CHAPTER 4

HYDRAULICS AND DRAINAGE STRUCTURES

4-1 HYDRAULICS

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

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

4-1.3 Problem Areas

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

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

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

4-1.4 Design

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

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

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

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

4-1.6 **Drainage Law**

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

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

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

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

4-2 AIRFIELDS

4-2.1 Drainage Pipe

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

4-2.1.2 **Selection of type of pipe**

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

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

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

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

Table 4-1. Roughness Coefficients for Various Pipes

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

4-2.1.5 **Minimum and maximum cover**

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

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

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

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

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

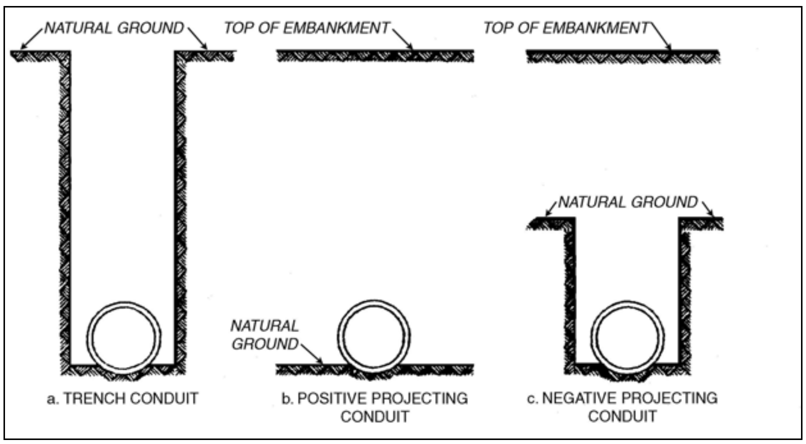
4-2.1.5.6 The tables in Appendix D identify the recommended minimum and maximum cover requirements for storm drains and culverts. These cover depths are valid for the specified loads and conditions, including average bedding and backfill. Deviations from these loads and conditions significantly affect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

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

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

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

Figure 4-1. Three Main Classes of Conduits



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

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

Figure 4-2. Free-Body Conduit Diagrams

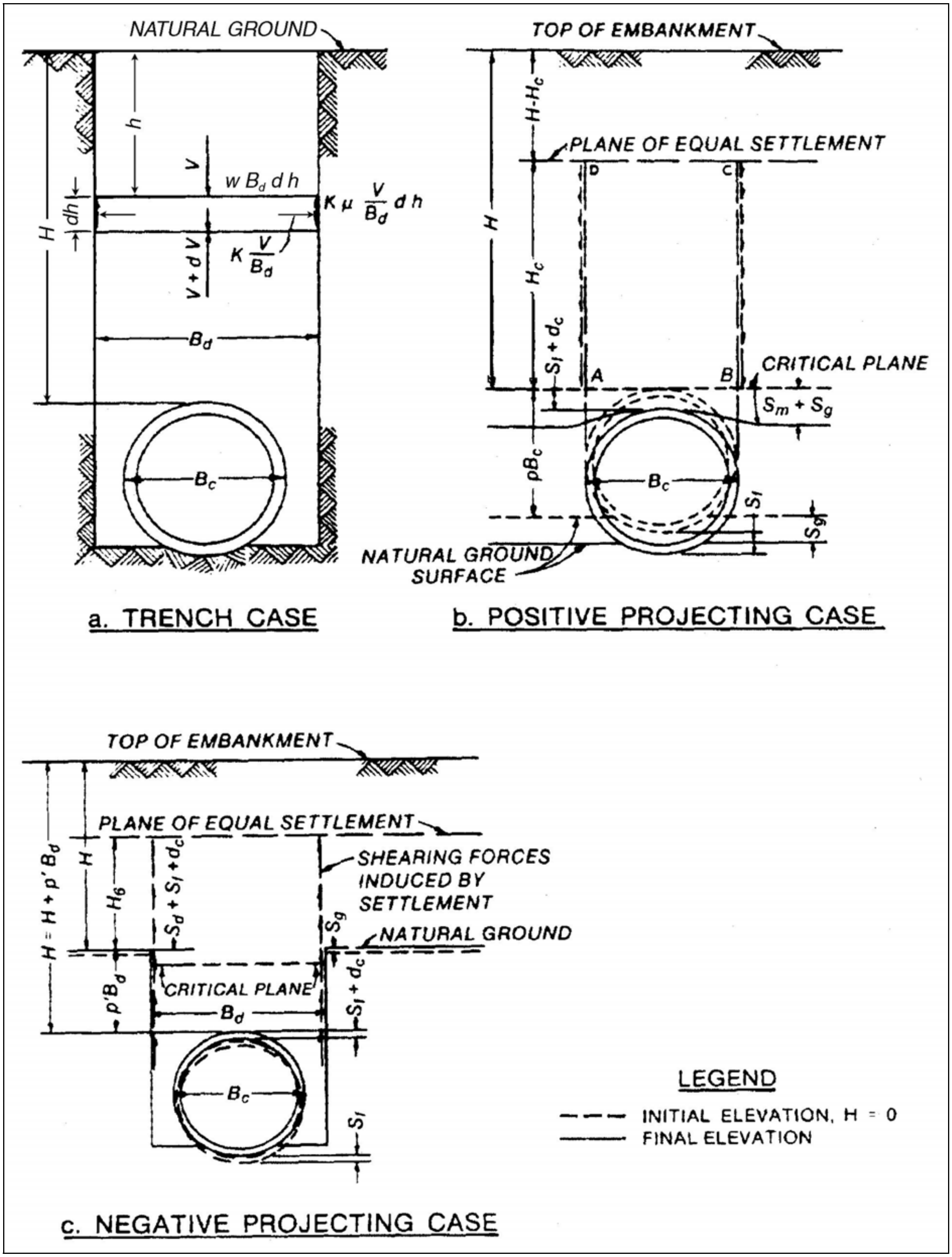


Figure 4-3. Trench Beddings for Circular Pipe

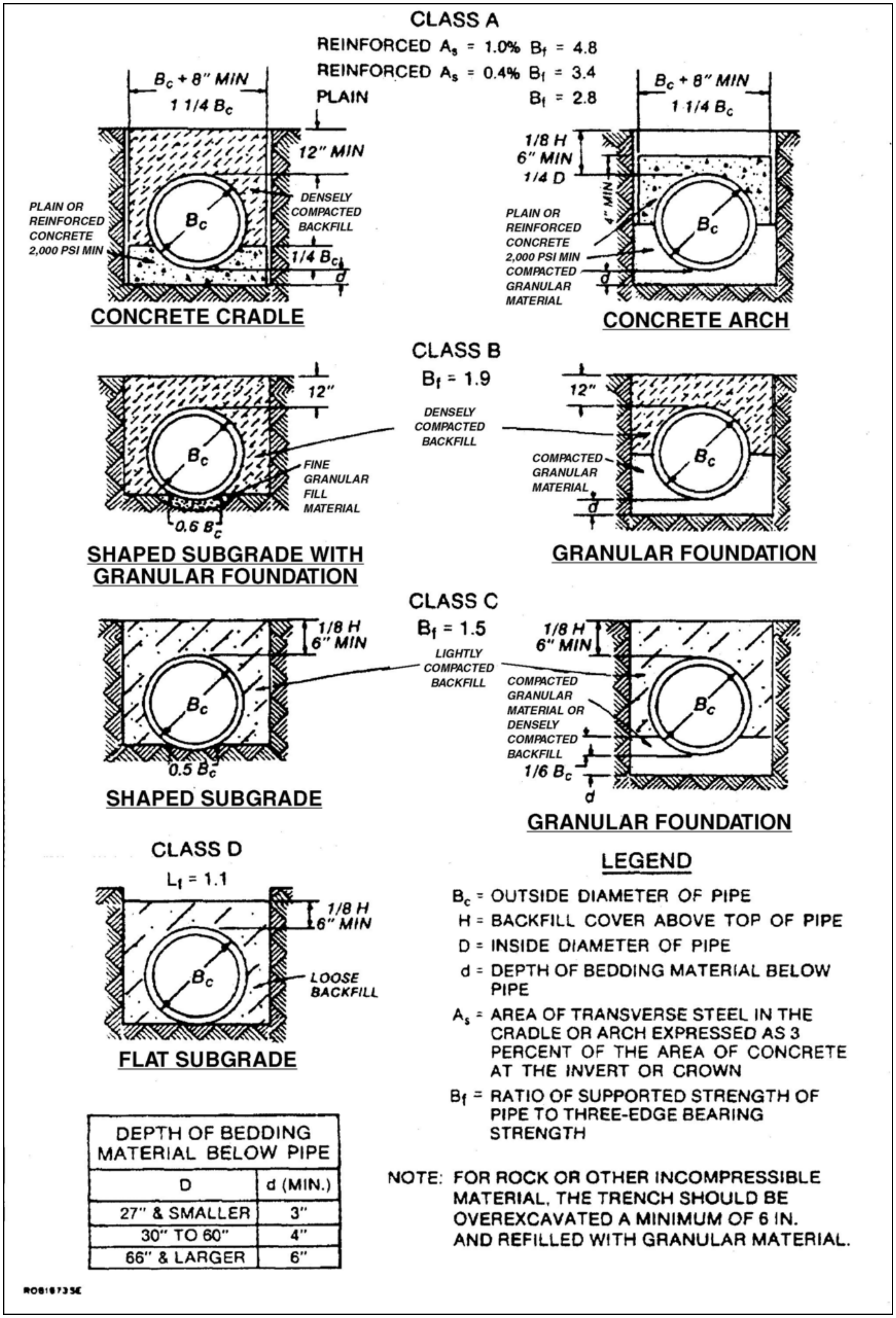


Figure 4-4. Beddings for Positive Projecting Conduits

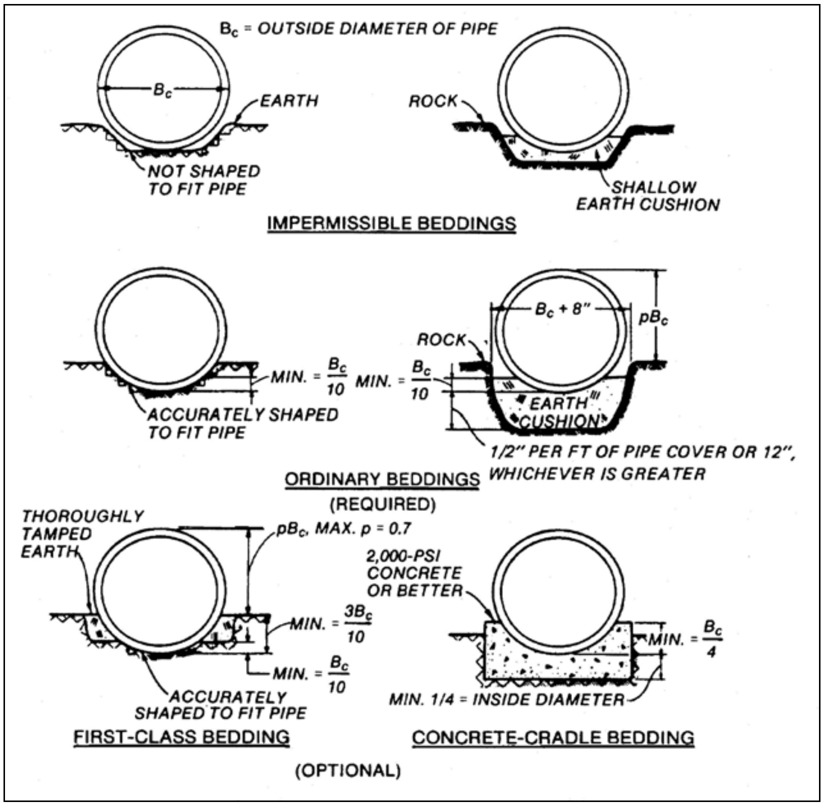
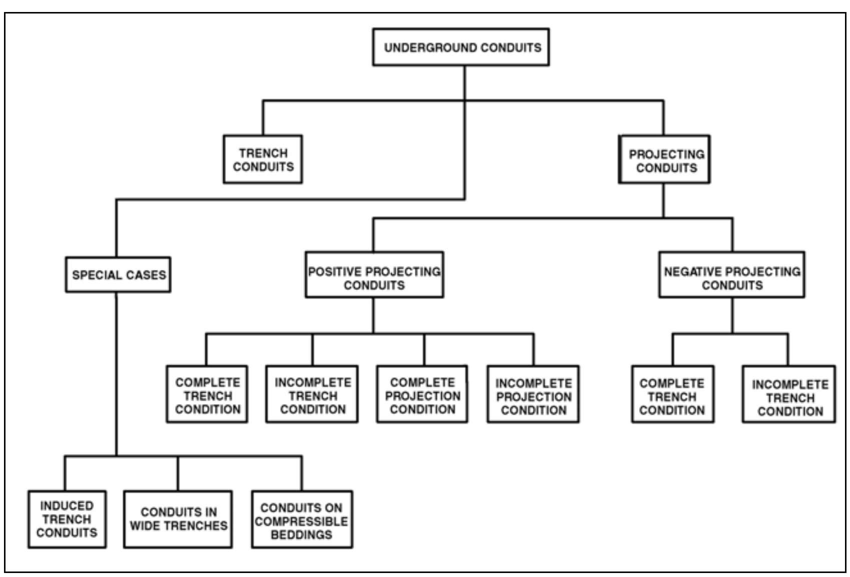


Figure 4-5. Installation Conditions Which Influence Loads on Underground Conduits



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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

4-2.2 **Inlets and Box Drains**

4-2.2.1 **General**

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

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

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

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

4-2.2.3 **Design features**

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

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

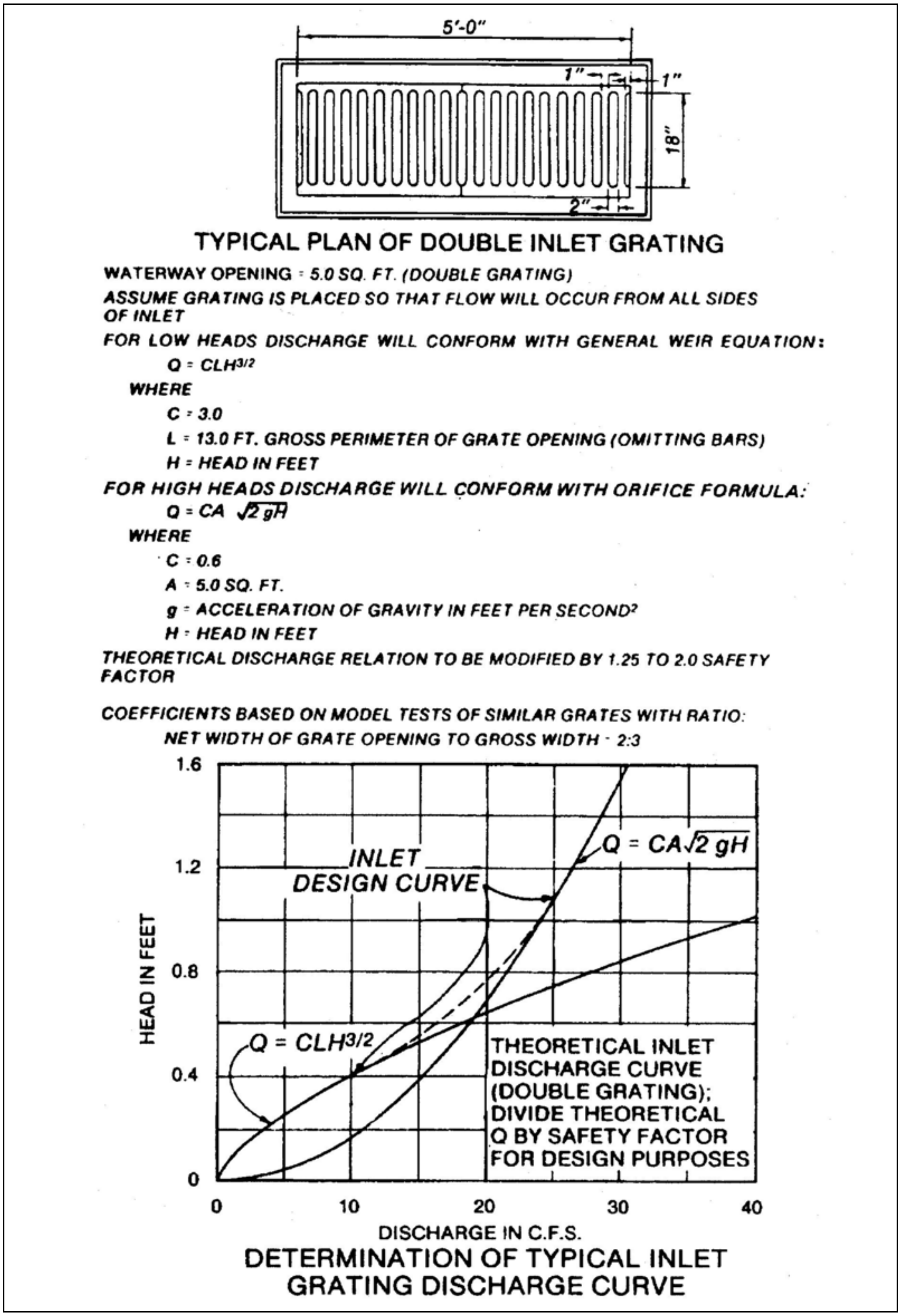
4-2.2.3.3 Drainage structures located in the usable areas on airports should be designed so that the grating does not extend above the ground level. The tops of such structures should be 0.2 of a foot below the ground line (finished grade) to allow for possible settlement around the structure, to permit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

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

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

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

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



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

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

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

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

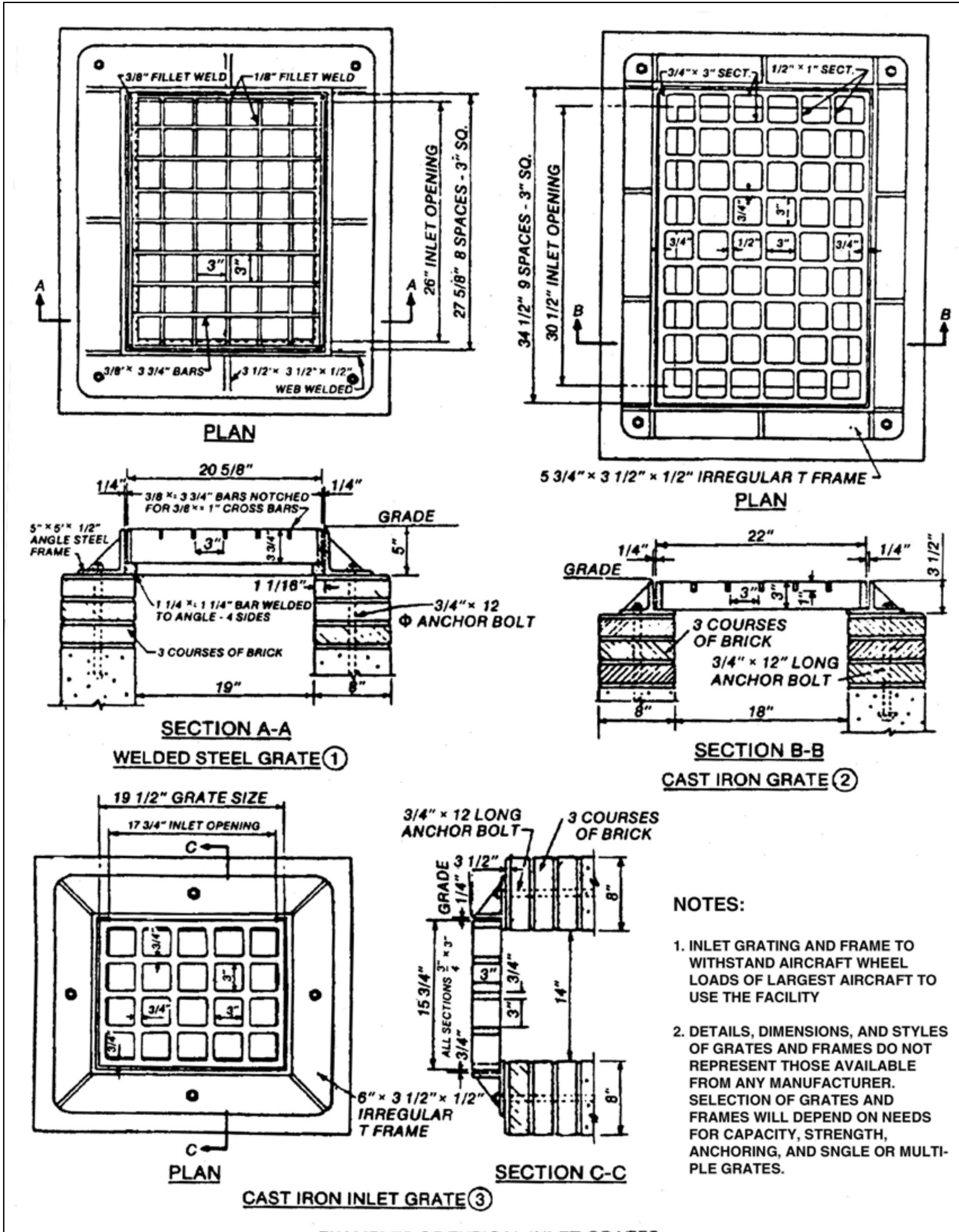
4-2.2.3.10 For rigid concrete pavements, grates may be protected by expansion joints around the inlet frames. Construction joints, which match or are equal to the normal spacing of joints, may be required around the drainage structure. The slab around the drainage structure should include steel reinforcements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

