(\frac{\text{Actual Length}}{\frac{\text{Desired n}}{\text{n (for nomograph)}}}\right)^2$$

(2) 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:

$$HW = h_o + H - LS_o$$

where S_o is the slope of the flow line in feet per foot and all terms are in feet. The determination of h_o for various flow conditions at the outlet is discussed below.

(a) When the pipe is running full (fig 11-1c) tail water (TW) depth equals the depth of the culvert (D) equals h_o.

(b) When the tailwater elevation is below the top of the crown of the culvert outlet (fig 11-1d), h_o is the greater of TW (depth of outlet channel fig 11-1d) or the following.

Table 11-1.

Entrance Loss Coefficients Outlet Control,
Full or Partly Full
Entrance Head Loss $H_e = K_e V^2/2g$

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 slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Boxed, 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° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*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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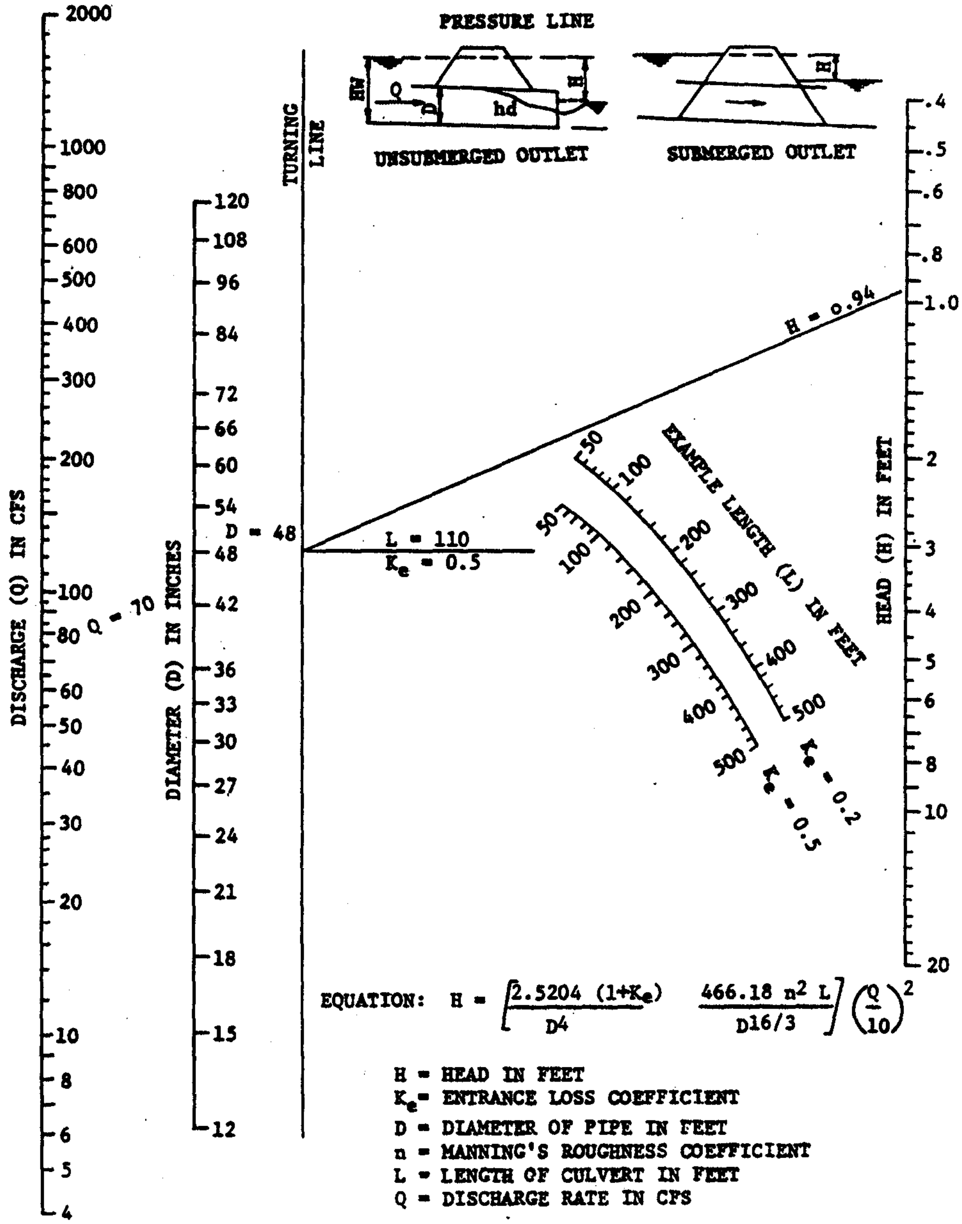

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FIGURE 11-5. HEAD FOR SMOOTH PIPE CULVERTS FLOWING FULL n = 0.012

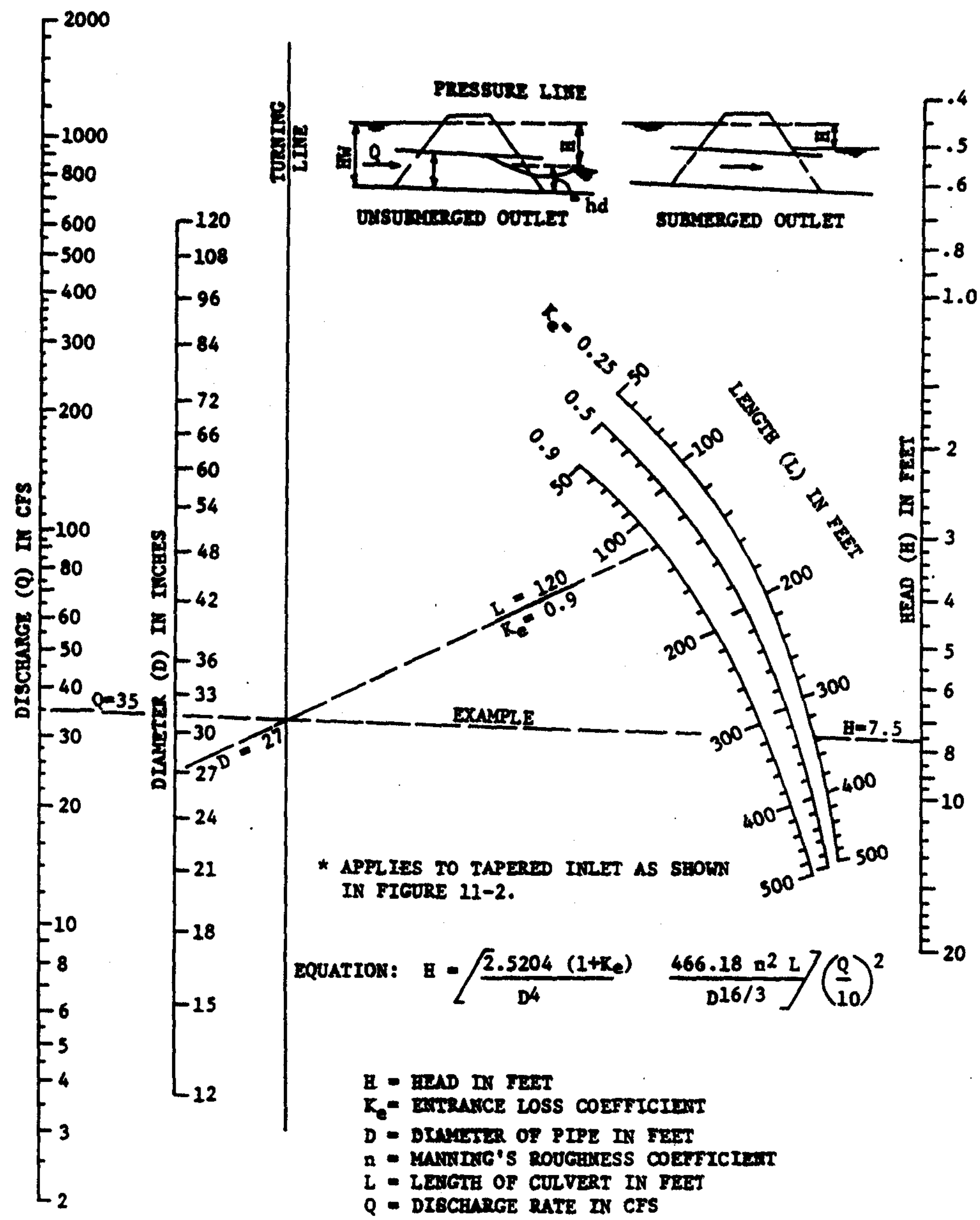

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FIGURE 11-6. HEAD FOR CORRUGATED PIPE CULVERTS FLOWING FULL n = 0.024

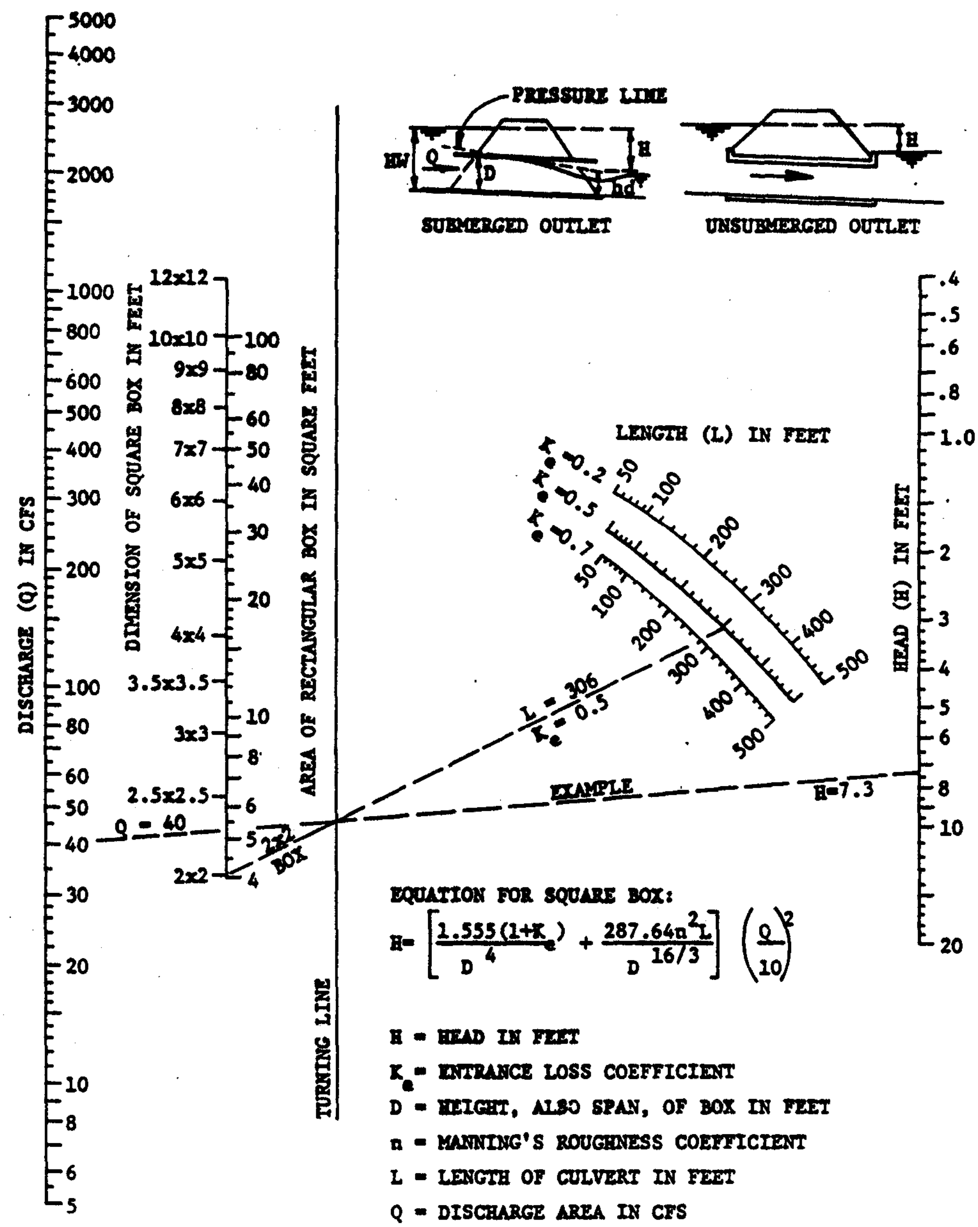

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FIGURE 11-7. HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n = 0.012

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

where d_c equals the critical depth as determined from figures 11-8 and 11-9. The value of d_c should never exceed D.

(3) If the headwater (HW), as determined by the equation $HW = h_o + H - LS_o$, is less than

$$HW = D + (1 + k_e)\ V^2/2g$$

and HW is less than 0.75D (similar to figure 11-1a) then a natural stream flow exists and water volume can be determined by using Manning's equation $V = (1.486/n)\ R^{2/3}\ S^{1/2}$. See table 11-2 for typical n values.

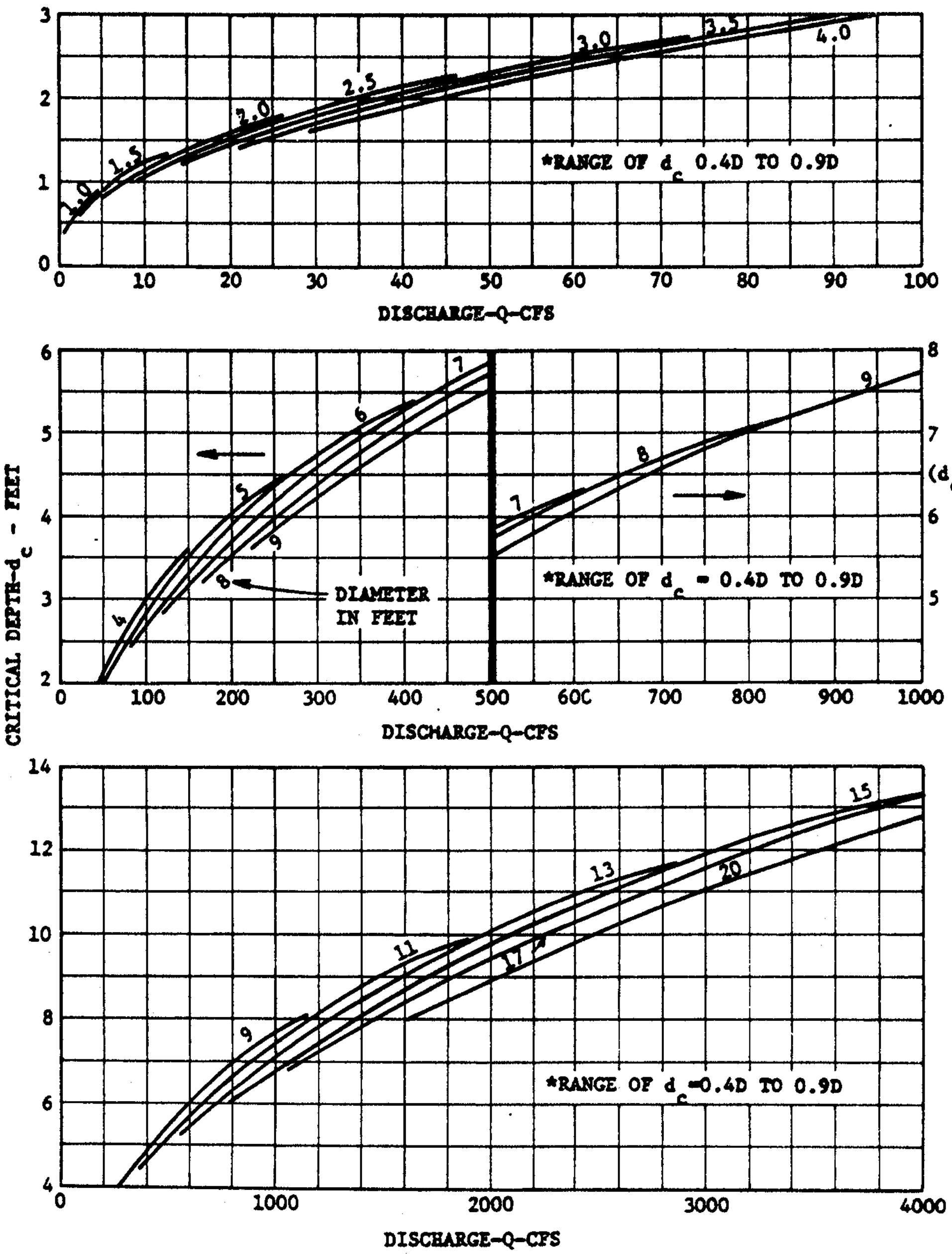
Table 11-2. Manning's n for Natural Stream Channels
(Surface width at flood stage less than 100 feet)

Channel Type	Values of n
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 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

11-3. Procedure for selection of culvert size.

Step 1: List given data

a. Design discharge Q, in cubic feet per second.



*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

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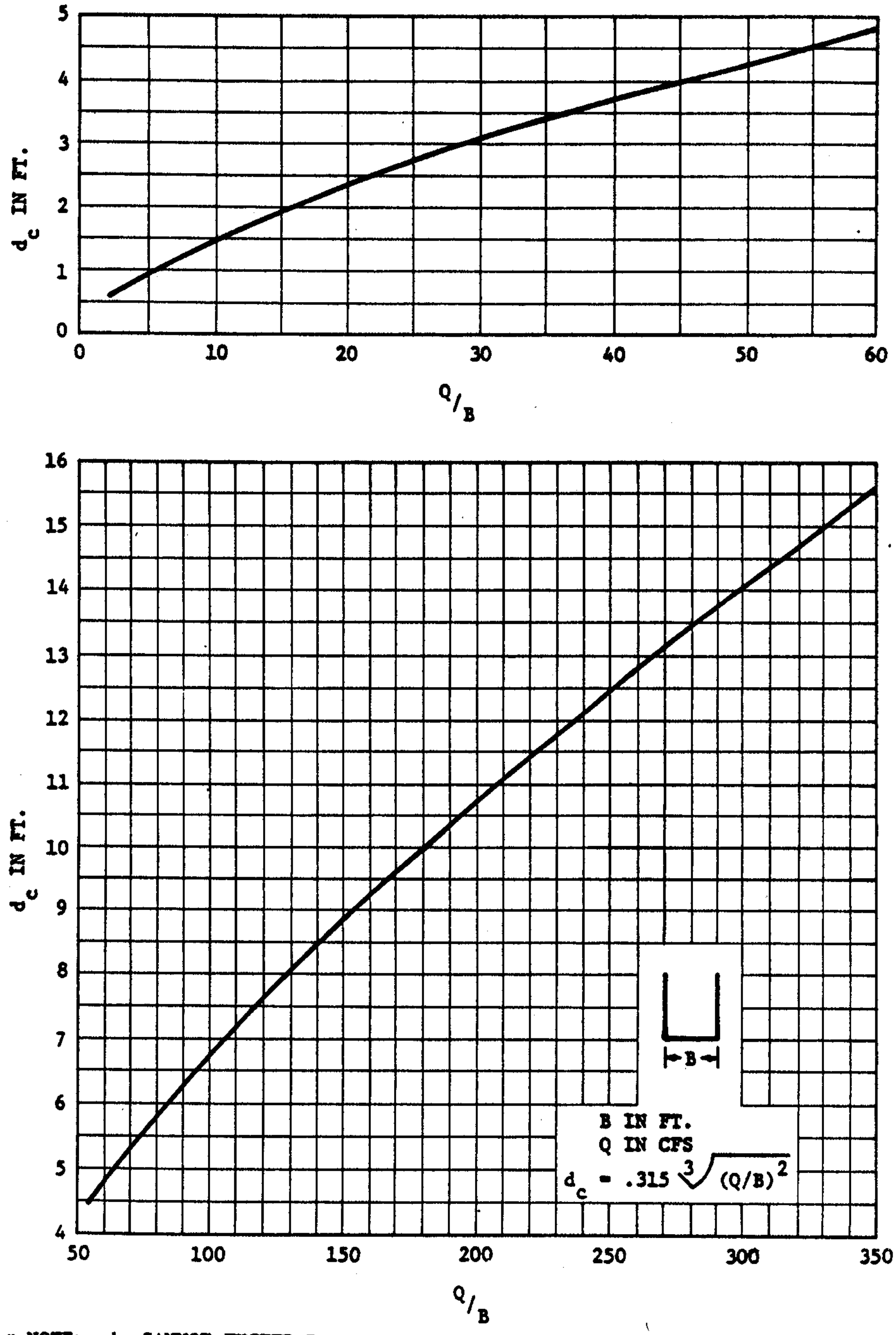
FIGURE 11-8. CIRCULAR PIPE CRITICAL DEPTH



* NOTE: d_c CANNOT EXCEED D

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FIGURE 11-9. CRITICAL DEPTH RECTANGULAR SECTION

b. Approximate length of culvert, in feet.

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

d. Type of culvert, including barrel material, barrel cross-sectional shape, and entrance type.

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

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

Step 2: Determine a trial-size culvert.

a. Refer to the inlet-control nomograph (figures 11-2, 11-3, and 11-4) for the culvert type selected.

b. Using an 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 lesser or greater relative depth of headwater in a particular case should exist, another value of HW/D may be used for this trial selection.

c. If the trial size for the culverts is obviously too large because of limited height of embankment or availability of size, try a different HW/D value or multiple culverts by dividing the discharge equally for the number of culverts used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Selection should be based on an economic analysis.

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

a. Determine and record headwater depth by use of the appropriate inlet-control nomograph (figures 11-2, 11-3, and 11-4). Tailwater conditions are to be neglected in this determination. Headwater in this case is found by simply multiplying HW/D obtained from the nomograph by D.

b. Compute and record headwater for outlet control as instructed below:

(1) 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.

(2) 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:

$$HW = h_o + H - S_oL$$

where:

HW = vertical distance in feet from culvert invert (flow line at entrance to pool surface upstream.
h_o = vertical distance in feet from culvert flow line at outlet to control point. (In this case h_o equals TW.)
H = head loss in feet as determined from the appropriate nomograph (figures 11-5, 11-6, and 11-7).
S_o = slope of barrel in feet per foot.
L = culvert length in feet.

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

$$HW = h_o + H - S_oL$$

where:

$h_o = \frac{d_c + D}{2}$ or TW, whichever is greater.

Note: where d_c exceeds D in a rectangular section, h_o should be set equal to D.

d_c = critical depth in feet (figure 11-8 and 11-9)

D = culvert height in feet.

Other terms are as defined in (2) above.

c. 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.

d. 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.

Step 4: Check outlet velocities for size selected.

a. 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 A corresponding to d_c or TW depth, whichever gives the greater area of flow.

b. 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.

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

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

11-4. Instructions for use of inlet-control nomographs.

a. To determine headwater.

(1) 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 straightedge on HW/D scale 1.

(2) If HW/D scale 1 represents entrance type used, read HW/D on scale 1. If some other entrance type is used, extend the point of intersection (see subparagraph 11-4a(1) above) horizontally to scale 2 or 3 and read HW/D.

(3) Compute headwater by multiplying HW/D by D.

b. To determine culvert size.

(1) Given an HW/D value, locate HW/D on scale for appropriate entrance type. If scale 2 or 3 is used, extend HW/D point horizontally to scale 1.

(2) Connect point on HW/D scale 1 as found in subparagraph 11-4b(1) to given discharge and read diameter, height, or size of culvert required.

c. To determine discharge.

(1) Given HW and D, locate HW/D on scale for appropriate entrance type. Continue as in subparagraph 11-4b(1).

(2) Connect point on HW/D scale 1 as found in subparagraph 11-4c(1) and the size of culvert on the left scale and read Q or Q/B on the discharge scale.

(3) If Q/B is read, multiply by B to find Q.

11-5. Instructions for use of outlet-control nomographs. These nomographs solve for head when culverts flow full with outlet control. They are also used in approximating the head for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater.

a. To determine head for given culvert and discharge.

(1) Locate appropriate nomograph for type of culvert selected.

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

(a) If the n value of the nomograph corresponds to that of the culvert being used, find the proper k_e from table 11-1 and on the appropriate nomograph locate starting point on length curve for that k_e. If a k_e curve is not shown for the selected k_e, see (b) below. If the n value for the culvert selected differs from that of the nomograph, see (c) below.

(b) 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.

(c) 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:

$$L_1 = L(n_1/n)^2$$

(See subparagraph b below for n values.)

(3) Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line." See instruction c below for size considerations for rectangular box culvert.

(4) Pivot the straightedge 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 $H = KQ^2$ where K is found by substituting values of H and Q from chart.

b. To find the n value for a culvert. To find the n value for the culvert selected, refer to the following tabulation:

n value for concrete		n value for corrugated metal			
		Small corrugations (1/2 by 2-2/3 in.)		Large corrugations (2 by 6 in.)	
Pipe	Boxes				
0.013	0.013	Unpaved	0.024	Unpaved	0.033
		25% paved	0.021	25% paved	0.027
		Fully paved	0.013		

c. To use the box-culvert nomograph. To use the box-culvert nomograph (fig 11-7) for full flow for other than square boxes:

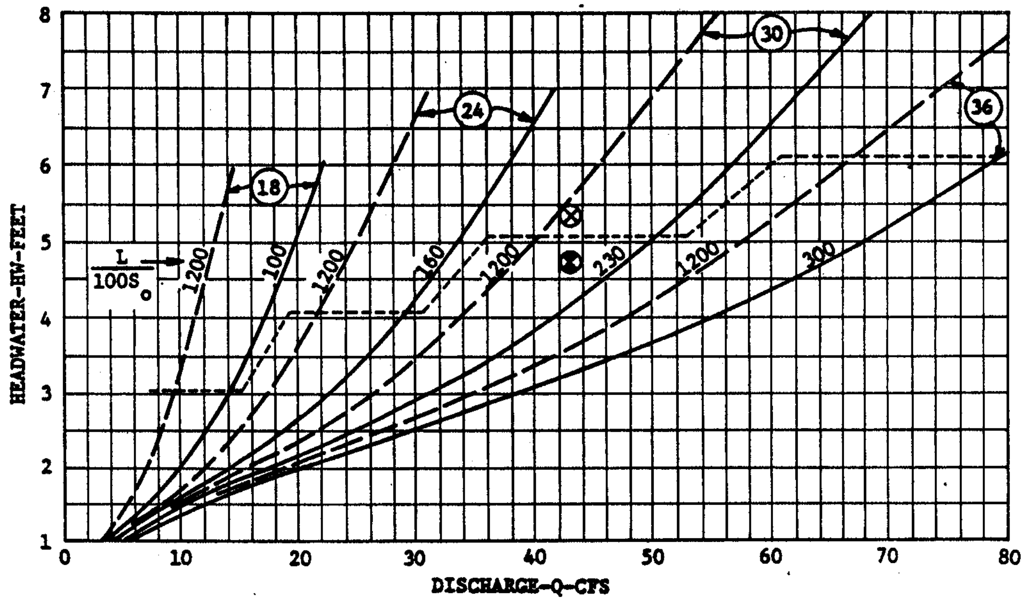
(1) Compute cross-sectional area of the rectangular box.

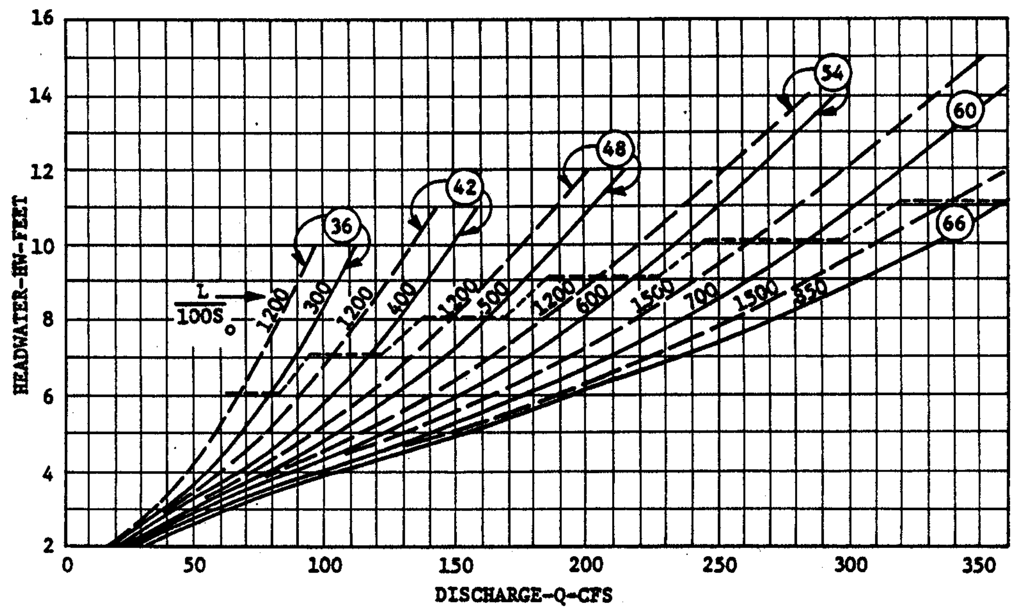
Note: The area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 2/3D. For other box proportions, use equation shown in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

(2) Connect proper point (see subparagraph 11-5a) on length scale to barrel area and mark point on turning line.

(3) Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head scale.

11-6. Culvert capacity charts. These charts, figures 11-10, 11-11, and 11-12, 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 $L/100S_o$ where L is culvert length in feet and S_o is culvert barrel slope in feet per foot. The length: slope ratio given on the solid line curve 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 $L/100S_o$ values is permitted in the range of headwater depths equal to or less than twice the barrel height. The upper limits of this range of headwater depths are designated by a horizontal dotted line on the charts. Values of $L/100S_o$ less than those given in the chart do not impose any limitation; merely read the solid-line 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





EXAMPLE

GIVEN:
43 CFS; AHW = 5.4 FT.
L=120 FT.; S_o=0.002

SELECT 30"
HW=4.7 FT.

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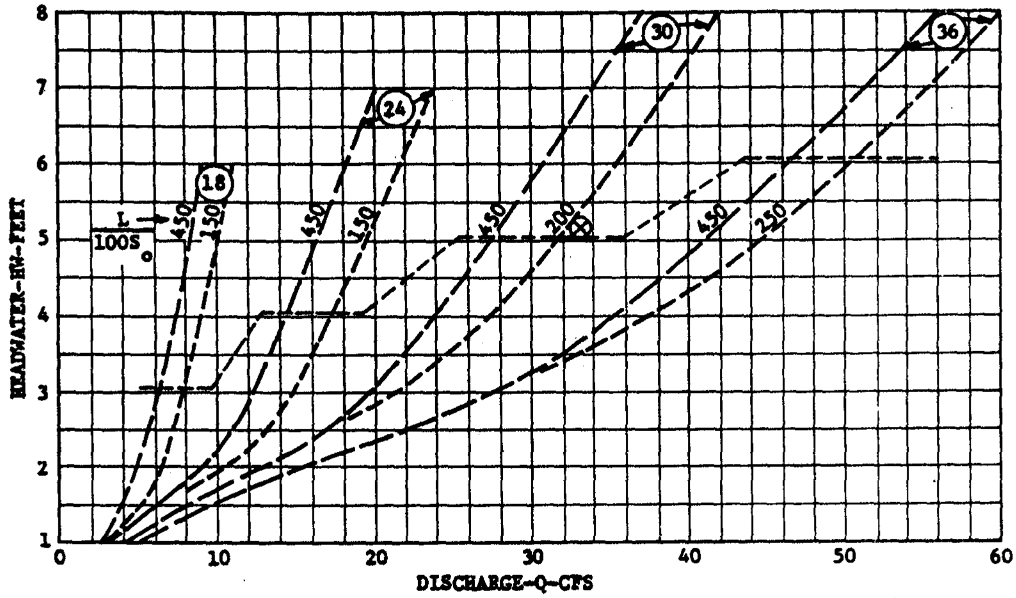
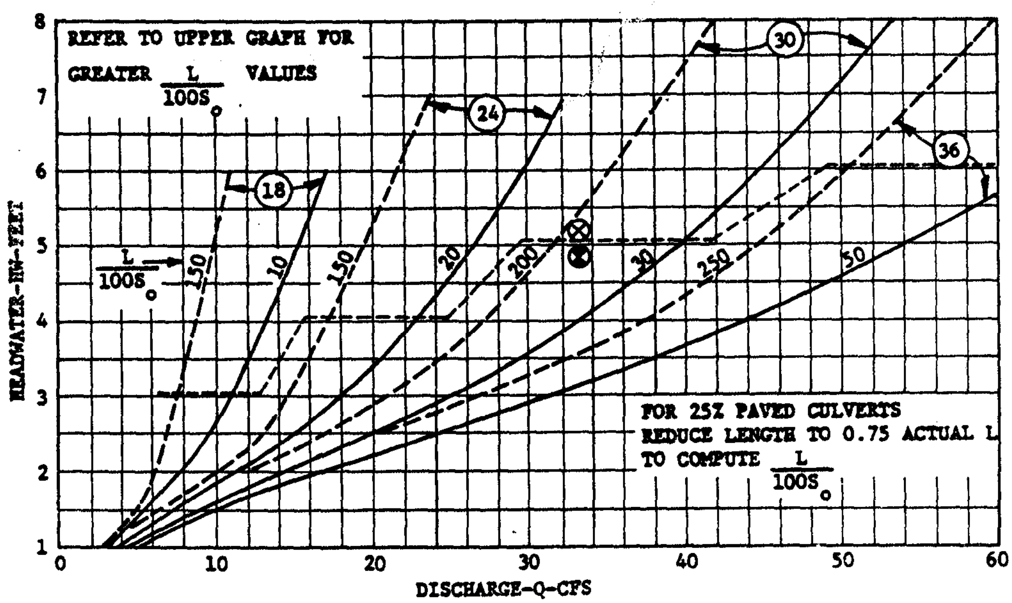
FIGURE 11-10. CULVERT CAPACITY CIRCULAR SMOOTH PIPE
GROOVE-EDGE ENTRANCE 18" TO 66"





EXAMPLE

⊗ **GIVEN:**
33 CFS; AHW=5.2 FT.
L=70 FT.; S_o=0.005

SELECT 30" UNPAVED
HW=4.9 FT.