4-2.2.4 **Box drains**

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

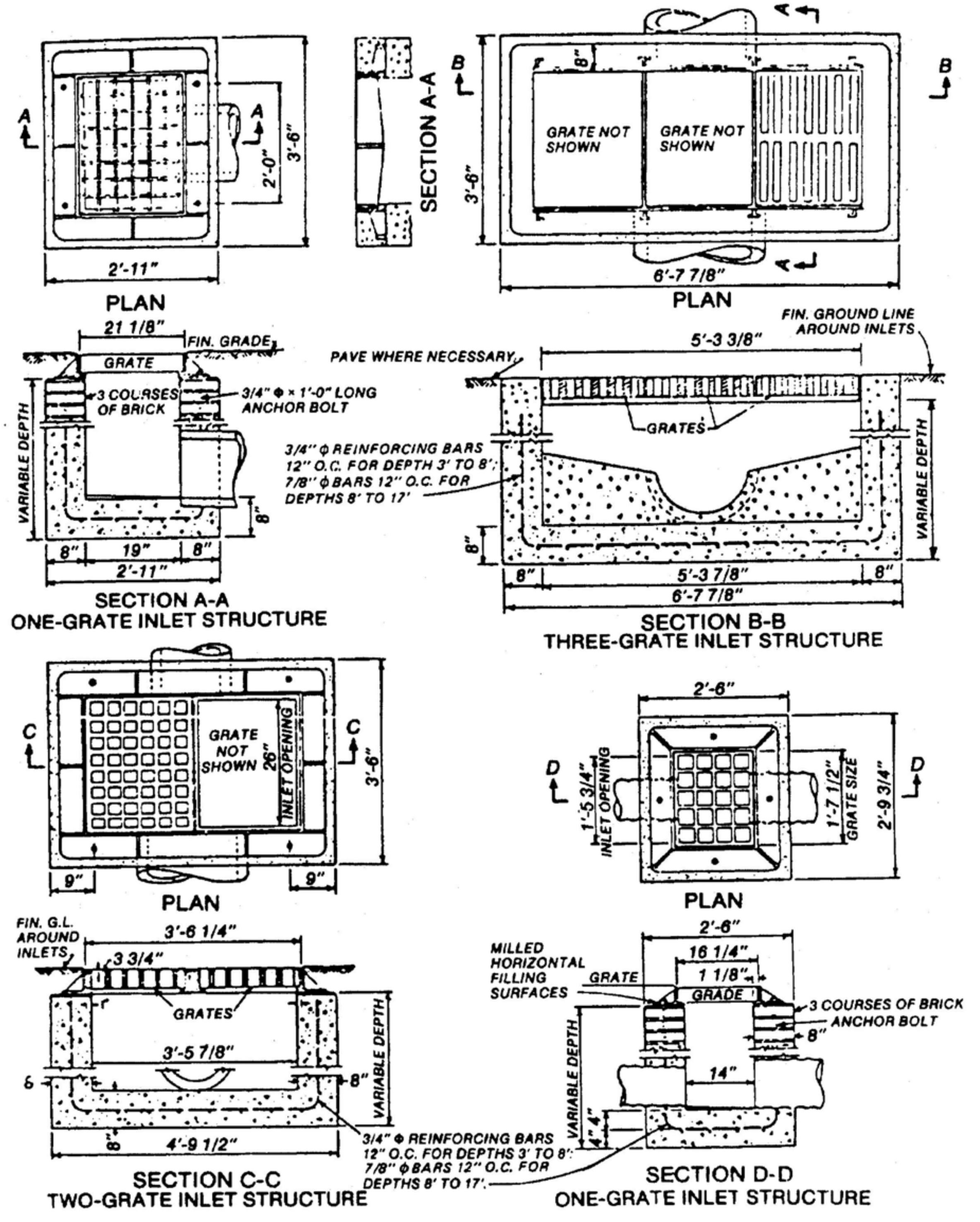
Grating for box drains can be built of steel, cast iron, or reinforced concrete with adequate strength to withstand anticipated loadings. Where two or more box drains are adjacent, they will be interconnected to provide equalization of flow and optimum hydraulic capacity.

Figure 4-7. Examples of Typical Inlet Grates



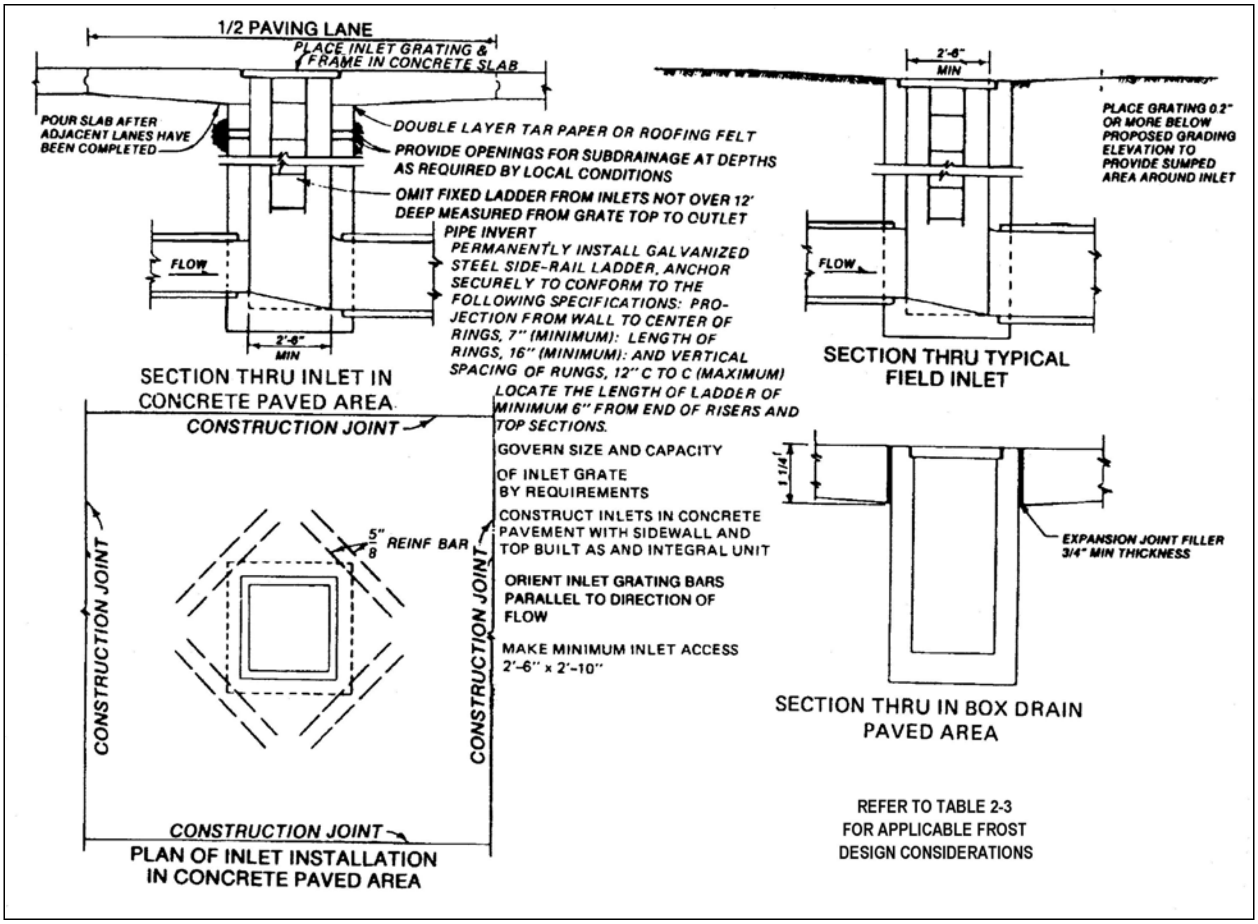
EXAMPLES OF TYPICAL INLET GRATES

Figure 4-8. Examples of Inlet Design



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

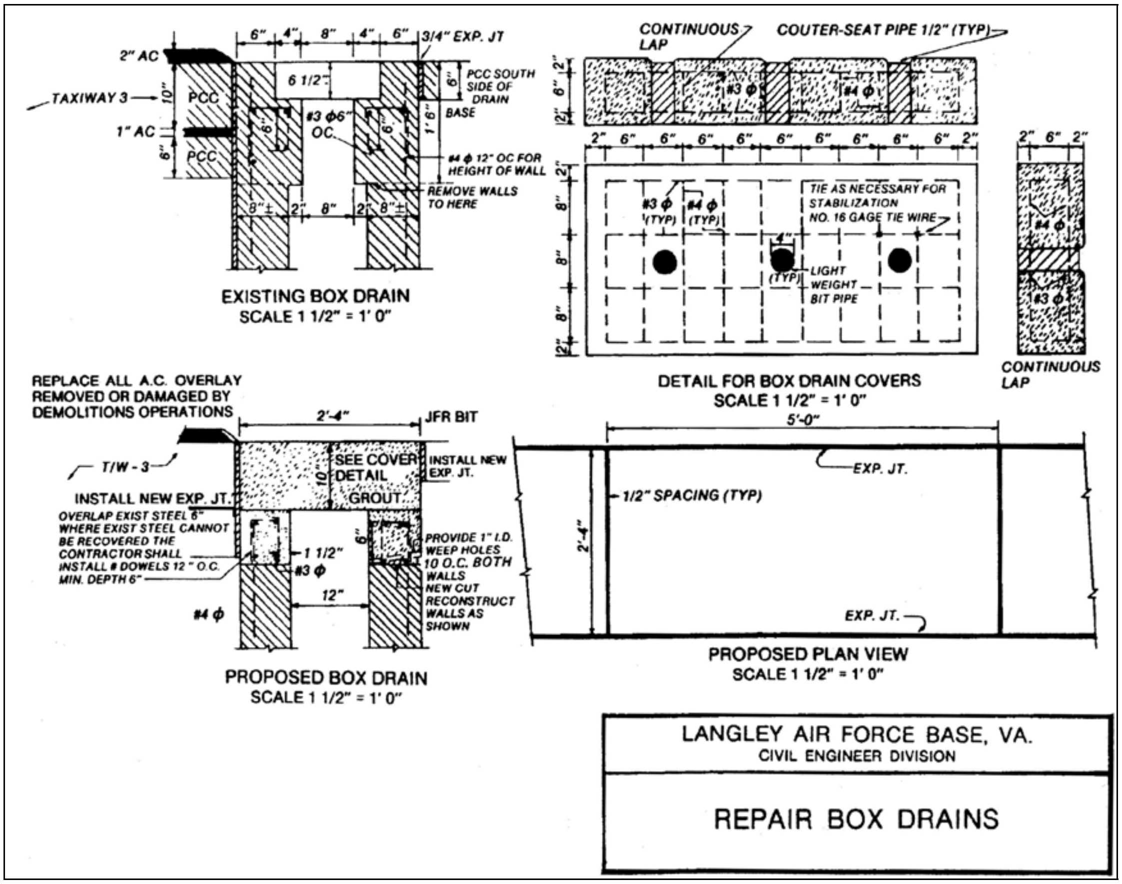
Figure 4-9. Typical Inlet and Box Drain Designs for Airfield and Heliport Storm Drainage Systems



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

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

Figure 4-10. Repair Box Drains



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

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

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

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

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

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

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

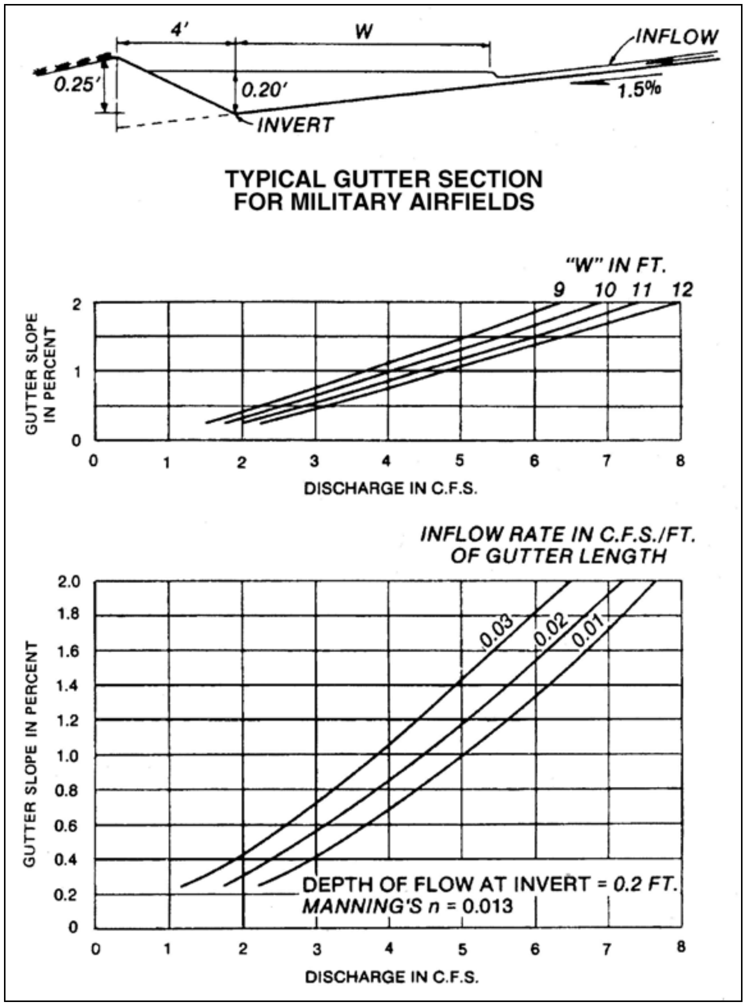
4-2.3 Gutters

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

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

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

Figure 4-11. Drainage Gutters for Runways and Aprons



4-2.4 **Storm Drains and Culverts**

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

4-2.4.2 **Headwalls and endwalls**.

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

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

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

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

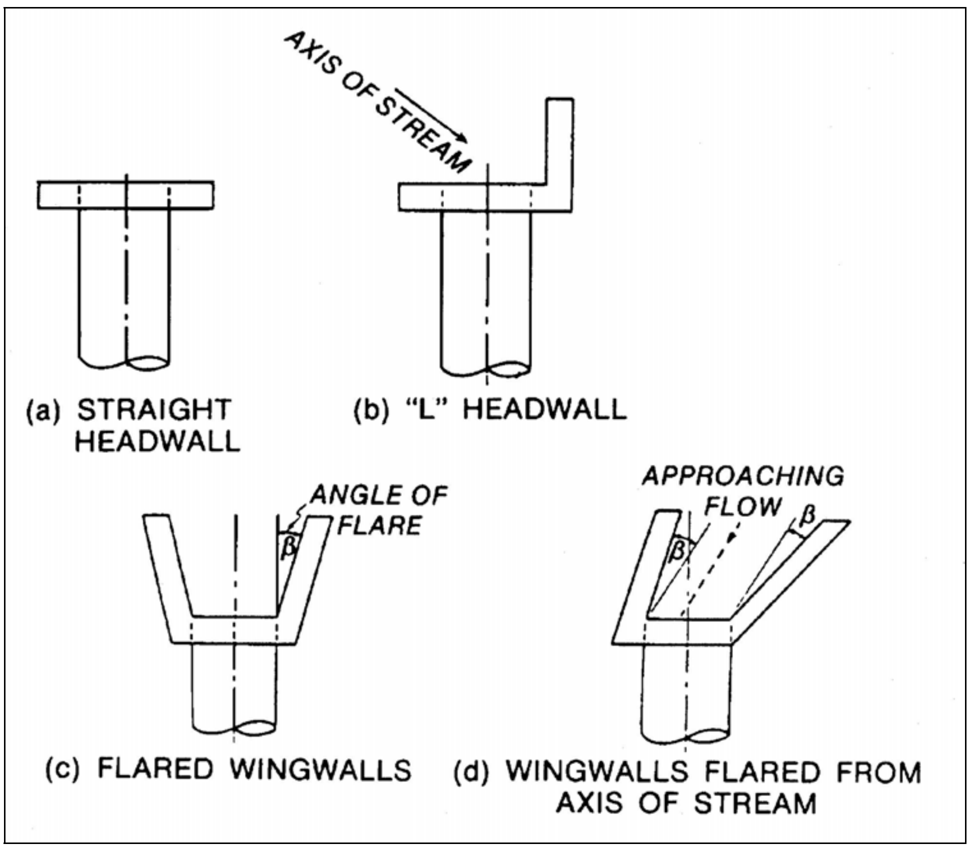
4-2.4.2.5 Entrance structures (headwalls and wingwalls) protect the embankment from erosion and, if properly designed, may improve the hydraulic characteristics of the culvert. The height of these structures should be kept to the minimum that is consistent with hydraulic, geometric, and structural requirements.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

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

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

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

Figure 4-12. Culvert Headwalls and Wingwalls



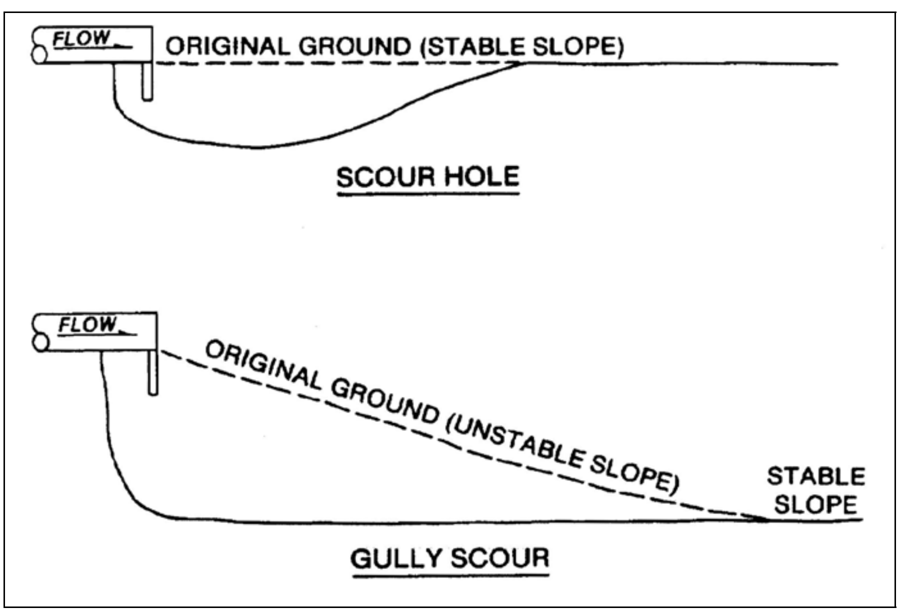
4-2.4.2.7 Headwalls and endwalls incorporating various designs of energy dissipators, flared transitions, and erosion protection for culvert outfalls are discussed in detail in subsequent sections of this chapter.

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

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

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

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



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

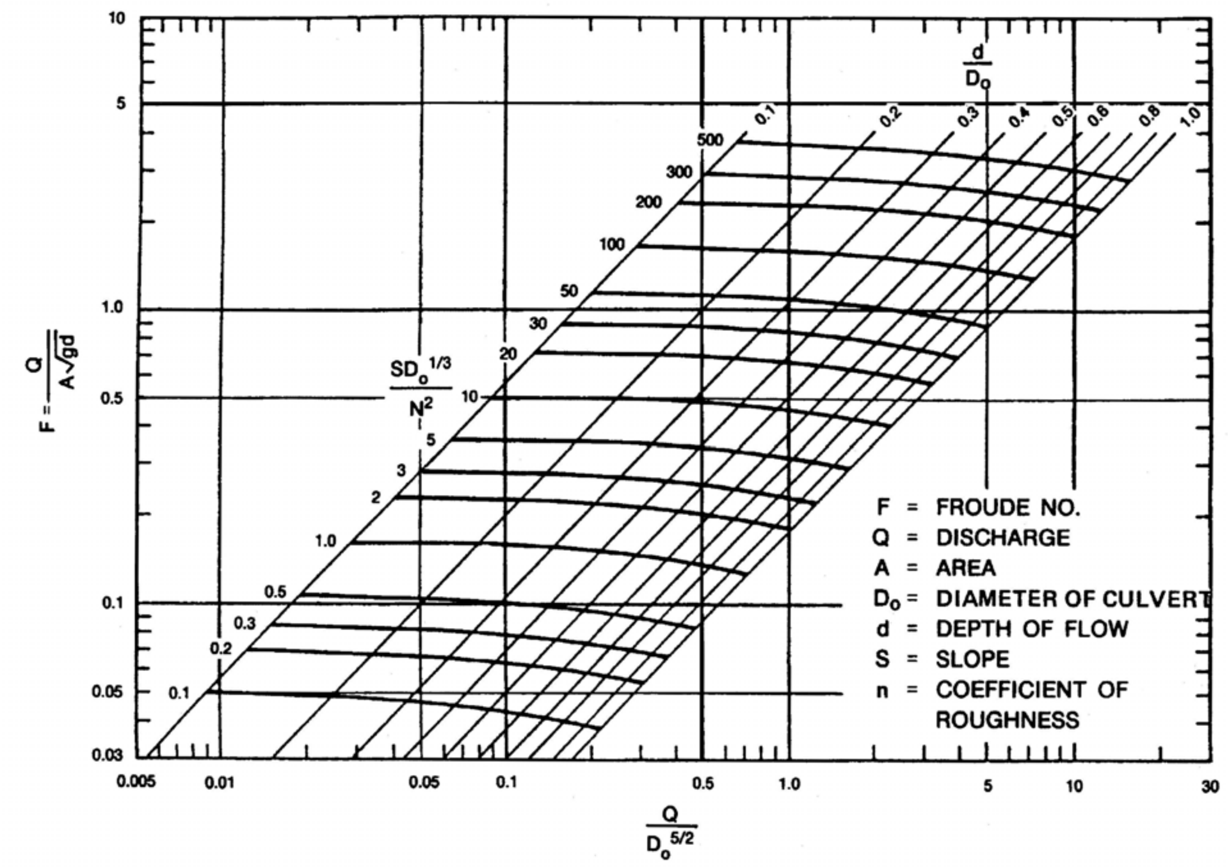
4-2.4.3.2 A scour hole or localized erosion can occur downstream of an outlet even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. In many situations, flow conditions can produce scour resulting in embankment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

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

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

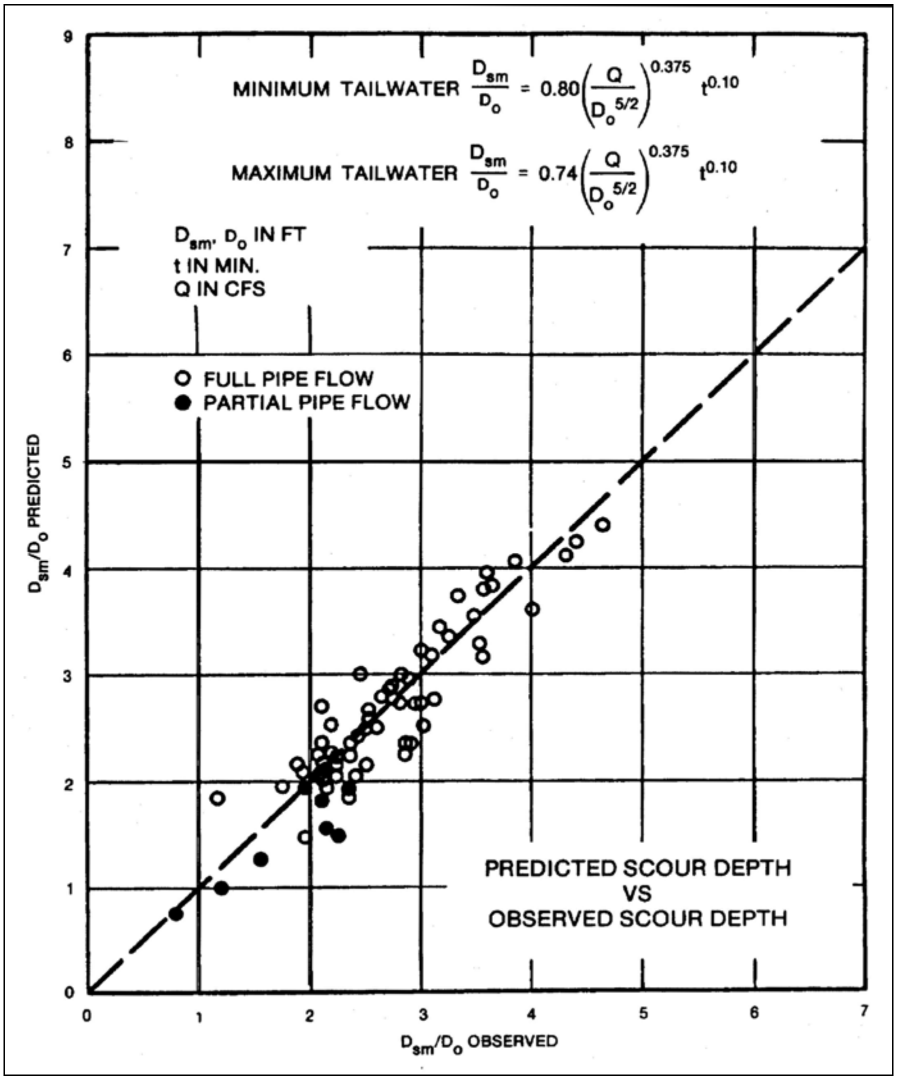
4-2.4.4.1 In some situations placement of riprap at the end of the outlet may be sufficient to protect the structure. The average size of stone (d_{50}) and configuration of a horizontal blanket of riprap at outlet invert elevation required to control or prevent localized scour downstream of an outlet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