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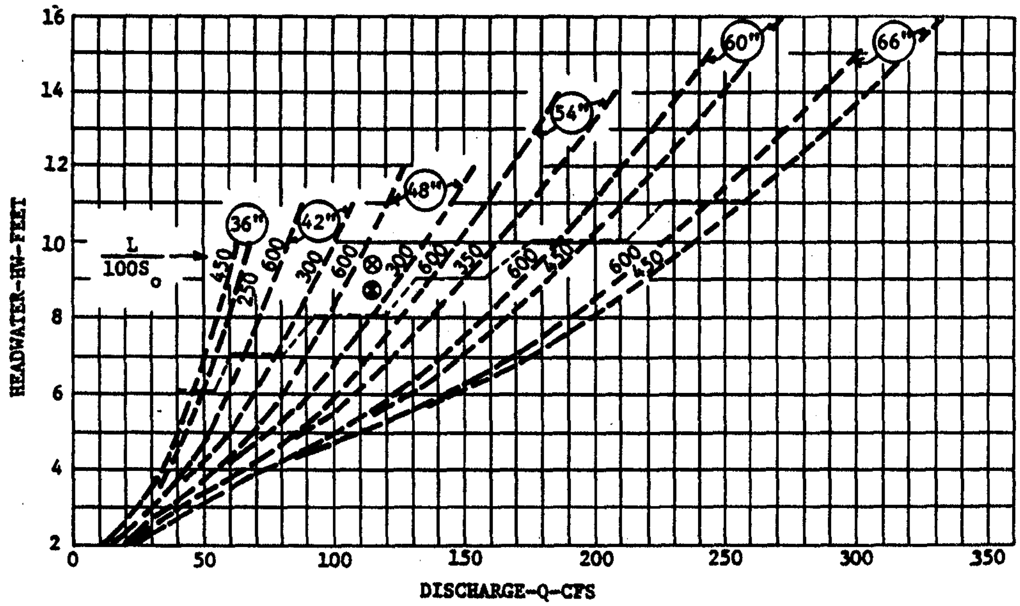
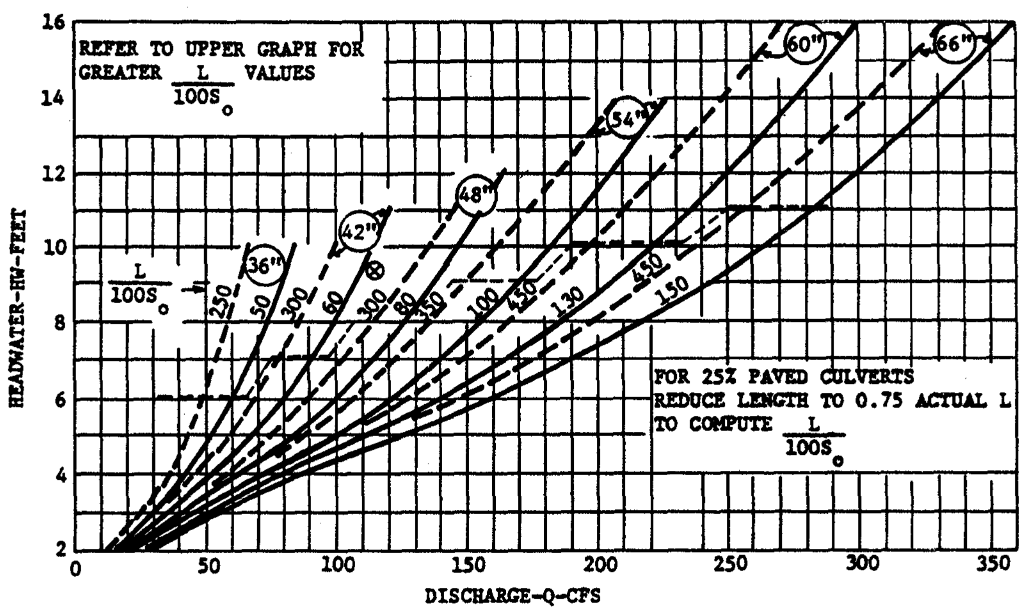
FIGURE 11-11. CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED PIPE PROJECTING ENTRANCE 18" TO 36"





EXAMPLE

⊗ **GIVEN:**
115 CFS; AHW=9.4 FT.
L=135 FT.; S_o=0.0034

● **SELECT 48" UNPAVED**
HW=8.6 FT.

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FIGURE 11-12. CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED PIPE PROJECTING ENTRANCE 36"-66" 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.

CHAPTER 12

CONSTRUCTION DRAINAGE

12-1. 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.

12-2. Planning. Efforts to control delays or damages arising from construction drainage must begin in the planning stage and carry through to 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.

12-3. Protective measures.

a. Runoff. Control of runoff problems during construction can be costly. Consideration of the following items will aid in maintaining satisfactory drainage during the construction period.

b. Natural drainage. 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.

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

d. Final facilities. Installation of final storm-drain facilities and backfilling operations will be planned and timed to render maximum use during the construction period. Random excavation will be held to a minimum and finished surfaces will be sodded or seeded immediately.

PART FOUR

EROSION CONTROL

CHAPTER 13

EROSION CONTROL REQUIREMENTS

13-1. General. Erosion control can be provided by crushed rock blankets for the more severe cases and by ground cover vegetation where no turbulant flows are predictable. Riprap and grass turf are covered in detail here. Other forms of protection such as rock filled baskets, precast slabs, or flowering folage may also be considered in special cases.

13-2. Design criteria.

a. Scour. 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.

b. Gully scour. 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.

c. Scour hole. 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.

13-3. Estimating erosion.

a. Empirical methods. Observations and empirical methods have been developed which provide specific guidance relative to the conditions that produce gully scour or only a localized scour hole as well as those required for stable channels.

b. Empirical equations. 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. However, the relationship between the Froude number of flow at the culvert outlet and a discharge parameter, $Q/D_o^{5/2}$, can be calculated for any shape of outlet and the discharge parameter is just as representative of flow conditions as is the Froude number. The relations between the two parameters for partial and full pipe flow in square culverts are shown in figure 13-1. Since the discharge parameter is easier to calculate and is suitable for application purposes, equations were determined for estimating the extent of localized scour to be anticipated downstream of culvert and storm-drain outlets.

c. Graphic solutions to empirical equations. The equations for the maximum depth, width, length, and volume of scour and comparisons of predicted and observed values are shown in figures 13-2 through 13-5. Minimum and maximum tailwater depths are defined as those less than 0.5 D_o and equal to or greater than 0.5 D_o respectively. Dimensionless profiles along the center lines of the scour holes to be anticipated with minimum and maximum tailwaters are presented in figures 13-6 and 13-7. 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 13-6 and 13-7.

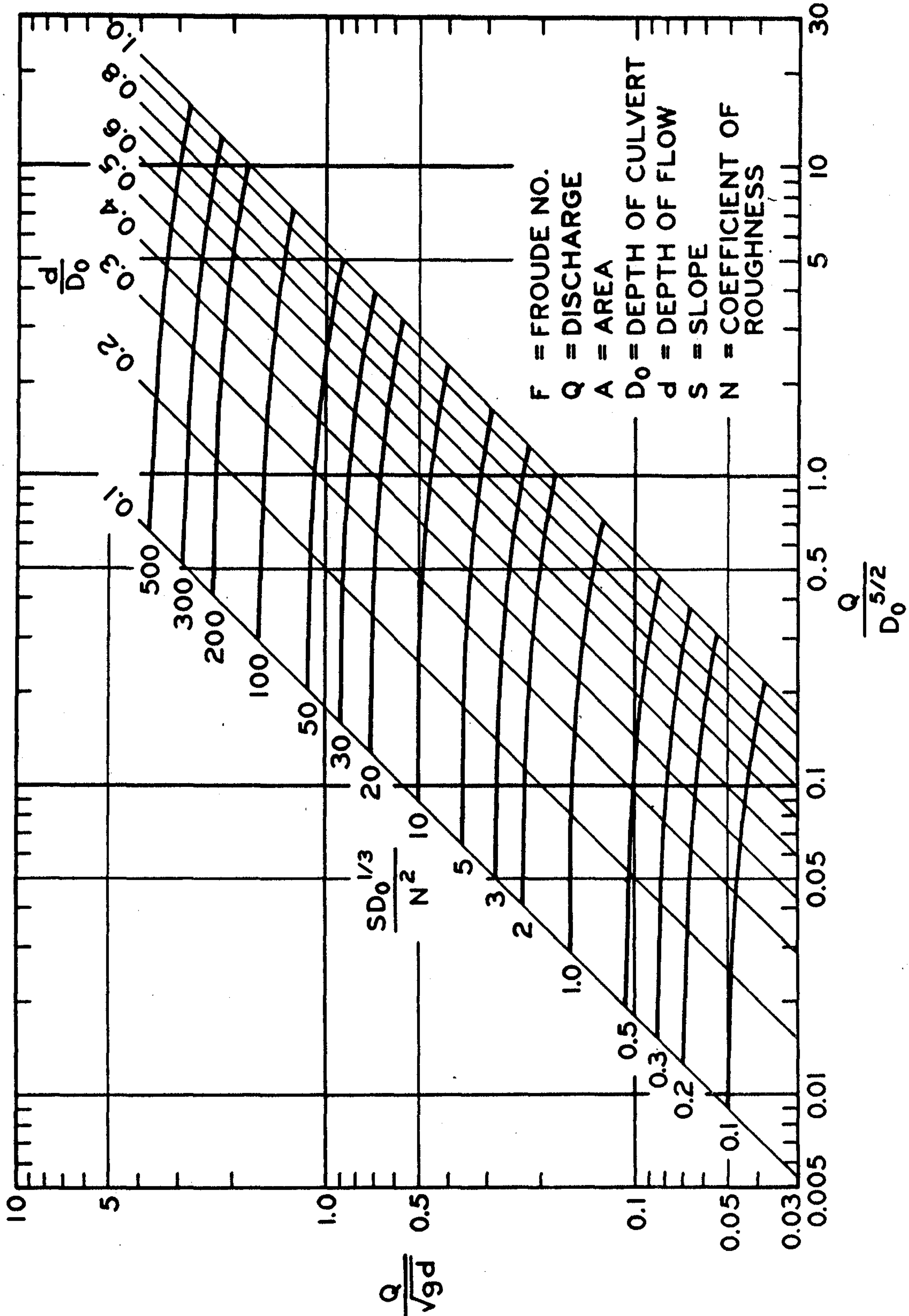

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FIGURE 13-1. SQUARE CULVERT-FROUDE NUMBER

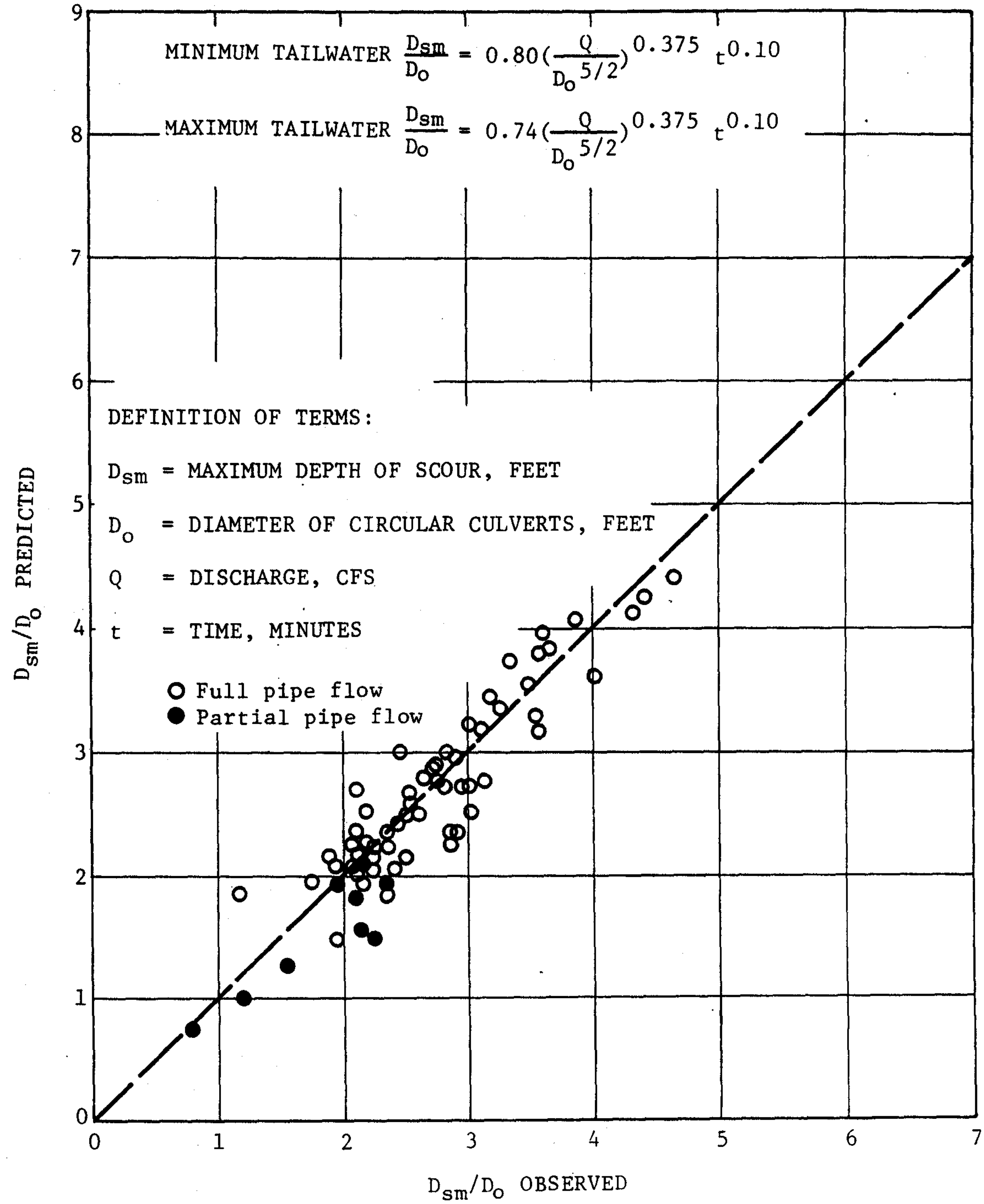

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FIGURE 13-2. PREDICTED SCOUR DEPTH VERSUS OBSERVED SCOUR DEPTH

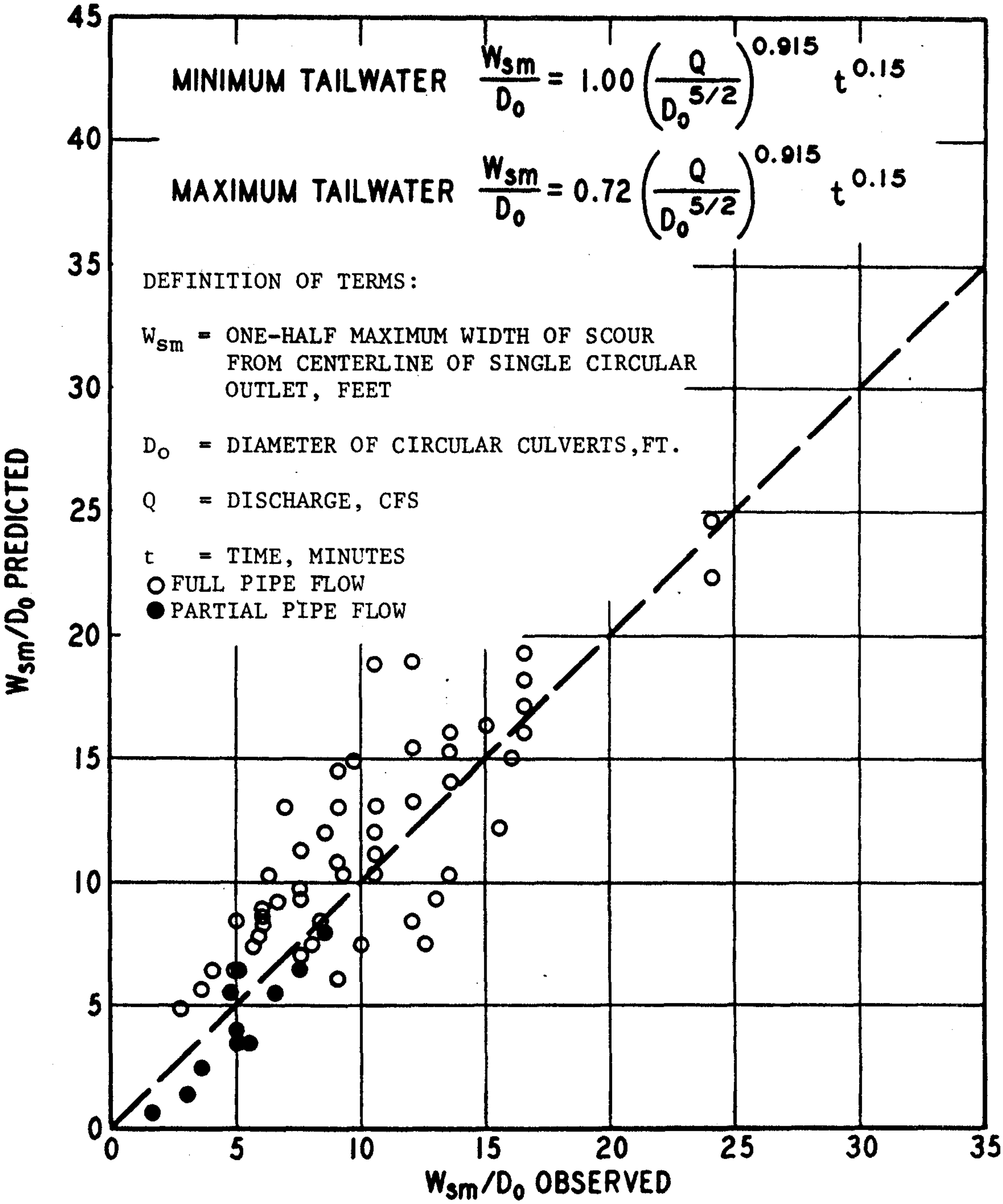

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FIGURE 13-3. PREDICTED SCOUR WIDTH VERSUS OBSERVED SCOUR WIDTH

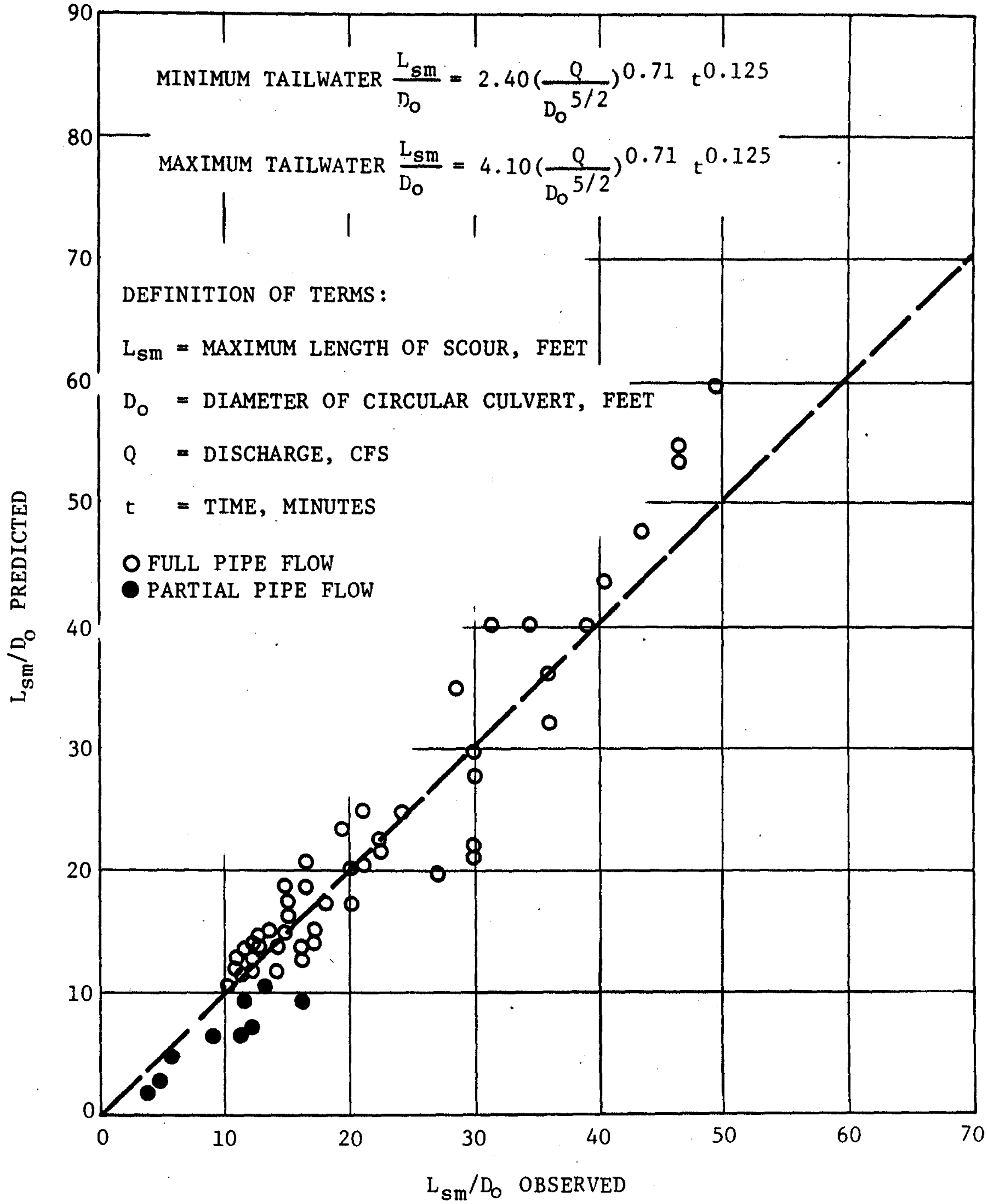

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FIGURE 13-4. PREDICTED SCOUR LENGTH VERSUS OBSERVED SCOUR LENGTH

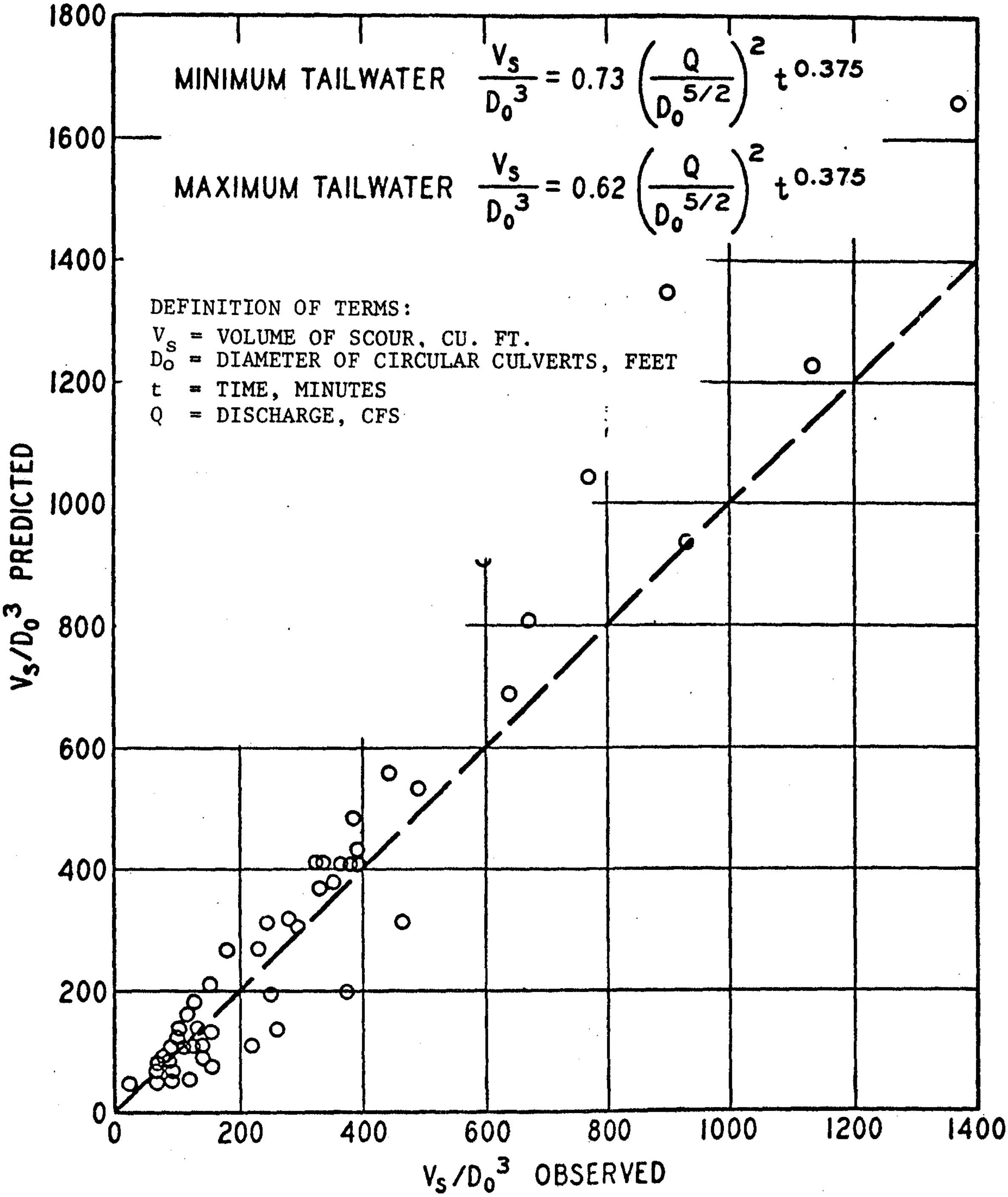

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FIGURE 13-5. PREDICTED SCOUR VOLUME VERSUS OBSERVED SCOUR VOLUME

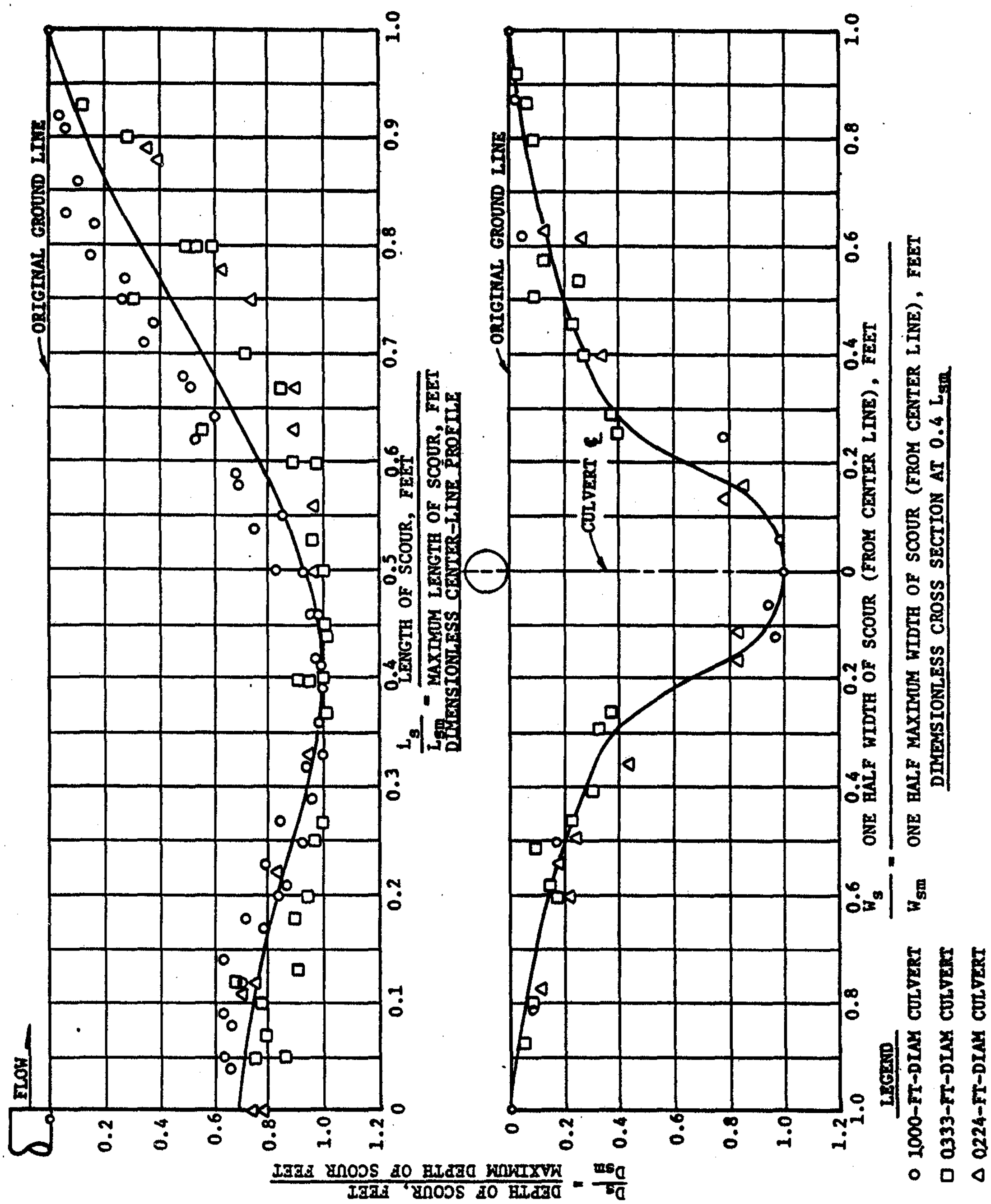

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FIGURE 13-6. DIMENSIONLESS SCOUR HOLE GEOMETRY FOR MINIMUM TAILWATER

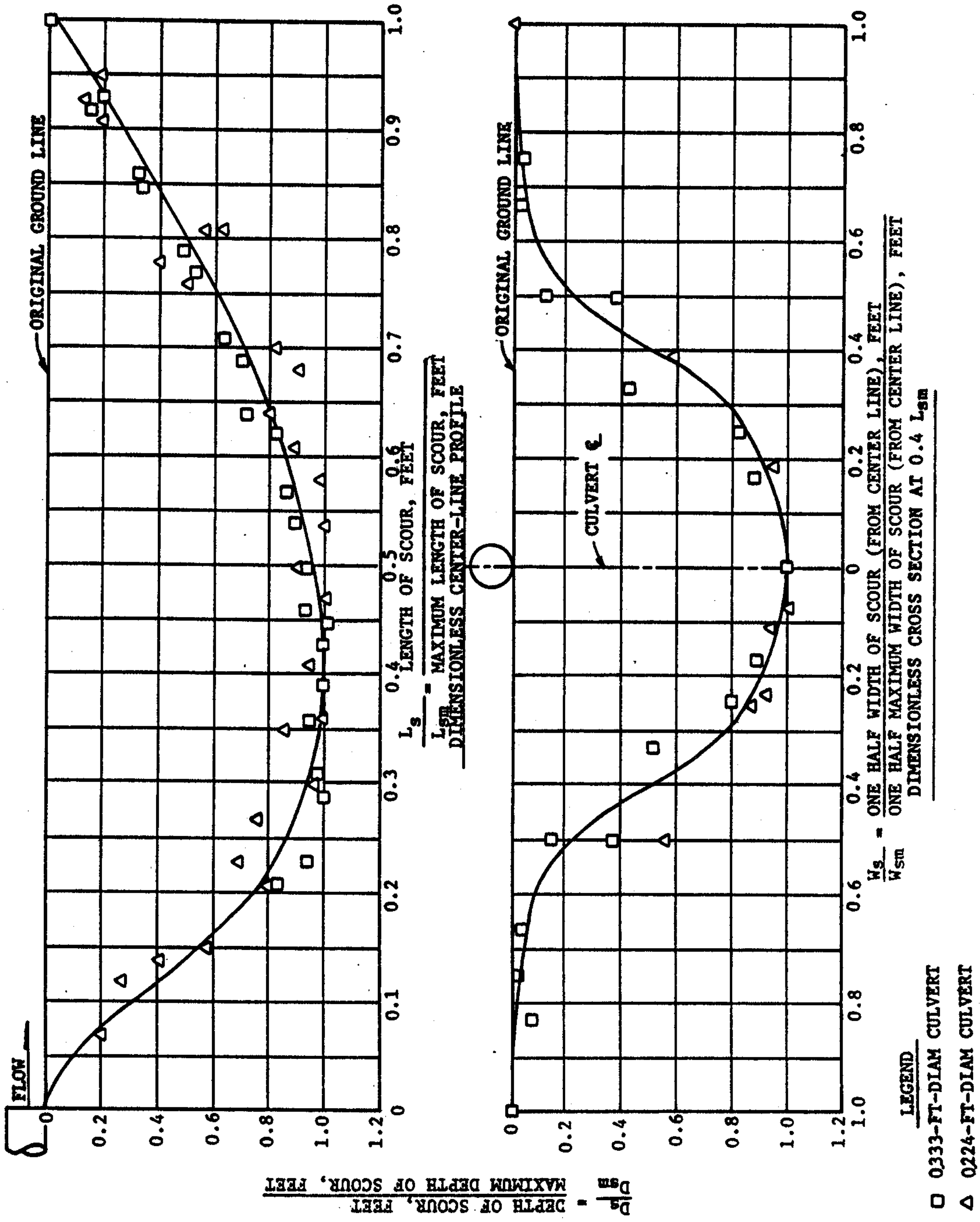

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FIGURE 13-7. DIMENSIONLESS SCOUR HOLE GEOMETRY FOR MAXIMUM TAILWATER

CHAPTER 14

RIPRAP

14-1. Riprap protection. Riprap protection should be provided adjacent to all hydraulic structures placed in erodable 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 also can 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.

14-2. Failures. 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 reveling 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.

14-3. Stone size.

a. Uses. Presented in figure 14-1 are curves for the selection of stone size required for protection with Froude numbers and depths of flow in the channel shown. 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 d_{50}/D value 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 shown in figure 14-2. The thickness of the riprap blanket should be equal to the longest dimension of the maximum size stone or 1.5 times d_{50}, whichever is greater. When the use of very large rock is desirable but impracticable, substitution of a grouted reach of smaller rock in areas of high velocities or turbulence may be appropriate. Grouted riprap should be followed by an ungrouted reach.

b. Gradation. A well-graded mixture of stone sizes is preferred to a relatively uniform size of riprap. A recommended gradation is shown

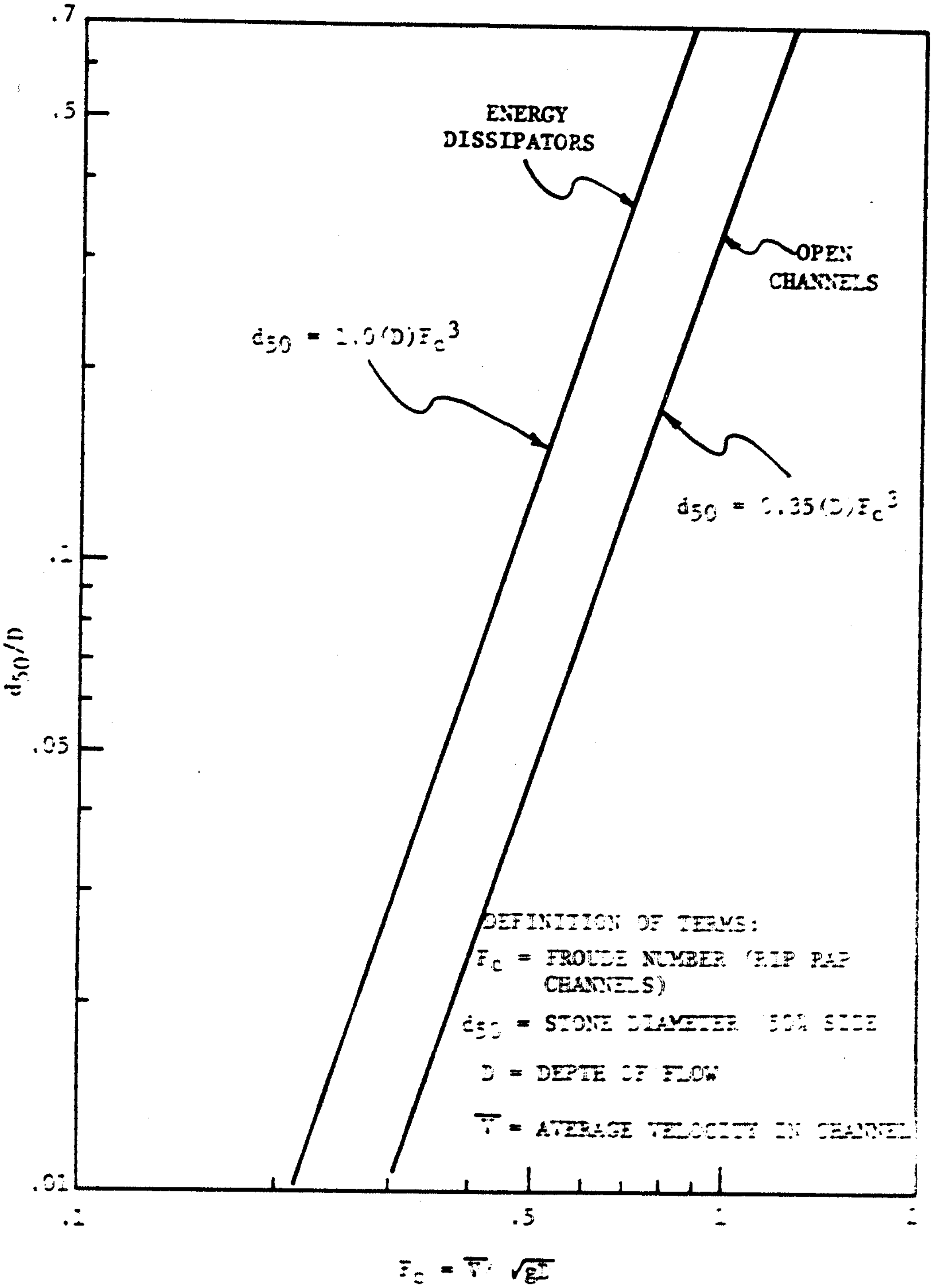

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FIGURE 1-1. RECOMMENDED RIPRAP SIZES, GULLY SCOUR

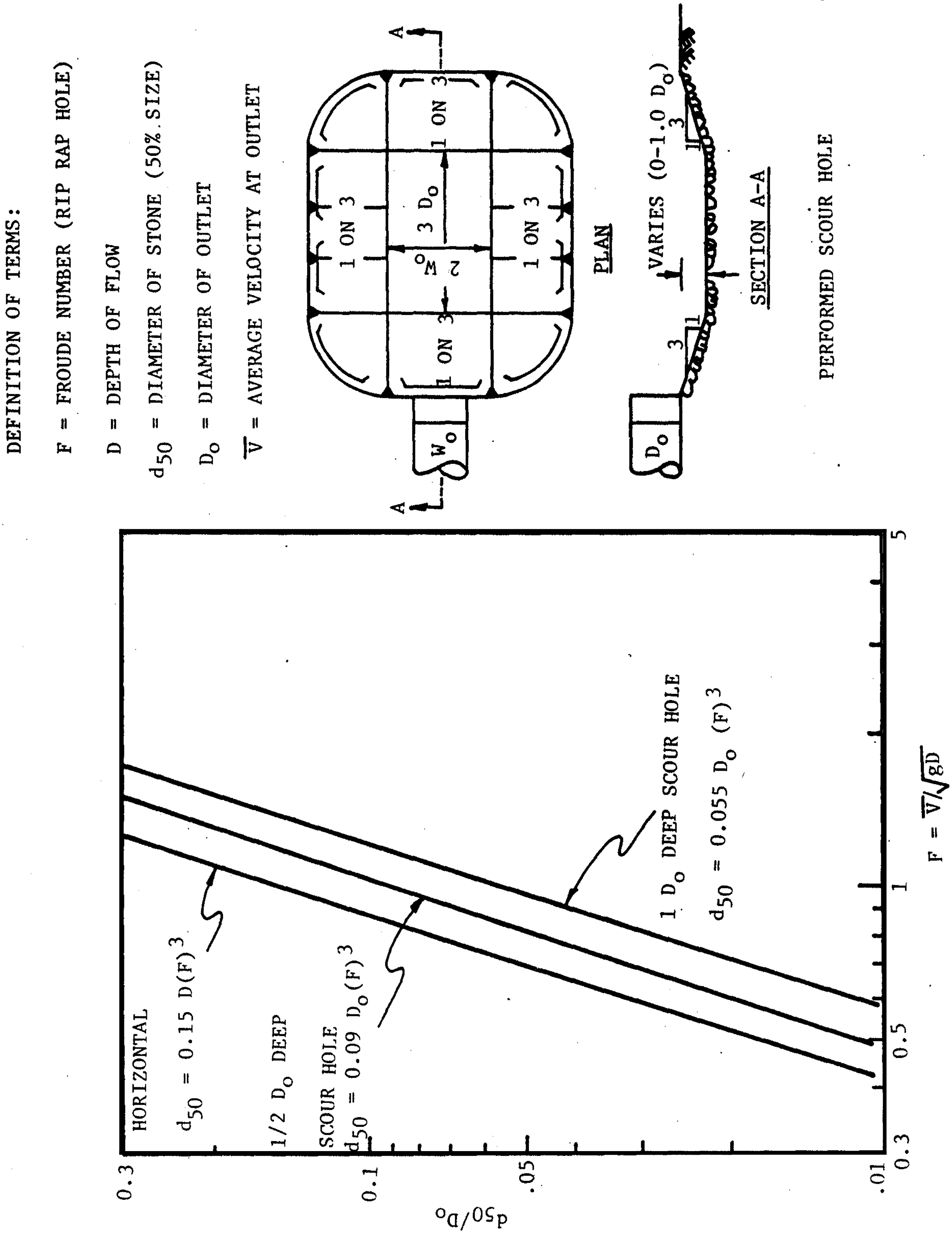

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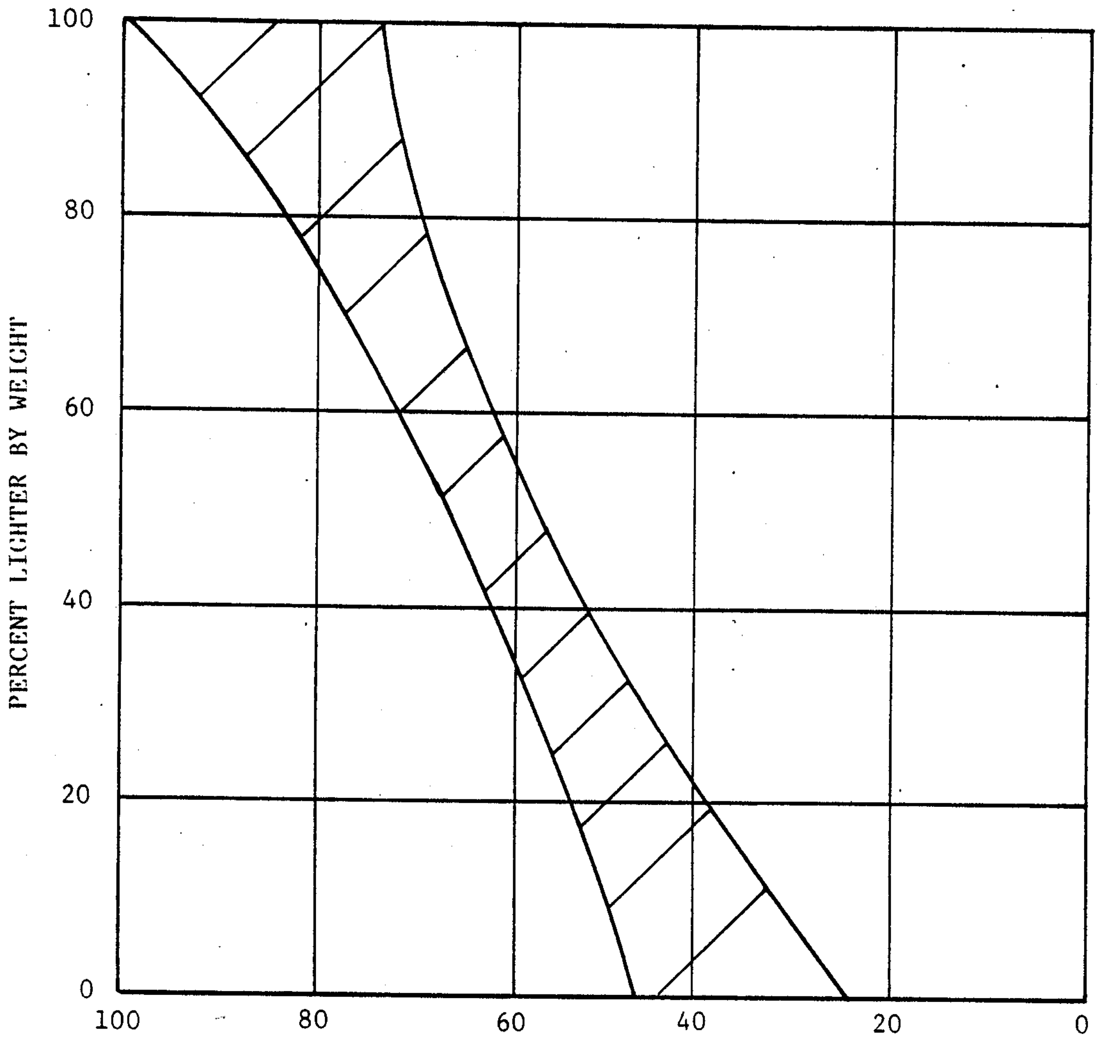
FIGURE 14-2. RECOMMENDED RIPRAP SIZES, SCOUR HOLE in figure 14-3. However, in certain locations the available material may dictate the gradation of riprap to be used. In such cases, the gradation should resemble as closely as possible the recommended mixture. Consideration should be given to increasing the thickness of the riprap blanket when locality dictates the use of gradations with larger percents of small stone than shown by the recommended plot. 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 inches, but greater thickness is sometimes necessary.

14-4. 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 stream bed to the estimated depth of bottom scour.



$$\frac{d}{d_m} = \frac{\text{Diameter of stone}}{\text{Maximum diameter of stone}}$$

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FIGURE 14-3. RECOMMENDED GRADATION

CHAPTER 15

VEGETATION

15-1. Planting zones. The United States has been divided into planting zones, as shown in figure 15-1, to provide a convenient grouping of treatments countrywide. While the following suggestions offer guidelines for developing turf, the use of any given grass species or mixtures should be confirmed by local experience as compiled by state highway commissions, county agents, and local nurserys.

15-2. Turf development.

a. Planting zone 1. Use 40 percent Kentucky bluegrass, 40 percent perennial rye grass, and 20 percent fescue north of latitude 40 within the zone, and Blando brome grass or Harding grass south of latitude 40.

b. Planting zone 2. Use Kentucky bluegrass east of the 97th meridian within the zone and crested wheatgrass in the remainder of the zone. In low rainfall areas (approximately 10 inches and less) or other sites where grasses do not thrive, do not use turf; instead use a 1 to 2 inch blanket of crushed rock or gravel aggregate (riprap). If irrigation is practicable in these low rainfall areas, use Kentucky bluegrass or crested wheatgrass.

c. Planting zone 3. Use a 50-50 mixture of Kentucky bluegrass and Chewings fescue. On sandy or shallow soils, especially those which will not be maintained under high fertility conditions, use 30 percent Kentucky bluegrass and 70 percent Chewings fescue.

d. Planting zones 4 and 6.

(1) East of the 98th meridian, use common Bermuda grass.

(2) West of the 98th meridian, use common Bermuda grass where irrigation is practicable; otherwise, use native species such as buffalo, blue grama, black grama, galleta, and sand dropseed within areas of recommended use. Use crested wheatgrass in elevations over 4,000 feet. Where average annual rainfall is less than 10 inches, and on other sites where grasses do not thrive, use crushed rock or gravel aggregate (riprap).

e. Planting zone 5. Use Bermuda grass. In eastern Texas and southern Oklahoma, Pensacola bahia may be substituted.

f. Planting zone 7. Use Harding grass or Blando brome grass.

g. Planting zone, Hawaii. Use locally adapted strains of Bermuda grass.

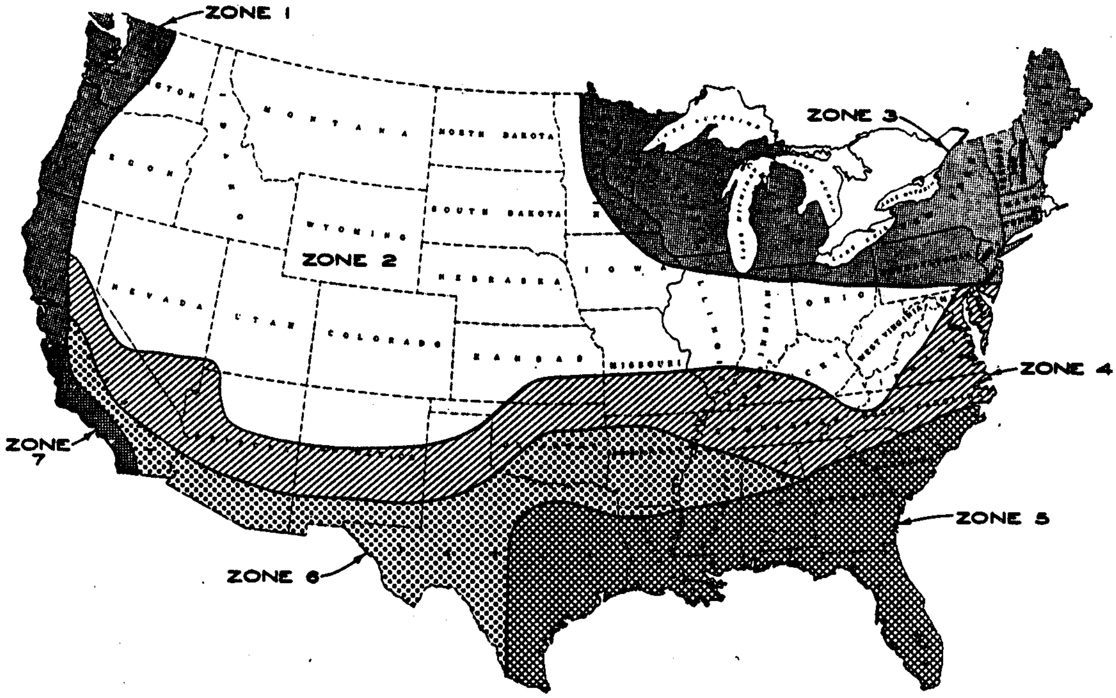

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FIGURE 15-1. PLANTING ZONES

h. Planting zone, Caribbean. Use Pensacola bahia grass.

i. Planting zone, Alaska. Use smooth brome grass from northern United States or Canada seed sources.

15-3. Terraces. Erosion control in nonuse areas can frequently be accomplished by terracing along with a well-developed turfing program. The terrace will consist of a low, broad-based earth levee constructed approximately parallel to the contours and designed to intercept overload flow before it achieves great erosive force and to conduct it to a suitable discharge point. A vigorous turf established on the terrace and channel will also aid in controlling erosion.

PART FIVE

DEFERRED DESIGN AND CONSTRUCTION

CHAPTER 16

REQUIREMENTS FOR DEFERRED STRUCTURES

16-1. General. The purpose for allowing deferred design and construction is to provide as much time as possible to be devoted to the essential parts of the drainage systems necessary for completion so initial operation of the facility can take place within the M plus 180 day schedule. Early in the project planning phase, a master schedule of both engineering and construction must be developed. As a part of the scheduling activity, each element of the drainage system must be evaluated as to its essentialness for initial operation of the airfield, road, railroad, or other facility. Once the priority list of drainage structures has been established, deferrable structures can be determined. In general it is anticipated that deferrable parts of the drainage system are related to "down stream" portions of the system and will include such structures as headwalls, drop structures, check dams, chutes, stilling basins, and large volume open channels. The principles of design for large channels are basically the same as those presented for smaller drainage channels located adjacent to roadways and around airfields. Channel design is detailed in part three chapter 10.

16-2. Repairs. For the most part structures listed as potentially deferrable are erosion protection structures for use in controlling large quantities of water. By allowing drainage over the paths where these structures are to be built, a certain amount of damage may occur. The amount of erosion damage will depend upon how long the work is deferred and the severity of storms during that time. All damage must be repaired and grades brought to design requirements as part of structure construction. Close observation of the natural water courses and the location of any erosion occurring from structure deferment may alter the preliminary requirements for size and placement of protective and control structures.

CHAPTER 17

HEADWALLS

17-1. General.

a. Purpose. 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 prevent disjointing due to excessive pressures, to control erosion and scour resulting from excessive velocities and turbulences and to prevent adjacent soil from sloughing into the waterway opening. Many of these functions can be provided for by other means and headwalls should only be used in special cases such as limited space or poor soils highly susceptible to erosion or sloughing.

b. Scouring. Where headwalls are used, provisions for drainage should be made over the center of the headwall to prevent scouring along the sides of the walls.

c. Requirements for usage. Erosion protection such as riprap, or sacked concrete with a sand-cement ratio of 9:1, may be used around the culvert entrance when a headwall is not used. The decision as to whether or not a headwall is necessary depends on the expected flow conditions and embankment stability.

17-2. Entrances. 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. In design of headwalls some degree of entrance improvement should always be considered. Several preformed flared or warped sections are available to increase inlet hydraulic efficiencies.

17-3. Types of headwalls.

a. Height. Headwall and wingwall heights should be kept to the minimum that is consistent with hydraulic, geometric, and structural requirements. Assuming a structure is required, typical applications of straight headwalls and winged headwalls consist of the following types.

b. Straight headwalls. Straight headwalls 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 alinement.

c. L headwalls. "L" headwalls are used for either gutter drainage or defined channels with low to moderate velocity where abrupt change of alinement is required at the culvert inlet.

d. Wing headwalls. Winged headwalls are used for channels with moderate velocity and medium drift (floating trash). Wings are best set flush with the edges of the culvert barrel, alined with respect to stream axis, and placed at a flare angle of 18 to 45 degrees.

17-4. Headwall construction. Headwalls are normally constructed of reinforced concrete or masonry and often include wingwalls and aprons. Drainage pipe and culvert manufacturers offer a variety of precast or preformed inlet and outlet sections to replace or simplify headwall construction.

17-5. Outlets and endwalls.

a. Undermining endwalls. 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 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. 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.

b. Endwall protection. Failure of the system will take place 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 result in washout of the structure. For corrugated metal (pipe or arch) culvert installations, reinforced pipe, and PVC pipe, use of prefabricated end sections may well prove desirable and economically feasible. When a 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.

c. Energy dissipators. Various designs of energy dissipators, flared transitions and erosion protection for culvert outfalls are discussed in detail in other chapters of this manual. See paragraphs 20-1 through 20-7 and part four, chapter 14.

17-6. Structural stability. The proposed structure will be adequate to withstand soil and hydrostatic pressures and in areas of seasonal freezing the effects of frost action. The structure will be designed to preclude detrimental heave or lateral displacement as the result of 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. Bedding requirements will be determined in accordance with procedures outlined in note 4, table 8-4.

17-7. Sloughing. The proposed structure 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.

17-8. Apron.

a. Protection. Paved aprons are probably the oldest and simplest form of culvert protection. 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 because of the hydraulic resistance provided by the apron.

b. Requirements. The necessity for an apron or stilling basin is determined largely by the soil characteristics of the adjacent open channel and by the anticipated maximum velocities and turbulence at the pipe outlet. Most culverts operate under free outfall conditions, i.e., controlling tailwater is absent, and the discharge possesses kinetic energy in excess of that naturally occurring in the waterway. This excess kinetic energy often must be dissipated to control damaging erosion. The extent to which protective works are required for energy dissipation depends on the amount of excess kinetic energy and the characteristics of the material in the outlet channel. The soil type will indicate the maximum permissible velocities for open channels. These velocities are given in part three table 10-1. The velocity may be regulated so far as it is feasible to vary the hydraulic gradient of the storm drain or outfall ditch. If excessive discharge velocities do occur, an apron of adequate design will be provided to reduce the velocities to permissible values. As an additional precaution, a cutoff wall will be provided to minimize the possibility of undermining the structure. Concrete aprons will be designed to preclude structural damage from differential movement caused by frost action during no-flow periods or by expansive subgrade soils.

c. Parameter calculations. Test results of recent studies for simple outlet transitions with the apron at the same elevation as the culvert invert are shown in figure 17-1. The maximum discharge

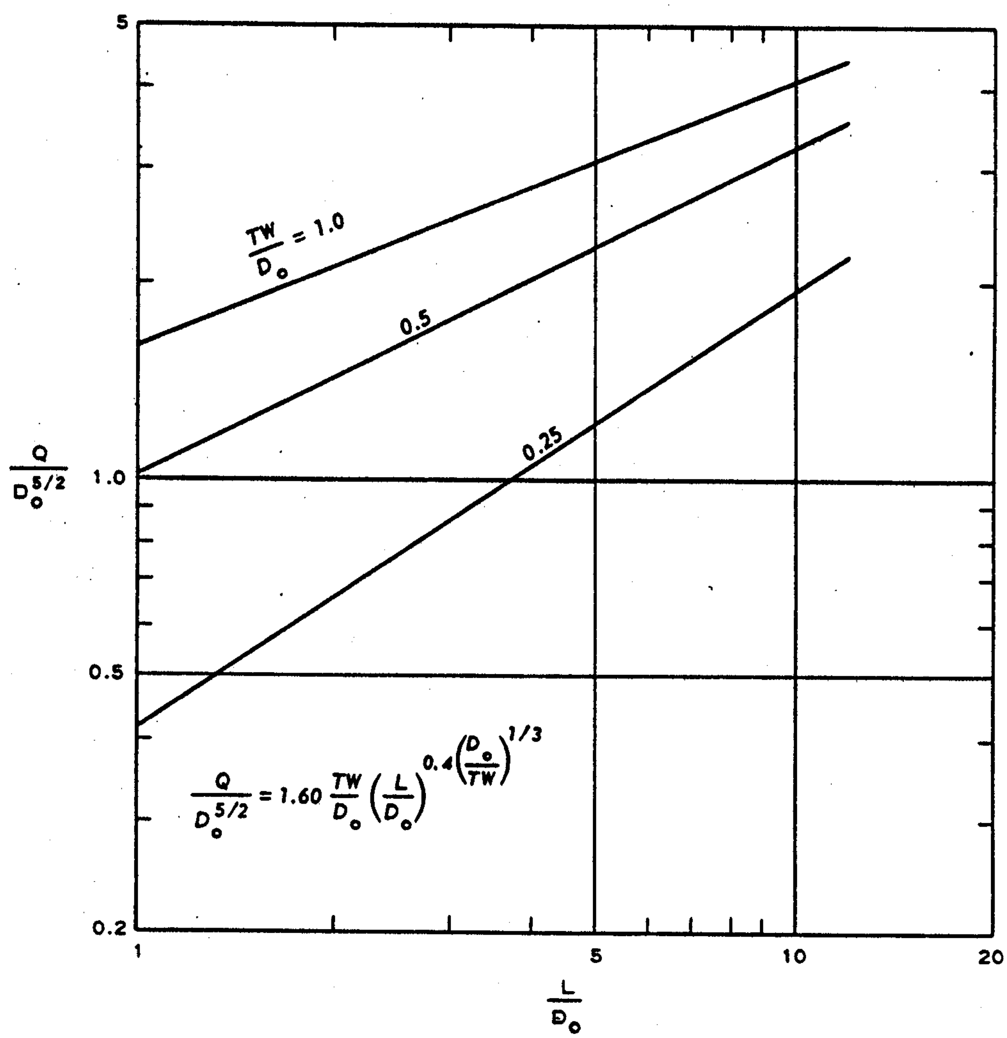

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FIGURE 17-1. MAXIMUM PERMISSIBLE DISCHARGE FOR VARIOUS LENGTHS OF FLARED OUTLET TRANSITION AND TAILWATERS parameter for a given culvert length of transition and tailwater can be calculated by the following equation:

$$\frac{Q}{D_o^{5/2}} = 1.60 \frac{TW}{D_o} \left(\frac{L}{D_o}\right)^{0.4(D_o/TW)^{1/3}}$$

where:

Q = Discharge, cfs
D_o = Diameter of circular culvert, feet
TW = Tailwater depth above invert of culvert outlet, feet
L = Length of apron, feet

Similarly, the length of transition for a given situation can be selected by the interrelations shown in figure 17-2, which is calculated by the following equation:

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_o^{5/2}}\right)^{2.5\,(TW/D_o)^{1/3}}$$

Variables show that this type of protection is satisfactory only for limited values of $Q/Do^{5/2}$ and TW/D_o. Arbitrary extent of scour depth equal to or less than $0.5D_o$ was used to classify satisfactory conditions.

d. Tailwater elevations. Figure 17-3 indicates that recessing the apron and providing an end sill at the downstream end did not significantly improve energy dissipation or increase the applicable maximum value of the discharge parameter, $Q/D_o^{5/2}$. The limiting values of the discharge parameter for various outlet transitions and tailwater elevations are listed in table 17-1.

e. Endwall elevations. Numerous endwall failures have occurred as a result of improper consideration of the relative elevation of the apron and outfall channel. If practicable, the apron elevation will be selected to insure that sufficient depth of backwater occurs over the apron during design flow conditions to prevent undesirable erosion, otherwise positive erosion protection measures will be required. Newly excavated channels erode slightly during the aging process, and proper allowance for this action must be included in establishing the apron elevation.

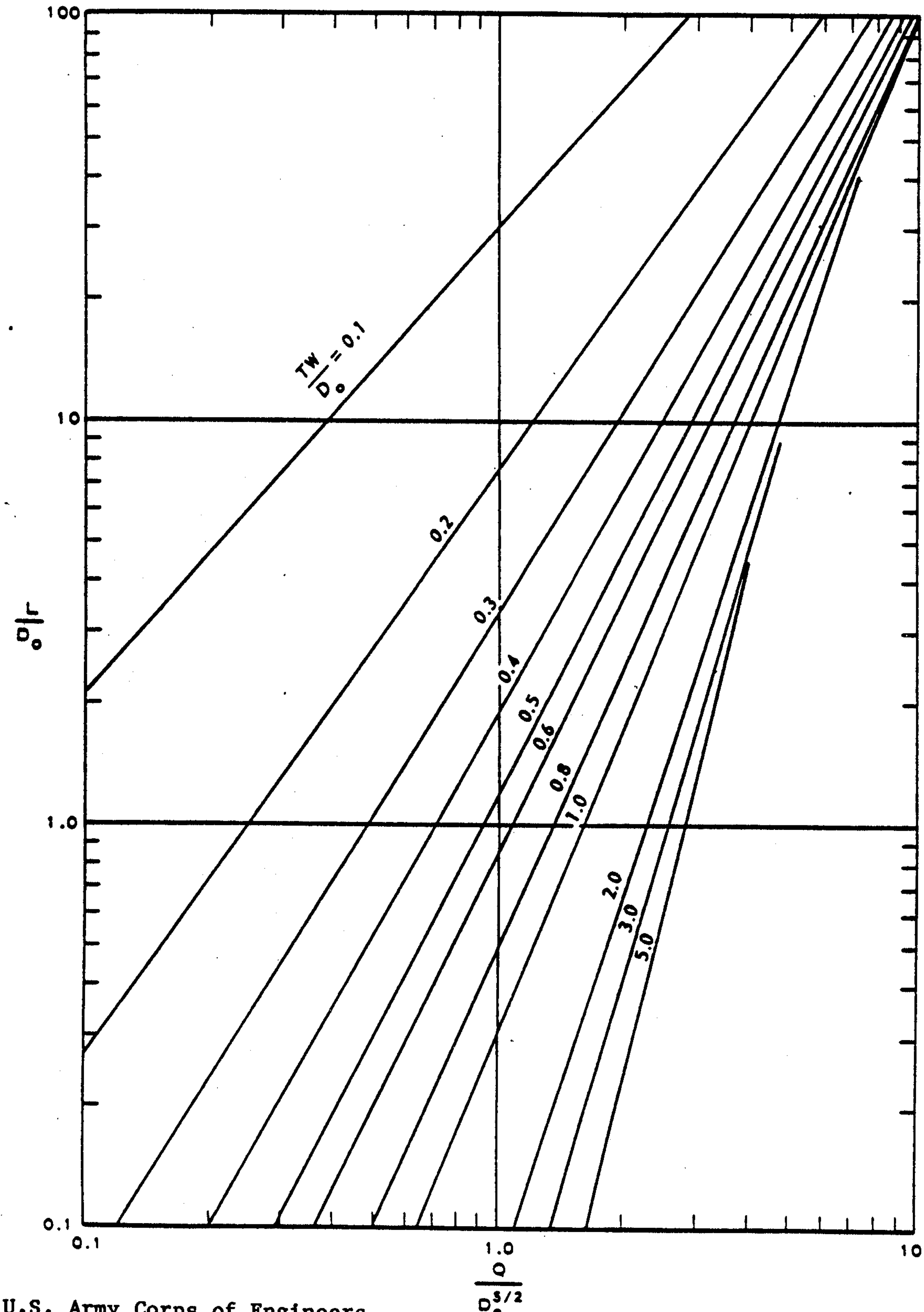

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FIGURE 17-2. LENGTH OF FLARED OUTLET TRANSITION RELATIVE TO DISCHARGE, TAILWATER, AND CONDUIT SIZE

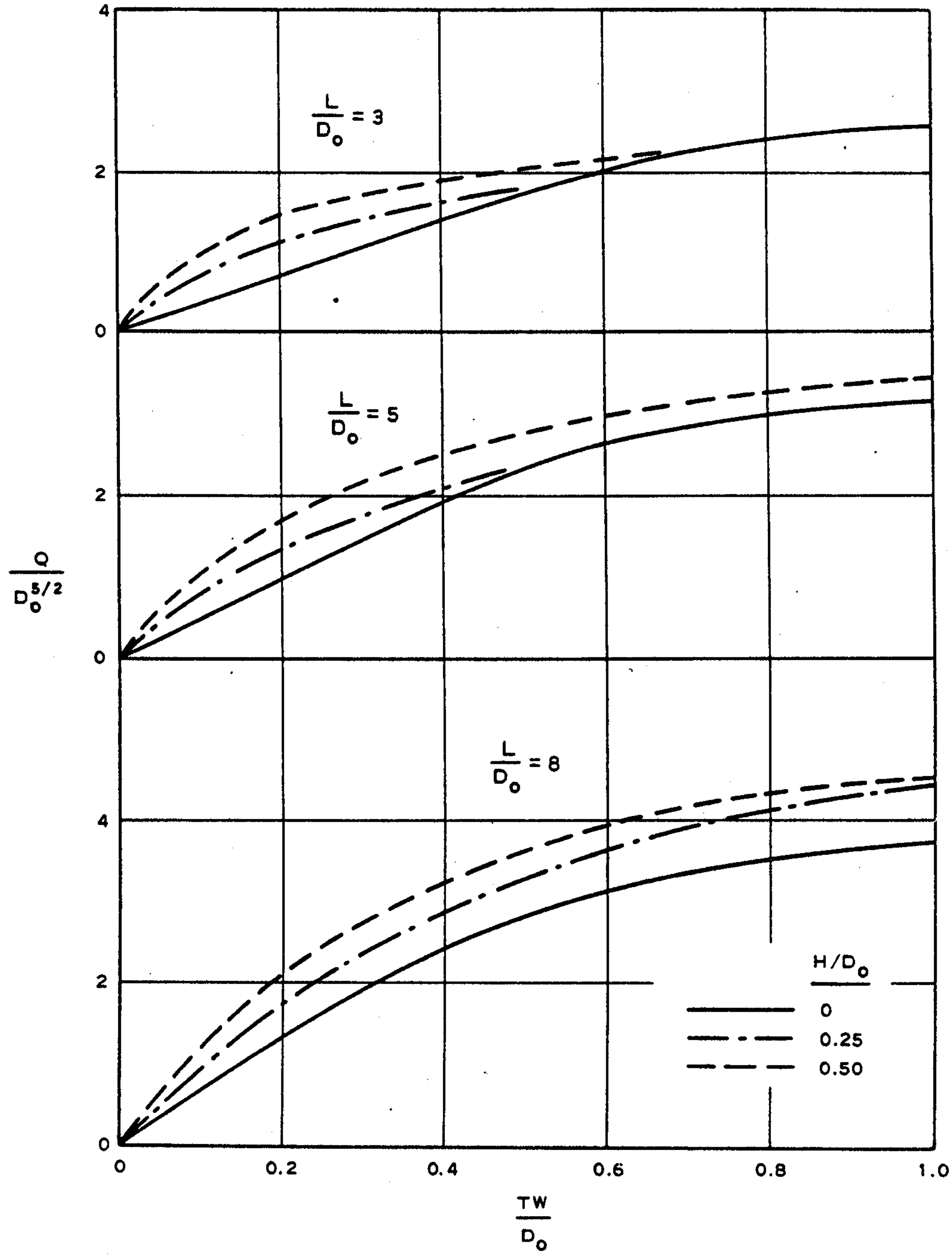

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FIGURE 17-3. RELATIVE EFFECTS OF RECESSED APRON AND END SILL ON PERMISSIBLE DISCHARGE

Table 17-1. Limiting Values of $Q/D_o^{5/2}$

L/D_o	H/D_o	TW/D_o	$Q/D_o^{5/2}$
3	0	0.25	0.88
3	0	0.50	1.78
3	0	1.00	2.56
3	0.25	0.25	1.28
3	0.25	0.50	1.78
3	0.25	1.00	2.56
3	0.50	0.25	1.58
3	0.50	0.50	2.00
3	0.50	1.00	2.56
5	0	0.25	1.20
5	0	0.50	2.40
5	0	1.00	3.20
5	0.25	0.25	1.58
5	0.25	0.50	2.78
5	0.25	1.00	3.47
5	0.50	0.25	1.47
5	0.50	0.50	2.77
5	0.50	1.00	3.46
8	0	0.25	1.68
8	0	0.50	2.40
8	0	1.00	3.75
8	0.25	0.25	2.17
8	0.25	0.50	3.36
8	0.25	1.00	4.44
8	0.50	0.25	2.46
8	0.50	0.50	3.65
8	0.50	1.00	4.55

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CHAPTER 18

DROP STRUCTURES AND CHECK DAMS

18-1. Description and purpose. Drop structures and check dams are designed to check channel erosion by controlling the effective gradient and to provide for abrupt changes in gradient by means of a vertical drop. They also provide satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 feet and over embankments higher than 5 feet 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.