Figure 4-14. Square Culvert-Froude Number



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

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



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

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

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

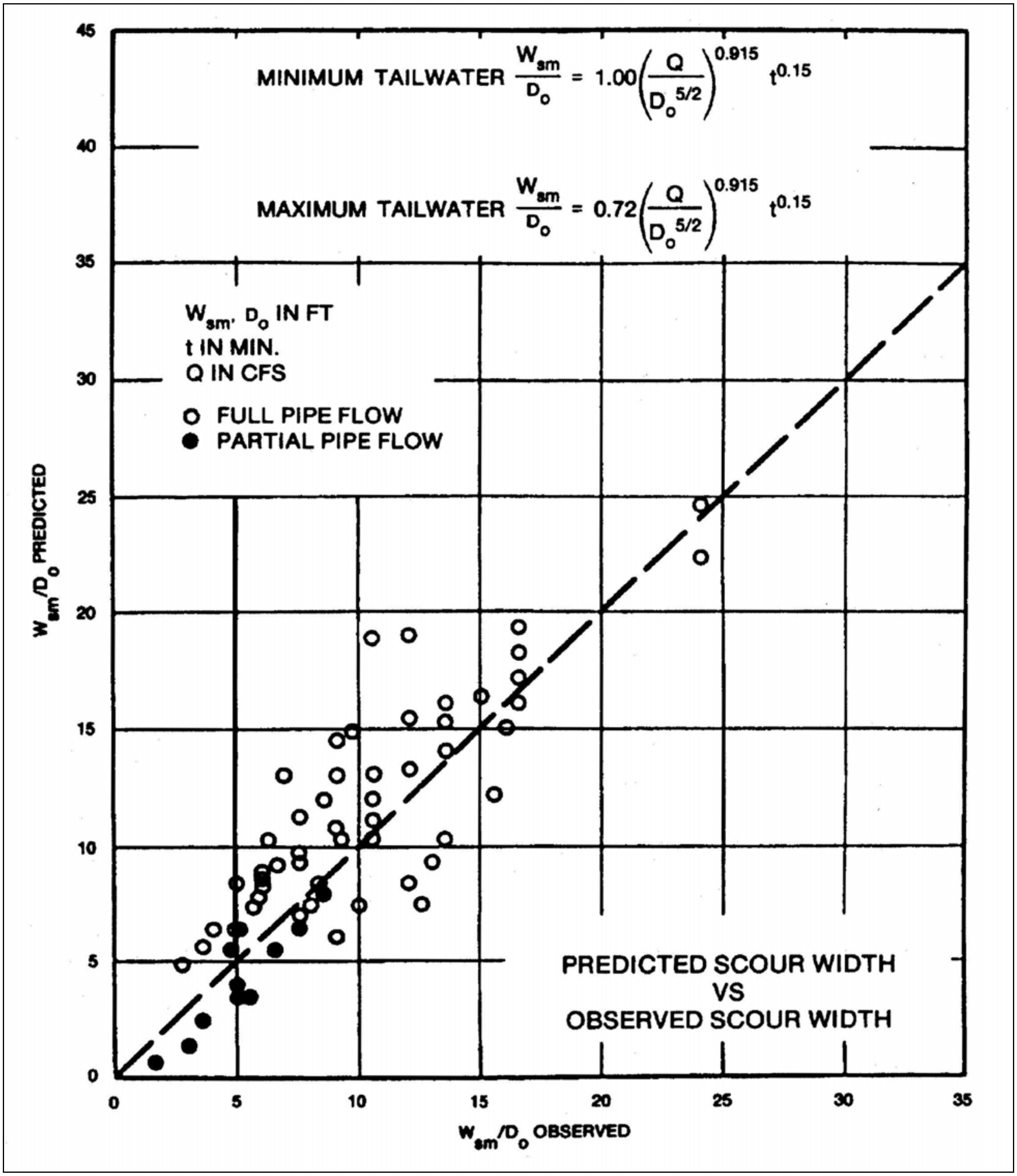


Figure 4-17. Predicted Scour Length Versus Observed Scour Length

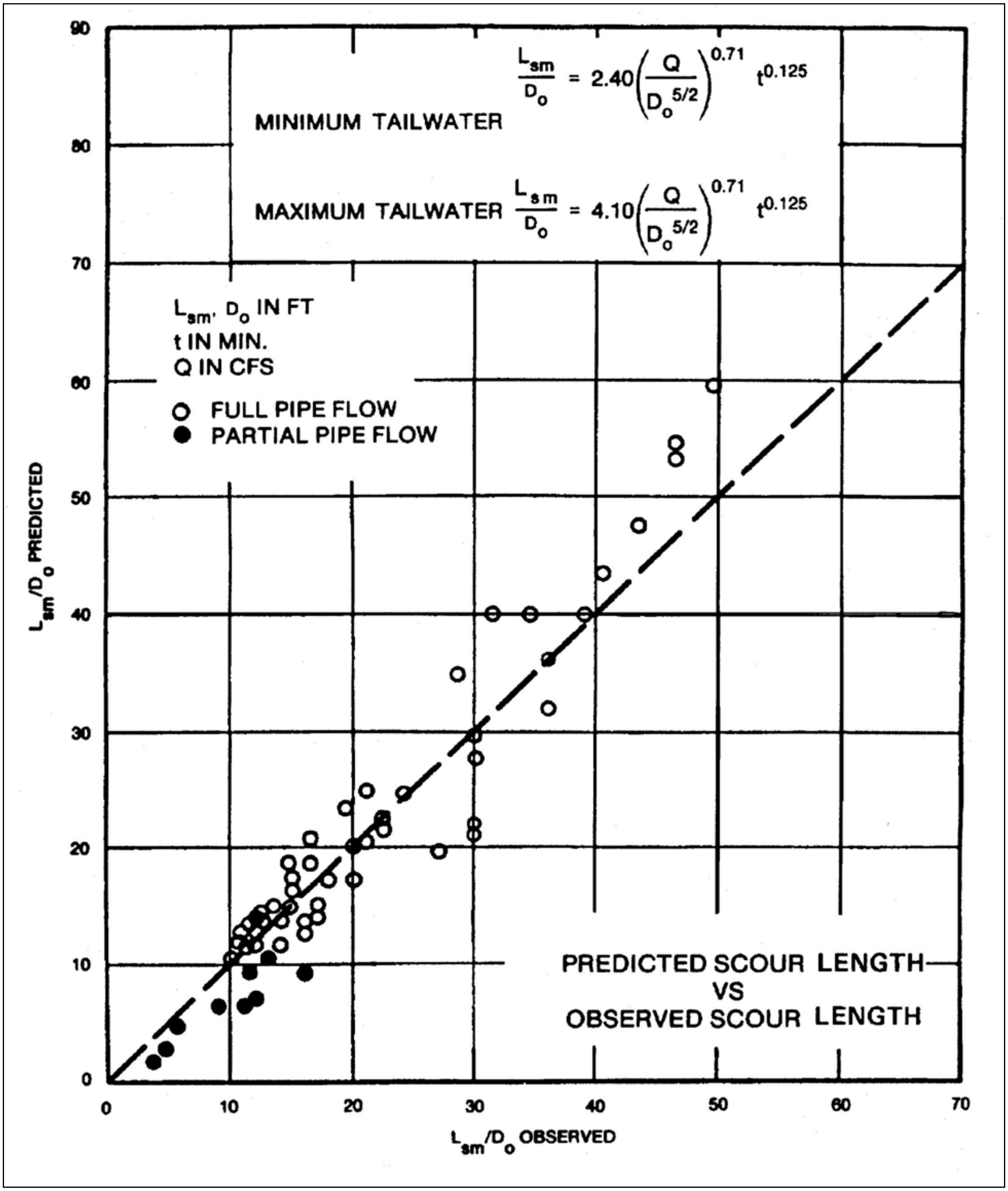


Figure 4-18. Predicted Scour Volume Versus Observed Scour Volume

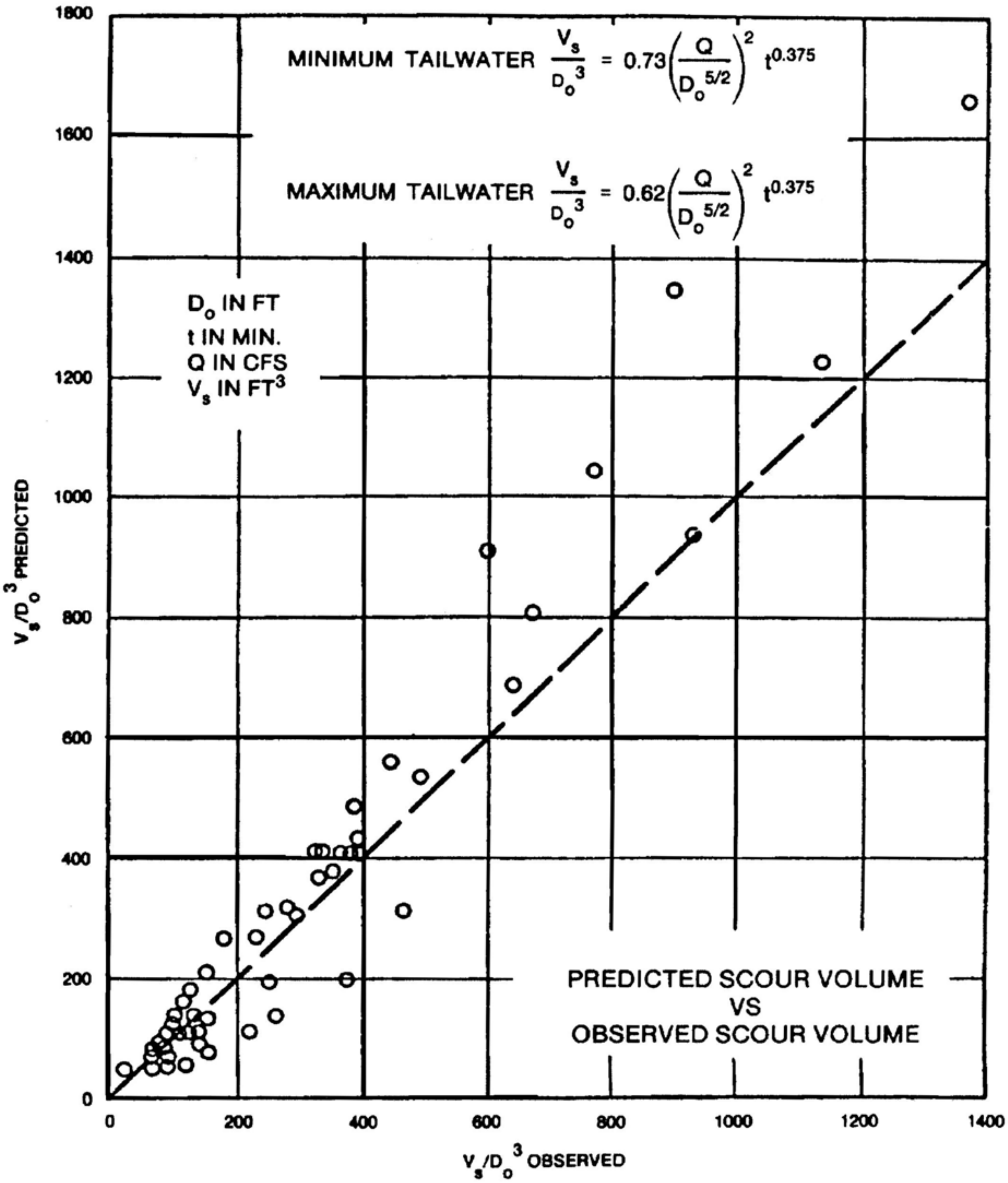


Figure 4-19. Dimensionless Scour Hole Geometry for Minimum Tailwater

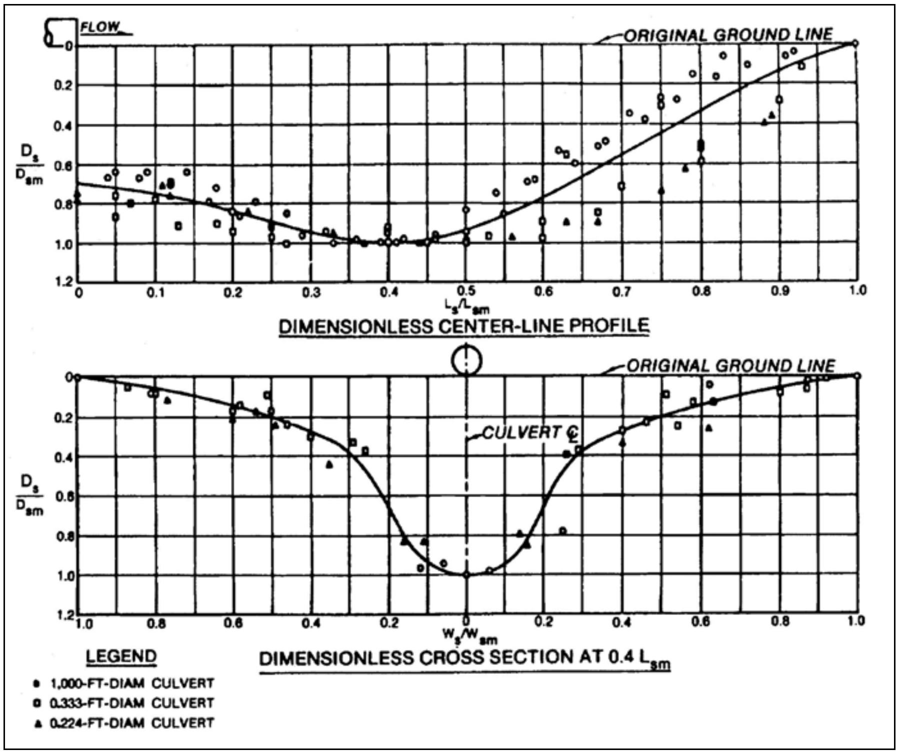


Figure 4-20. Dimensionless Scour Hole Geometry for Maximum Tailwater

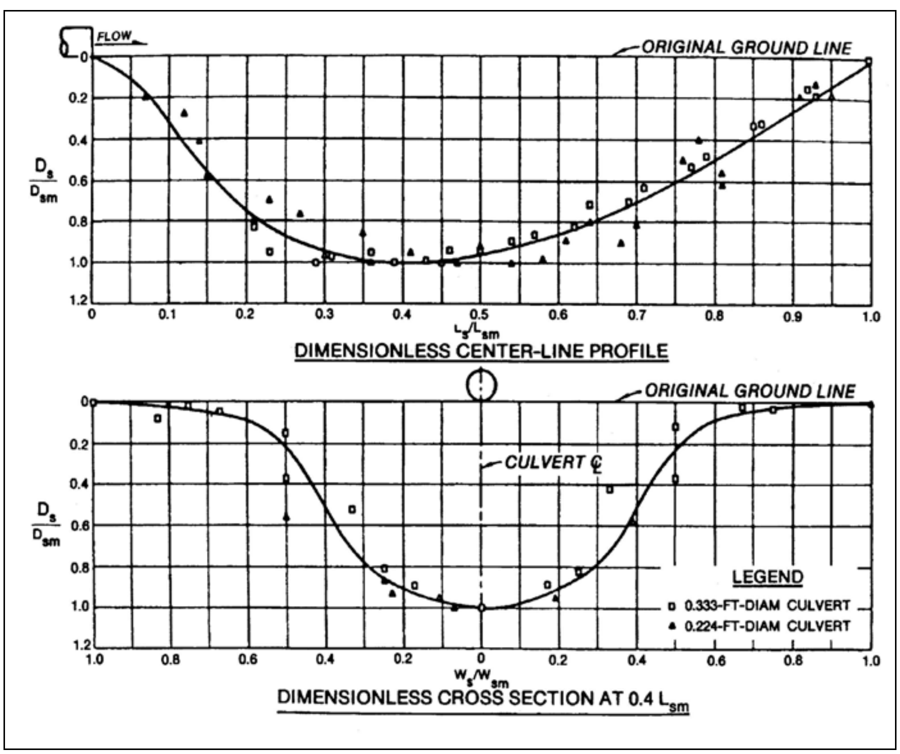
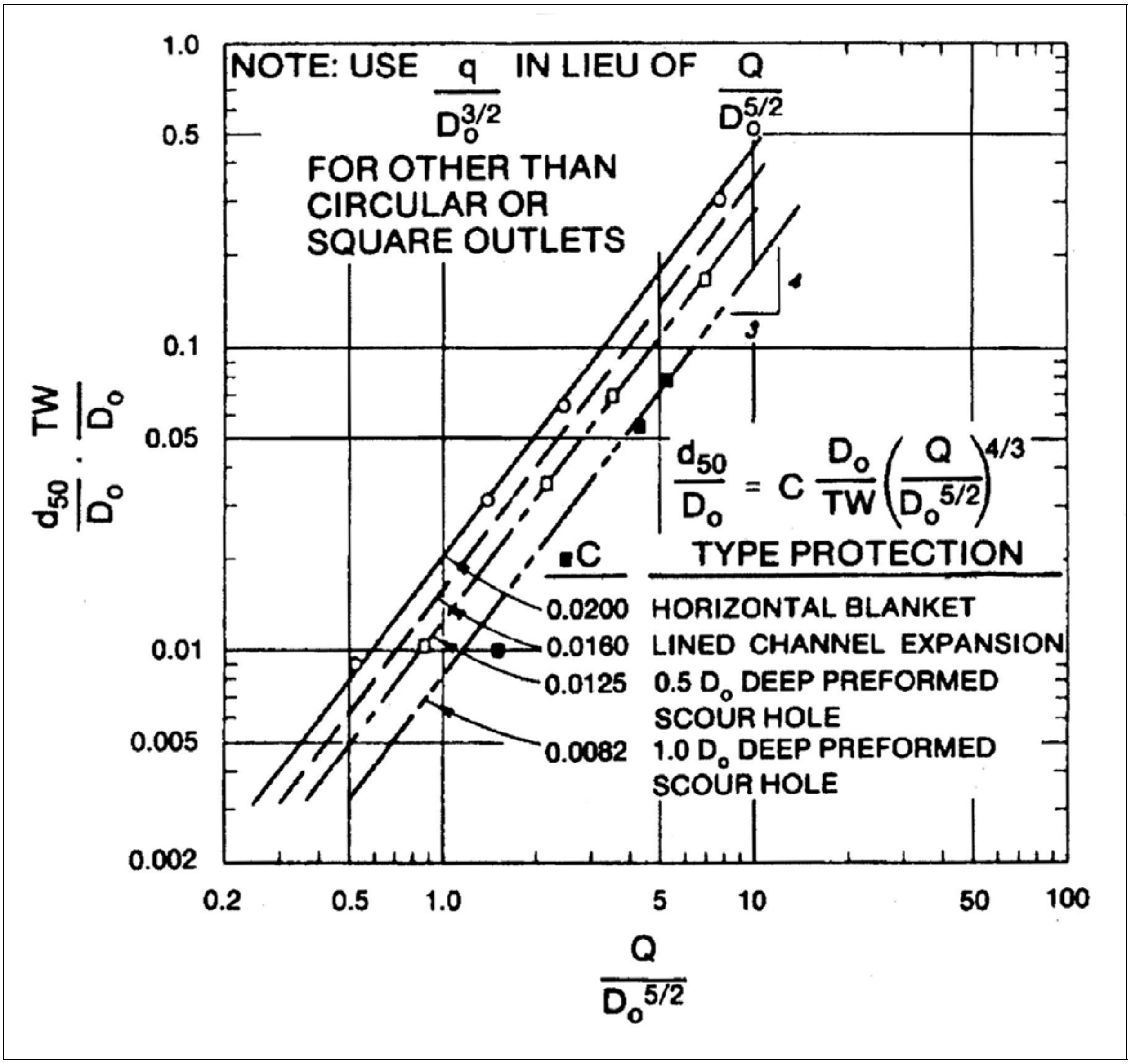


Figure 4-21. Recommended Size of Protective Stone



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

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

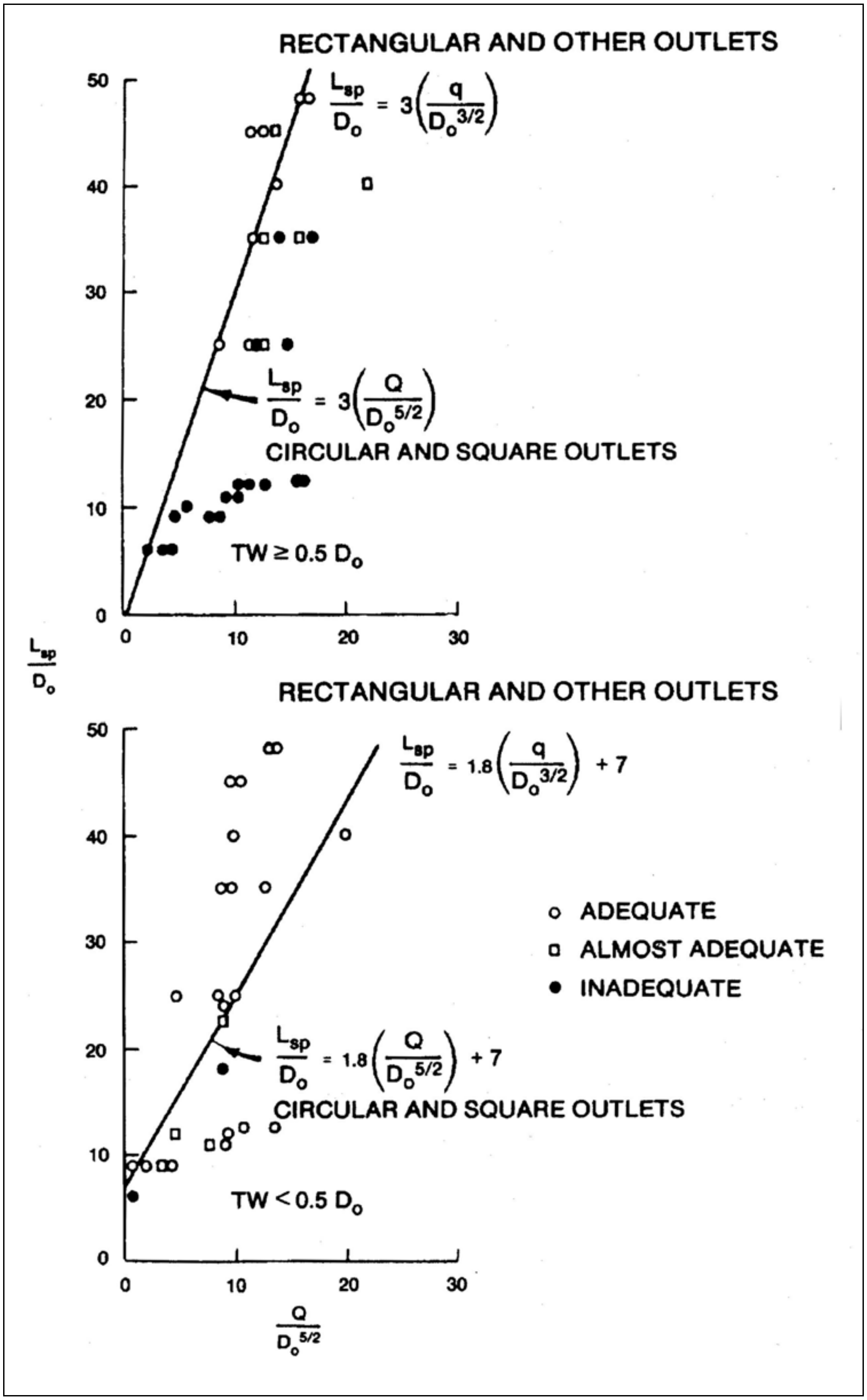


Figure 4-23. Recommended Configuration of Riprap Blanket Subject to Minimum and Maximum Tailwaters

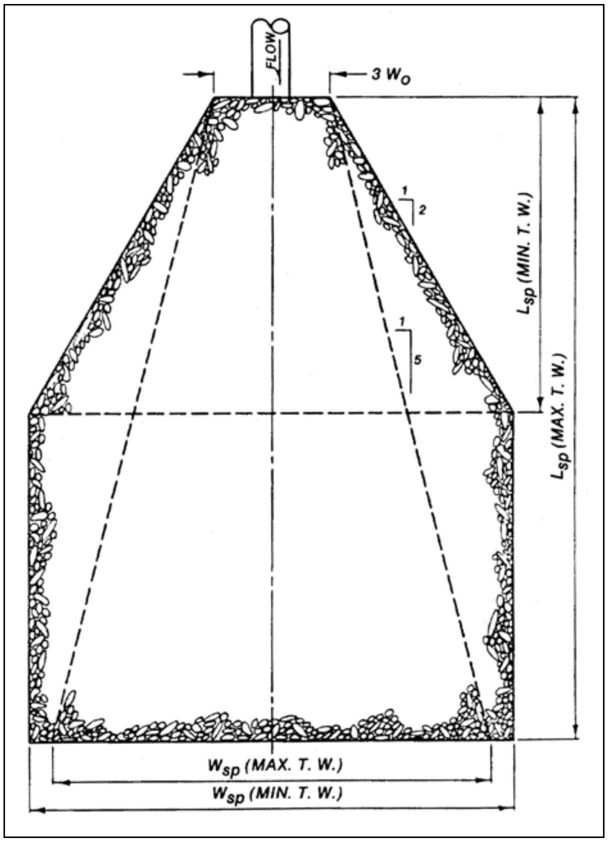


Figure 4-24. Preformed Scour Hole

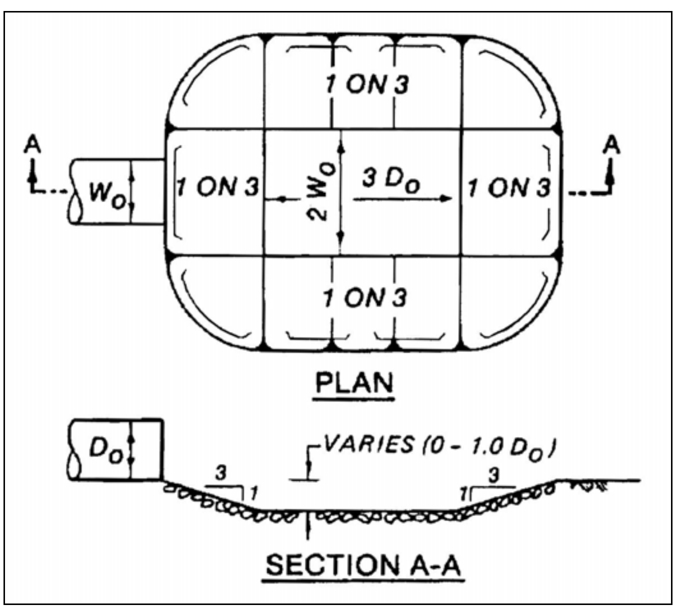


Figure 4-25. Culvert Outlet Erosion Protection, Lined Channel Expansion

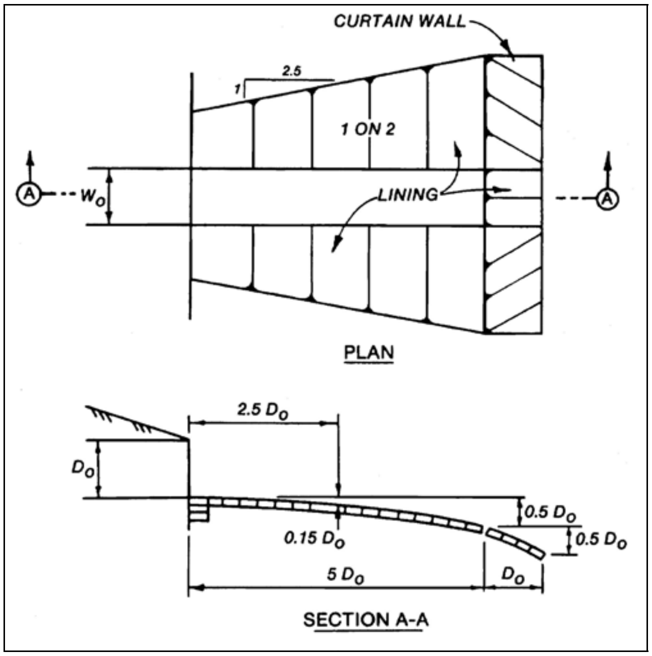
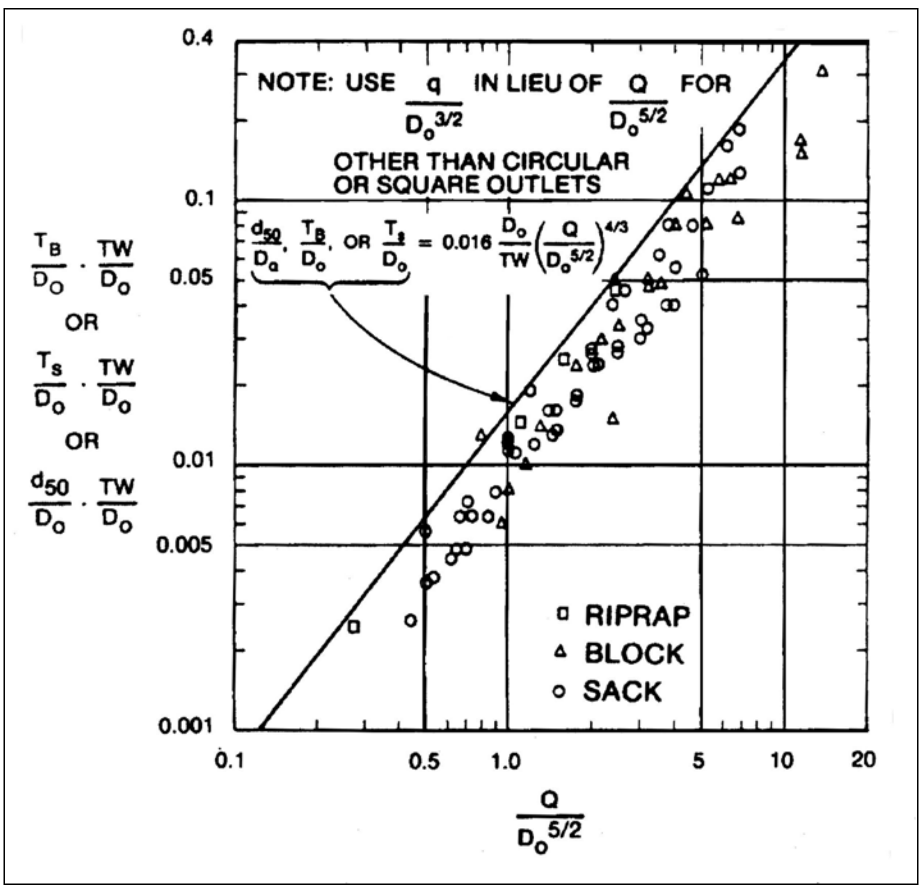


Figure 4-26. Maximum Permissible Discharge for Lined Channel Expansions



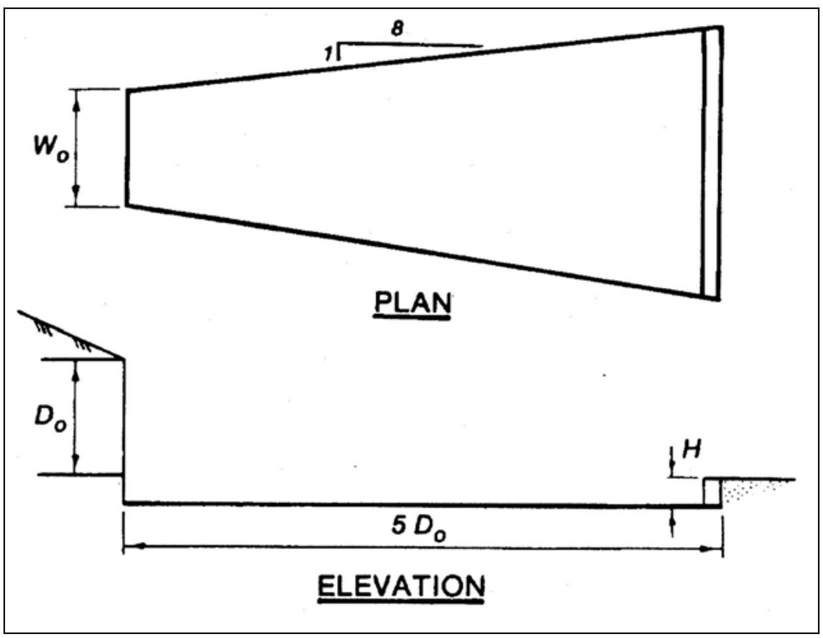
4-2.4.4.6 The simplest form of energy dissipator is the flared outlet transition. Protection is provided to the local area covered by the apron, and a portion of the kinetic energy of flow is reduced or converted to potential energy by hydraulic resistance provided by the apron. A typical flared outlet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

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

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

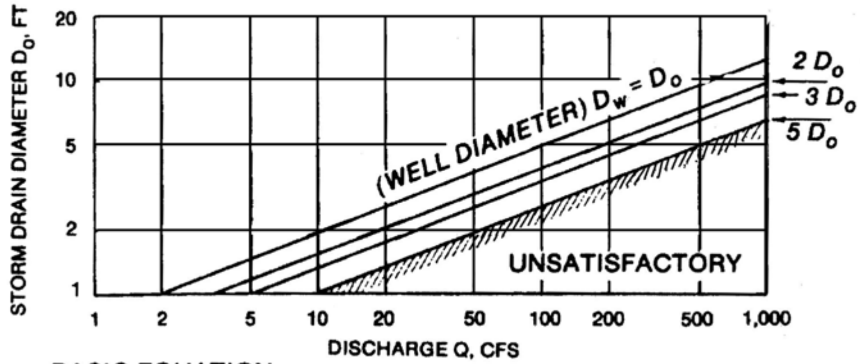
Recessing the apron and providing an end sill will not significantly improve energy dissipation.