18-2. Design.

a. Typical drop structure. Pertinent features of a typical drop structure are shown in figure 18-1a. The hydraulic design of these structures can be divided into two general phases: design of the notch 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, and 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. 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.

b. Notch design. The design of the notch or weir can be determined by use of figure 18-1b. As illustrated in the example below, a trial-and-error procedure will be used to balance the design of the notch with the ditch cross section. To minimize erosion in the approach channel, the length of the notch will be adjusted to maintain a head on the notch (d_c) equivalent to the depth of flow (d_s) in the channel. The controlling features of the stilling basin are determined by the following criteria.

$$d_c = (Q/5.67\ b_n)^{2/3}$$
$$L = C_L(hd_c)^{1/2}$$
$$h_t = d_c/2$$
$$h_l = 3d_c/8$$
$$b_n/b_s$$

where:

d_c = Critical depth (depth of head on notch), feet
Q = Normal channel discharge to drop structure, cfs

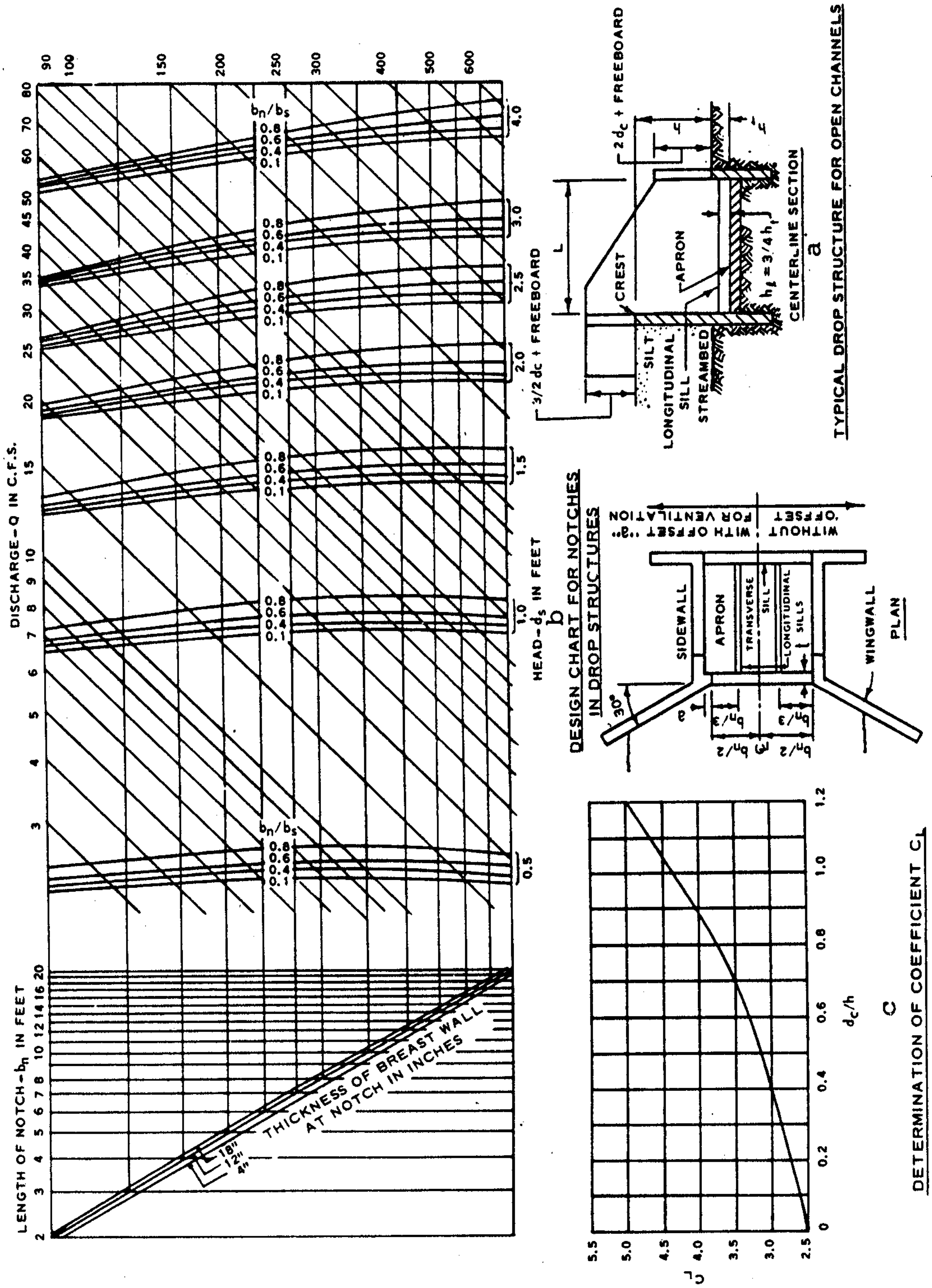

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FIGURE 18-1. DESIGN OF DROP STRUCTURES FOR OPEN CHANNELS

b_n = Width of notch (weir), feet
L = Length of drop structure apron, feet
h = height of fall or drop in structure, feet
C_L = Coefficient, obtained from figure 18-1c for condition of $d_c/h = 1$
h_t = height of transverse end sill, feet
h_l = height of longitudinal sill, feet
b_s = Bottom width of regular channel or bottom width of transition section if required, feet

18-3. Typical design problem. The use of the above-mentioned equations is illustrated in the following example. Design a drop structure for a discharge of 40 cfs and a drop of 4 feet. The channel in which the structure is to be built has a 2-foot bottom width, 1 foot on four side slopes, and a design depth of flow of 2 feet. Maximum permissible velocity due to soil conditions is 2 fps.

18-4. Solution - design of notch.

a. From figure 18-1b with a head of 2 feet and b_n/b_s of 0.8, it is evident that b_n must be approximately 4 feet, which is greater than the bottom width of the regular channel and requires that a transition from the regular channel be provided immediately upstream from the drop structure. The transition should be of sufficient length to provide a gradual change between the regular channel and the drop structure. In this problem the transition is made to a section with a 5 foot bottom width, 1 on 2 side slopes and a 2-foot depth. Using the above revised channel elements, enter figure 18-1b, with a head of 2 feet and an assumed $b_n/b_s = 0.8$ ($b_n = 4.0$). From the intersection of $b_n/b_s = 0.8$ and Q = 40 cfs follow the horizontal line to intersection with t = 12 inches, the assumed thickness of the breast wall and thence vertically to read b_n = 4 feet. This checks the assumed b_n and no further adjustment is necessary.

b. To design the stilling basin calculate the critical depth at notch, d_c:

$$d_c = (Q/5.67b_n)^{2/3} = (40/5.67 \times 4.0)^{2/3} = 1.46 \text{ feet}$$

Length of stilling basin: In figure 18-1c for a value of d_c/h = 1.46/4.0 = 0.37, C_L = 3.0

$$L = C_L \quad hd_c = 3.0 \quad 4.0 \times 1.46 = 7.25 \text{ feet}$$

(use 7 feet 3 inches)

Height of transverse end sill, h_t

$$h_t = d_c/2 = 1/2 \times 1.46 = 0.73 \text{ foot}$$

(use 0 foot 9 inches)

Height of longitudinal sills, h_l

$$h_l = 3\ d_c/8 = 3/8 \times 1.46 = 0.55 \text{ foot}$$

(use 0 foot 7 inches)

spacing of longitudinal sills: An average spacing of sills equal to $b_n/3$ is satisfactory.

18-5. Alternate design.

a. Alternate drop structure. Pertinent features of an alternate drop structure are shown in figure 18-2. Notations used in the design of the structure are defined in appendix B.

b. Weir calculations. Discharge over the weir should be computed from the equation:

$$Q = CWH^{3/2}$$

where:

Q = discharge, cfs
C = discharge coefficient (for alternate design use C = 3.0)
W = length of weir, feet
H = head (depth of flow), feet

The length of the weir should be such as to obtain maximum use of the available channel cross section upstream from the structure. A trial-and-error procedure should be used to balance the weir height and width with the channel cross section.

c. Stilling basin dimensions. Stilling basin length and end sill height should be determined from the design curves in figure 18-2.

d. Riprap. Riprap probably will be required on the side slopes and below the end sill immediately downstream from the structure.

e. Toe slopes. The toe of the side slopes of the channel downstream from the structure should be offset from the basin walls in order to prevent scouring of side slopes immediately downstream from the end sill.

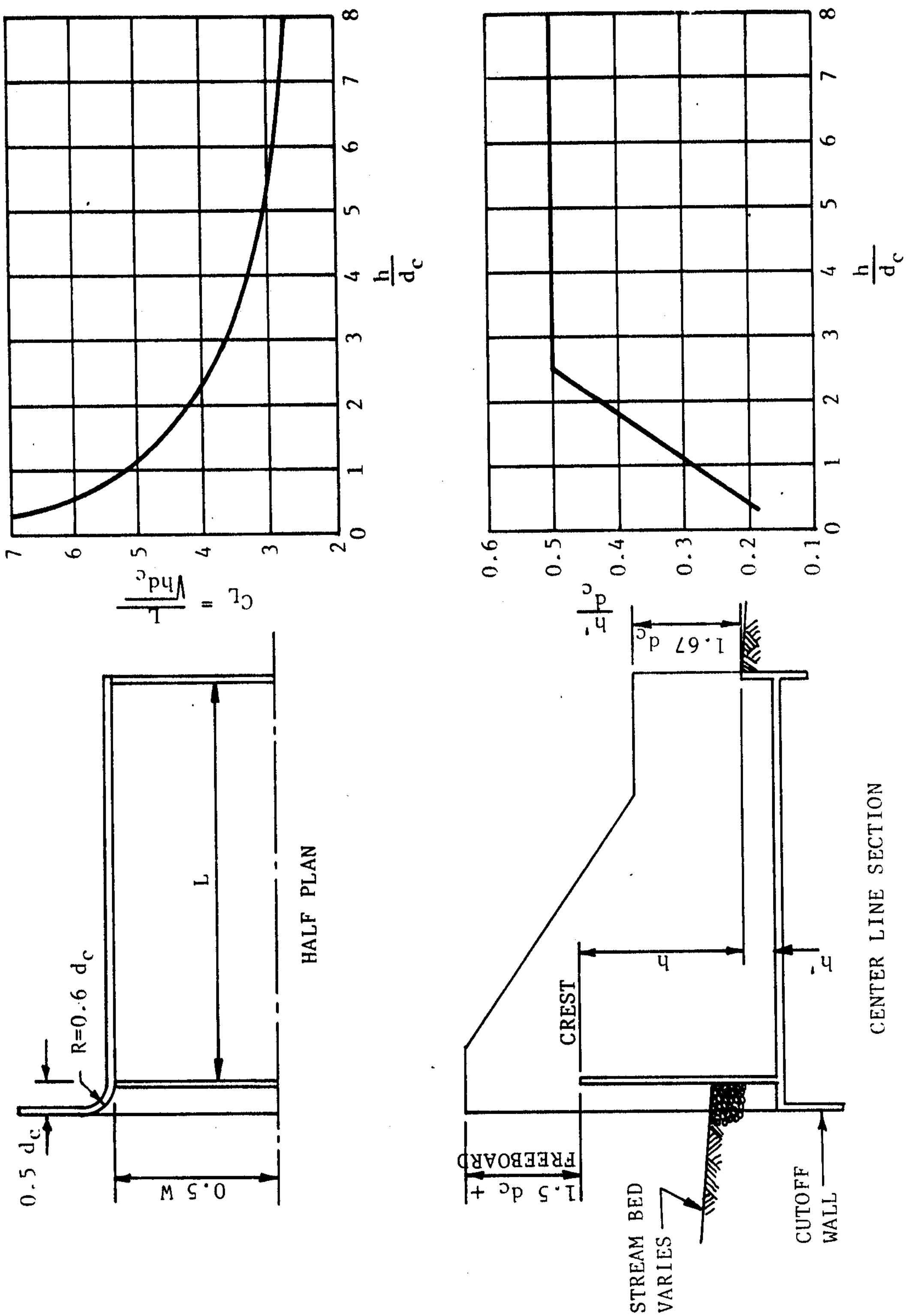

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FIGURE 18-2. DETAILS AND DESIGN CHART FOR TYPICAL DROP STRUCTURE

CHAPTER 19

CHUTES

19-1. General.

a. Description. 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 19-1 and design charts for chutes constructed of concrete for various gradients and discharges are shown in figure 19-2. Frost penetration beneath the structure will be restricted to nonfrost-susceptible materials using procedures outlined in table 8-4, 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.

b. Configuration. 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. This depth is also normally adequate to contain any swelling effect that may be present due to air entrainment. Turfed berm slopes will not be steeper than one vertical to three horizontal because they cannot be properly mowed with gang mowers.

c. Construction. 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.

d. The following equation will provide a satisfactory determination of the discharge for the chute shown in figure 19-1.

$$Q = 3.75\ W^{0.90}\ H^{1.6}$$

where:

Q = Discharge, cfs
W = Width of flume, feet
H = Head, feet

Adequate freeboard is most important in the design of a concrete chute. The critical section where most failures have been experienced 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

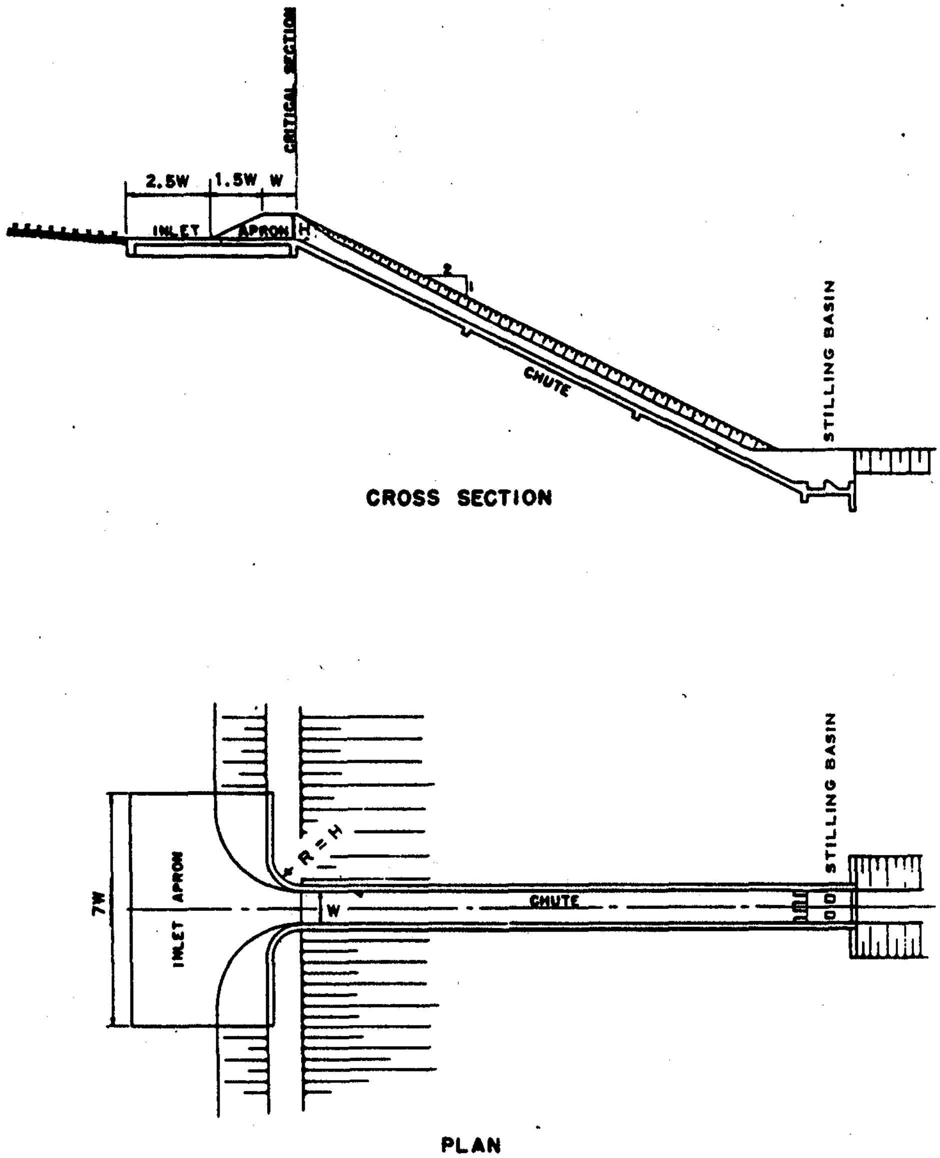

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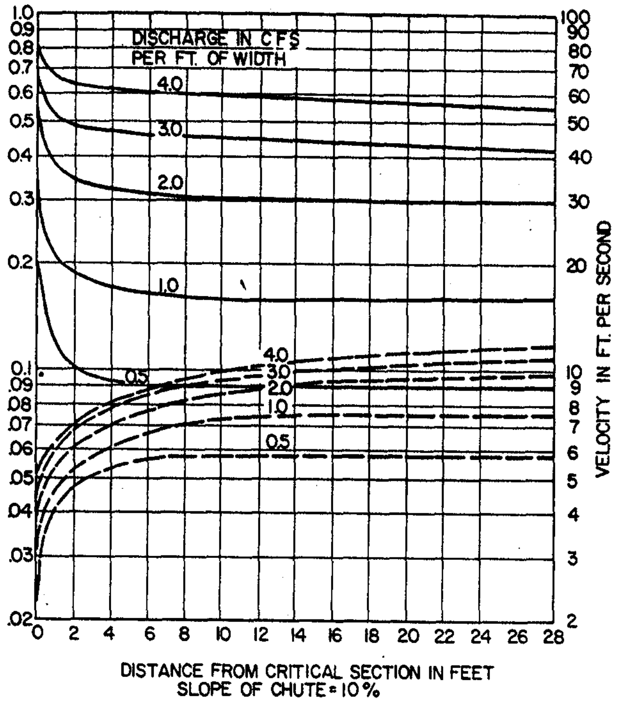
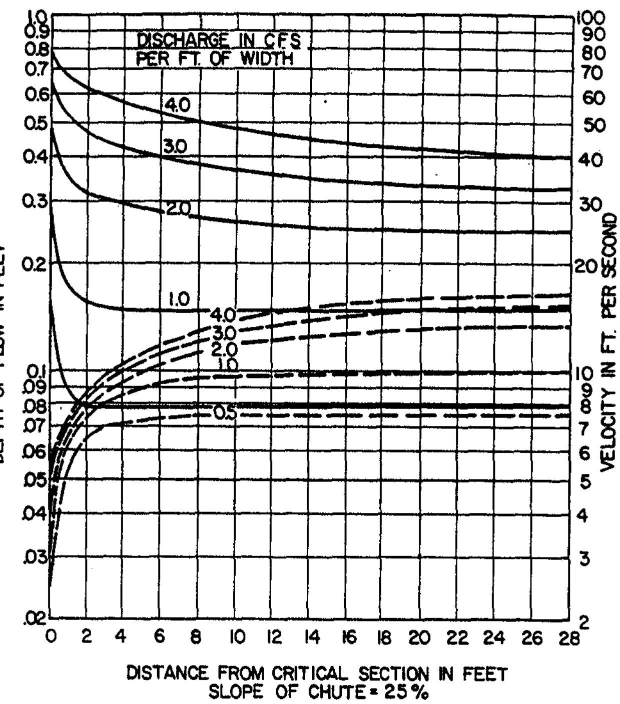
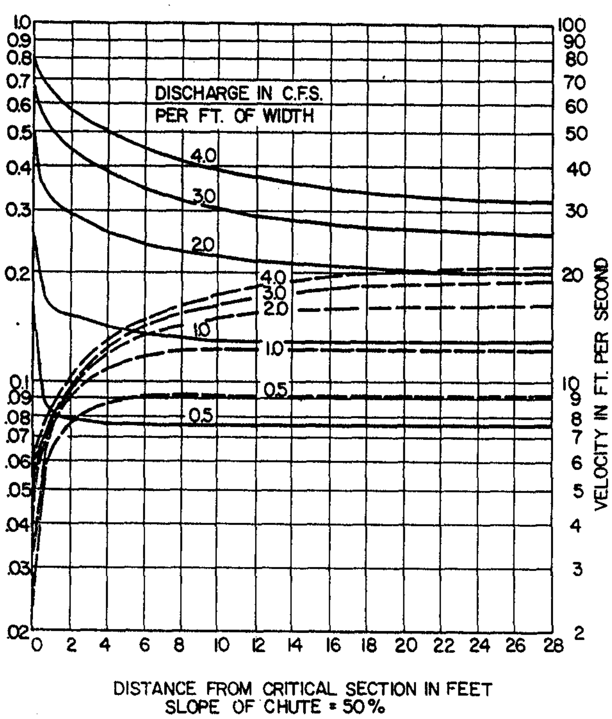
FIGURE 19-1. DETAILS OF TYPICAL DRAINAGE CHUTE







——— DEPTH RELATION CURVES

– – – VELOCITY RELATION CURVES

DEPTH CURVES INDICATE DEPTH OF WATER INCLUDING CORRECTION FOR AERATION RECOMMEND MINIMUM 50% INCREASE IN HEIGHT OF WALL WITH MINIMUM VALUE OF 3 INCHES. VELOCITY CURVES INDICATE COMPUTED VELOCITY. TO DETERMINE d_1 FOR STILLING BASINS USE:

$d_1 = q/V$

WHERE:

d_1 = DEPTH OF FLOW UPSTREAM FROM HYDRAULIC JUMP, FEET

Q = DISCHARGE PER FOOT OF WIDTH, CFS/FEET

V = AVERAGE VELOCITY OF FLOW, FEET/SECOND

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FIGURE 19-2. DESIGN CHARTS FOR CONCRETE DRAINAGE CHUTE computed depth of flow is recommended. A minimum depth of 3 inches is suggested for the chute. Minor irregularities in the finish of the chute frequently result in major flow disturbances and may even cause over topping 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 will be provided at the end of the chute.

19-2. Design problem. Design a concrete chute for a discharge of 8 fps, if the chute is to have a 25 percent slope and is to be 30 feet long.

19-3. Solution. Assume width of chute to be 2 feet. Then discharge Q = $3.75W^{0.9}\ H^{1.6}$ or

$$H = (Q/3.75W^{0.9})^{1.6} = (8.0/3.75 \text{ X } 2.0^{0.9})^{1.6} = 1.09 \text{ feet}$$

Provide berm at least 1.5 feet high.

From figure 19-2, the depth of flow at various stations along the chute can be determined from the depth-relation curve for q = 4.0 cfs per foot as follows:

Station feet	Depth of flow d_1 feet	Minimum height of sidewall ($1.5 \times d_1$) feet	Adopted height of sidewall inches
0+00	0.80	1.20	18
0+02	0.63	0.95	12
0+05	0.56	0.84	10
0+10	0.49	0.74	9
0+20	0.42	0.63	8
0+30	0.40	0.60	8

CHAPTER 20

STILLING BASINS

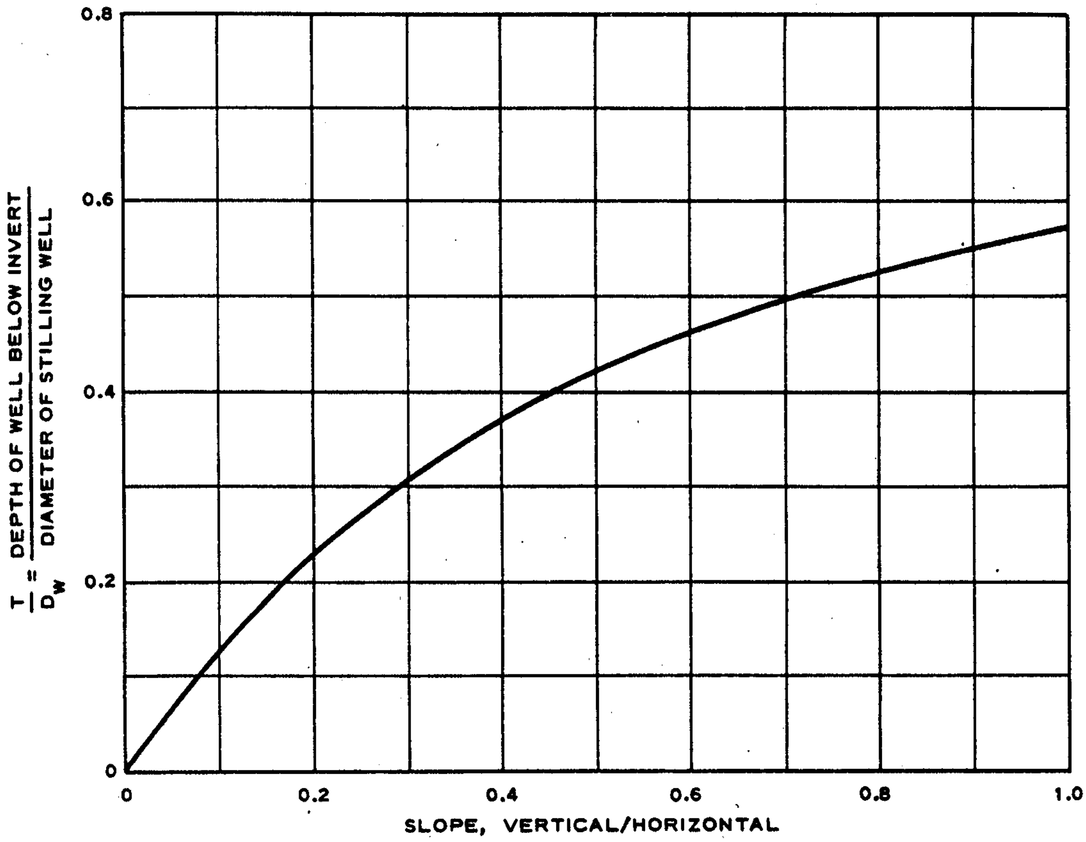
20-1. General. A stilling basin is a channel structure of mild slope, placed at the outlet of a spillway, chute or other high-velocity flow channel, the purpose of which is to dissipate some of the high kinetic energy of flow in a hydraulic jump. Stilling basins or other energy-dissipating devices are almost always necessary in such circumstances to prevent bed scour and undermining of the structure when the high velocity stream is discharged into the downstream channel. There are many types of these devices available such as hydraulic jump basins, roller buckets, flip buckets, impact energy dissipating devices and stilling wells. In unusual cases involving major structures, use of a special type of device should be considered. Three of the most commonly used energy dissipators, namely, a stilling well, the US Bureau of Reclamation (USBR), Type VI Basin, and the St. Anthony Falls (SAF) stilling basin will be reviewed in subsequent sections. The discussion that follows will be confined to energy dissipators used in conjunction with circular storm-drain-outlets. It is possible that energy dissipating devices may be necessary at the end of other types of outlets.

20-2. Stilling well.

a. Description. The stilling well consists of a vertical section of circular pipe affixed to the outlet end of a storm-drain outfall. Components of a typical stilling well are shown in figure 20-1. In order to be effective, the top of the well must be located at the elevation of the invert of a stable natural drainage basin or an artificial channel. The area adjacent to the top of the well, including the side slopes and outfall ditch, is usually protected by riprap or paving.

b. Hydraulics. Energy dissipation is accomplished by the expansion of flow that occurs in the well, the impact of the flow on the base and wall of the stilling well opposite the pipe outlet, and the change in momentum resulting from redirection of the flow. Important advantages of an energy dissipator of this type are that energy loss is accomplished without the necessity of maintaining a specified tailwater depth in the vicinity of the outlet and construction is both simpler and less expensive because the concrete formwork necessary for a conventional basin is eliminated.

c. Well depth. The stilling wells suggested were recommended from tests conducted on a number of model stilling wells. The recommended height of stilling wells above the invert of the incoming pipe is two times the diameter of the incoming pipe, D_o. The recommended depth of well below the invert of the incoming pipe, T is dependent on the slope of the incoming pipe and the diameter of the stilling well, D_w, and can



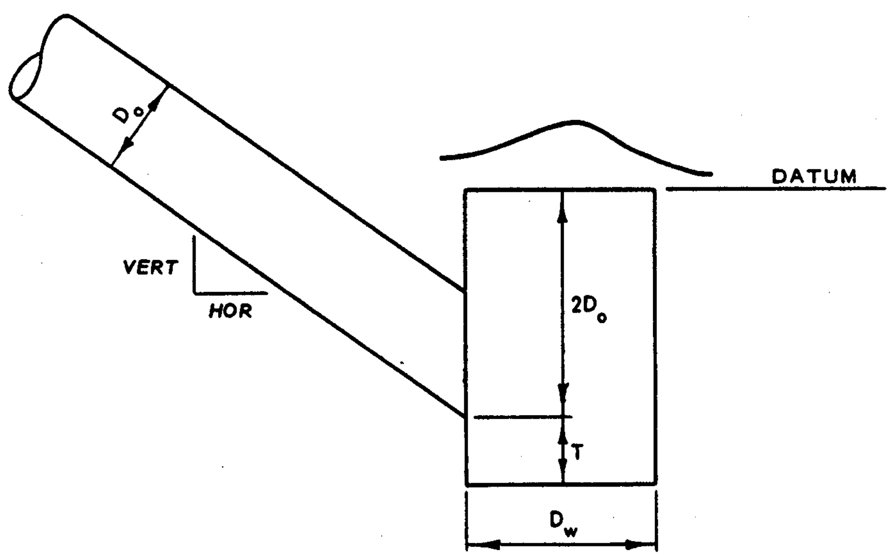


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FIGURE 20-1. STILLING WELL

be determined from the plot shown in figure 20-1. The model investigations indicated that satisfactory performance could be maintained for $Q/D_o^{5/2}$ ratios as large as 2.0, 3.5, 5.0, and 10.0, respectively, with stilling well diameters of one, two, three, and five times that of the incoming storm drain. The stated ratios were used to calculate the relations among actual storm-drain diameter, well diameter, and maximum discharge recommended for selection and design of stilling wells and shown in figure 20-2.

20-3. USBR Type VI basin.

a. Hydraulics. The USBR impact energy dissipator is an effective stilling device even with deficient tailwater. Dissipation is accomplished by the impact of the incoming jet on the vertical hanging baffle and by eddies formed by changing the direction of the jet after it strikes the baffle. Best hydraulic action is obtained when the tailwater elevation approaches, but does not exceed, a level half the height of the baffle. Excessive tailwater, on the other hand, will cause some flow to pass over the top of the baffle, which should be avoided, if possible. With velocities less than 2 fps, the incoming jet could possibly discharge underneath the hanging baffle. Thus, this basin is not recommended with velocities less than 2 fps. To prevent the possibility of cavitation or impact damage to the baffle, it is believed that an entrance velocity of 50 fps should not be exceeded with this device. The general arrangement of the Type VI basin and the dimensional requirements based on the width of the structure are shown in figure 20-3.

b. Model tests. The model used to test the limitations of the type VI basin was reported (by Beichley). The results of test on this particular model which had a width four times the diameter of the incoming pipe indicated that the limiting $Q/D_o^{5/2}$ value was 7.6. This value is slightly less than that recommended (by Beichley) in terms of the Froude number at the storm-drain outlet. The recommended relations between discharge, outlet diameters, and basin widths are shown in figure 20-4. With the discharge and size of the incoming pipe known, the required width of the basin can be determined from the design curves; other dimensions of the basin can be computed from the equations in figure 20-3.

20-4. SAF basin.

a. Description. The SAF stilling basin is a hydraulic jump type basin. All the dimensions of this basin are related in some way to the hydraulic jump. A reduction in the basin length from that of a natural hydraulic jump is achieved through the use of appurtenances consisting of chute blocks, floor blocks or baffle piers, and an end sill. General details of the SAF basin are shown in figure 20-5. Dimensions of the chute blocks and floor blocks may be modified slightly to

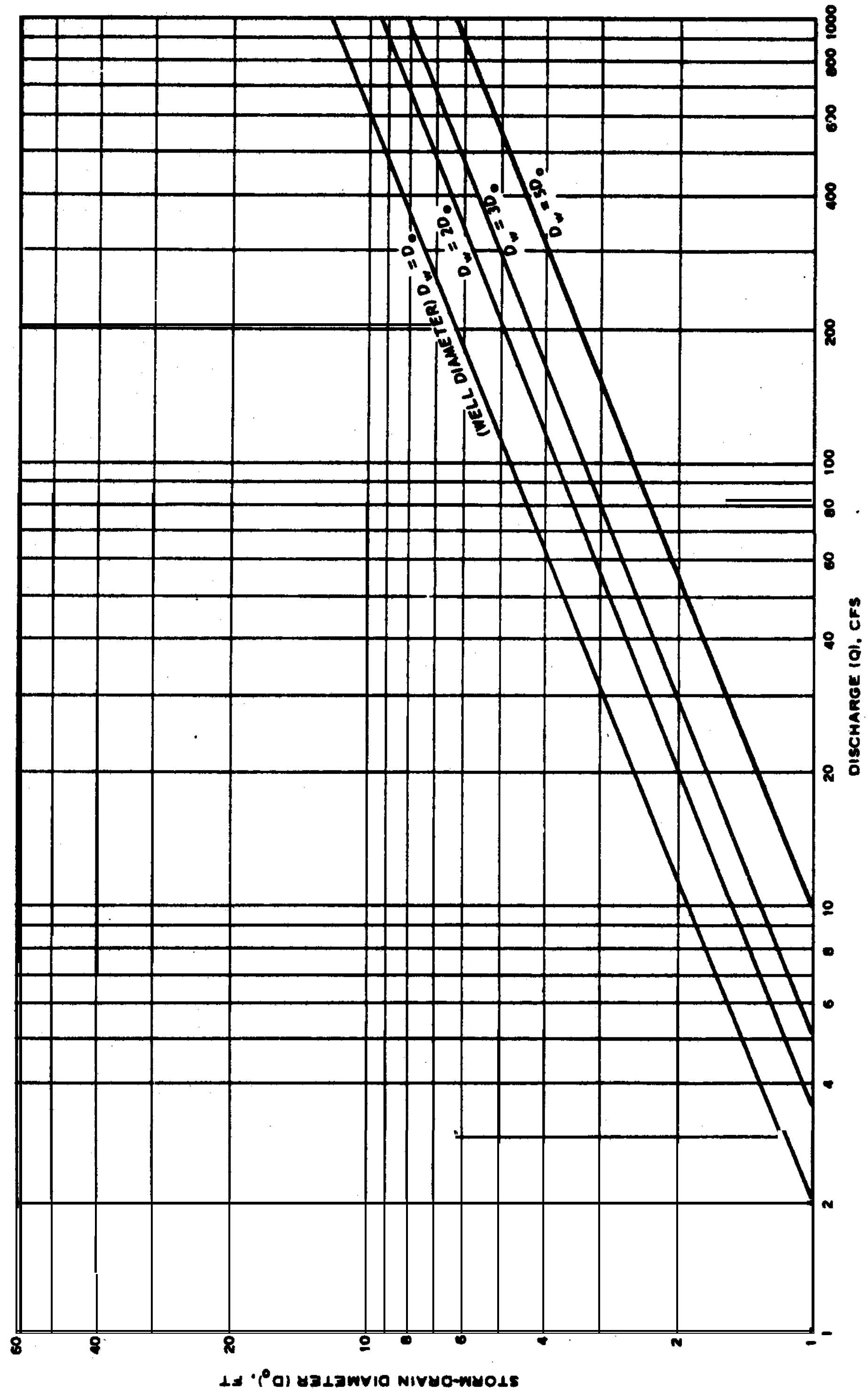

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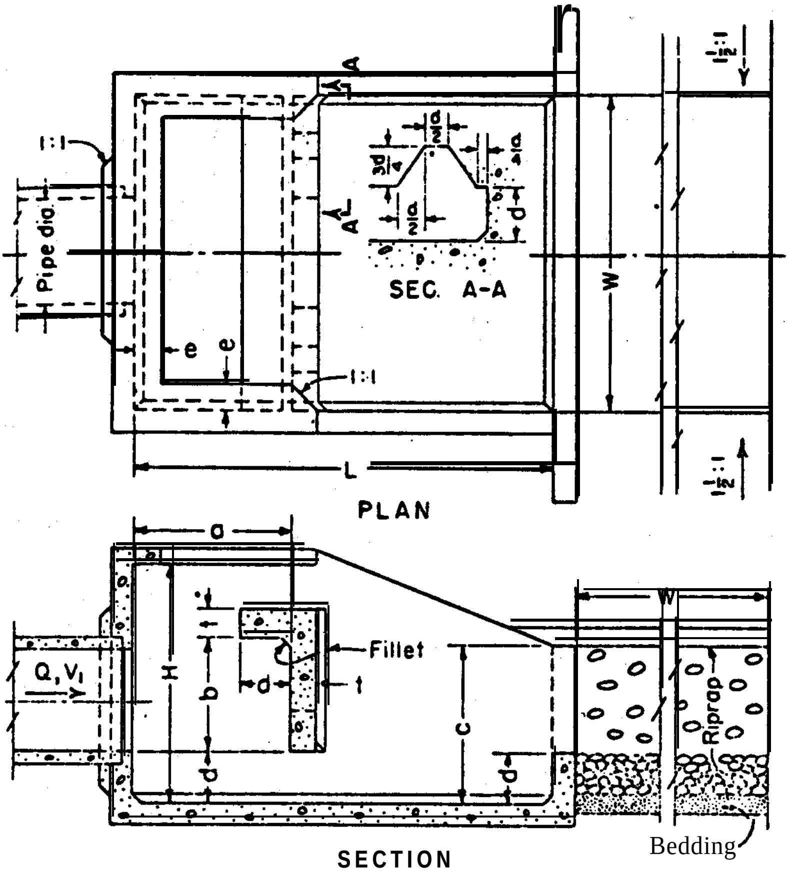
FIGURE 20-2. STORM-DRAIN DIAMETER VERSUS DISCHARGE STILLING WELL



STILLING BASIN DESIGN

H = 3/4 (W)
L = 4/3 (W)
a = 1/2 (W)
b = 3/8 (W)
c = 1/2 (W)
d = 1/6 (W)
e = 1/12 (W)
t = 1/12 (W), SUGGESTED MINIMUM
RIPRAP STONE SIZE DIAMETER = 1/20 (W)

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FIGURE 20-3 , USBR TYPE VI BASIN

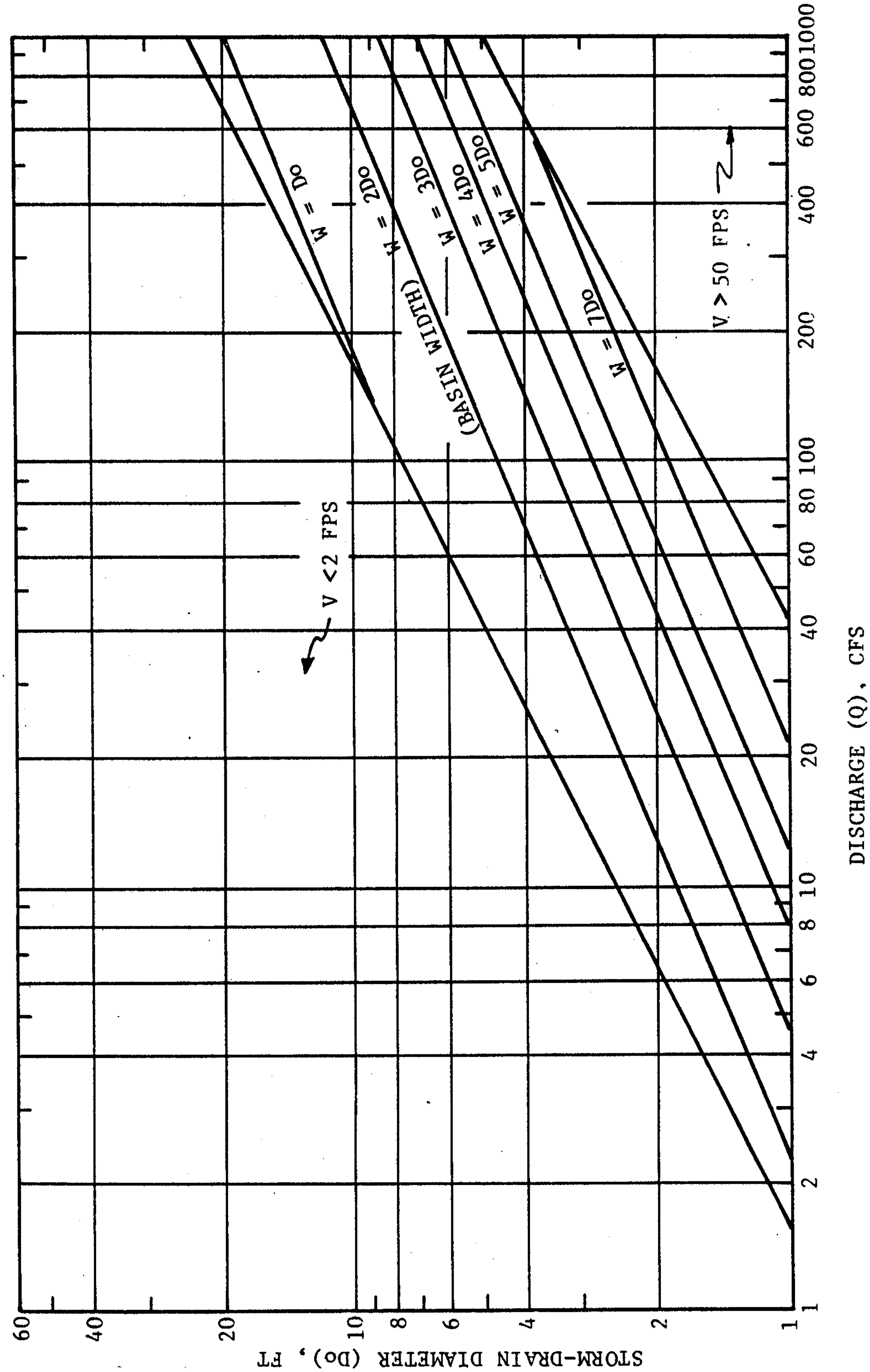

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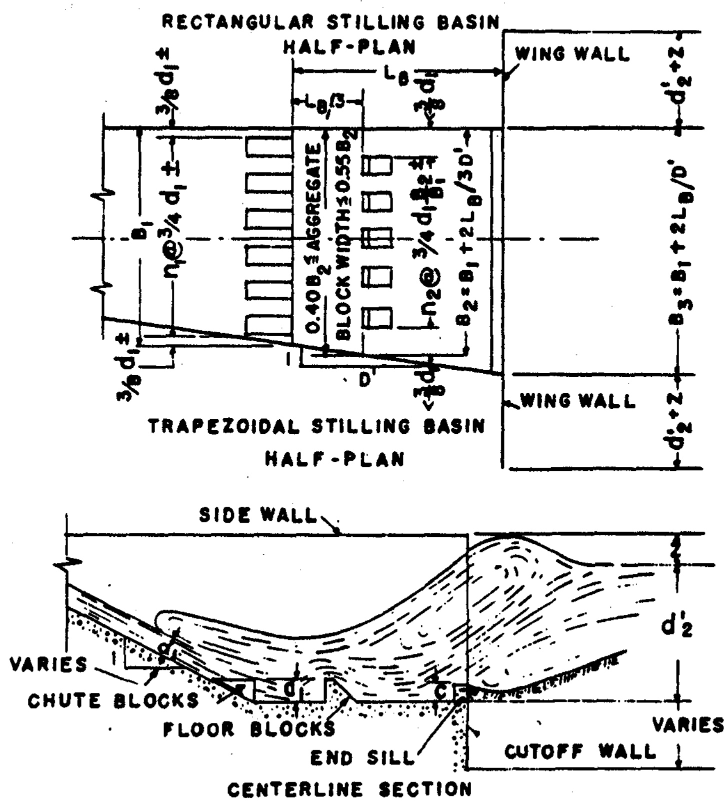
FIGURE 20-4. STORM-DRAIN DIAMETER VERSUS DISCHARGE USBR TYPE VI BASIN



DESIGN EQUATIONS

(1) $F = \frac{V_1^2}{gd_1}$ (2) $d_2 = \frac{d_1}{2}(-1 + \sqrt{8F + 1})$

(3a) $F = 3$ TO 30 $d'_2 = (1.10 - F/120)\, d_2$

(3b) $F = 30$ TO 120 $d'_2 = 0.85 d_2$

(3c) $F = 120$ TO 300 $d'_2 = (1.00 - F/800)\, d_2$

(4) $L_B = \frac{4.5 d_2}{F^{0.38}}$ (5) $Z = \frac{d_2}{3}$ (6) $c = 0.07 d_2$

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FIGURE 20-5. PROPORTIONS OF SAF STILLING BASIN provide reasonable construction dimensions without materially affecting the efficiency of the structure.

b. Model tests. Several different models were constructed according to recommendations made from tests at the St. Anthony Falls Hydraulic Laboratory. Stilling basins one, two, and three times as wide as the outlet were tested with drops from the invert of the outlet to basin floor of one-half and two times the outlet diameter. The basins with widths of two and three times the outlet diameter were flared one on eight with respect to the center line of the structure. The size of the basin elements and the basin length were adjusted for the two apron elevations according to the depth of flow entering the basin. Comparisons of flow conditions for the various discharges with each basin were made with tailwater depths only sufficient to produce a hydraulic jump in the basin. Within the limits investigated, the drop from the invert of the outlet to the basin apron had little effect on the limiting $Q/D_o^{5/2}$ ratios. Maximum values of 3.5, 7.0, and 9.5 were indicated for $1D_o$, $2D_o$, and $3D_o$, $2D_o$, and $3D_o$ wide SAF stilling basins, respectively. These were used to determine the relations recommended for design and shown in figure 20-6.

20-5. Design summary.

a. Relationship of Froude number. The general design practice that has developed in recent years relative to highway culverts results in the conclusion that most of these structures convey discharges up to four or five times the diameter of the culvert raised to the five-halves power. The energy dissipator magnitude of this parameter will vary depending on the particular site or structure, but it is useful for classifying the relative design capacity of such structures. It is also related to the Froude number of flow commonly used in open channel hydraulics. For example, the Froude number of full pipe flow at the outlet of a circular pipe is unity for a $Q/D_o^{5/2}$ ration of 4.5

b. Widths versus maximum discharge. The range of applicability of maximum discharge capacity for various widths of the three commonly used energy dissipators relative to the diameter of the incoming culvert or storm-drain outlet, D_o is summarized in table 20-1. Based on these values of the relative maximum discharge capacity for comparable relative widths of the three energy dissipators, the stilling well is particularly suited to the lower range of discharges, the USBR Type VI basin to the intermediate range of discharges, and the SAF stilling basin to the higher range of discharges. However, all three of the energy dissipators are applicable for general drainage and erosion control practice. Comparative cost analyses will indicate which of the devices is the most economical energy dissipator for a given installation.

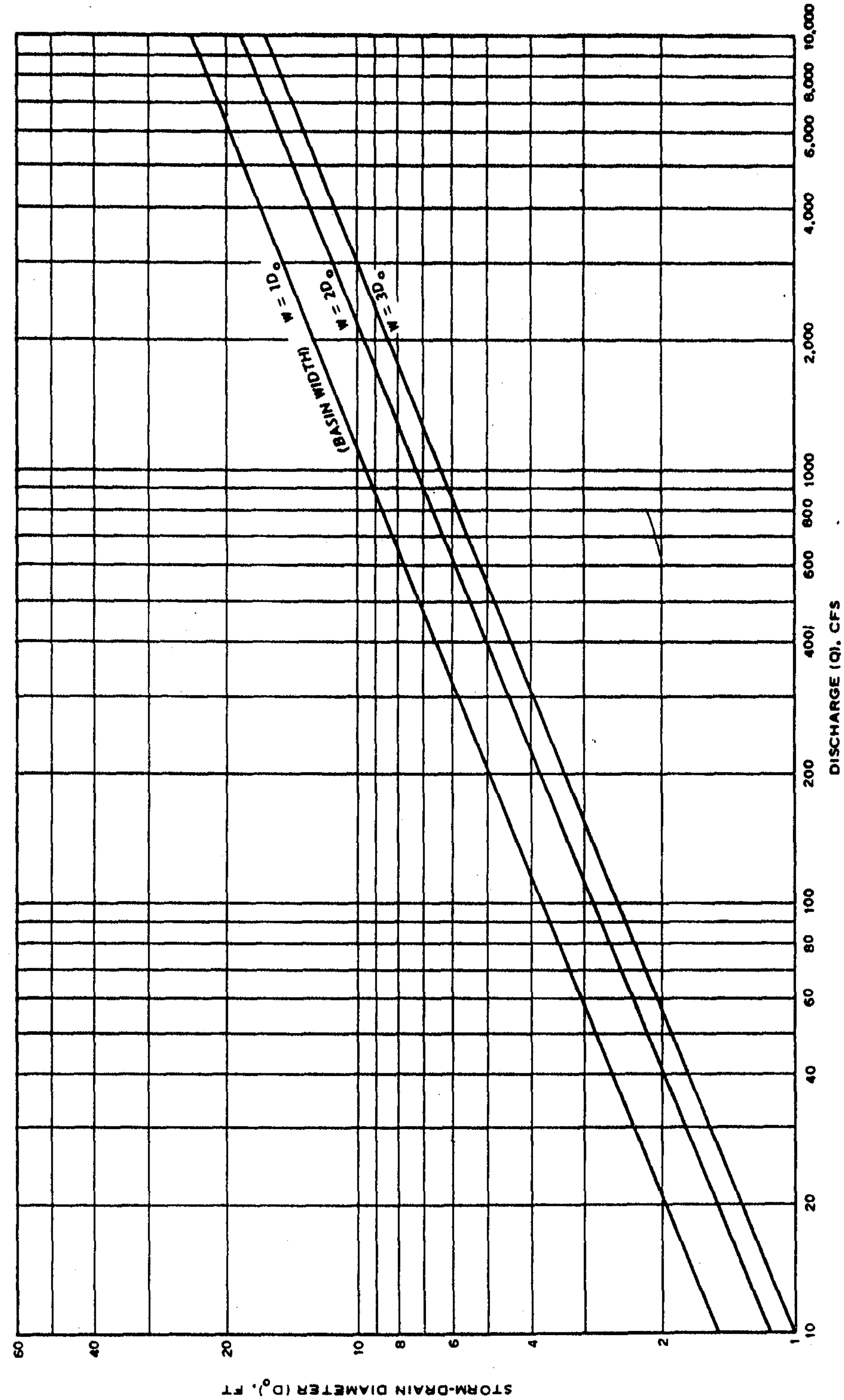

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FIGURE 20-6. STORM-DRAIN DIAMETER VERSUS DISCHARGE SAF STILLING BASIN

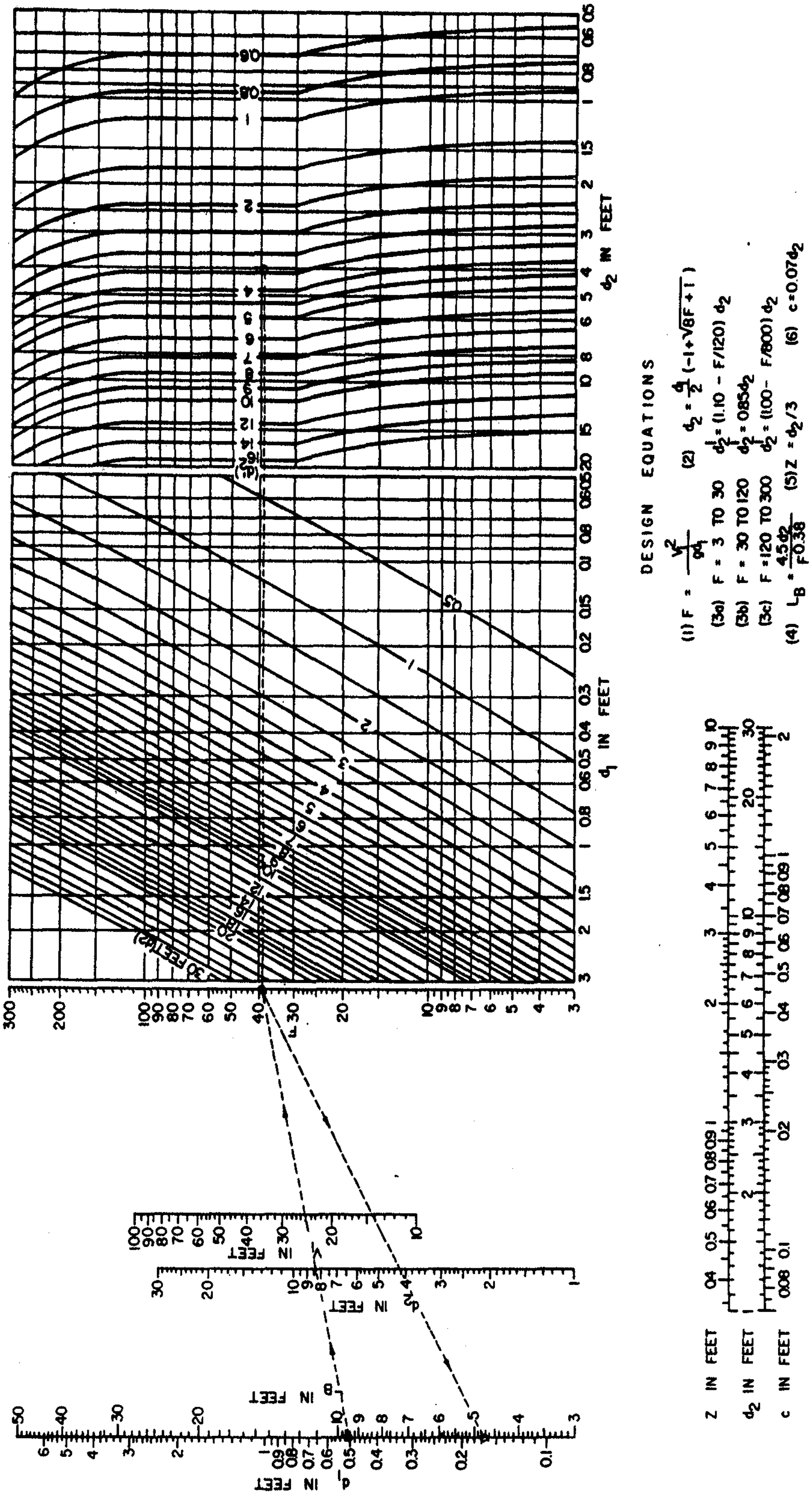
Table 20-1. Maximum Discharge Recommended for Various Types and Sizes of Energy Dissipators

Relative width and type of energy dissipator	Maximum $Q/D_o^{5/2}$
Stilling Well	
1 D_o diameter	2.0
2 D_o diameter	3.5
3 D_o diameter	5.0
5 D_o diameter	10.0
USBR Type VI Basin	
1 D_o wide	0.6
2 D_o wide	2.2
3 D_o wide	4.5
4 D_o wide	7.6
5 D_o wide	11.5
7 D_o wide	21.0
SAF Stilling Basin	
1 D_o wide	3.5
2 D_o wide	7.0
3 D_o wide	9.5

20-6. Comparison of various stilling basins. Using the given design curves for the three energy dissipators, the designer can determine the applicability and necessary dimensions of each type of energy dissipator. In some cases, more than one type of dissipator may be applicable and in such cases local terrain, tailwater conditions, and cost analyses will determine the most practical energy dissipator for protecting the outlet. For example, with a 60 inch diameter culvert and a design discharge of 290 cfs, either a 10-foot wide ($2D_o$) SAF stilling basin, or a 20-foot wide ($4D_o$) USBR Type VI basin, or a 20-foot diameter ($4D_o$) stilling well could be used. With a 48-inch diameter culvert and a design discharge of 110 cfs, a 4-foot wide ($1D_o$) SAF stilling basin or an 8-foot diameter ($2D_o$) stilling well or a 10-foot wide ($2.5D_o$) USBR Type VI basin could be used.

20-7. Design problem.

a. Use of SAF stilling basin design chart. The use of the SAF stilling basin chart (fig 20-7) is illustrated by the following problem. It will demonstrate that the dimensions of the chute blocks and floor blocks may be modified without materially affecting the efficiency of the structure.



BULLETIN SAF STILLING BASIN BY SCS IN COOPERATION WITH MINNESOTA AGRICULTURAL EXPERIMENT STATION

FIGURE 20-7. DESIGN CHART FOR SAF STILLING BASIN

b. Design approach. The indicated design is for a rectangular stilling basin to be constructed at the outlet of a 4-foot-wide rectangular chute. The depth and velocity of the flow at the end of the chute are 0.5 foot and 25 fps, respectively, and the depth of flow in the outlet channel is 2 feet for the design discharge of 50 cfs.

20-8. Solution.

a. Principal dimensions. Reading the principal dimensions from the design chart it is found that d_2 = 4.2 feet, d_2' = 3.5 feet, L_B = 4.7 feet, Z = 1.4 feet, and c = 0.294 foot. In order to simplify construction, use L_B = 4.75 feet and c = 6 inches. (The height of end sill will be a minimum of 6 inches.) The height of the sidewall is d_2 + Z = 4.90 feet (use 5 feet) which places the top of the wall 1.5 feet above the tailwater elevation.

b. Alternate arrangements. Several arrangements of the 6-inch-high chute and floor blocks are possible, the floor blocks being placed one-third of the height of the sidewall or 1.67 feet downstream from the upper end of the basin. The width of the chute and floor blocks and the spaces between can be made 0.5 x 3/4 = 0.375 foot, or 4.5 inches. This gives 48/4.5 = 10+ spaces across the stilling basin. Locate 3.75-inch-wide chute blocks at either side of the flume and space remaining 4.5-inch-wide blocks on 9-inch centers across the chute.

c. Configuration of floor blocks. Floor blocks can be located downstream from spaces between chute blocks except that a floor block cannot be placed adjacent to sidewalls. This will result in 5 floor blocks or 5 x 4.5 x 100/48 = 47 percent of the width of the basin occupied by floor blocks. Considerable flexibility is permitted in designing chute and floor blocks, and a satisfactory arrangement can be developed using a block width of 6 inches with a chute block placed adjacent to the sidewalls.

d. Wingwalls. A wingwall will be provided with a height equal to that of the sidewall and a length determined by the side slopes of the outfall channel. A cutoff wall having a depth of 2 feet or more will be used. The outfall channel will be riprapped for a distance of at least L_B = 5 feet downstream from the stilling basin.

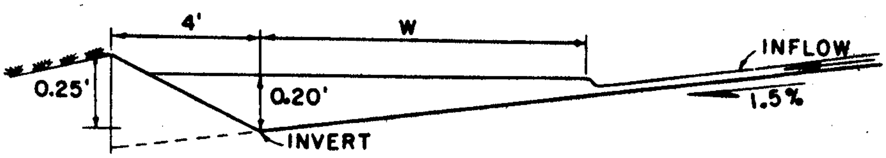
CHAPTER 21

GUTTERS

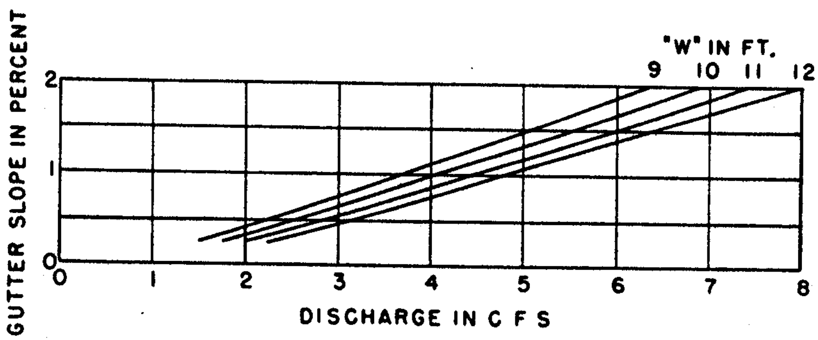
21-1. General. The use of gutters for mobilization construction, even when deferred is discouraged. However, the use of shallow paved gutters may prove to be an expedient solution to a drainage problem or an effective remedial action under deferred conditions. Gutters should be considered for special cases only not as a standard plan of street, road, or airstrip design.

21-2. 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 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 the Manning equation.

21-3. Design charts. A cross section of a typical runway gutter and the design charts are shown in figure 21-1. 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 3 inches.



TYPICAL GUTTER SECTION
FOR ARMY AIRFIELDS



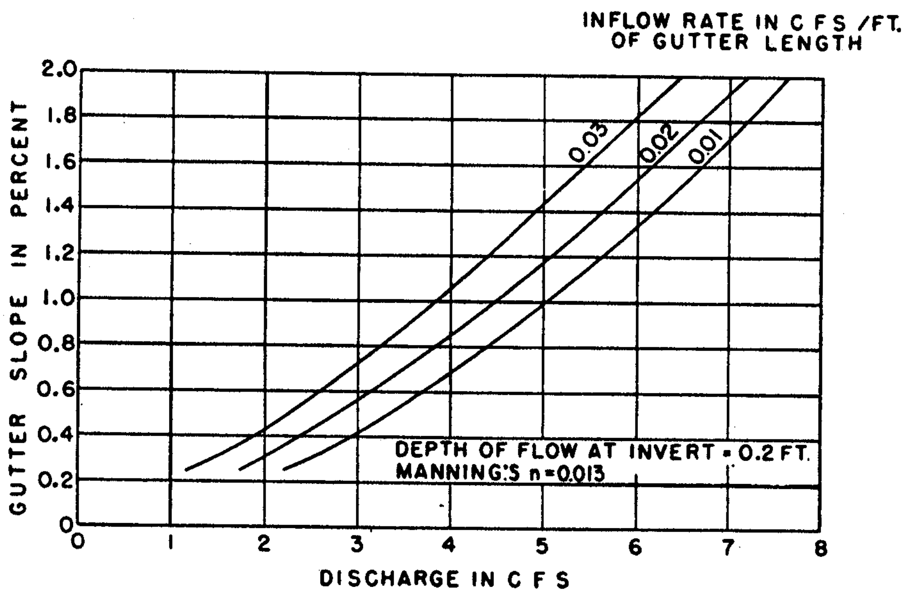


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FIGURE 21-1. DRAINAGE GUTTERS FOR RUNWAYS AND APRONS

APPENDIX A

SUPPORTING INFORMATION FOR CULVERT DESIGN

The following graphs are supplementary information to those presented in part three chapter 5 of this manual. The graphs listed below provide design and for special shaped, special inlet conditions, and additional materials for culverts.

Inlet Control nomographs - figures A-1 through A-5.

Outlet Control head (H) nomographs - figures A-6 through A-9.

Outlet Control Critical Depth (d_c) graphs - figures A-10 through A-13.

Culvert Capacity Charts - figures A-14 through A-28.

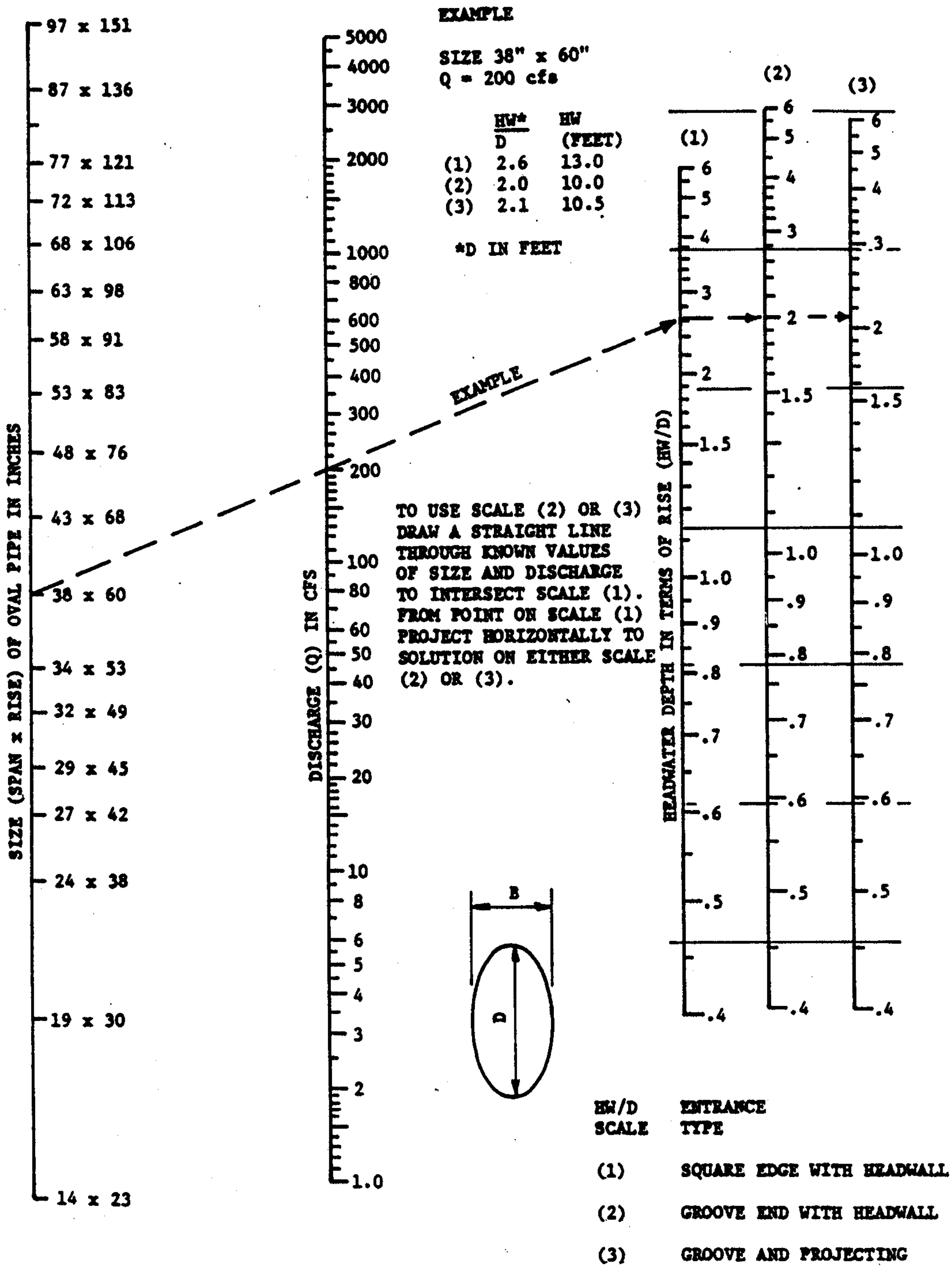

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FIGURE A-1. HEADWATER DEPTH FOR OVAL SMOOTH PIPE CULVERTS, LONG AXIS VERTICAL WITH INLET CONTROL

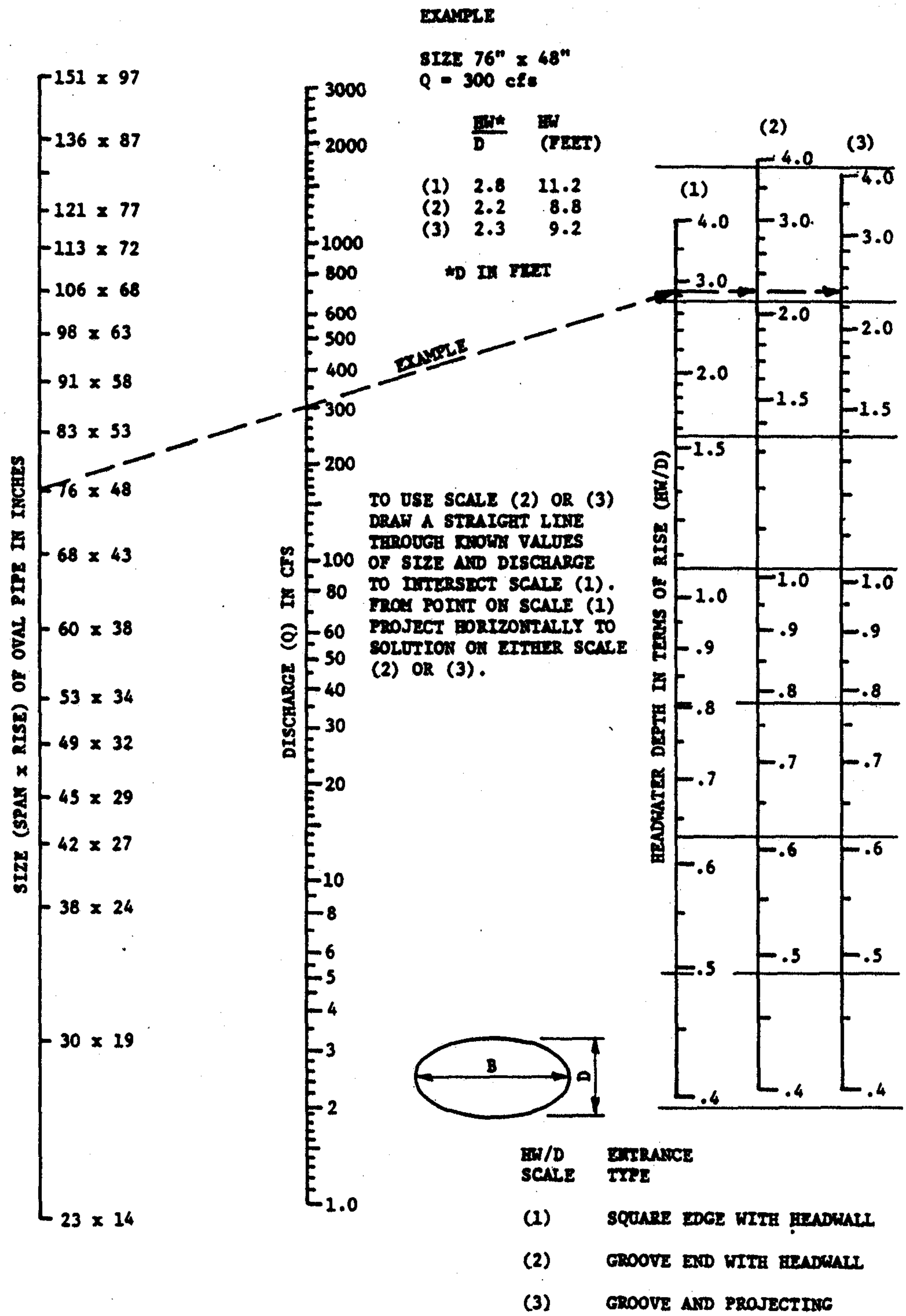

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FIGURE A-2. HEADWATER DEPTH FOR OVAL SMOOTH PIPE CULVERTS, LONG AXIS HORIZONTAL WITH INLET CONTROL