Figure 4-27. Flared Outlet Transition



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

Figure 4-28. Stilling Well

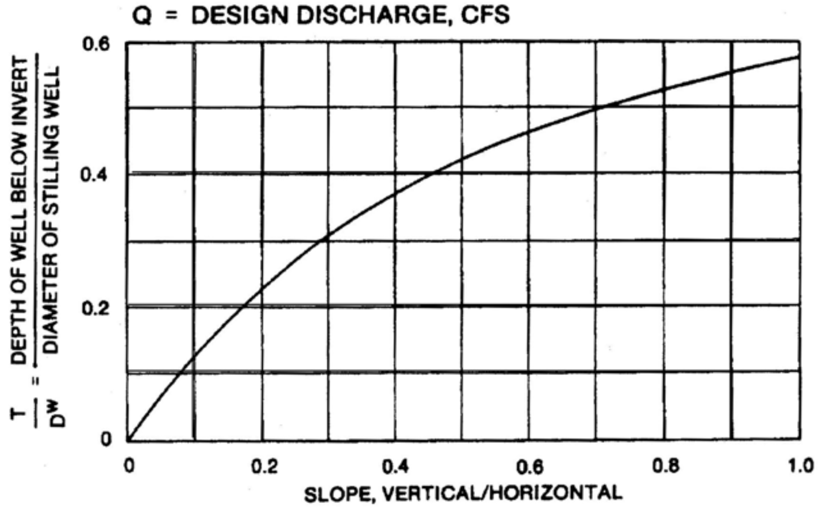


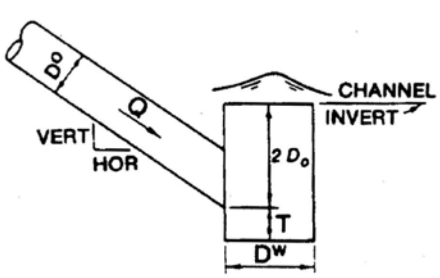
BASIC EQUATION

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

WHERE:

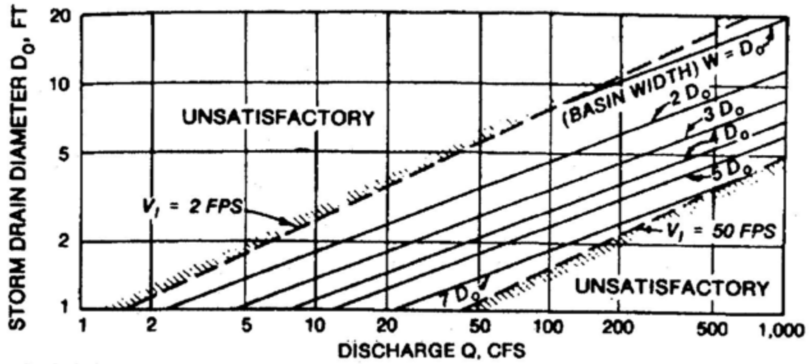
D^w = STILLING WELL DIAMETER, FT
D^o = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS





ELEVATION

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

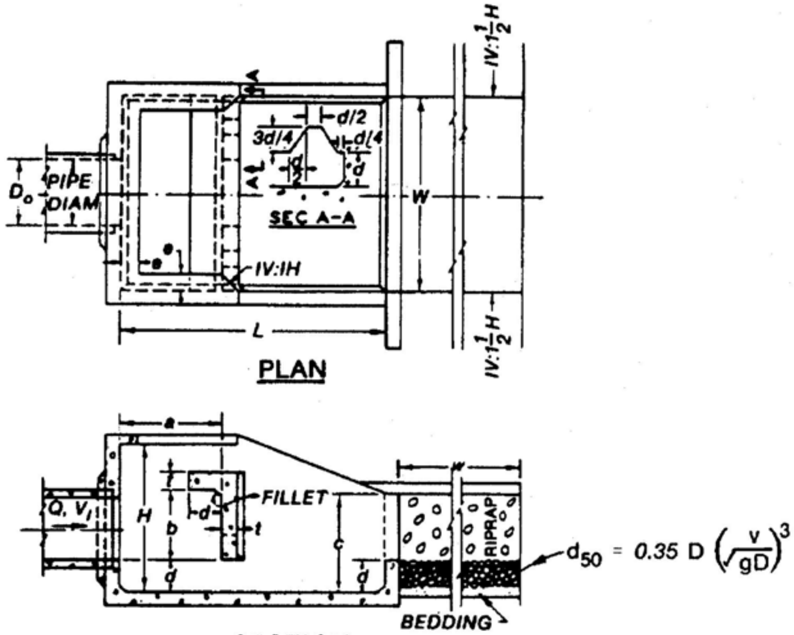


BASIC EQUATION

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

WHERE:

W = BASIN WIDTH, FT
D° = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS
V_i = PIPE VELOCITY, FPS



STILLING BASIN DESIGN

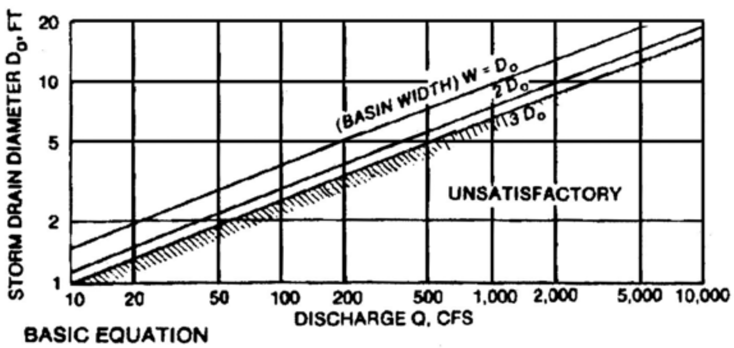
$H = \frac{3}{4}$ (W) $c = \frac{1}{2}$ (W)

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

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

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

Figure 4-30. Saint Anthony Falls Stilling Basin



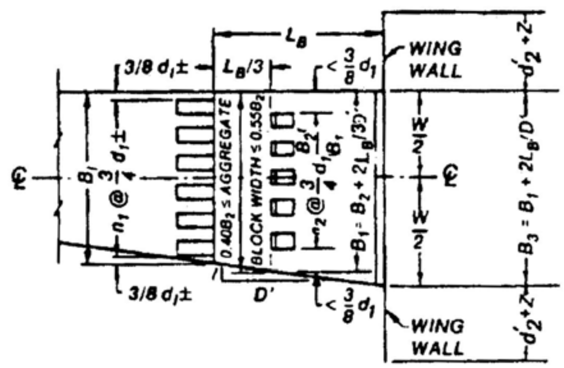
BASIC EQUATION

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

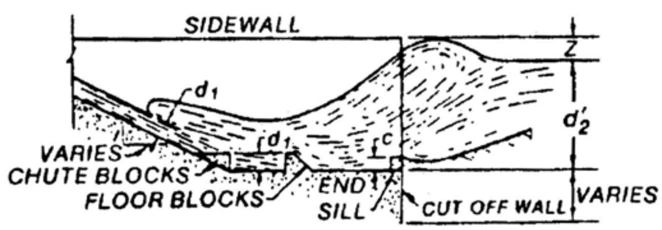
WHERE:

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

STILLING BASIN HALF-PLAN



TRAPEZOIDAL STILLING BASIN HALF-PLAN



CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{gd_1} \quad (1) \qquad L_B = \frac{4.5\,d_2}{F^{0.38}} \quad (4)$$

$$d_2 = \frac{d_1}{2}\left(-1 + \sqrt{8F + 1}\right) \quad (2) \qquad Z = \frac{d_2}{3} \quad (5)$$

$$F = 3 \text{ TO } 30 \qquad d_2 = (1.10 - F/120)d_2 \quad (3a) \qquad c = 0.07d_2 \quad (6)$$

$$F = 30 \text{ TO } 120 \qquad d_2 = 0.85d_2 \quad (3b)$$

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

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

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

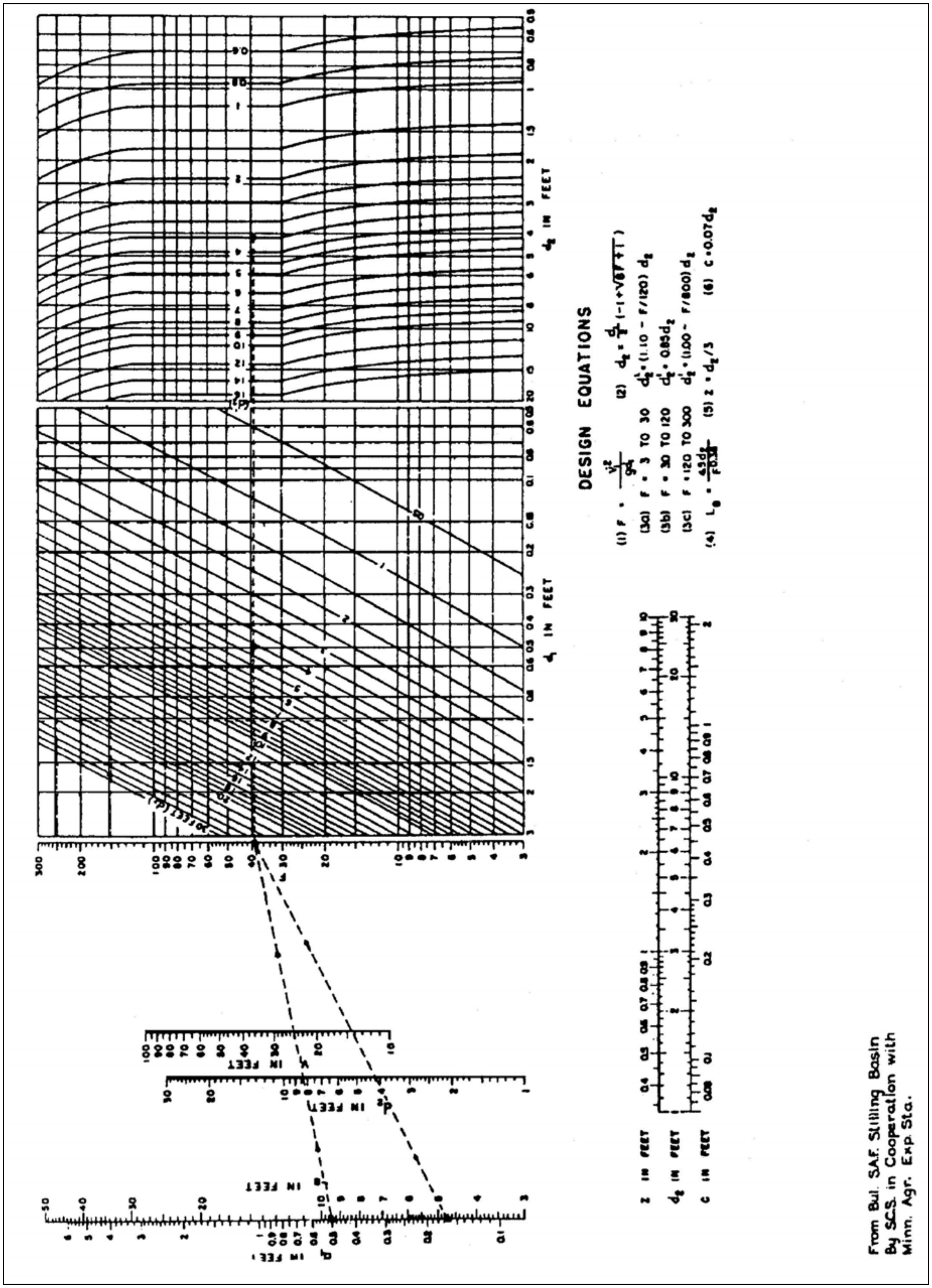
4-2.4.4.10 The Saint Anthony Falls (SAF) stilling basin shown in Figure 4-30 is a hydraulic jump energy dissipator. To function satisfactorily this basin must have sufficient tailwater to cause a hydraulic jump to form. Design equations for determining the dimensions of the structure in terms of the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

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

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

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

Figure 4-31. Design Chart for SAF Stilling Basin



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

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

Trapezoidal channels

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

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

Wide channel bottom or horizontal scour hole

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

½ D deep scour hole

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

D deep scour hole

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

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

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

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

Figure 4-32. Recommended Riprap Sizes

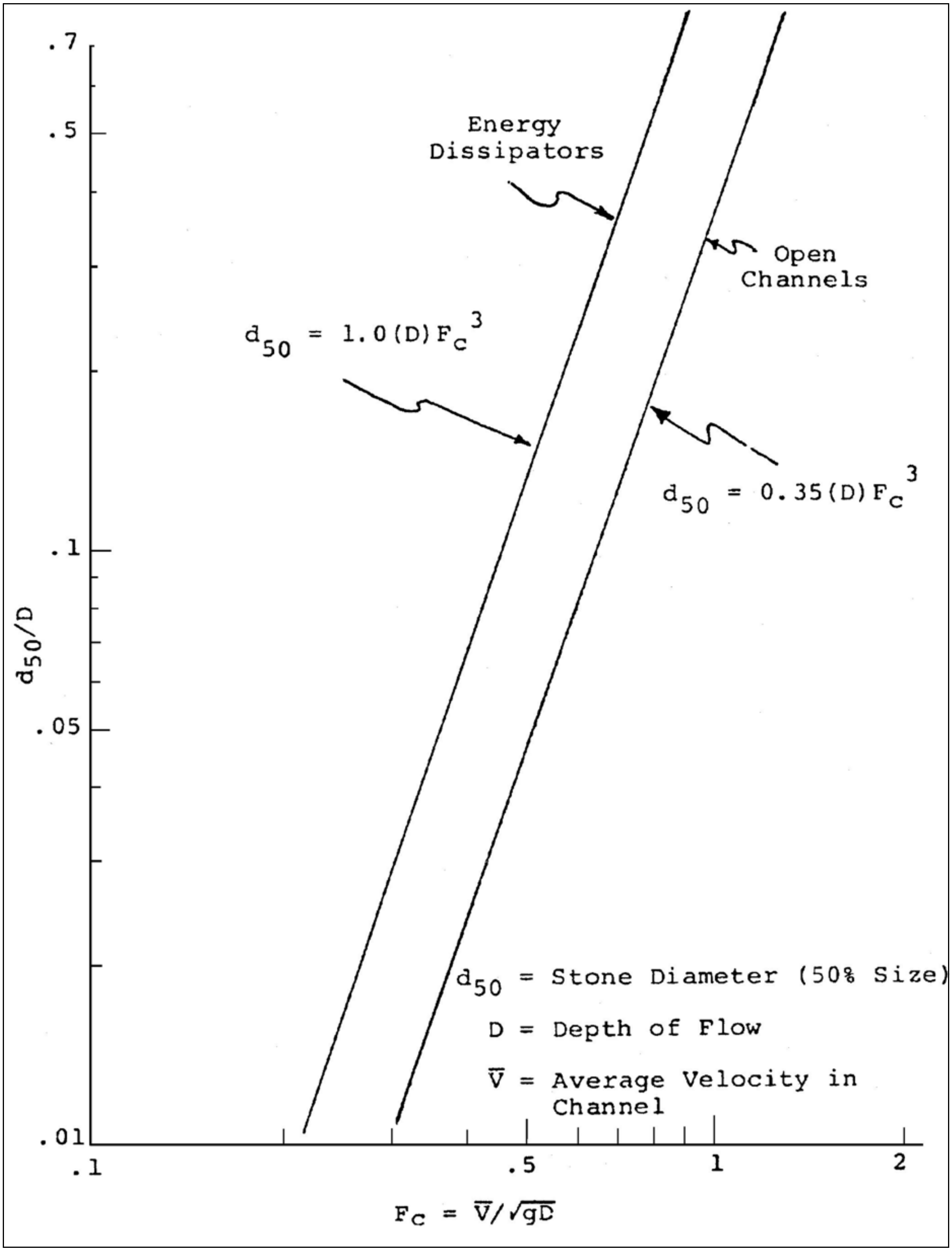
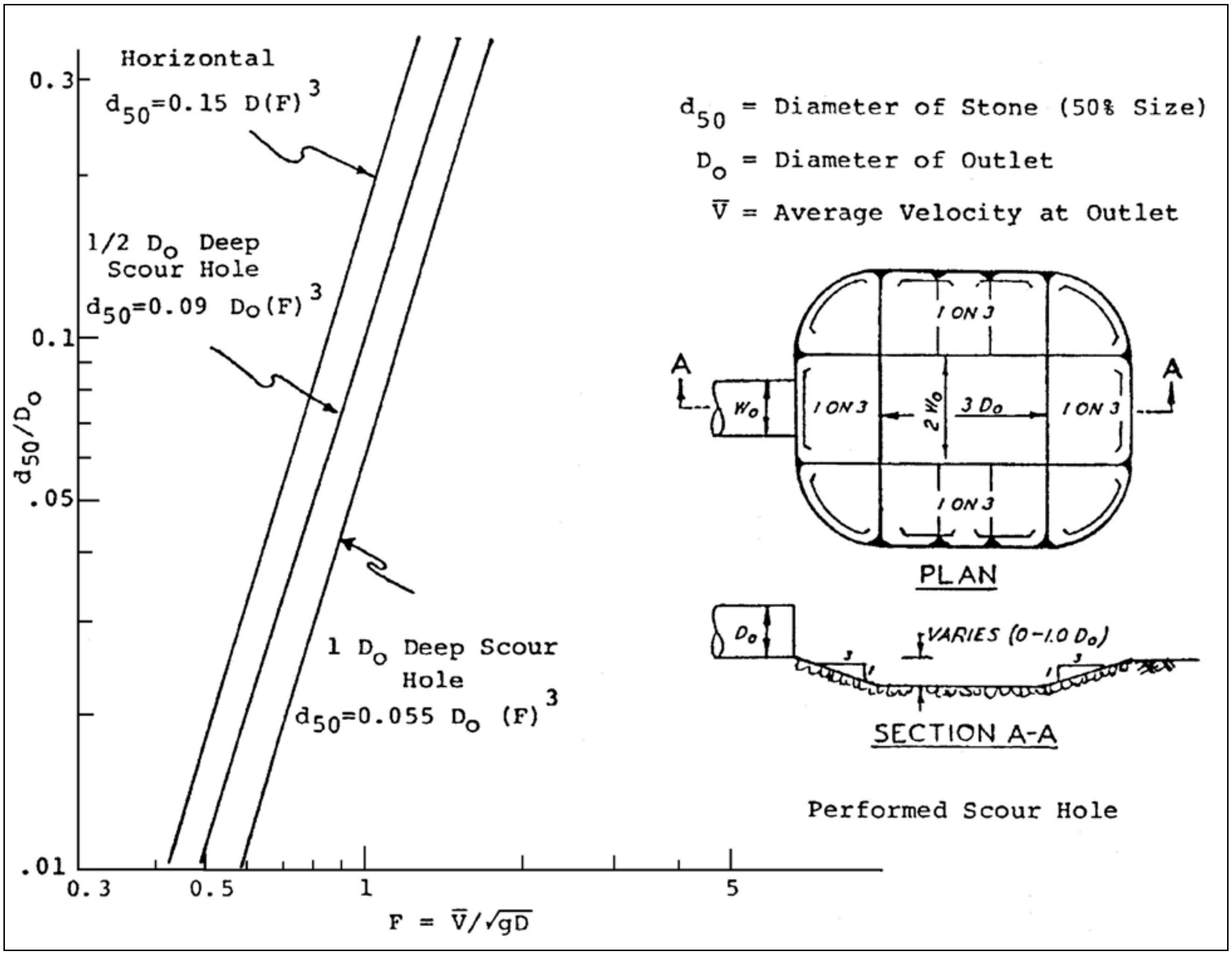