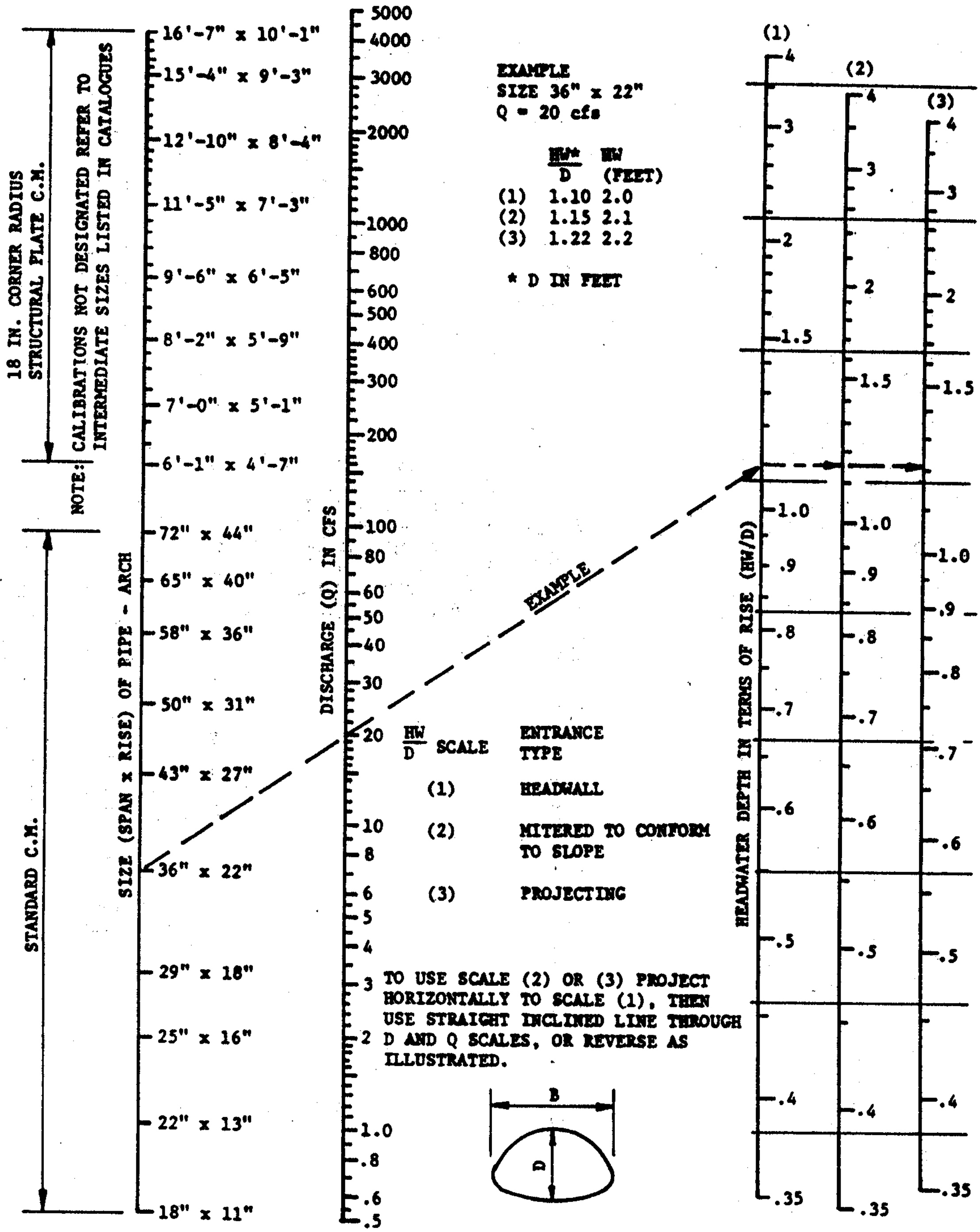

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FIGURE A-3. HEADWATER DEPTH FOR STRUCTURAL PLATE AND CORRUGATED PIPE ARCH CULVERTS WITH INLET CONTROL

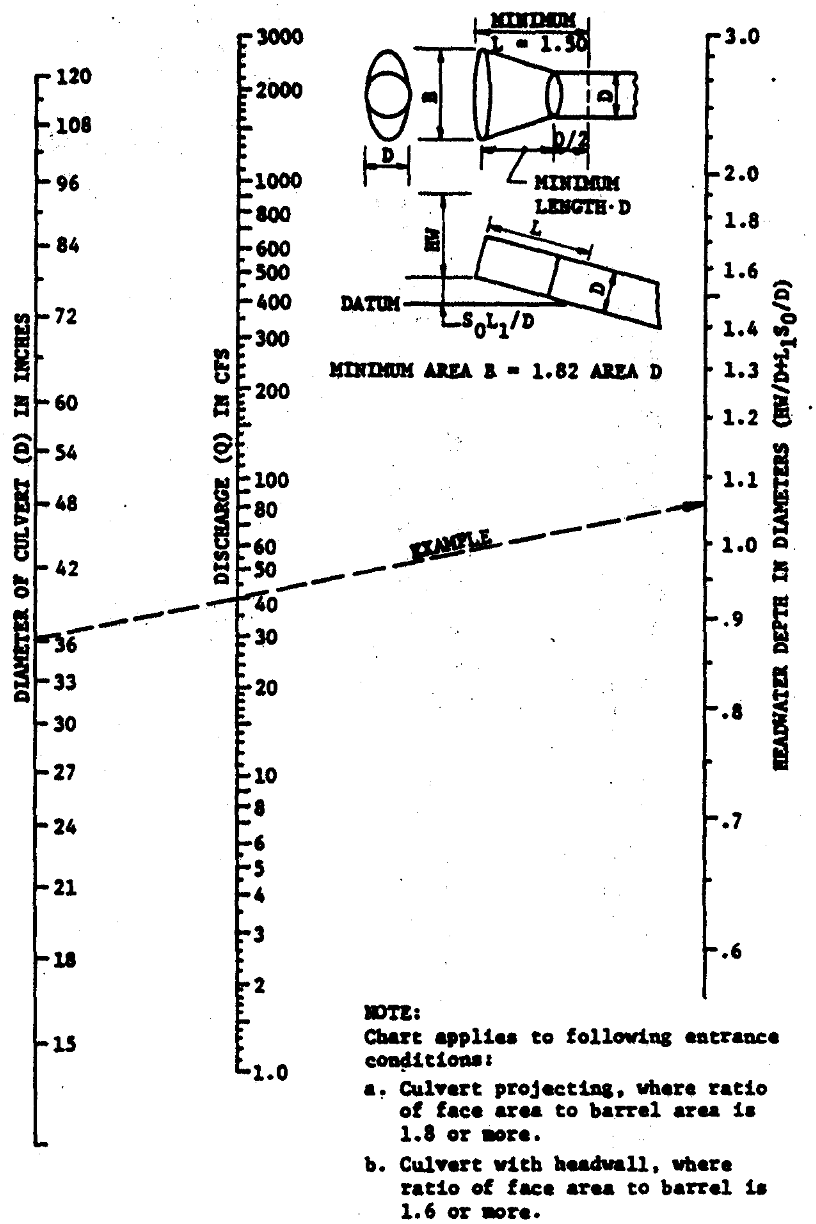

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FIGURE A-4. HEADWATER DEPTH FOR CORRUGATED PIPE CULVERTS WITH TAPERED INLET, INLET CONTROL

$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.063	A
0.063	0.125	0.042	0.125	B

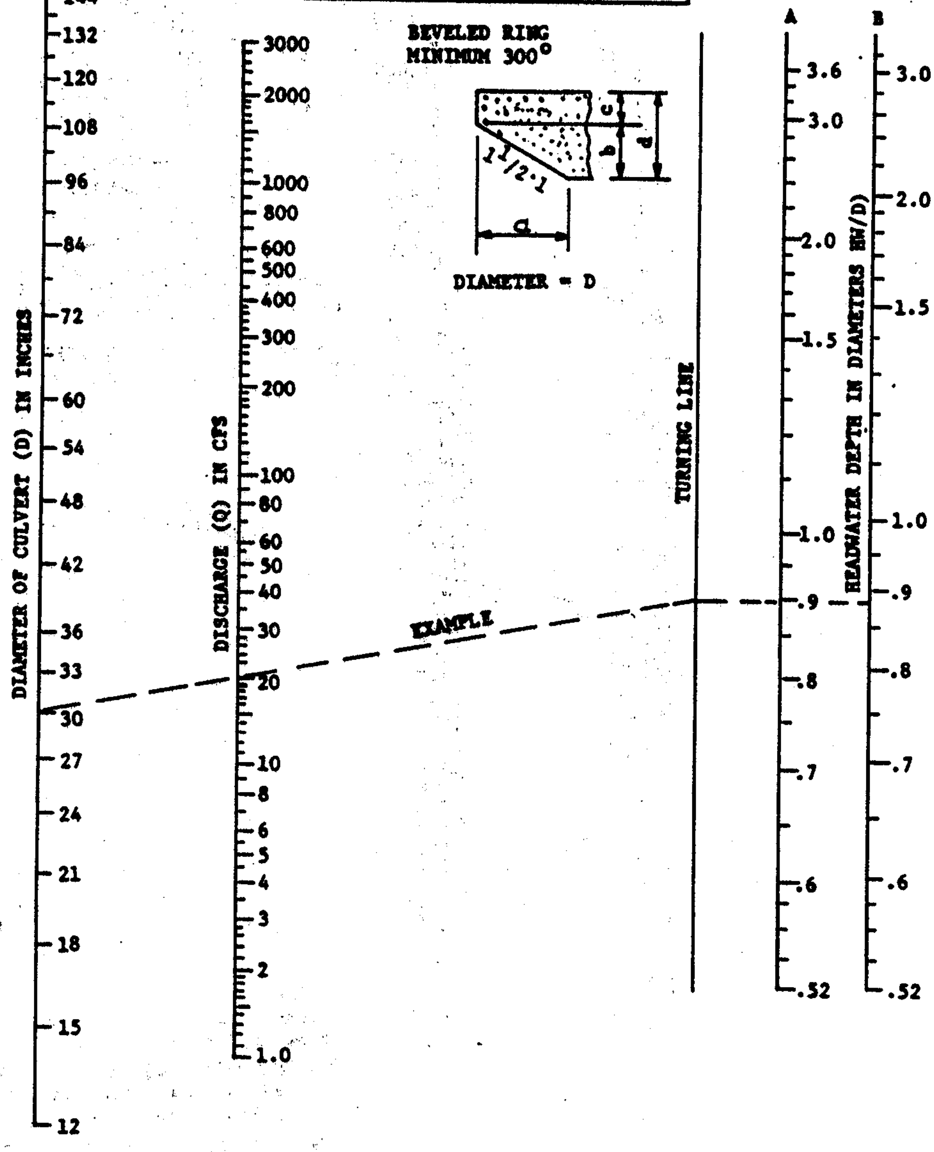

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FIGURE A-5. HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING, INLET CONTROL

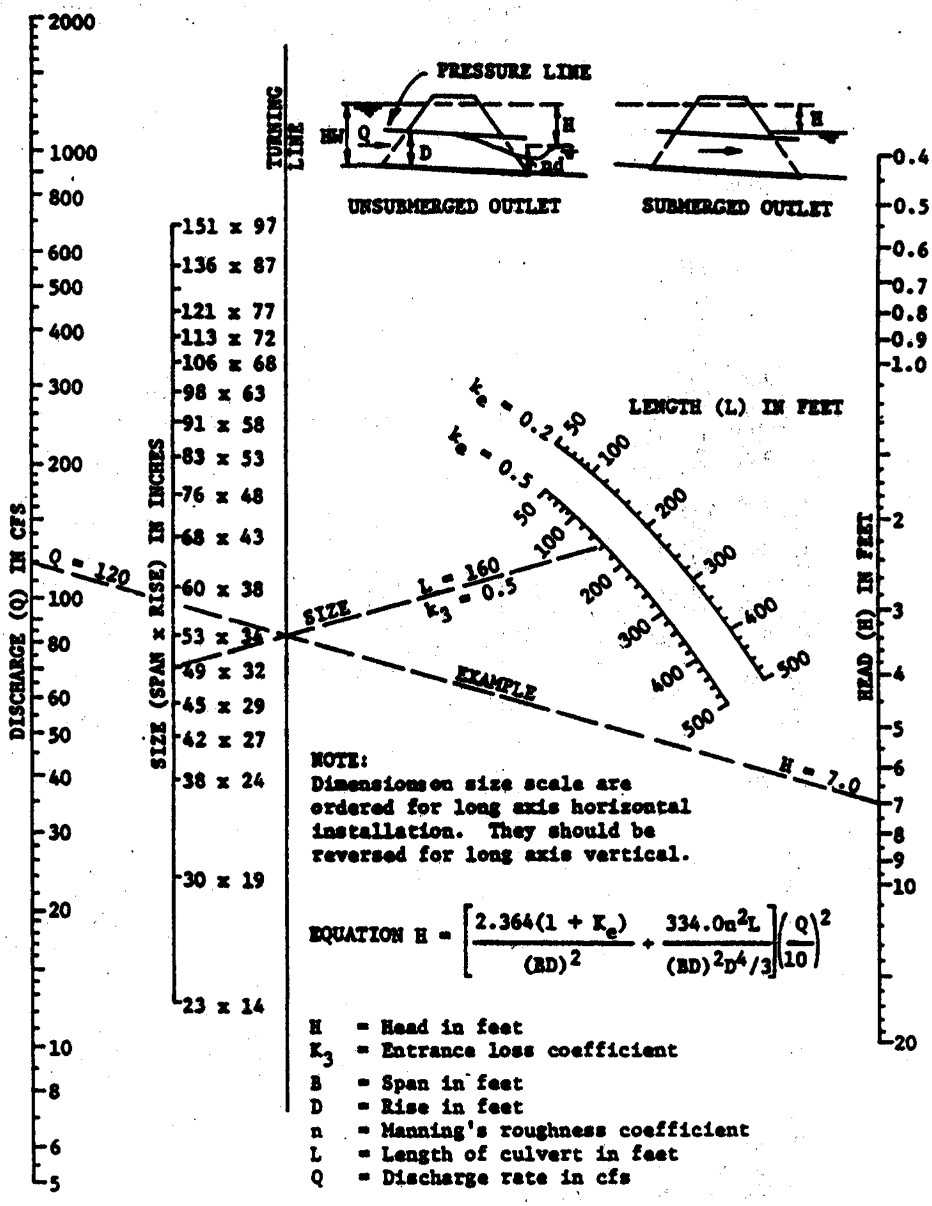

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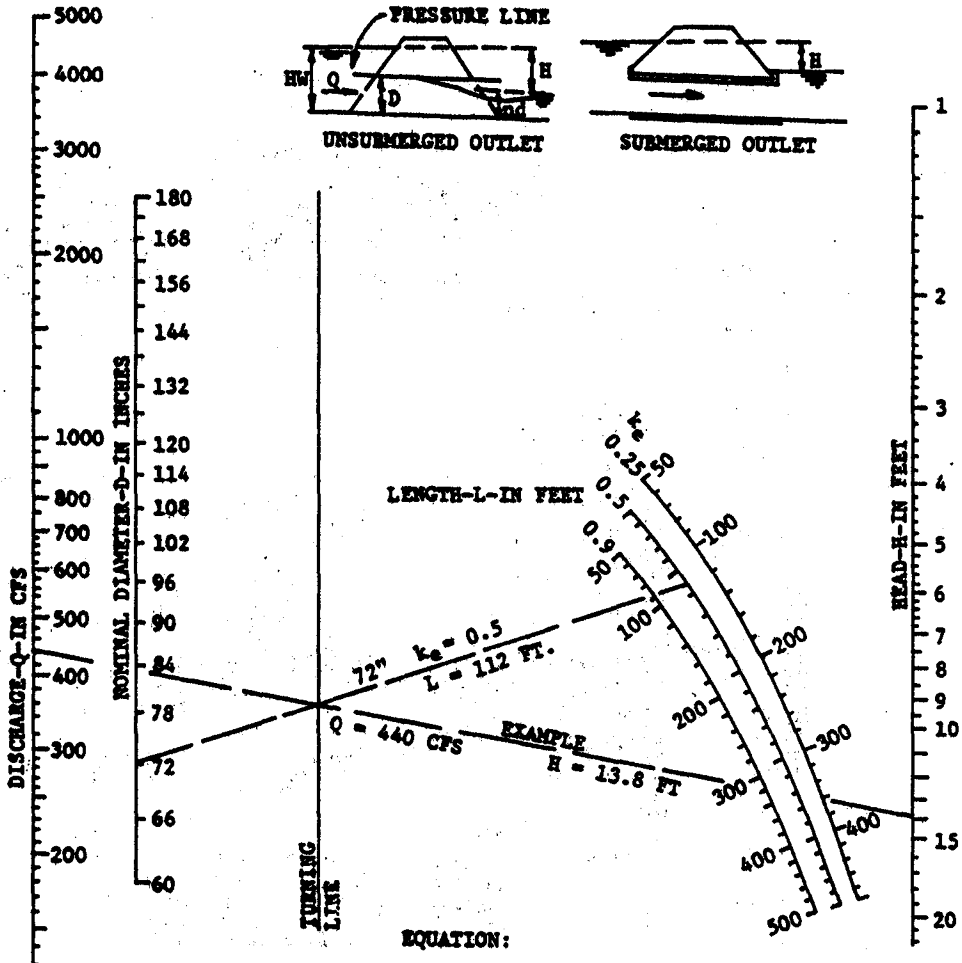
FIGURE A-6. HEAD FOR OVAL SMOOTH PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL n = 0.012



EQUATION:

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

H = Head in feet
K_3= 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

NOTE:
Computed for actual diameters given in fabricators' catalogs.

Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

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FIGURE A-7. HEAD FOR STRUCTURAL PLATE PIPE CULVERTS FLOWING FULL n = 0.0328 TO 0.0302

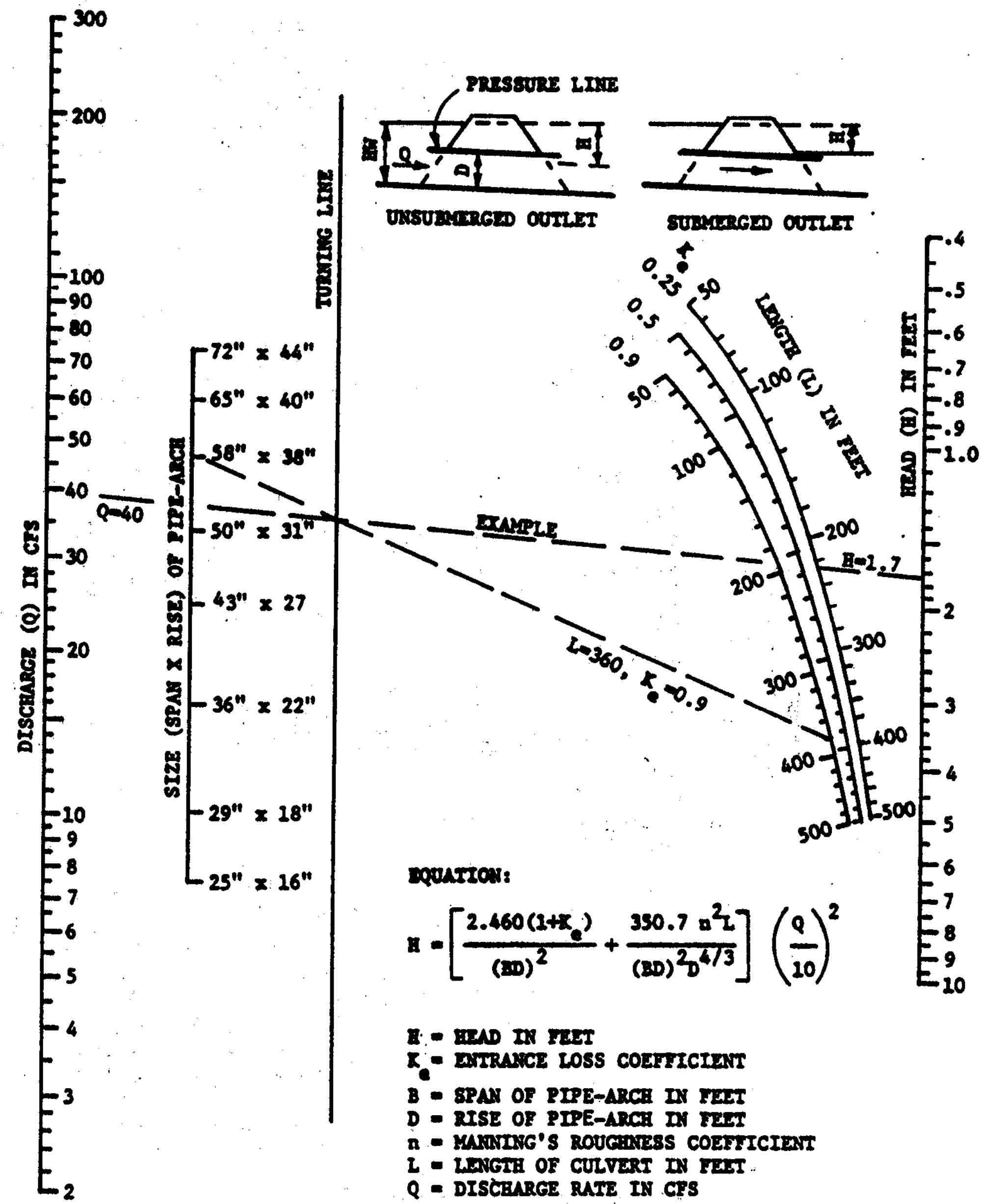

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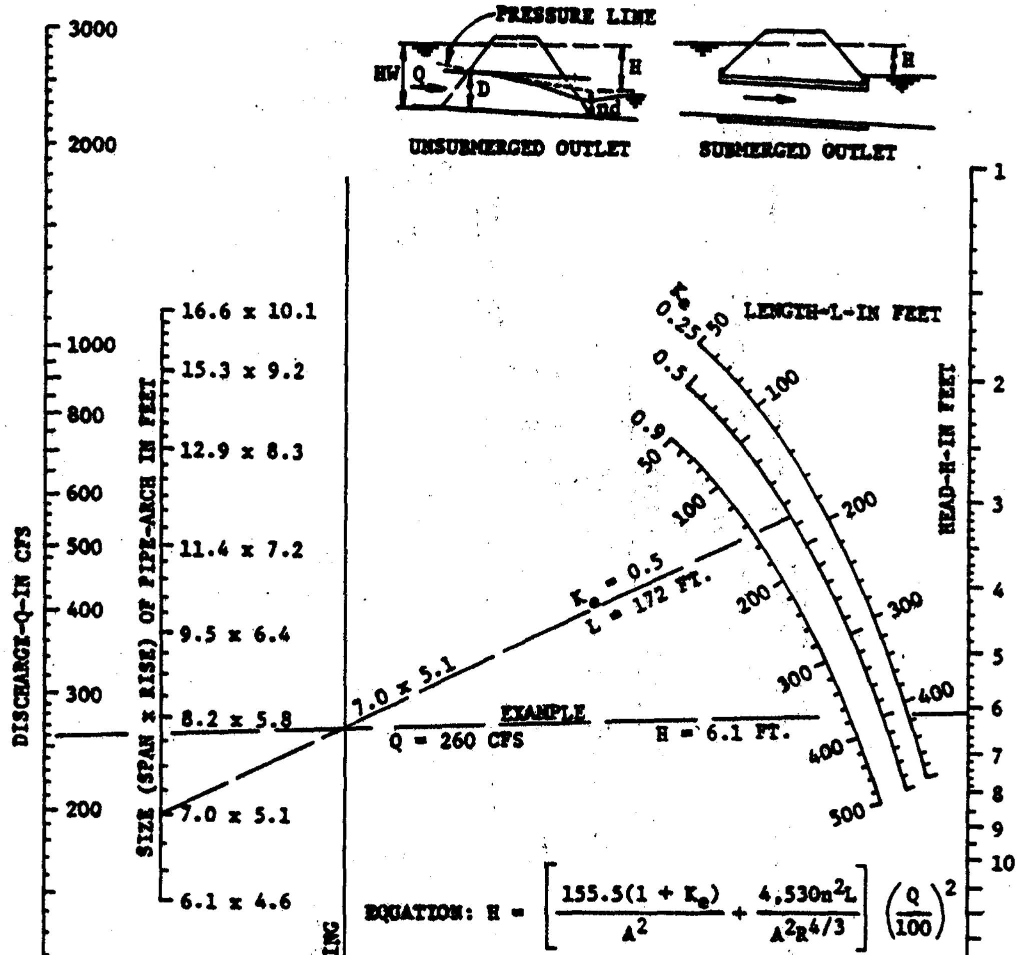
FIGURE A-8. HEAD FOR CORRUGATED PIPE-ARCH CULVERT FLOWING FULL n = 0.024



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

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 x 4.6	0.0327
8.2 x 5.8	0.0321
11.4 x 7.2	0.0315
16.6 x 10.1	0.0306

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FIGURE A-9. HEAD FOR FIELD-BOLTED STRUCTURAL PLATE PIPE-ARCH CULVERTS 18-INCH CORNER RADIUS FLOWING FULL n = 0.327 TO 0.0306

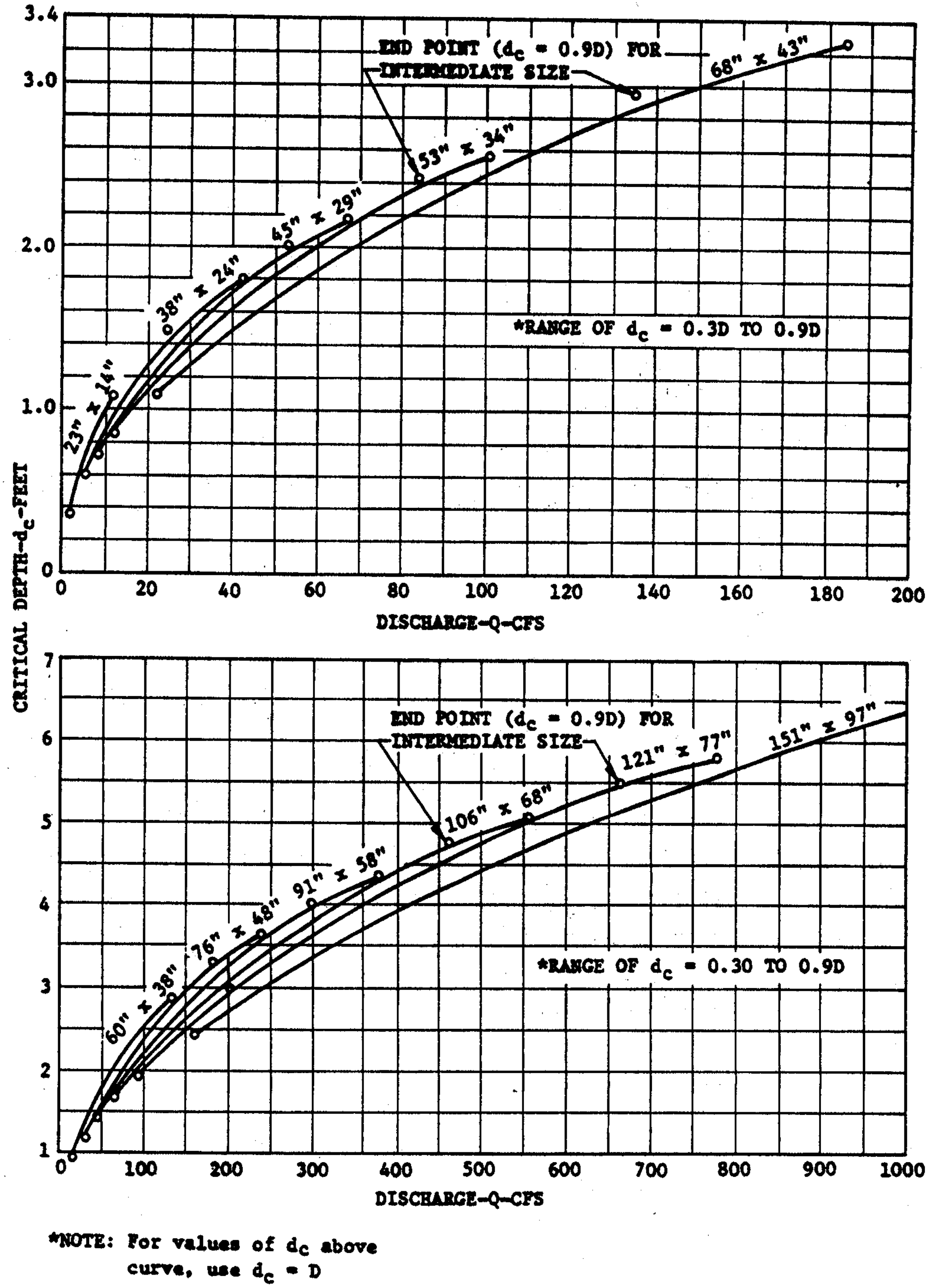

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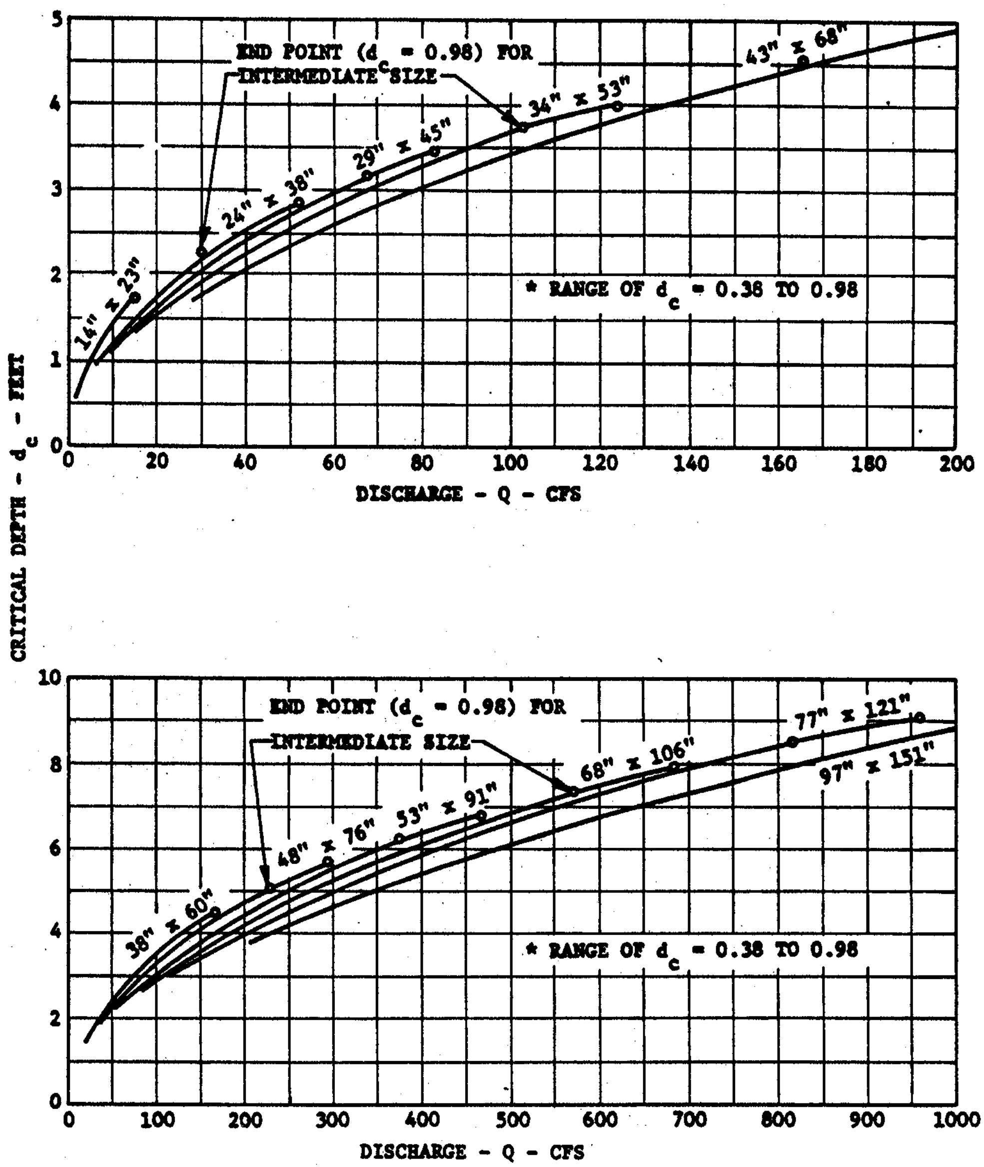
FIGURE A-10. OVAL SMOOTH PIPE LONG AXIS HORIZONTAL CRITICAL DEPTH



* NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

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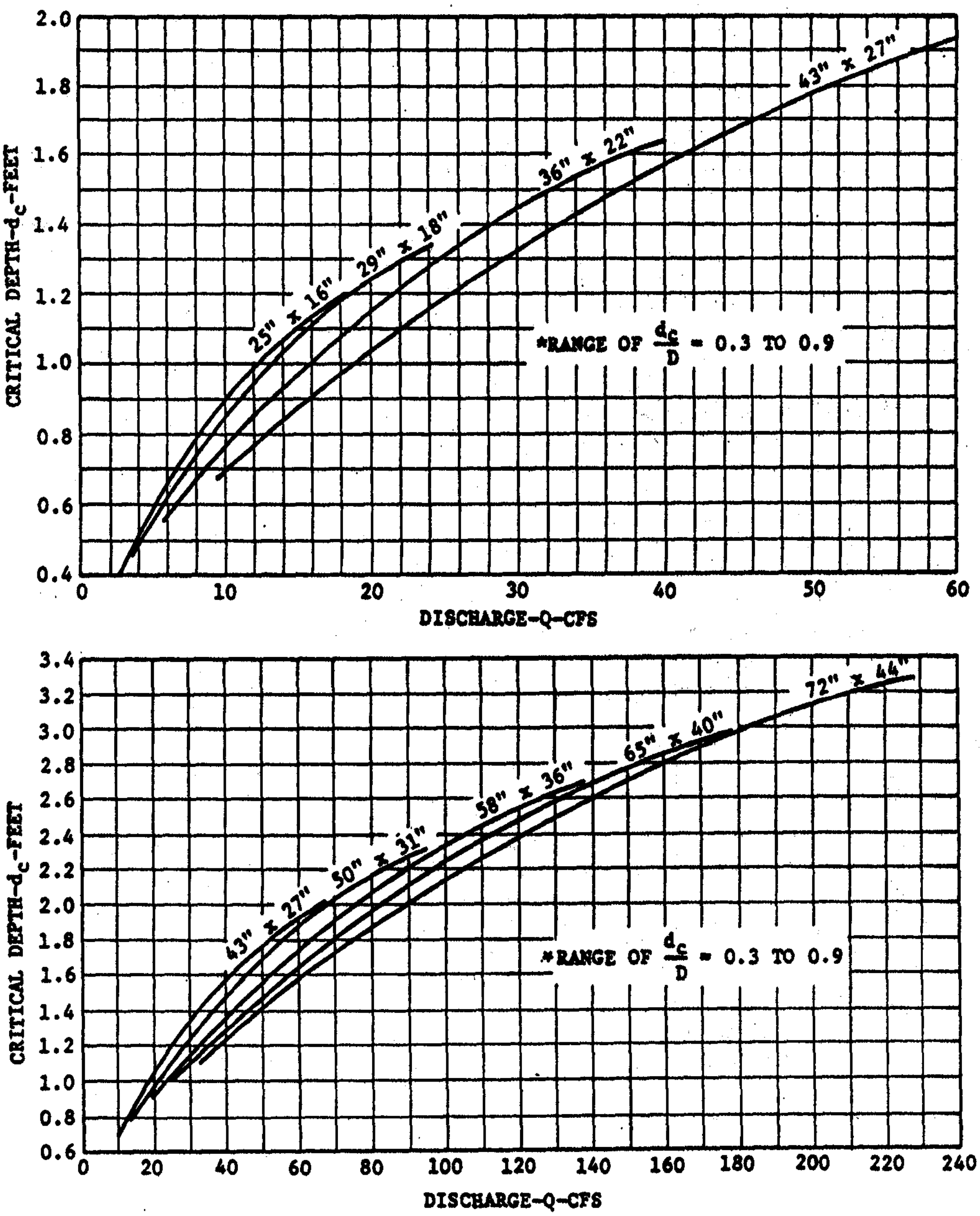
FIGURE A-11. OVAL SMOOTH PIPE LONG AXIS VERTICAL CRITICAL DEPTH