Figure 4-33. Scour Hole Riprap Sizes



4-2.5 Open Channels

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

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

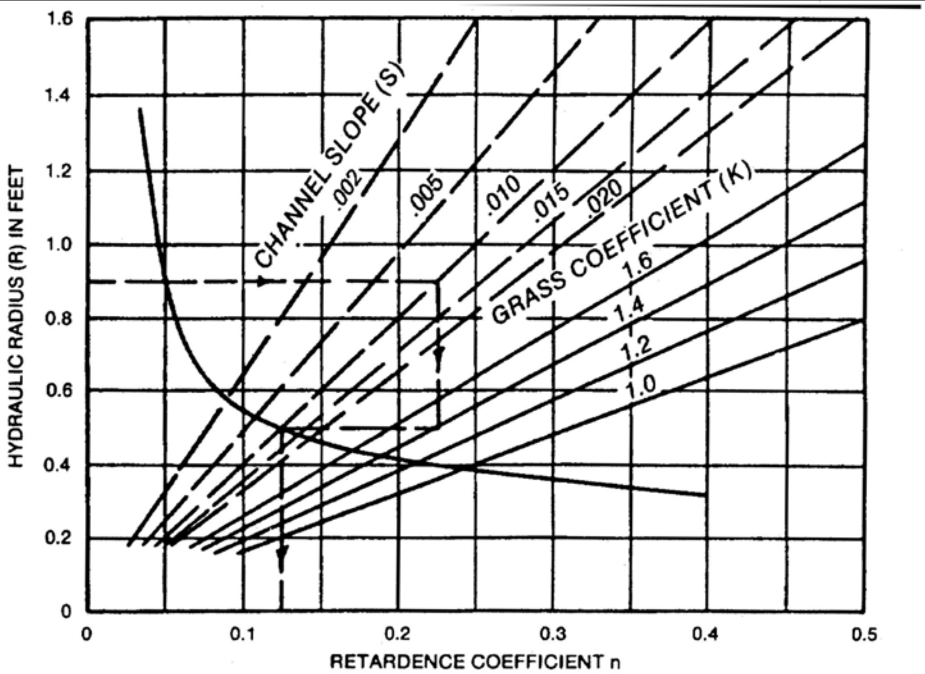
4-2.5.2.1 The hydraulic characteristics of the channel may be studied by using an open-channel formula such as Manning's. Suggested retardance coefficients and maximum permissible velocities for nonvegetated channels are given in Table 4-3. Retardance coefficients for turf-lined channels are a function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

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

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

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

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



GRASS COEFFICIENTS (K) FOR DENSE AIRFIELD TURF

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

EXAMPLE:

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

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

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

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

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

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

4-2.5.3 Design procedure

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

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

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

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

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

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

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

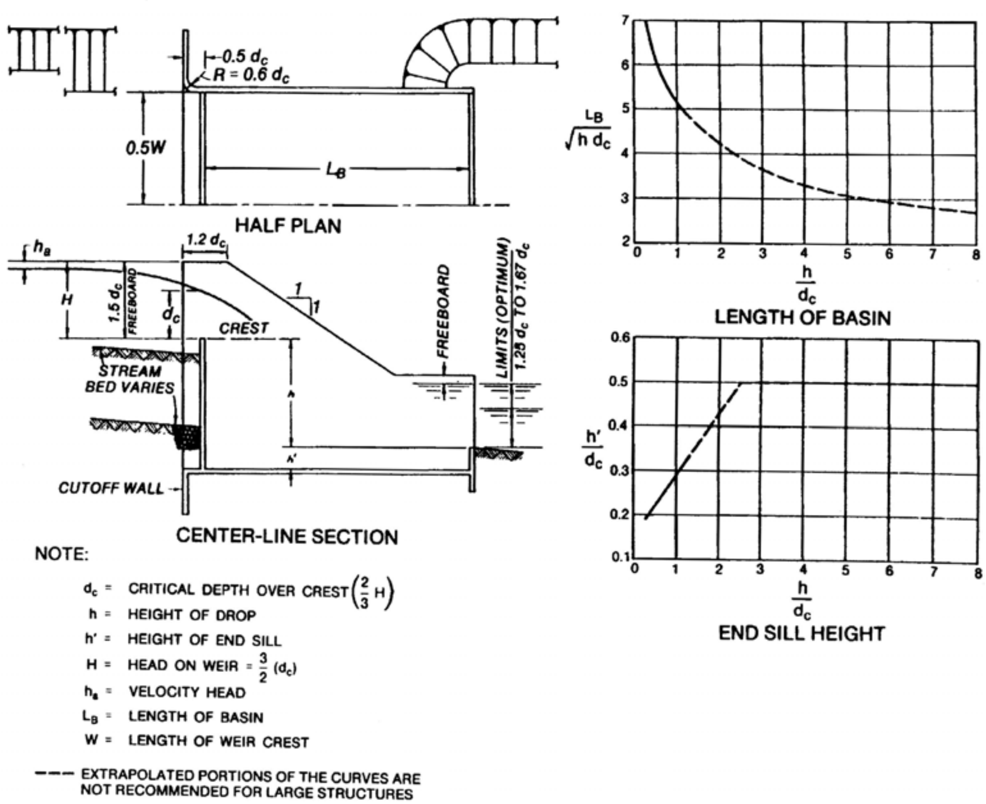
a. Discharge over the weir should be computed from the equation $Q = CWH^{3/2}$ using a C value of 3.0. To minimize erosion and obtain maximum use of the available channel cross section upstream from the structure, the length of the weir should be adjusted to maintain a head on the weir equivalent to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

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

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

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

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



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

Width of Crest, W:

$$Q = CWH^{3/2} \qquad \text{(eq. 4-9)}$$

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

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

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

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

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

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

Critical depth, d_c:

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

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

From Figure 4-35:

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

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

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

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

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

Riprap design:

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

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

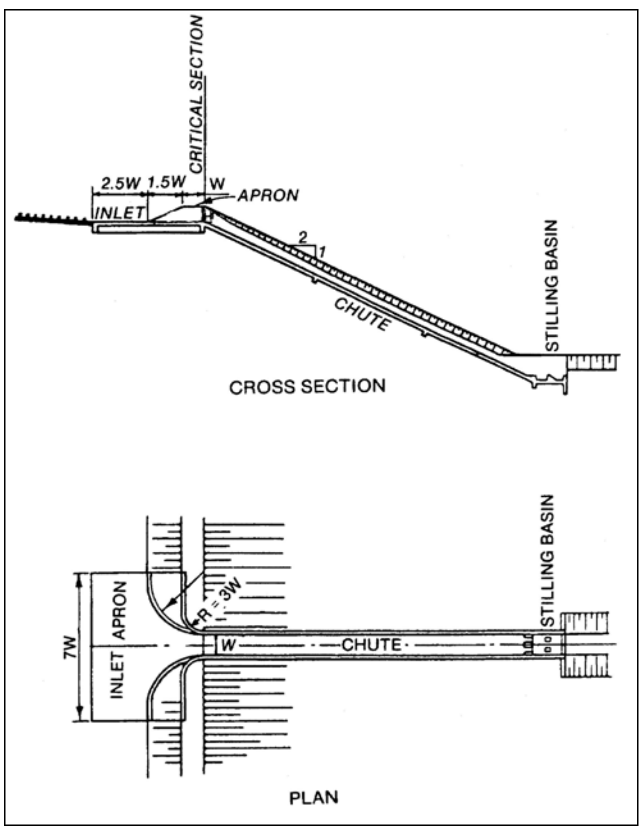
Riprap should extend approximately 10 times depth of flow downstream from structure (10 × 5 = 50 ft).

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

4-2.6 **Chutes**

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

Figure 4-36. Details and Typical Drainage Chute



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

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

4-2.6.2 **Design**

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

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

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

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

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

where:

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

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

Figure 4-37. Details of Typical Drop Intake

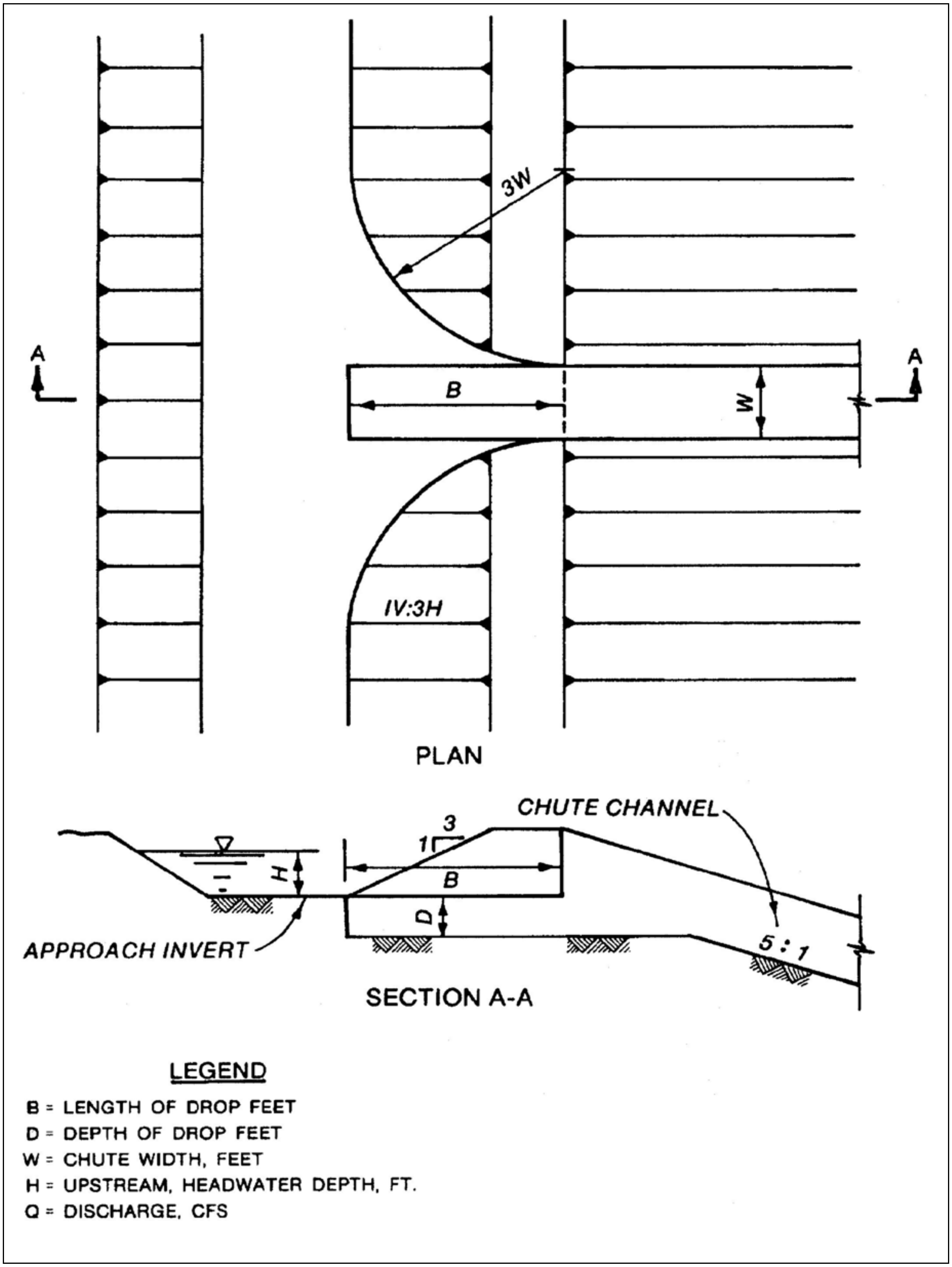


Figure 4-38. Drop Structure Calibration Curve

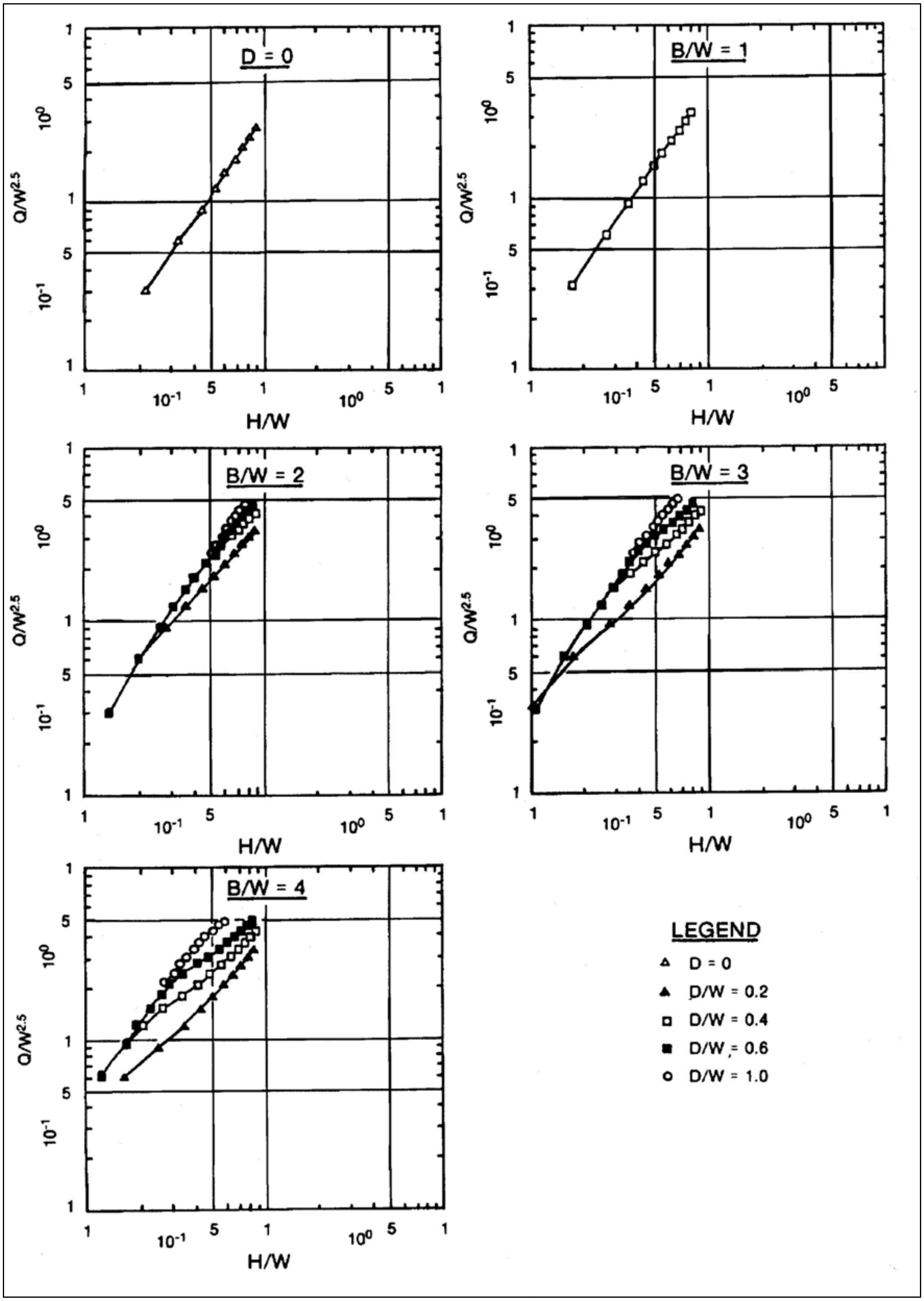
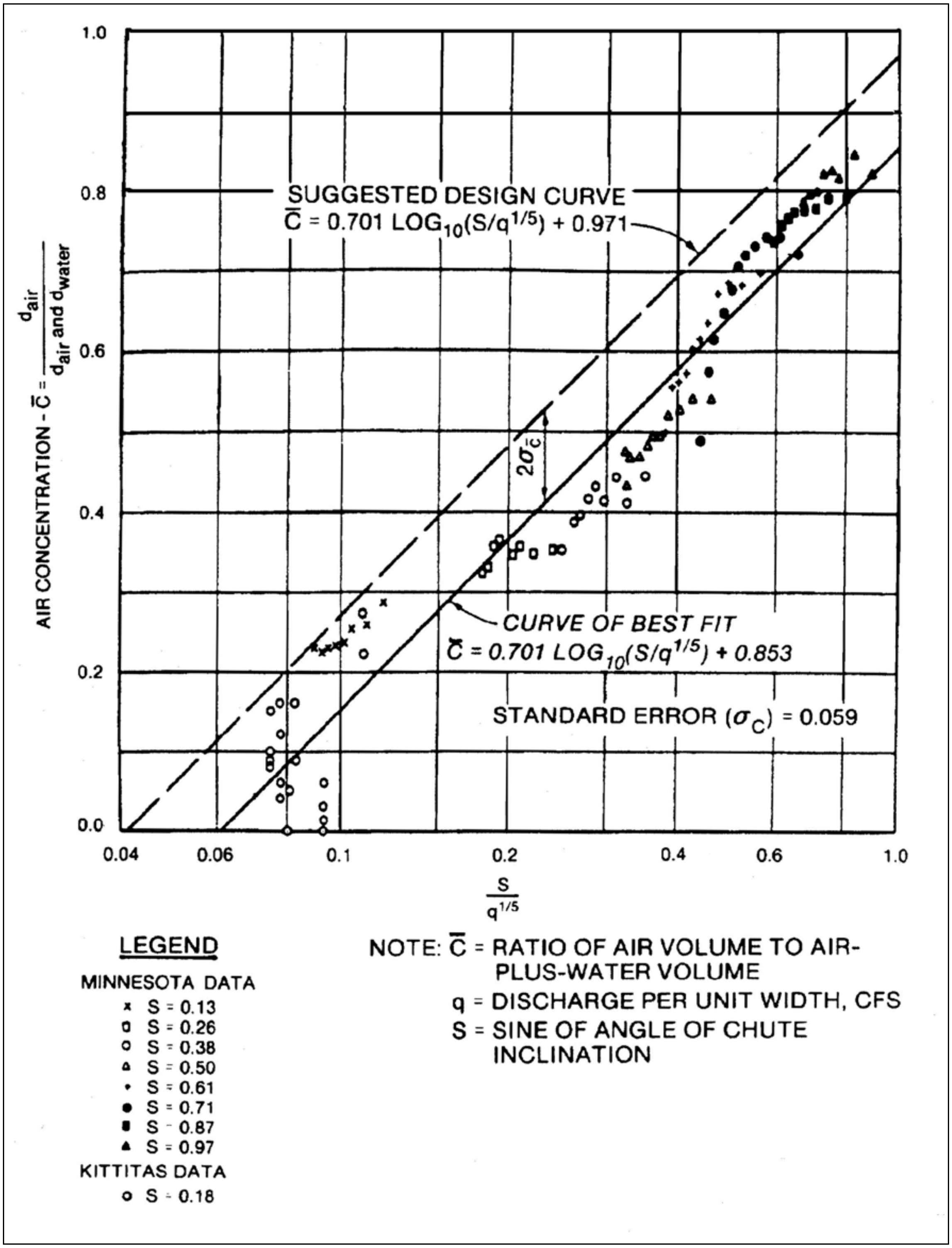


Figure 4-39. Air Entrainment in Chute Flow



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

4-2.7 **Construction Drainage**

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

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

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

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

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

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

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

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

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

4-2.8 **Notation**

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

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

h Height of fall or drop in structure, ft

h_1 Height of longitudinal sill, ft

h_t Height of transverse end sill, ft

h' Height of end sill

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

L_s Length of scour, ft

L_{sm} Maximum length of scour, ft

L_{sp} Length of stone protection

n Manning's roughness coefficient

Q Discharge, cfs

q Discharge per foot of width, cfs/ft

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

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

TW Tailwater depth above invert of culvert outlet, ft

T Top width of flow in channel, ft

T_s Thickness of sack revetment

T_B Thickness of cellular blocks

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

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

V_s Volume of scour, ft^3

W Length of weir, width of flume, ft

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

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

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

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

4-4 AREAS OTHER THAN AIRFIELDS

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

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

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

4-4.2 **Channels**.

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

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

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

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

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

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

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

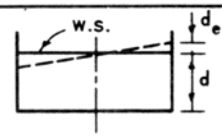
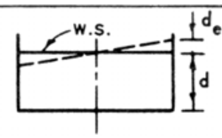
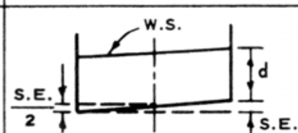
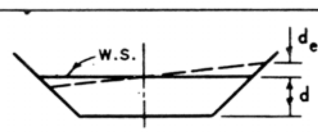
$$L_t = 15\frac{V^2T}{R_c g} \qquad \text{(eq. 4-23)}$$

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

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

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

Figure 4-40. Superelevation Formulas

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

LEGEND

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

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

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

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

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

4-4.3 **Bridges**

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

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

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

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

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

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

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

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

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

4-4.4 **Curb-and-Gutter Sections**

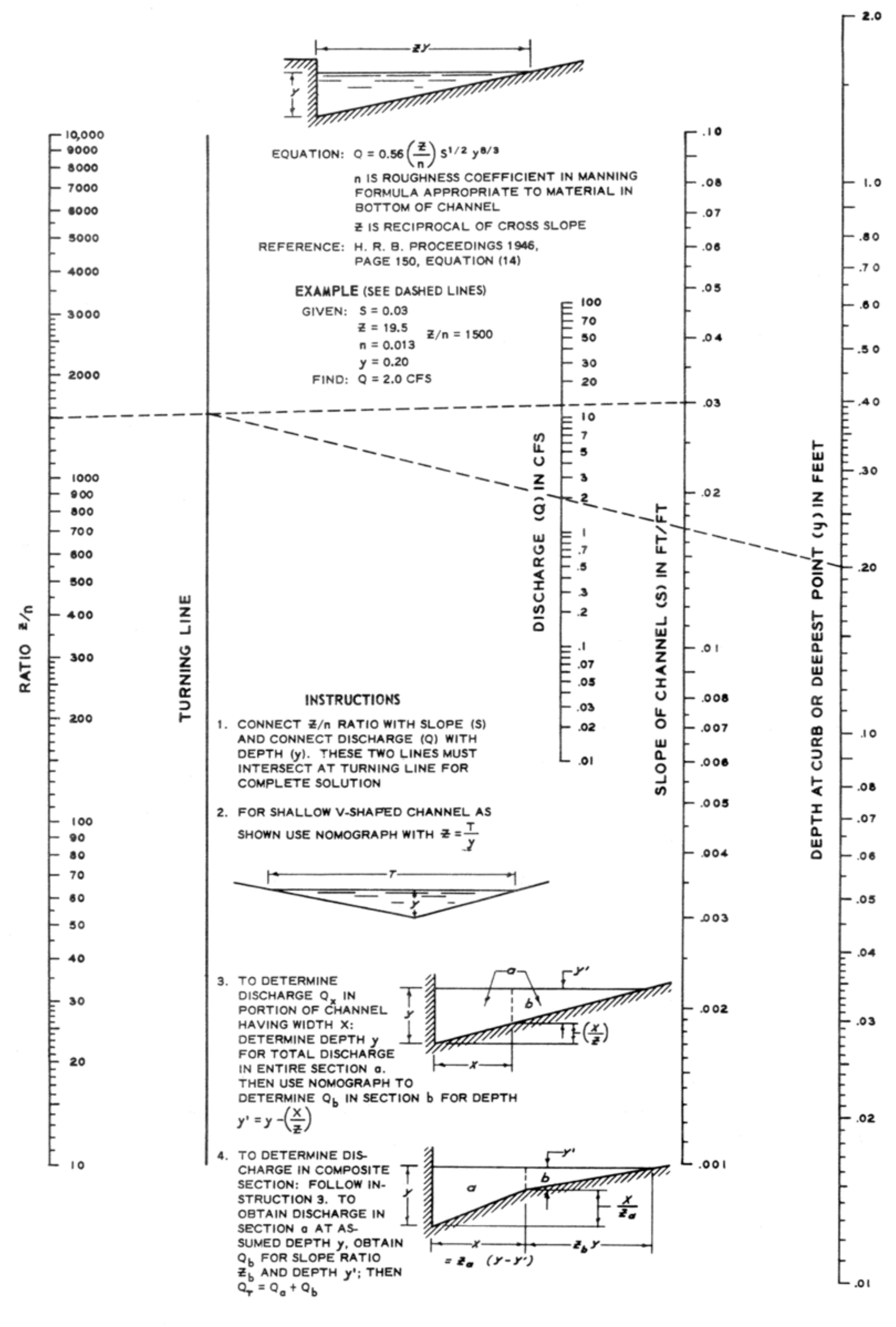
4-4.4.1 Precipitation which occurs upon city streets and adjacent areas must be rapidly and economically removed before it becomes a hazard to traffic. Water falling on the pavement surface itself is removed from the surface and concentrated in the gutters by the provision of an adequate crown. The surface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

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

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

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

Figure 4-41. Nomograph for Flow in Triangular Channels



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4-4.5 **Culverts**

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

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

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

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

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

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

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

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

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

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

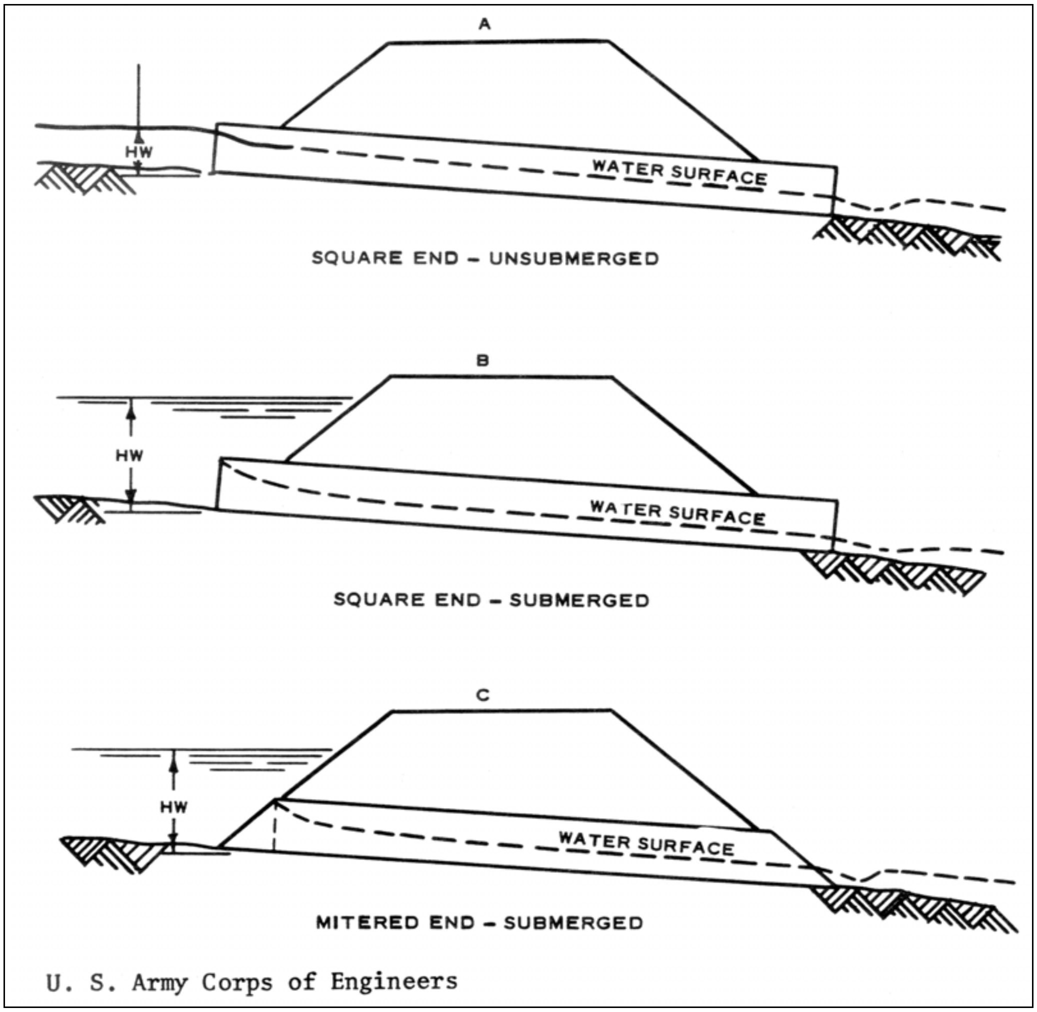
4-4.5.9 **Hydraulic design data for culverts**

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

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

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

Figure 4-42. Inlet Control



4-4.5.11 **Outlet control**

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

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

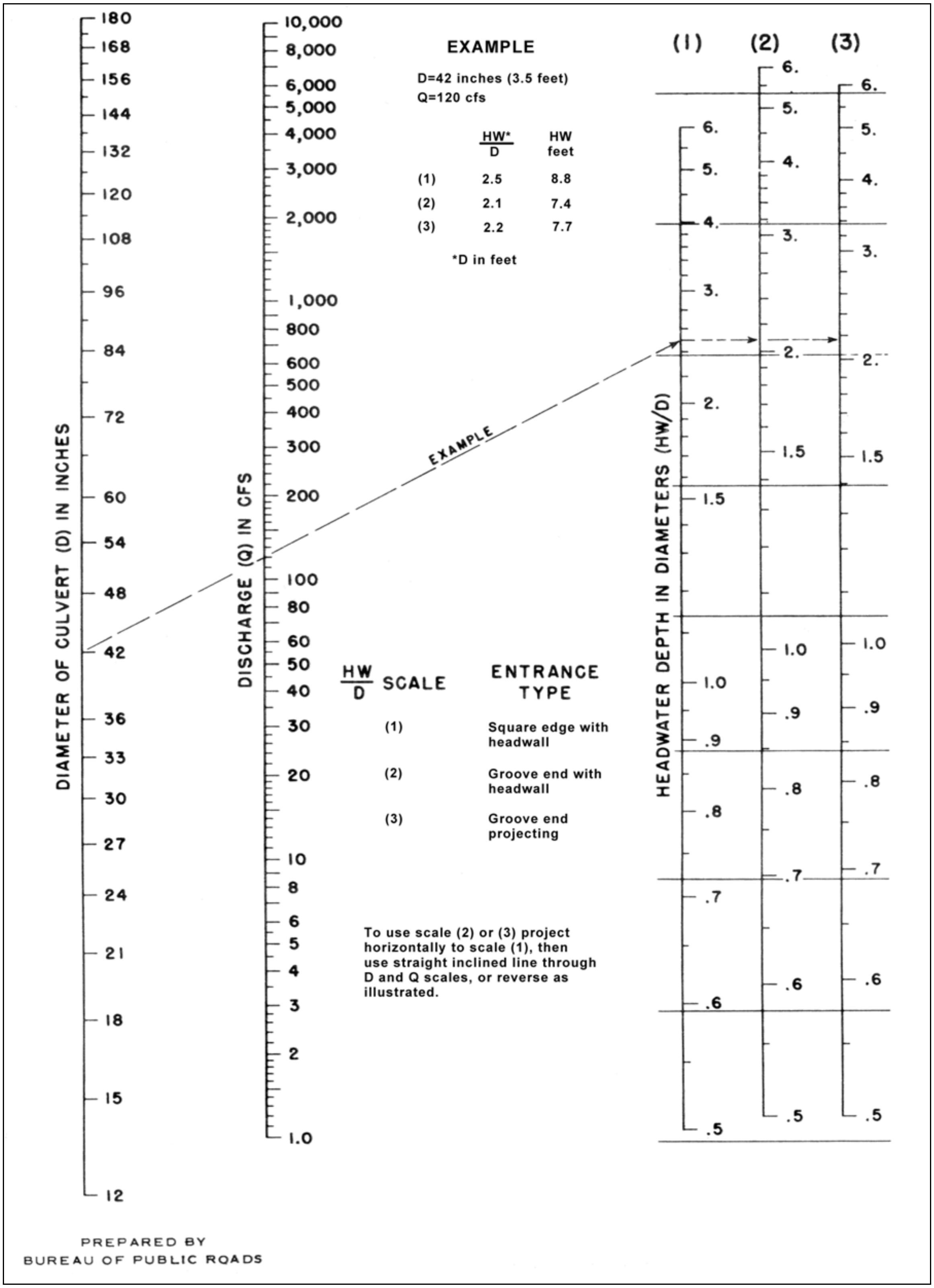


Figure 4-44. Headwater Depth for Oval Concrete Pipe Culverts Long Axis Vertical with Inlet Control

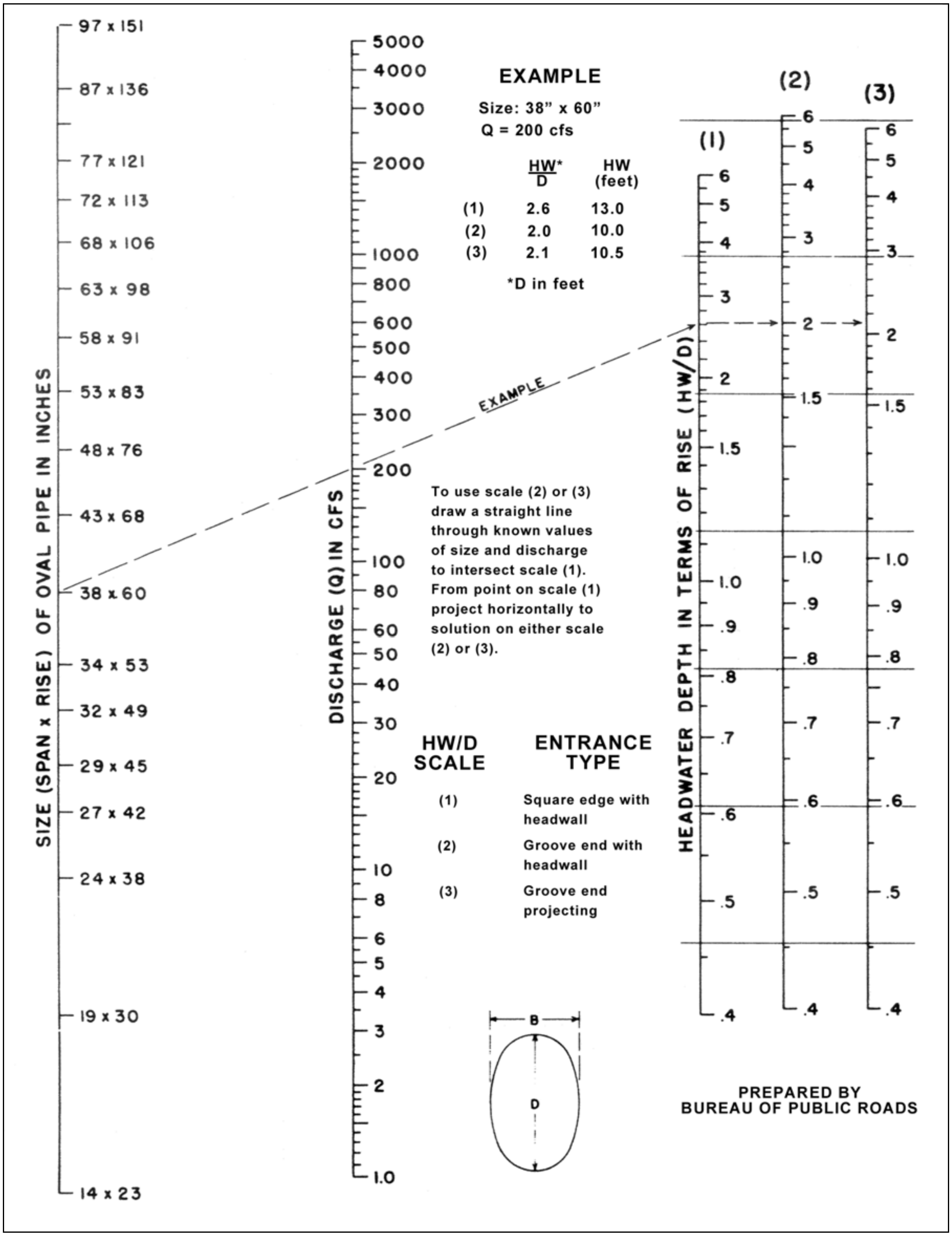


Figure 4-45. Headwater Depth for Oval Concrete Pipe Culverts Long Axis Horizontal with Inlet Control

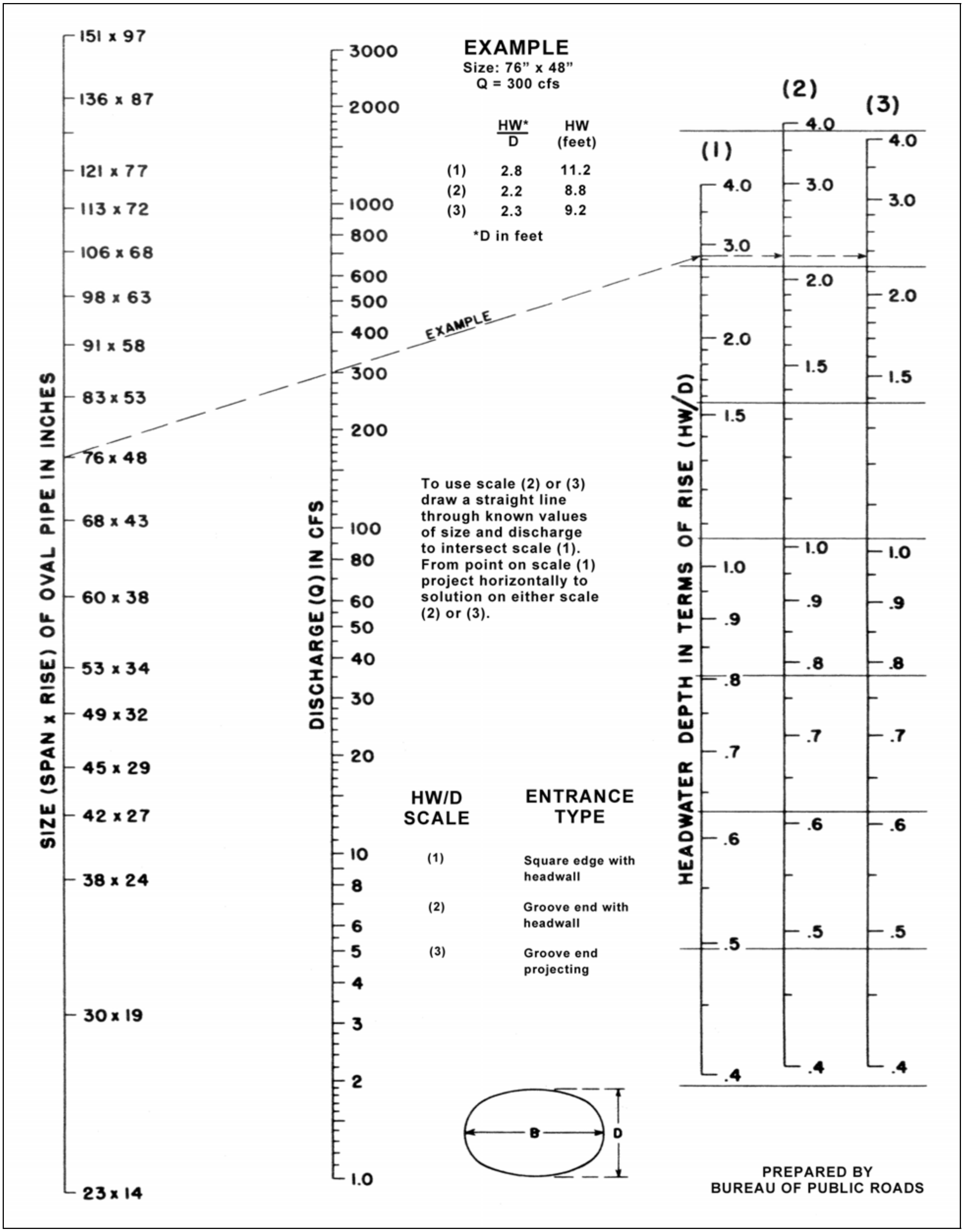


Figure 4-46. Headwater Depth for Corrugated Metal Pipe Culverts with Inlet Control