*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

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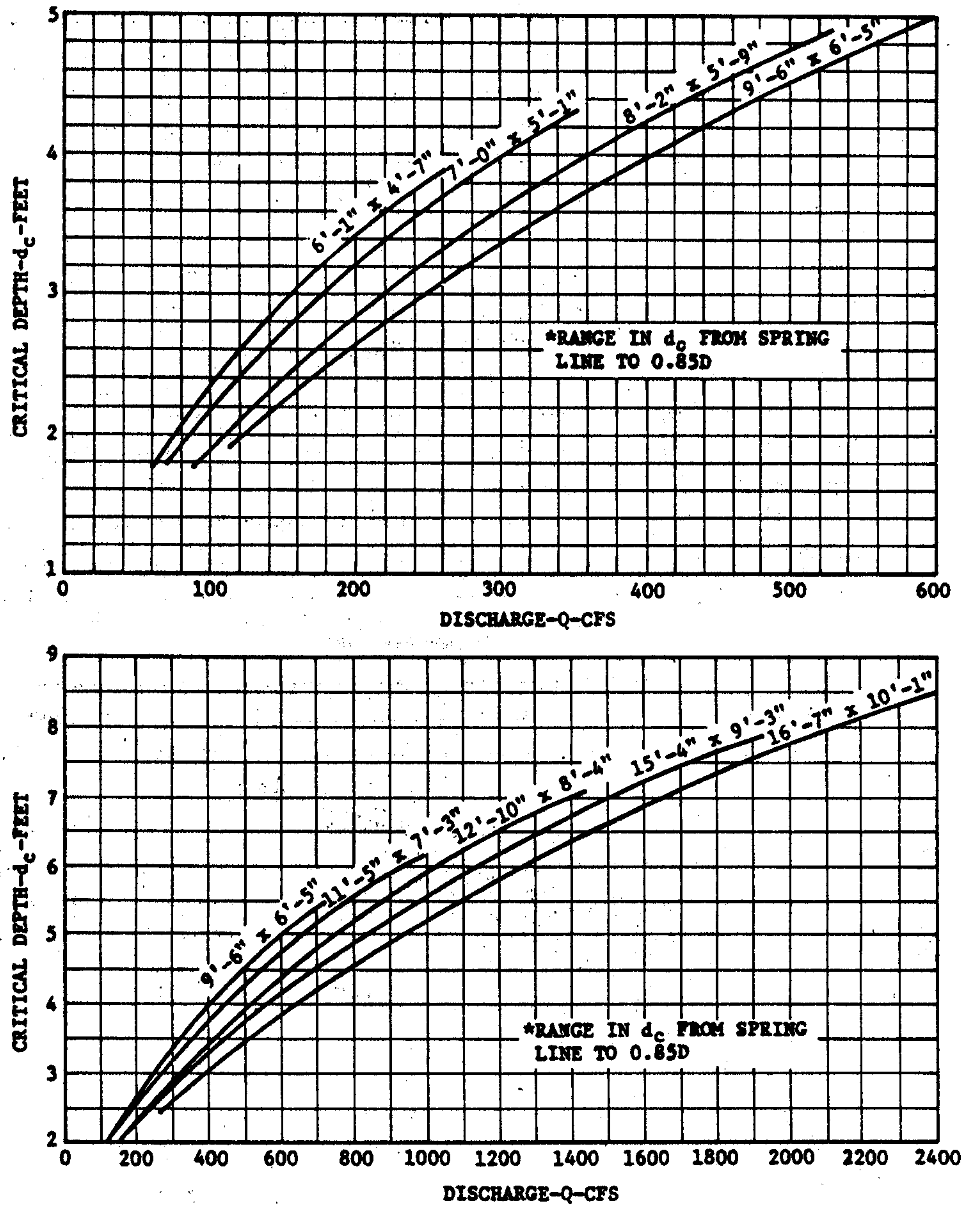
FIGURE A-12. CORRUGATED PIPE-ARCH CRITICAL DEPTH



*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

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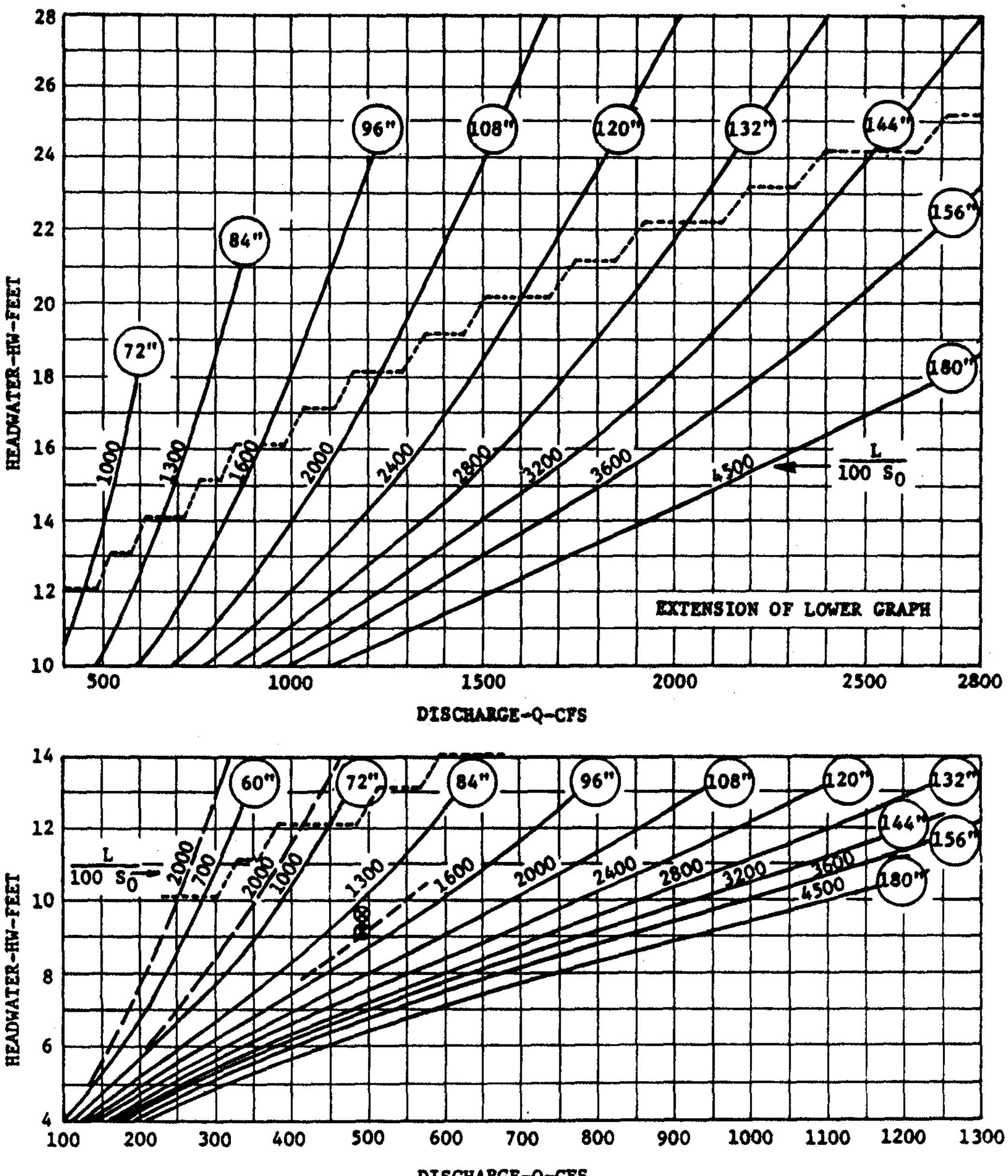
FIGURE A-13. STRUCTURAL PLATE PIPE-ARCH CRITICAL DEPTH



EXAMPLE:

⊗ **GIVEN:**
490 CFS; AHW = 9.6 FT.
L = 60 FT.; S_0 = 0.000

SELECT 90" ($\frac{L}{D}$ = 8)
HW = 9.2 FT.

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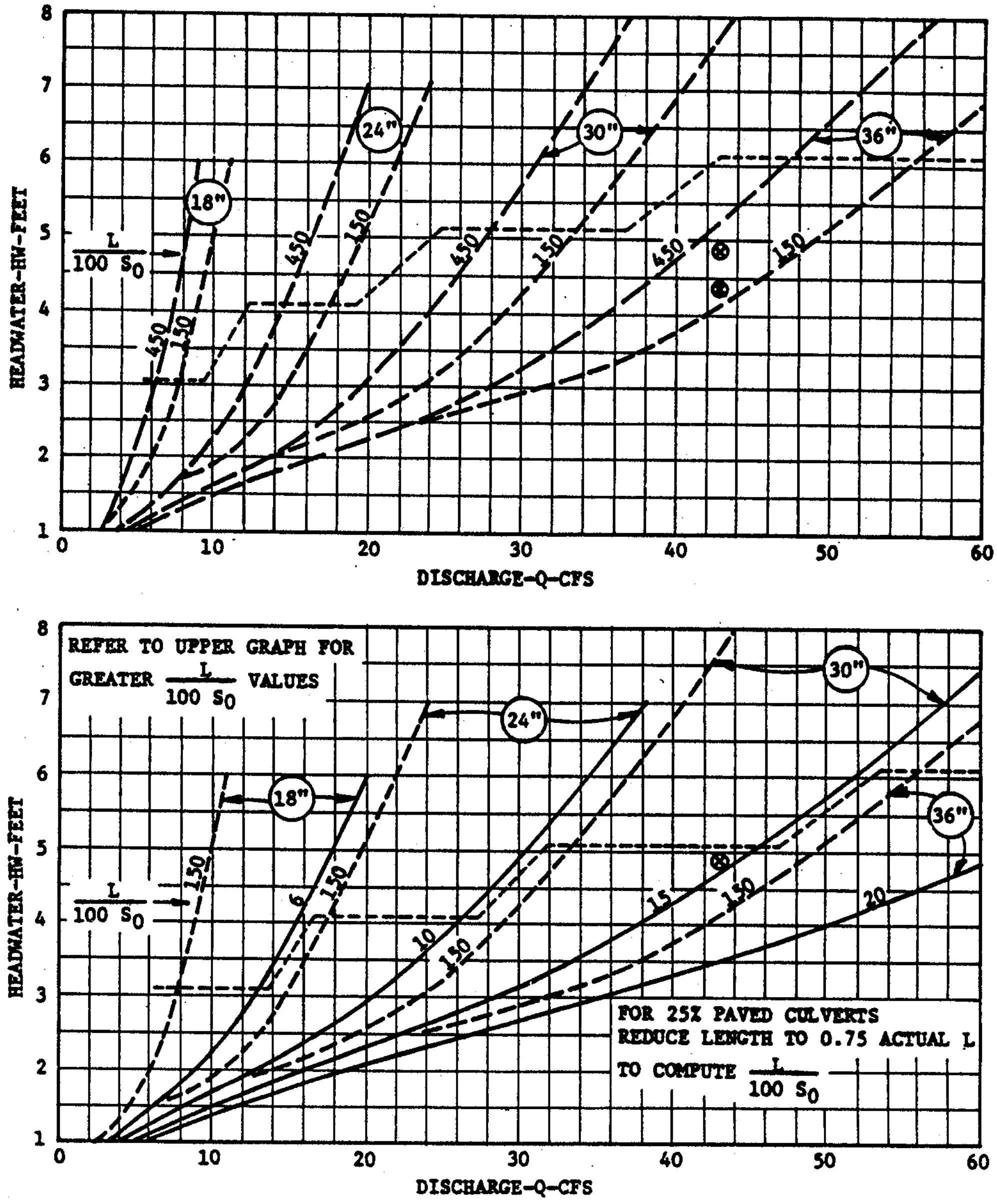
FIGURE A-14. CULVERT CAPACITY CIRCULAR SMOOTH PIPE GROOVE-EDGED ENTRANCE 60" TO 180"



EXAMPLE:
⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_0 = 0.003

⊕ SELECT 36" UNPAVED
HW = 4.4 FT.

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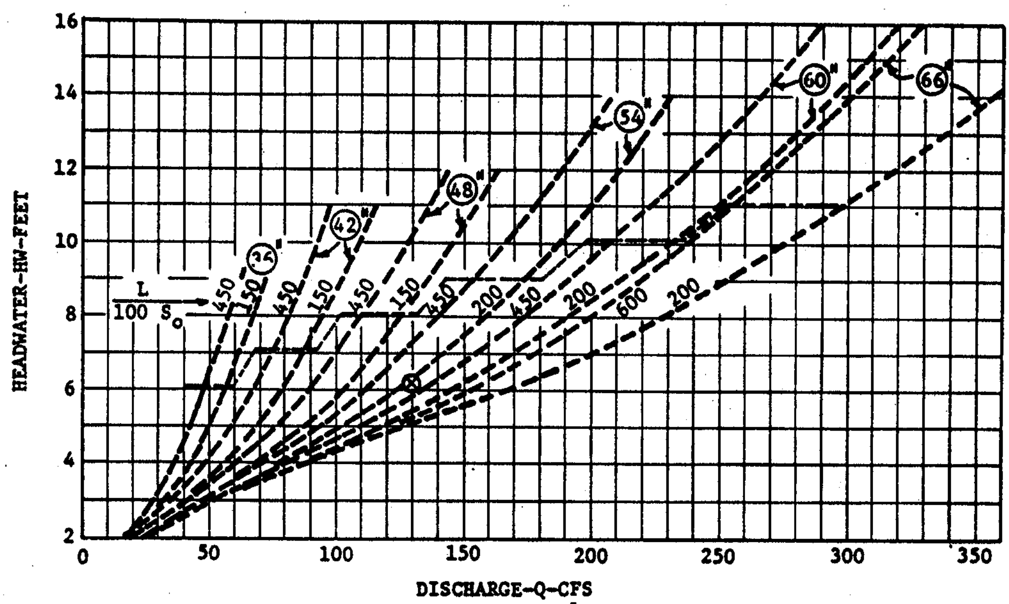
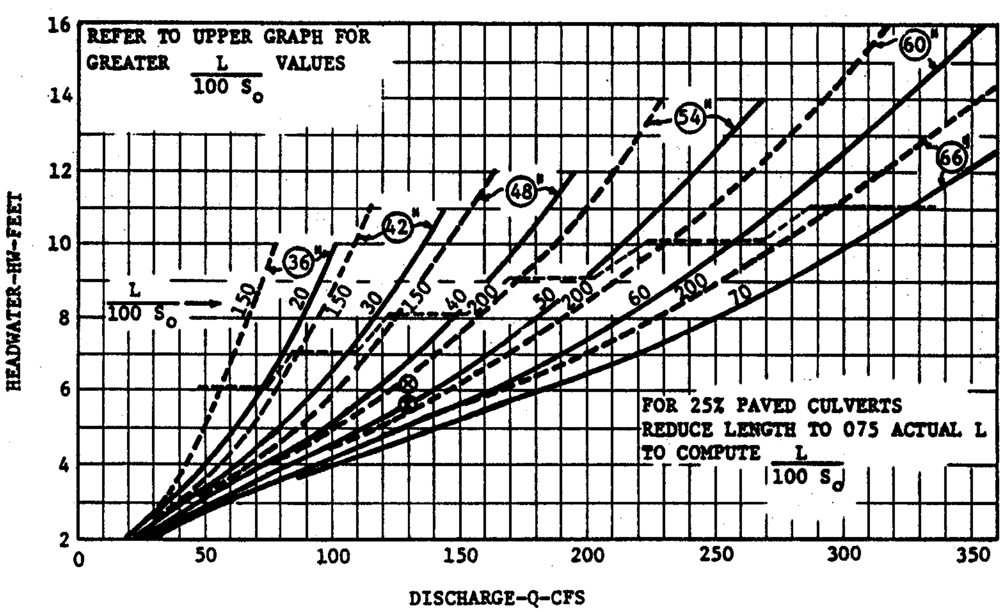
FIGURE A-15. CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED PIPE HEADWALL ENTRANCE 18" TO 36"





EXAMPLE

⊗ GIVEN:
130 CFS; AHW=6.2 FT.
L=120 FT; S_0=0.0025

● SELECT 54" UNPAVED
HW = 5.6 FT.

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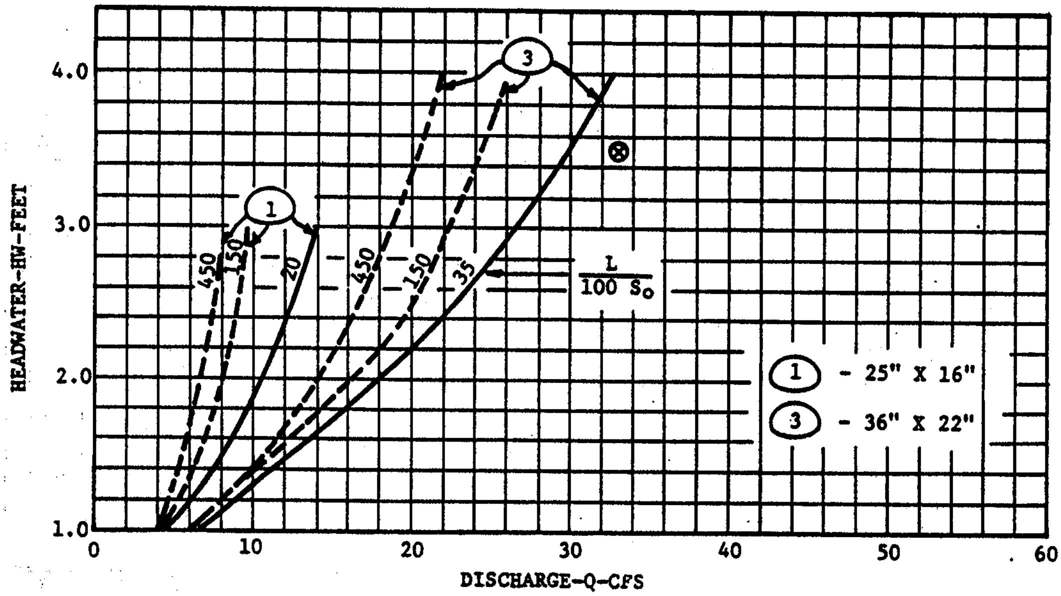
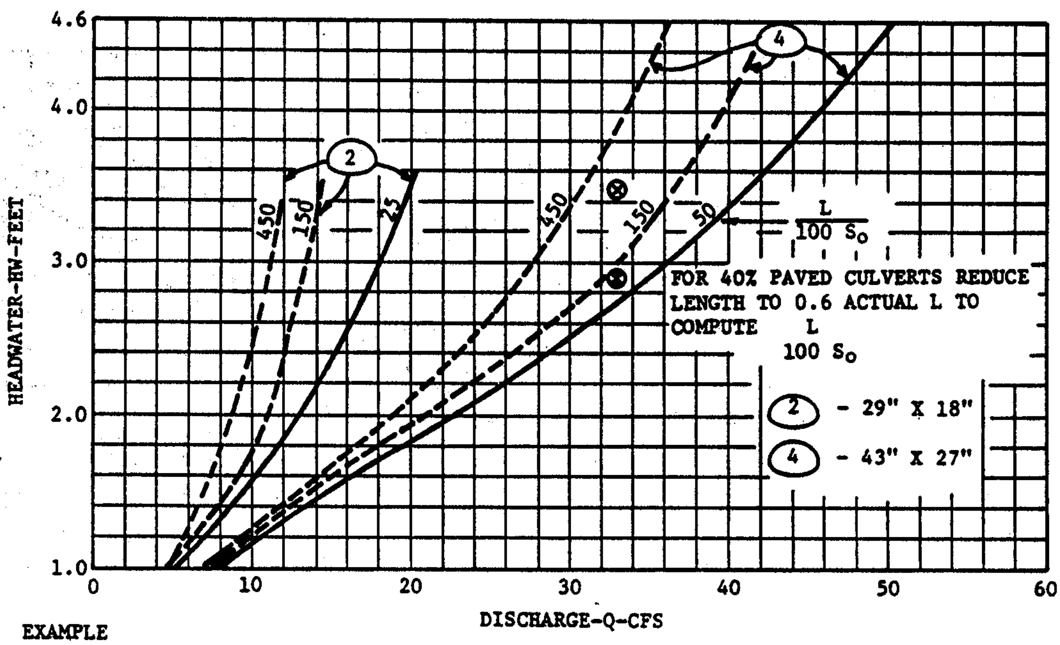
FIGURE A-16. CULVERT CAPACITY STANDARD CIRCULAR CORRUGATED PIPE HEADWALL ENTRANCE 36" TO 66" ○





EXAMPLE

⊗ GIVEN:
33 CFS; AHW=3.5 FT.
L = 60 FT.; S_o = 0.005

⊗ SELECT NO. 4, 43" X 27"
HW = 2.9 FT.
UNPAVED INVERT

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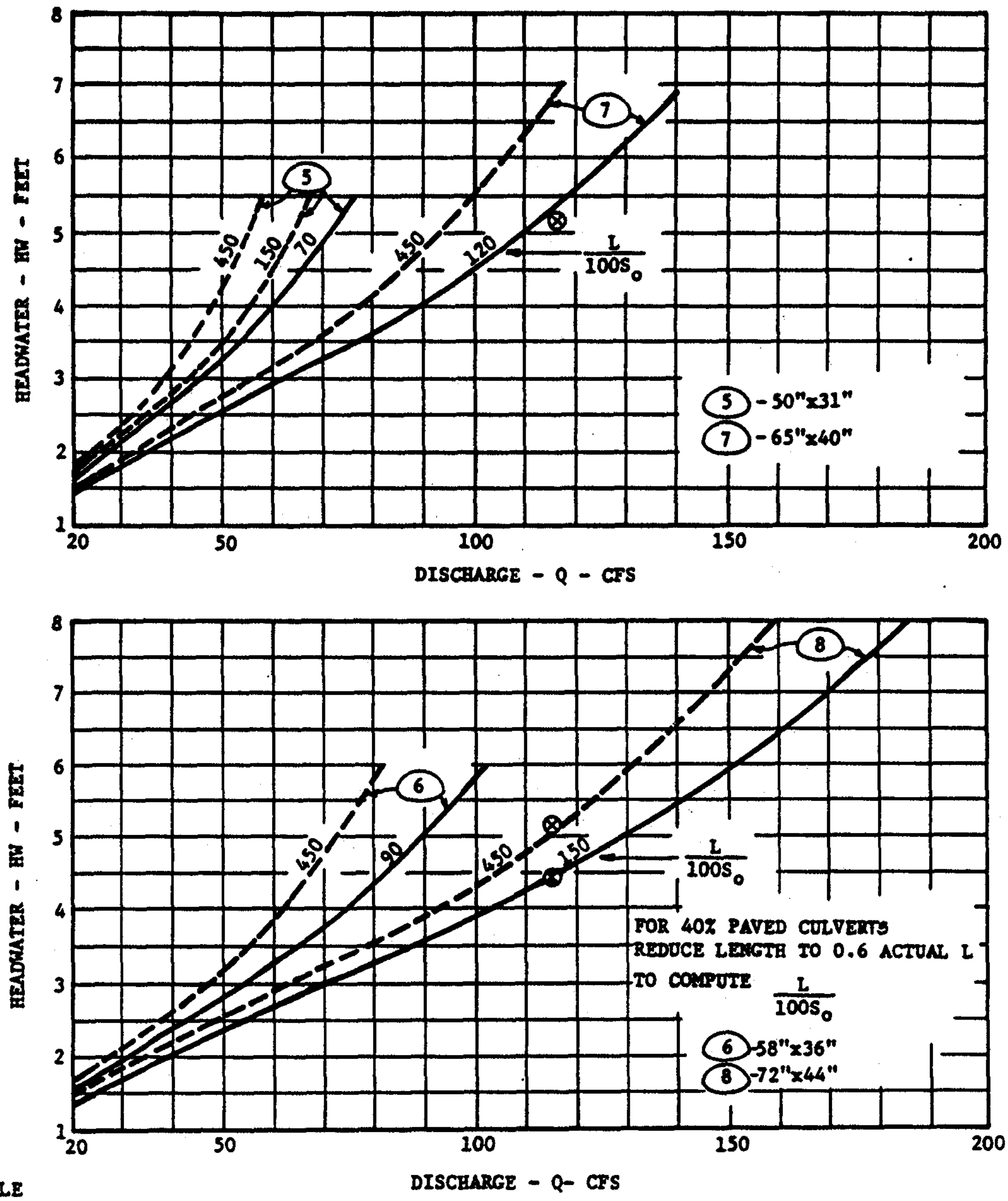
FIGURE A-17. CULVERT CAPACITY STANDARD CORRUGATED PIPE-ARCH PROJECTING ENTRANCE 25" BY 16" TO 43" BY 27"



EXAMPLE

⊗ GIVEN:
115 CFS; AHW = 5.2 FT
L = 110 FT, S_o = 0.0055

⊗ SELECT NO. 8, 72"x44"
HW = 4.5 FT.
PAVED INVERT

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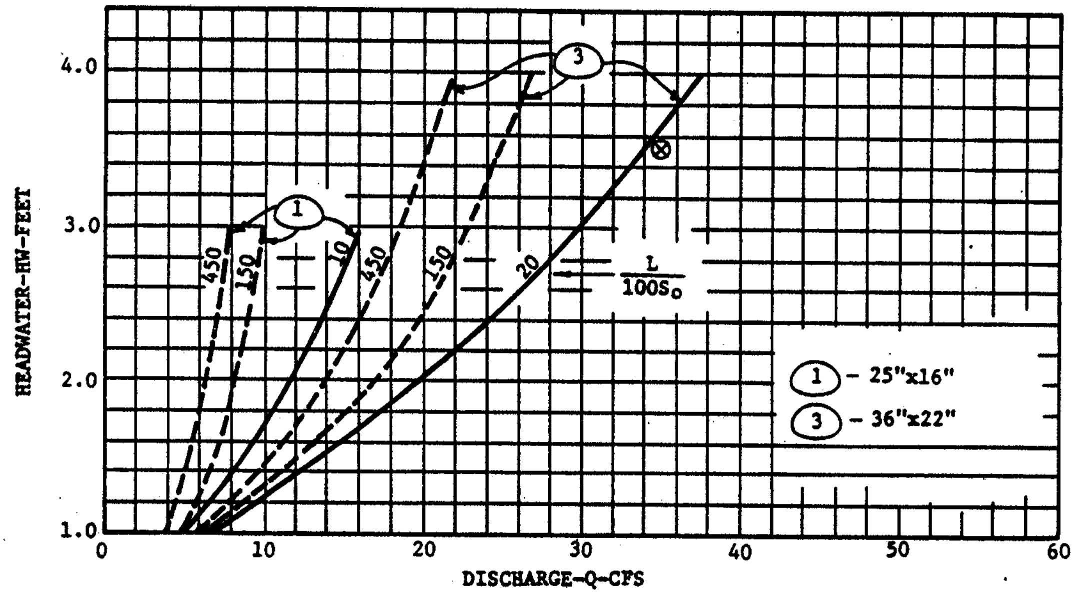
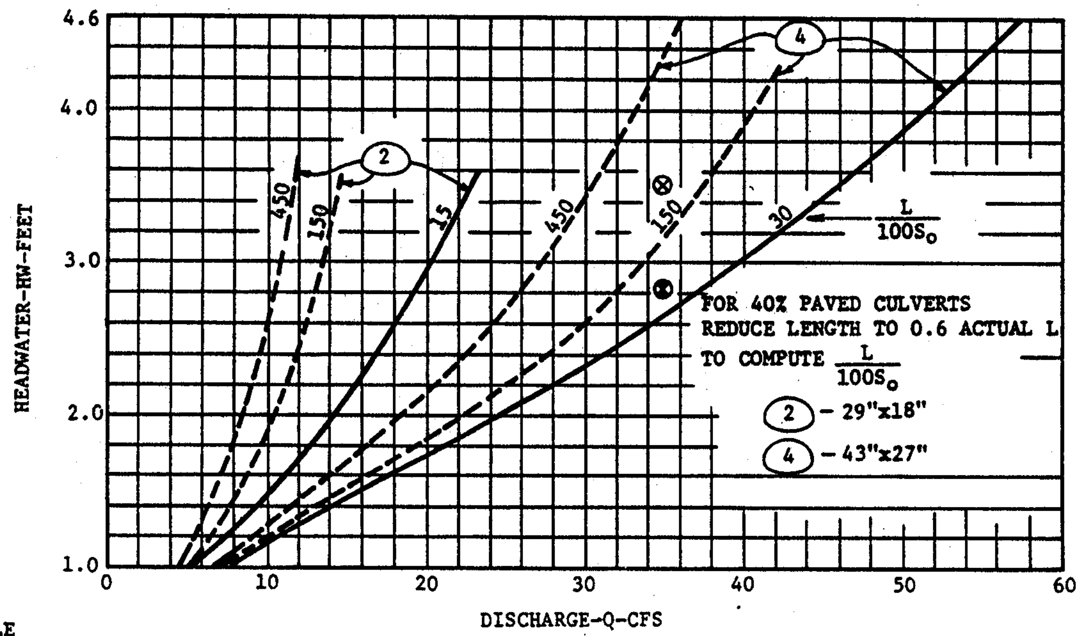
FIGURE A-18. CULVERT CAPACITY STANDARD CORRUGATED PIPE-ARCH PROJECTING ENTRANCE 50" BY 31" TO 72" BY 44"





EXAMPLE

⊗ GIVEN:
35 CFS, AHW = 35 FT.
L = 145 FT., S_o = 0.020

⊗ SELECT NO. 4, 43"x27"
HW = 2.8 FT.
UNPAVED INVERT

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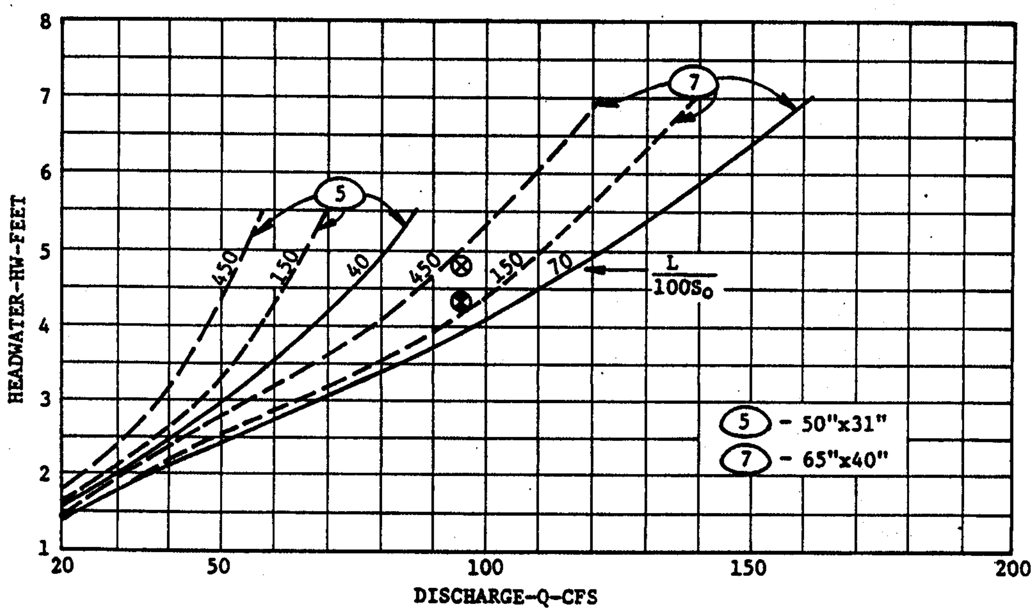
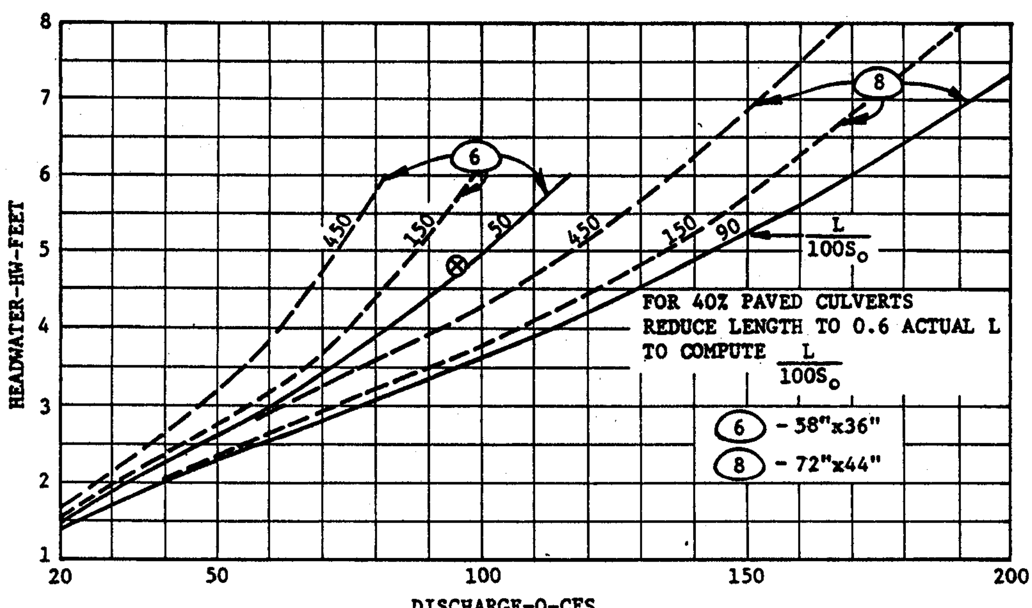
FIGURE A-19. CULVERT CAPACITY STANDARD CORRUGATED PIPE-ARCH HEADWALL ENTRANCE 25" BY 16" TO 43" BY 27"





EXAMPLE

⊗ GIVEN:
95 CFS, AHW = 4.8 FT.
L = 240 FT; S_o = 0.012

⊗ SELECT NO. 7, 65"x40"
HW = 4.3 FT.
UNPAVED INVERT

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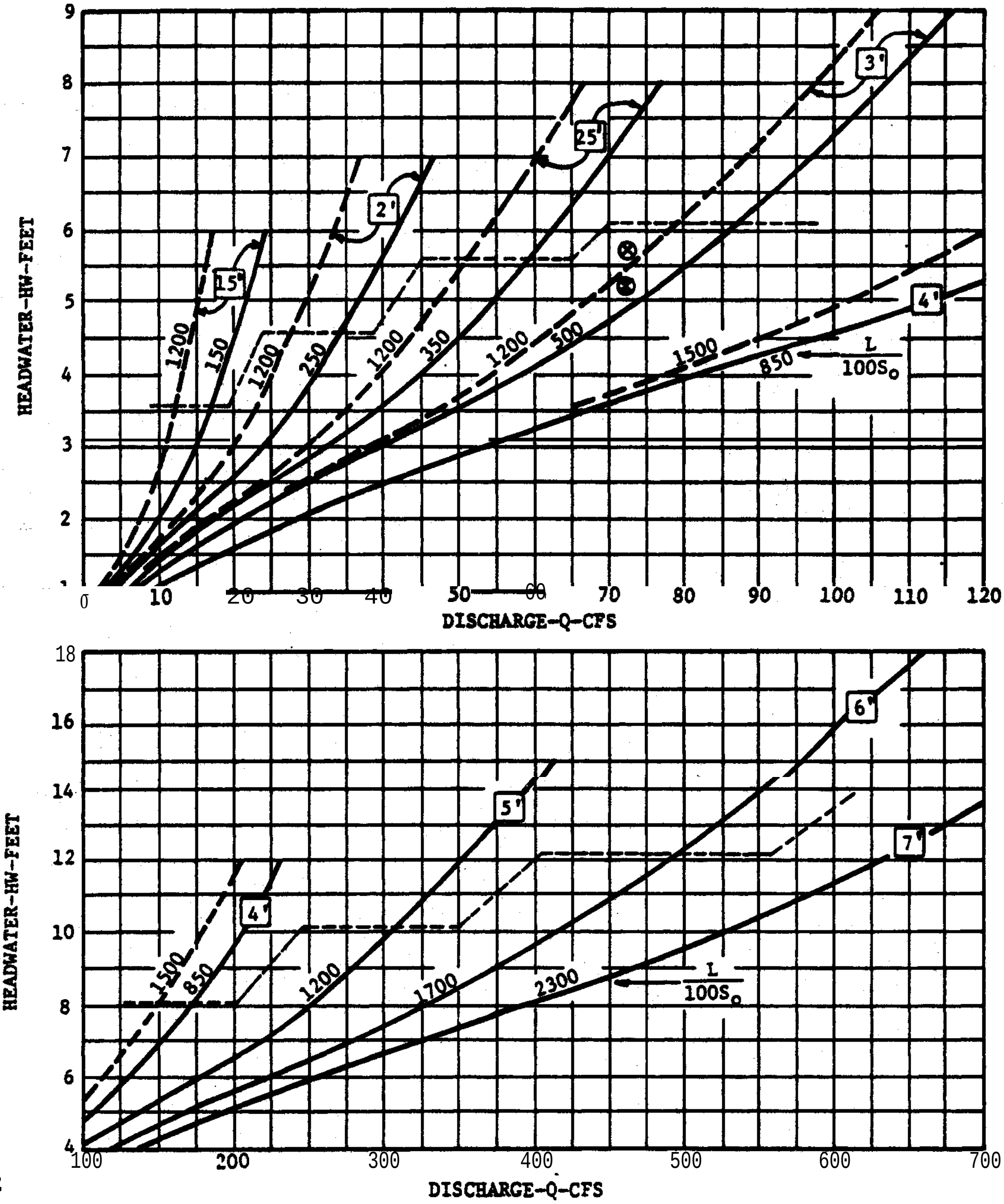
FIGURE A-20. CULVERT CAPACITY STANDARD CORRUGATED PIPE-ARCH HEADWALL ENTRANCE 50" BY 31" TO 72" BY 44"



EXAMPLE

⊗ GIVEN:
72 **CFS; AHW** = 5.7 FT.
L = **200 FT; S_o** = 0.002

⊗ **SELECT** 3' **x** 3'
HW = 5.2 **FT**

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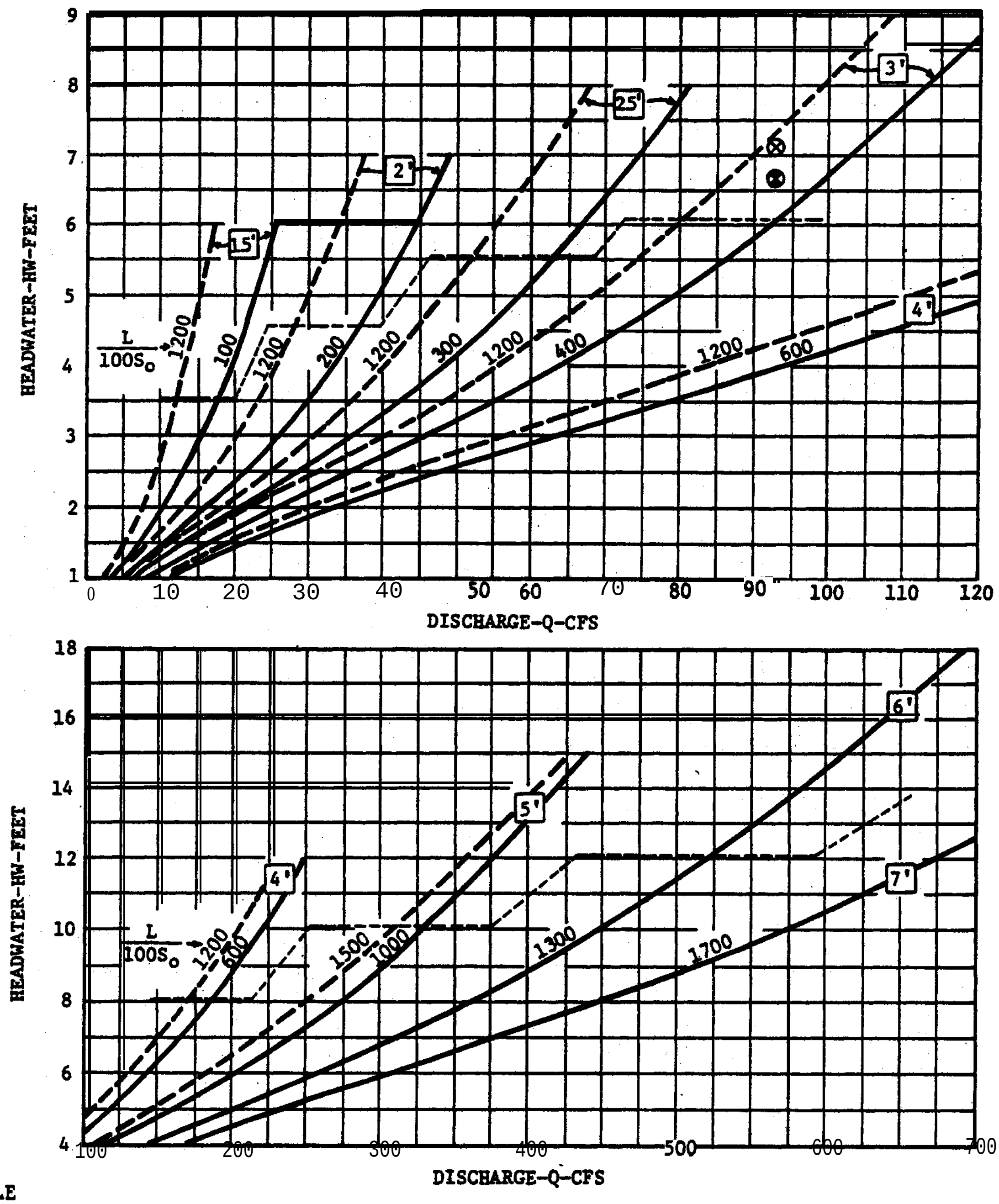
FIGURE A-21. CULVERT CAPACITY SQUARE CONCRETE BOX 90 DEGREES AND 15 DEGREES WINGWALL FLARE 1.5' BY 1.5' TO 7' BY 7'



EXAMPLE

⊗ **GIVEN:**
72 CFS; **AHW** = 5.7 FT.
L = 200 FT; S_o = 0.002

⊗ SELECT **3' x 3'**
?iW = S.2 FT

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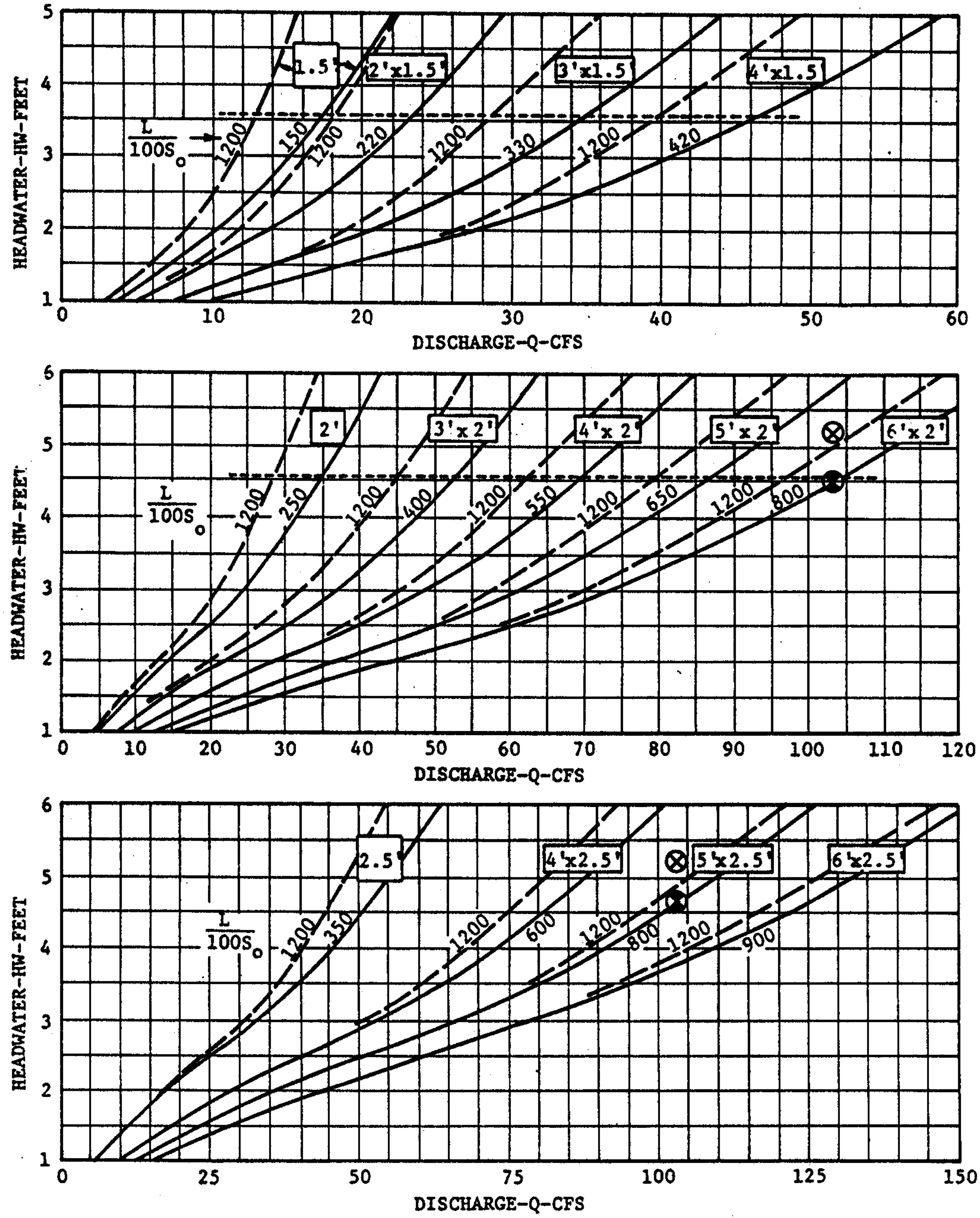
FIGURE A-22. CULVERT CAPACITY SQUARE CONCRETE BOX 30 DEGREES AND 75 DEGREES WINGWALL FLARE 1.5' BY 1.5' TO 7' BY 7'



EXAMPLE:

⊗ GIVEN:
103 CFS; AHW = 5.2 FT
L = 150 FT; S_o = 0.015

SELECT: 6' x 2', HW = 4.5 FT.
OR 5' x 2.5'; HW = 4.6 FT.

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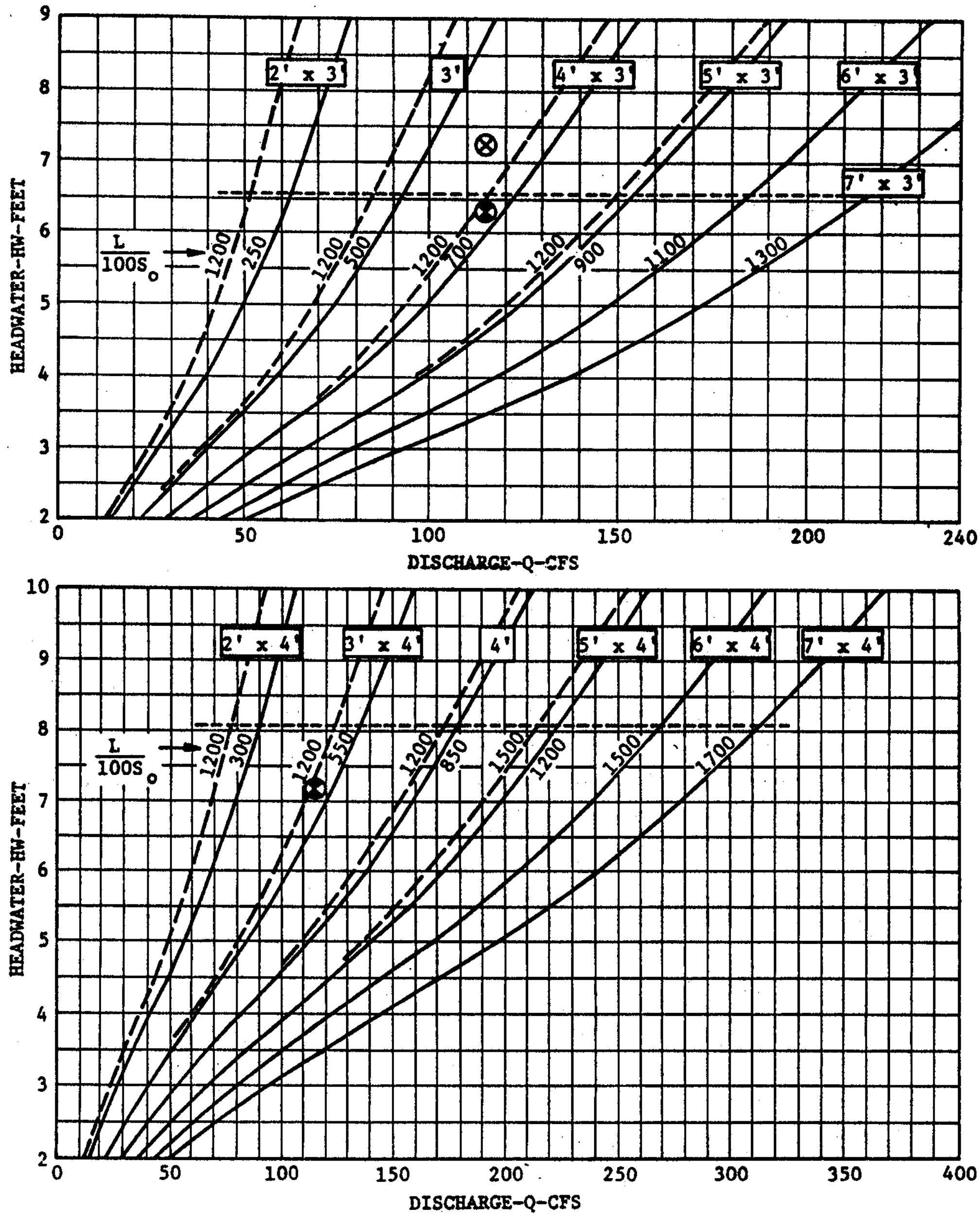
FIGURE A-23. CULVERT CAPACITY RECTANGULAR CONCRETE BOX 90 DEGREES AND 15 DEGREES WINGWELL FLARE 1.5', 2.0' AND 2.5' HEIGHTS



EXAMPLE:

⊗ GIVEN:
115 CFS, AHW = 7.2 FT
L = 300 FT, S_o = 0.003

SELECT 4' x 3', HW = 6.3 FT
OR 3' x 4', HW = 7.2 FT

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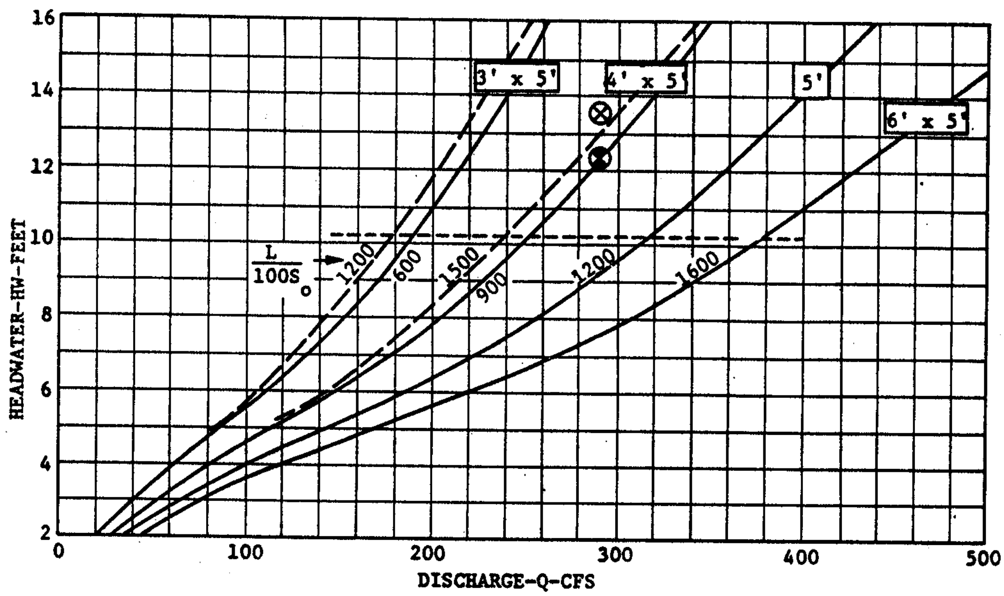
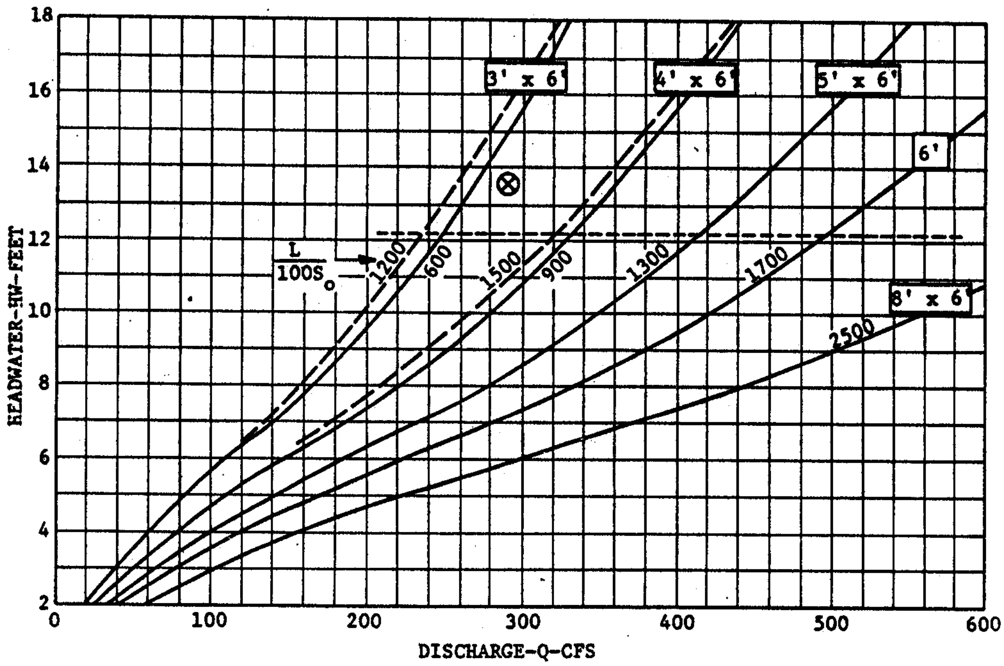
FIGURE A-24. CULVERT CAPACITY RECTANGULAR CONCRETE BOX 90 DEGREES AND 15 DEGREES WINGWALL FLARE 3' AND 4' HEIGHTS &





EXAMPLE:

⊗ **GIVEN:**
290 CFS; AHW = 13.5 FT.
L = 250 FT; S_o = 0.010

⊗ **SELECT 4' x 5'**
HW = 12.3 FT.

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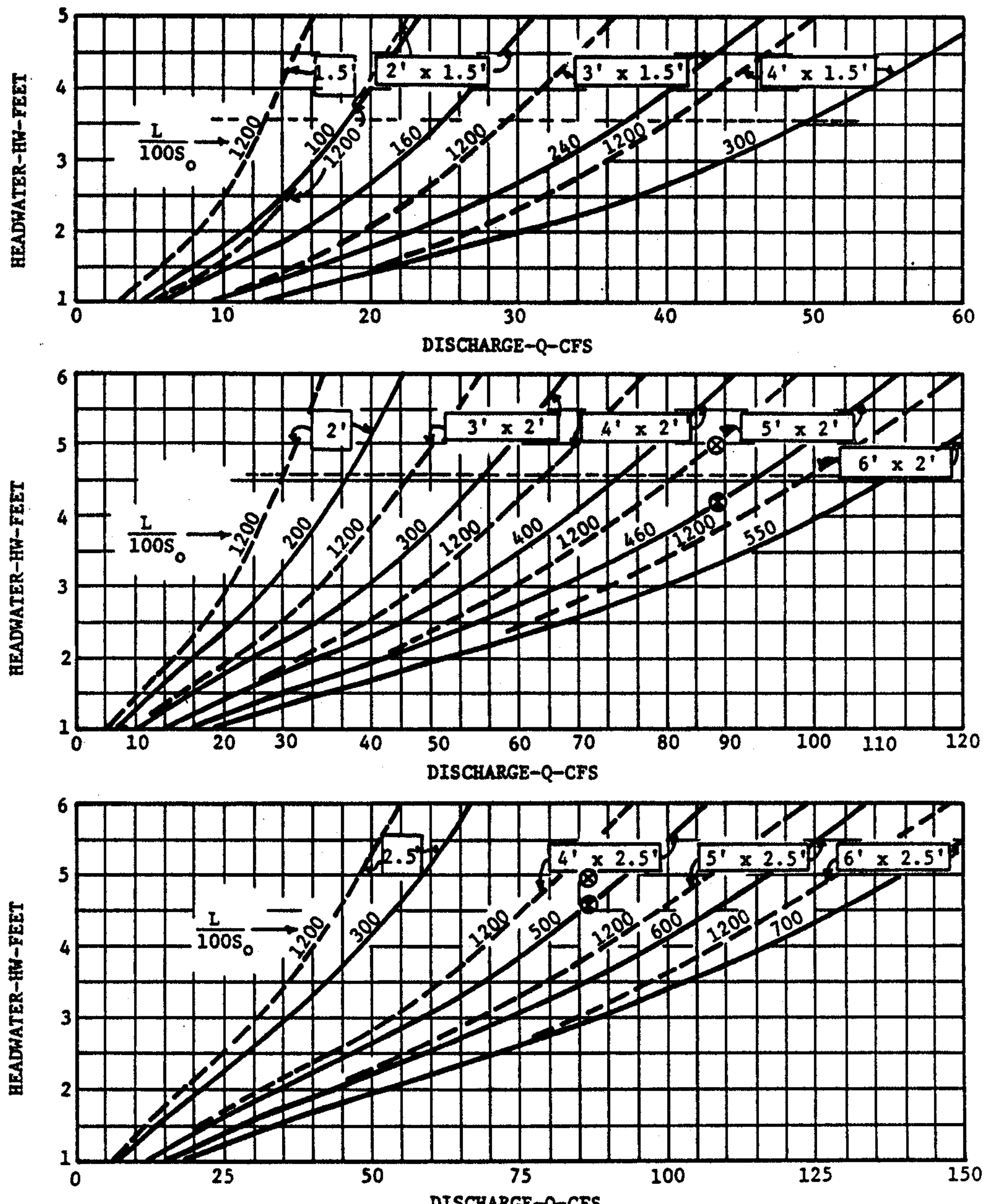
FIGURE A-25. CULVERT CAPACITY RECTANGULAR CONCRETE BOX
90° AND 15° WINGWALL FLARE 5' AND 6' HEIGHTS ▯ & ▭



EXAMPLE

⊗ **GIVEN:**
87 CFS, AHW = 5.0 FT.
L = 120 FT., S_o = 0.003

⊗ **SELECT 5' x 2', HW = 4.2 FT.**
OR 4' x 2.5', HW = 4.6 FT.

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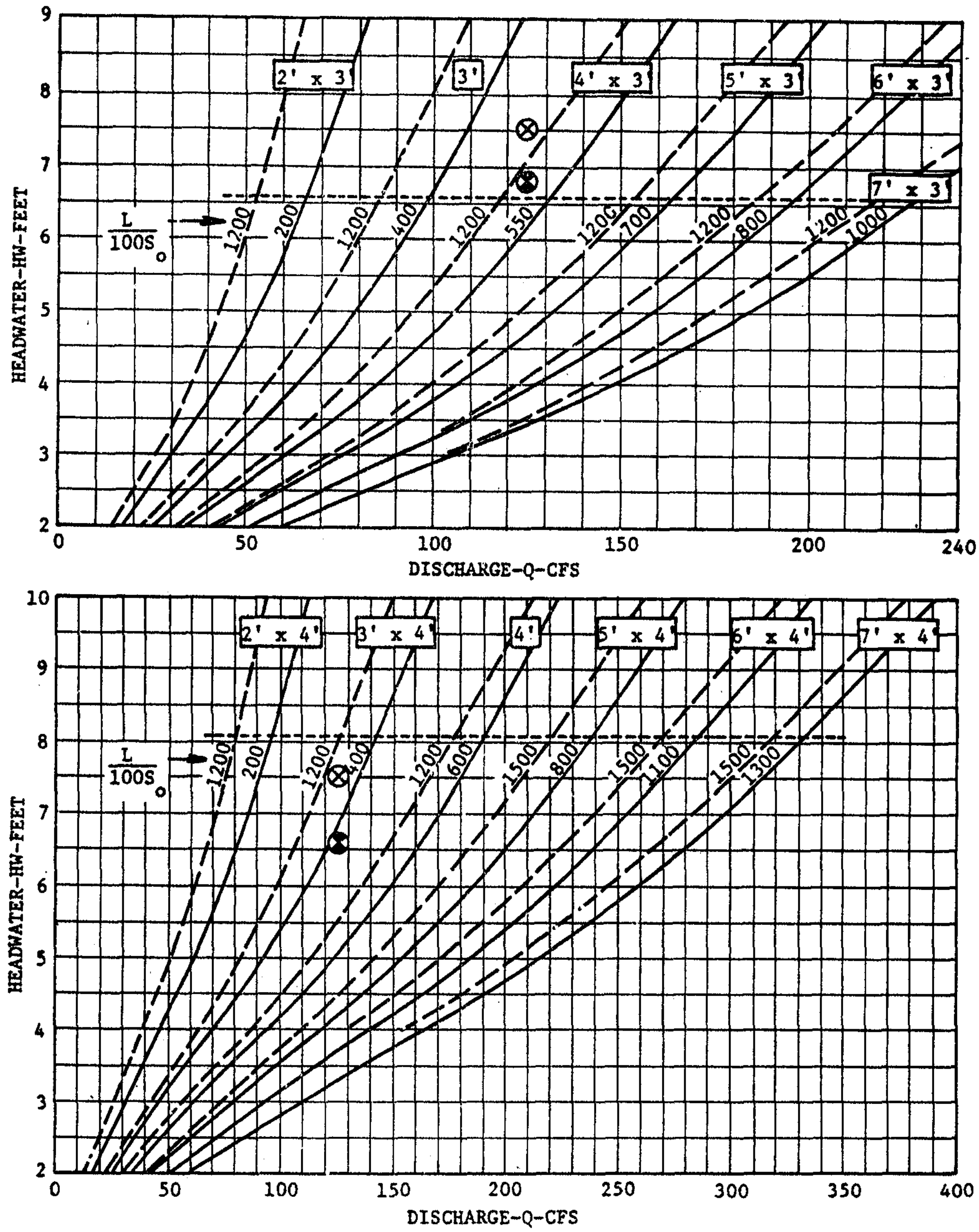
FIGURE A-26. CULVERT CAPACITY RECTANGULAR CONCRETE BOX 30° AND 75° WINGWALL FLARE 1.5', 2.0' AND 2.5' HEIGHTS



EXAMPLE:

⊗ GIVEN:
125 CFS, AHW = 7.5 FT.
L = 400 FT.; S_o = 0.004

SELECT 4' x 3', HW = 6.8 FT.
OR 3.5 x 4'; HW = 6.6 FT.

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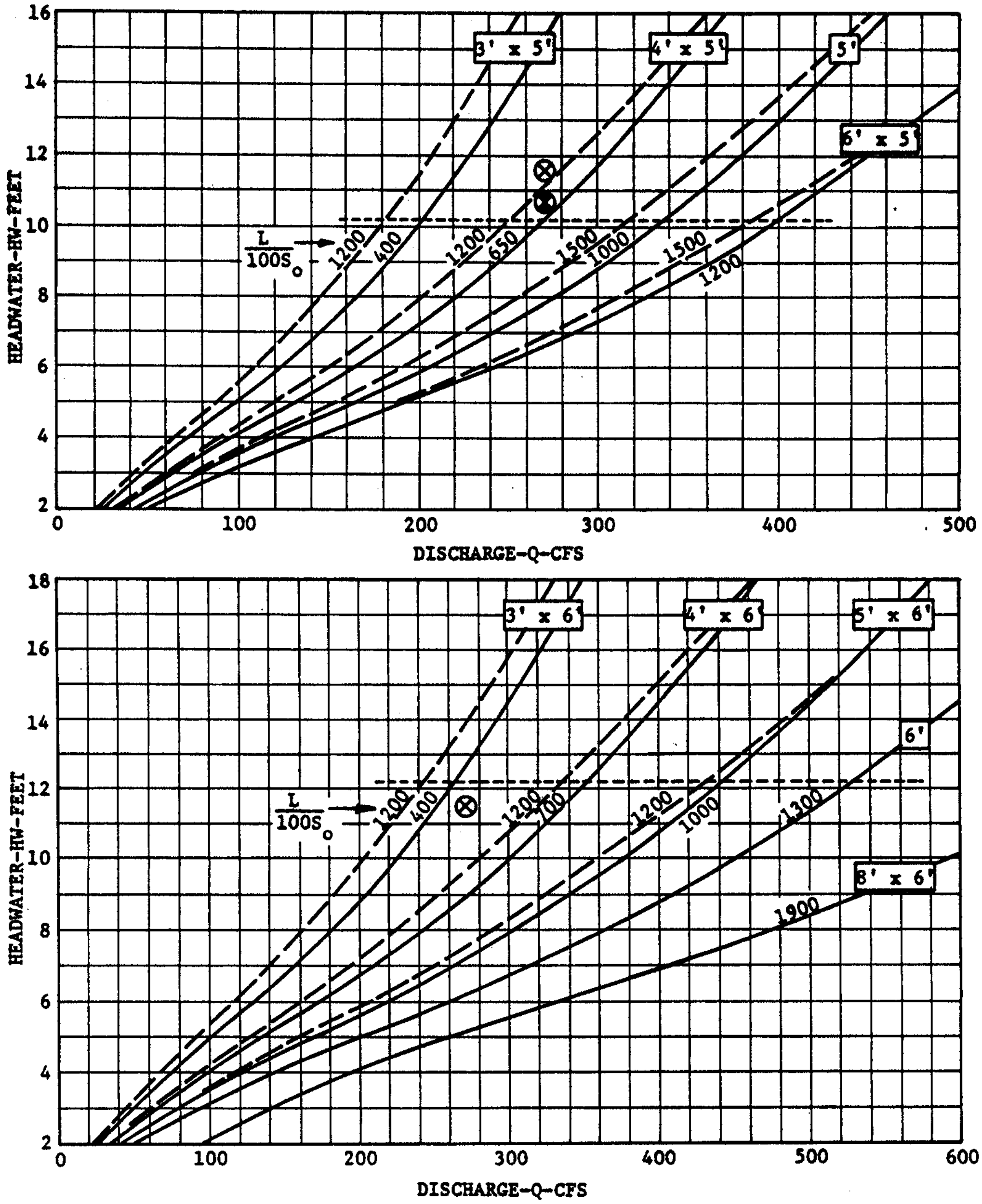
FIGURE A-27. CULVERT CAPACITY RECTANGULAR CONCRETE BOX 30° AND 75° WINGWALL FLARE 3' AND 4' HEIGHTS ▯ & ▭



EXAMPLE:

⊗ **GIVEN:**
270 CFS, AHW = 11.5 FT.
L = 450 FT., S_o = 0.0005

⊗ **SELECT 4' x 5'**
HW = 10.7 FT.

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FIGURE A-28. CULVERT CAPACITY RECTANGULAR CONCRETE BOX 30° AND 75° WINGWALL FLARE 5' AND 6' HEIGHTS

APPENDIX B

REFERENCES

Government Publications.

Department of the Army.

EM 1110-3-130 Geometrics for Roads, Streets, Walks and Open Storage Areas.

EM 1110-3-131 Flexible Pavements for Roads, Streets, Walks and Open Storage Areas.

EM 1110-3-132 Rigid Pavements for Roads, Streets, Walks and Open Storage Areas.

EM 1110-3-138 Pavement Criteria for Seasonal Frost Conditions.

EM 1110-3-141 Airfield Flexible Pavement.

EM 1110-3-142 Airfield Rigid Pavement.

EM 1110-3-152 Railroads.

Federal Highway Administration, Bureau of Public Roads, Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402

Hydraulic Design Series No. 3 (1979) "Design Charts for Open Channel Flow."

Hydraulic Engineering Circular No. 5 (1965) "Hydraulic Charts for the Selection of Highway Culverts."

Nongovernment Publications.

National Clay Pipe Institute NCPI

Clay Pipe Engineering Manual

Beichley, G. L. Research Report No. 24, U.S. Department of the Interior, Bureau of Reclamation, 1971, Hydraulic Design of Stilling Basin for Pipe or Channel Outlets.

Casagrande, A. and Rutledge, P. C. — Abstract of Soil Mechanics Lecture Notes of Dr. Arthur Casagrande, Harvard University, with additions by Professor P. C. Rutledge, Northwestern University, March 1946, as abstract appeared in TM 5-820-2 with changes in August 1965.

Taylor, D. W. — Fundamentals of Soil Mechanics, John Wiley & Sons, London, 1948.

Terzaghi, K. — "Der Grundbruch an Stauwerken und seine Verhutung" (The Failure of Dams by Piping and its Prevention), Die Wasserkraft, Vol. 17, pp. 445-449, 1922. Reprinted in From Theory to Practice in Soil Mechanics, New York, John Wiley and Sons, pp. 114-118, 1960.

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