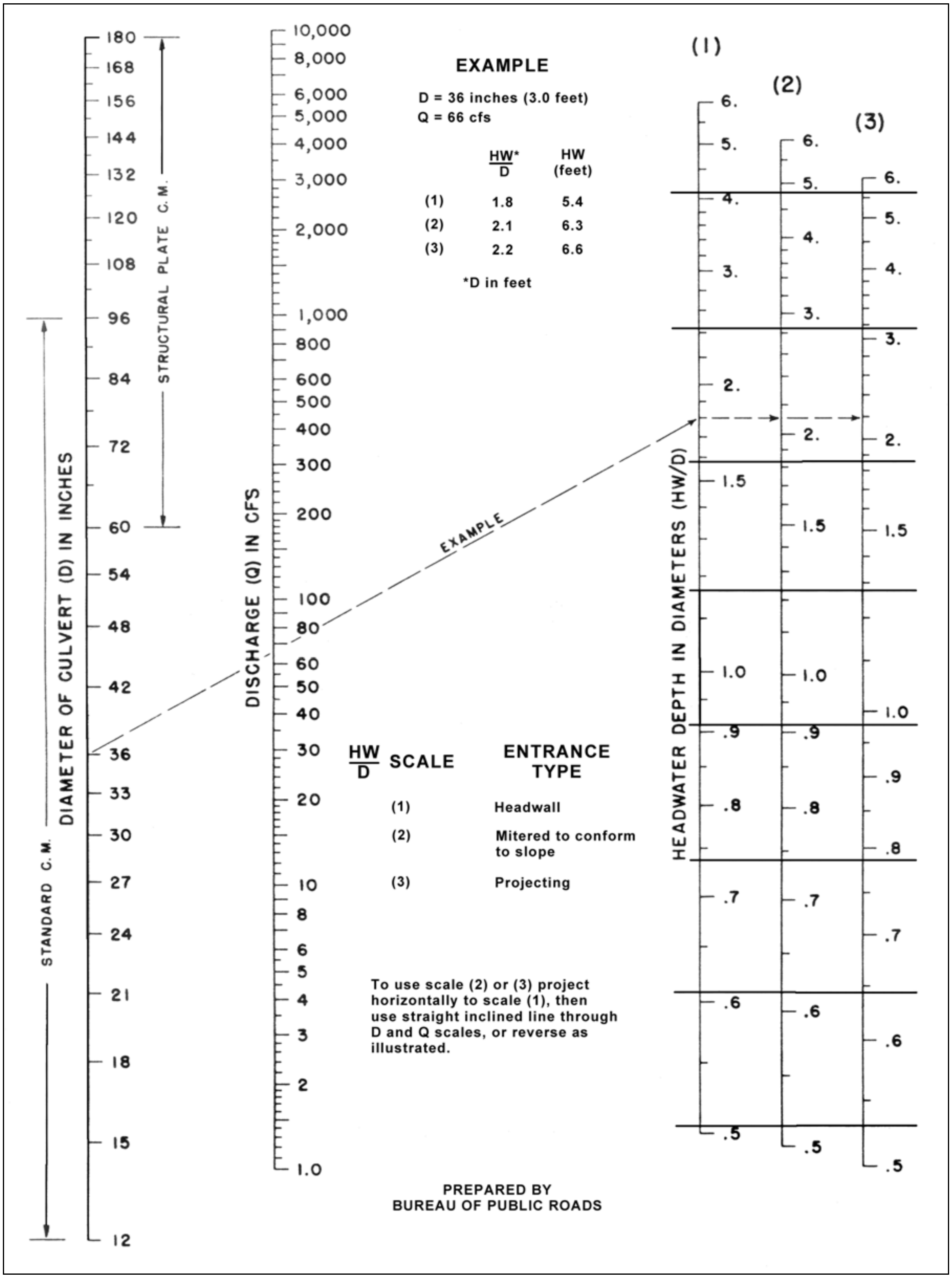


Figure 4-47. Headwater Depth for Structural Plate and Standard Corrugated Metal Pipe-Arch Culverts with Inlet Control

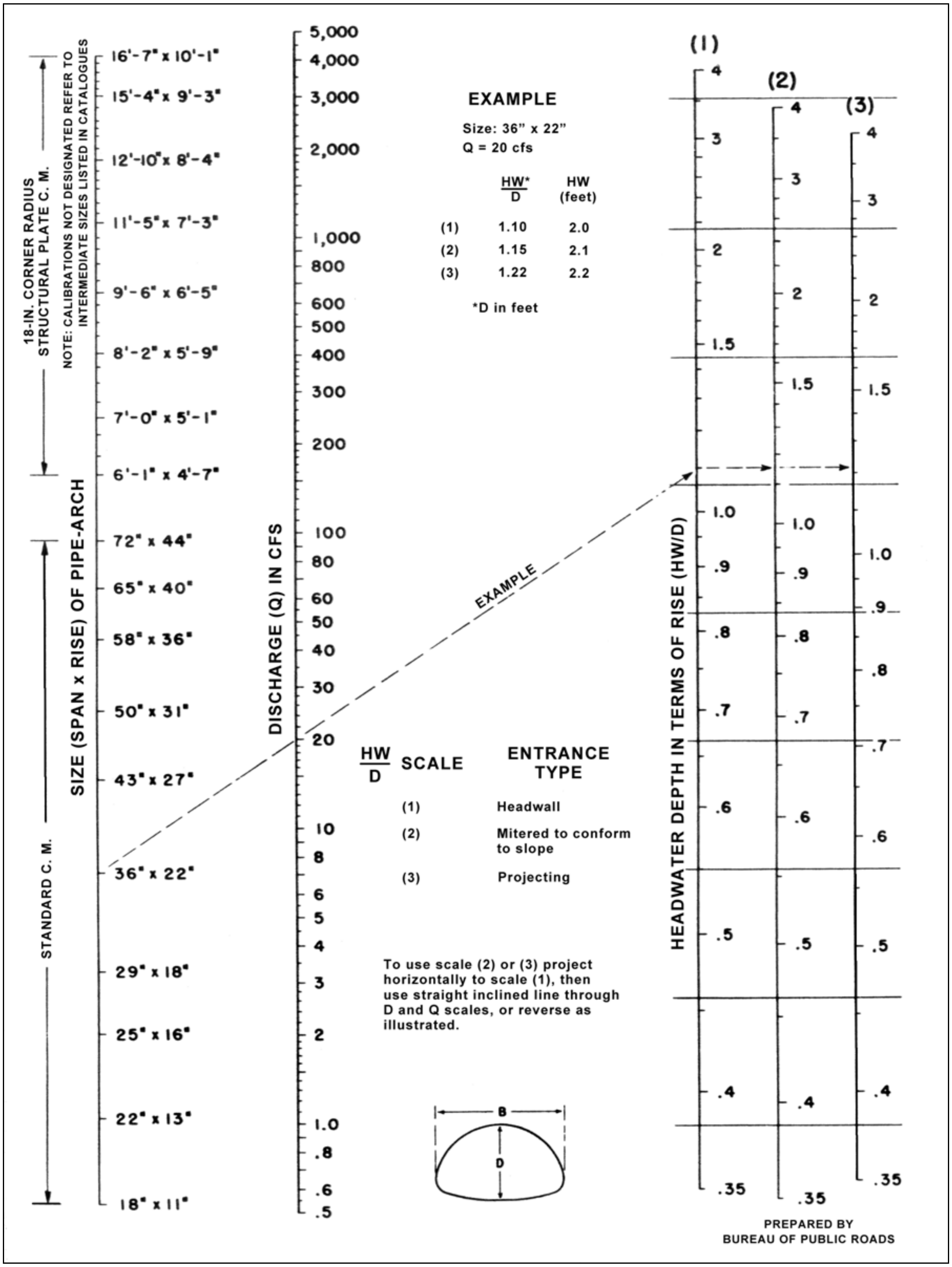


Figure 4-48. Headwater Depth for Box Culverts with Inlet Control

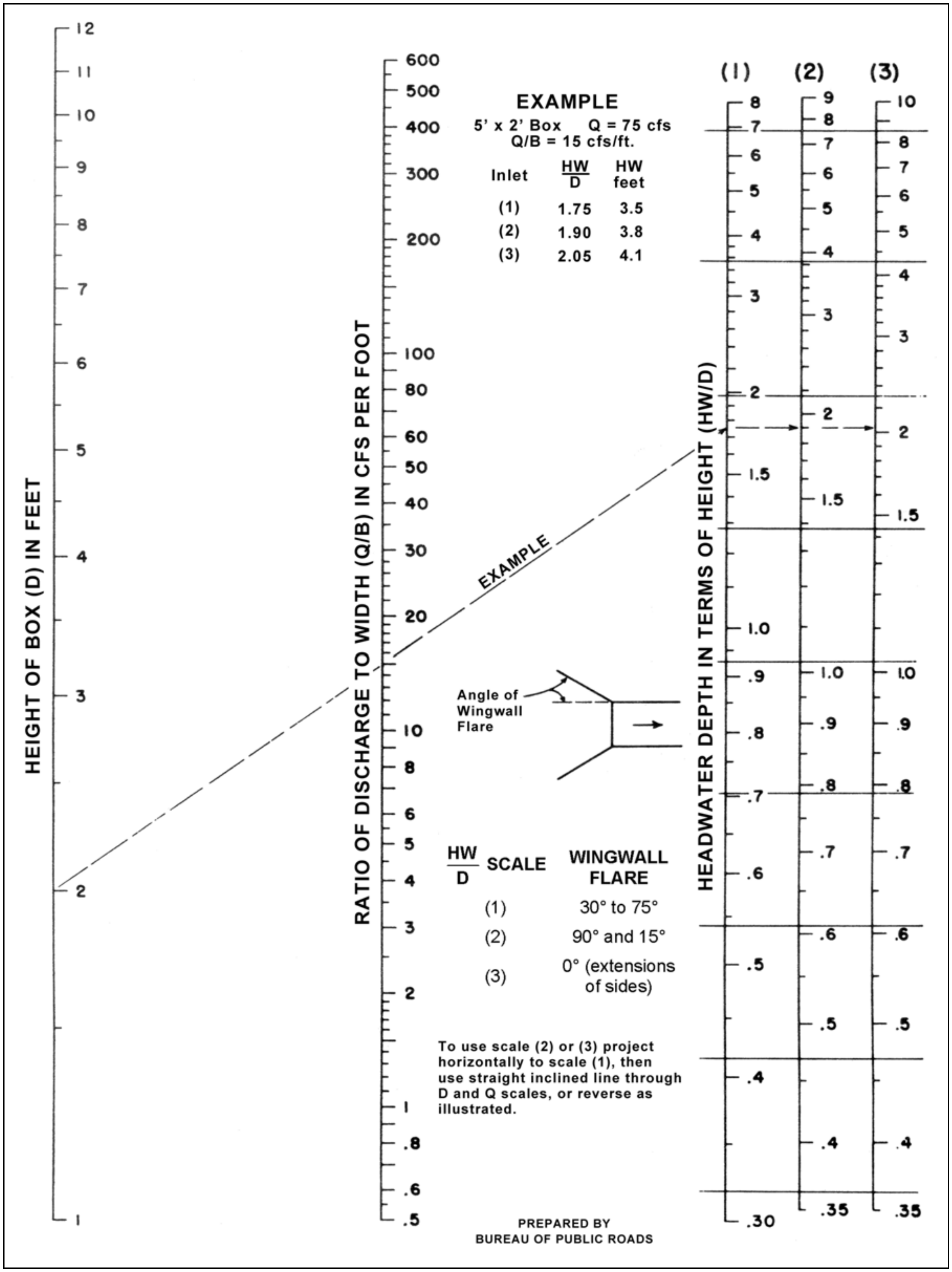


Figure 4-49. Headwater Depth for Corrugated Metal Pipe Culverts with Tapered Inlet-Inlet Control

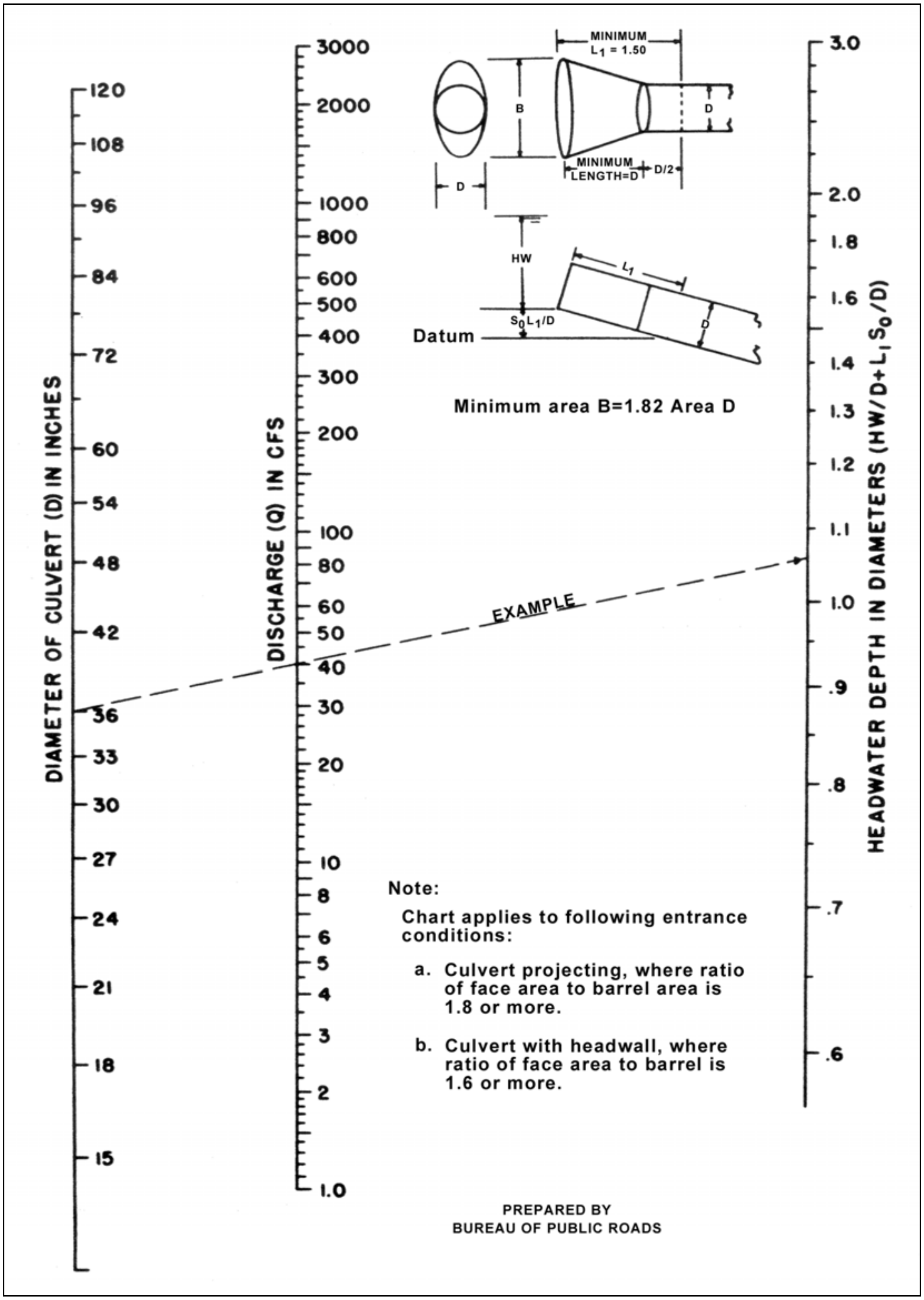


Figure 4-50. Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control

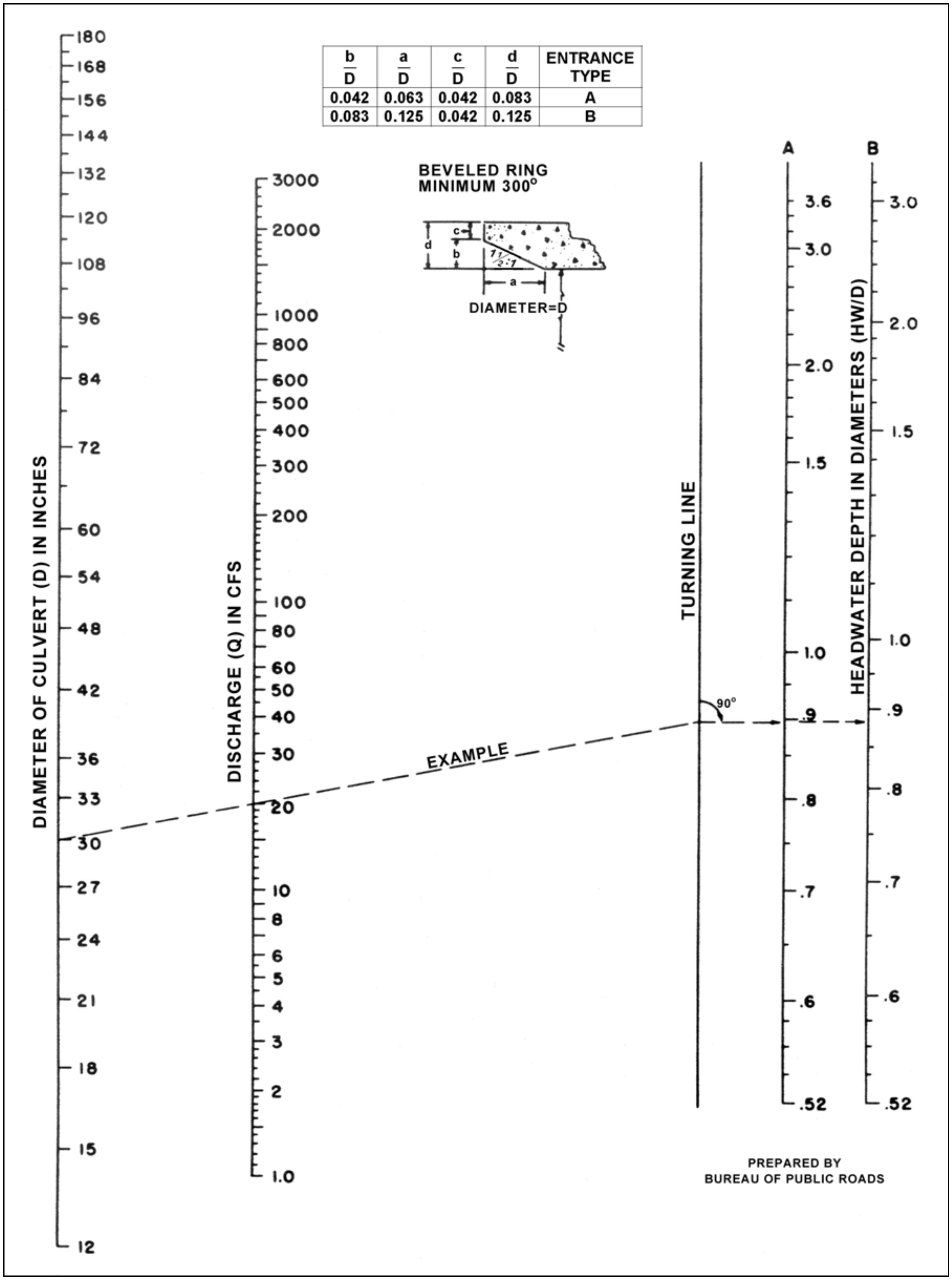
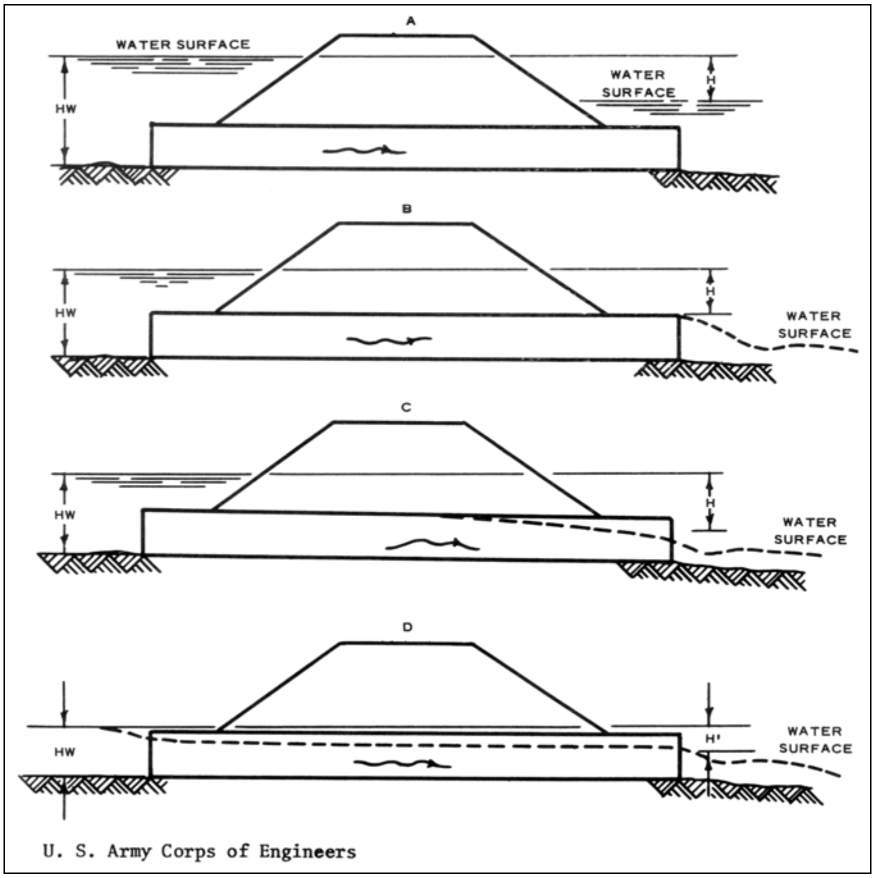


Figure 4-51. Outlet Control



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

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

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

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

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

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

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

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

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

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

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

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

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

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

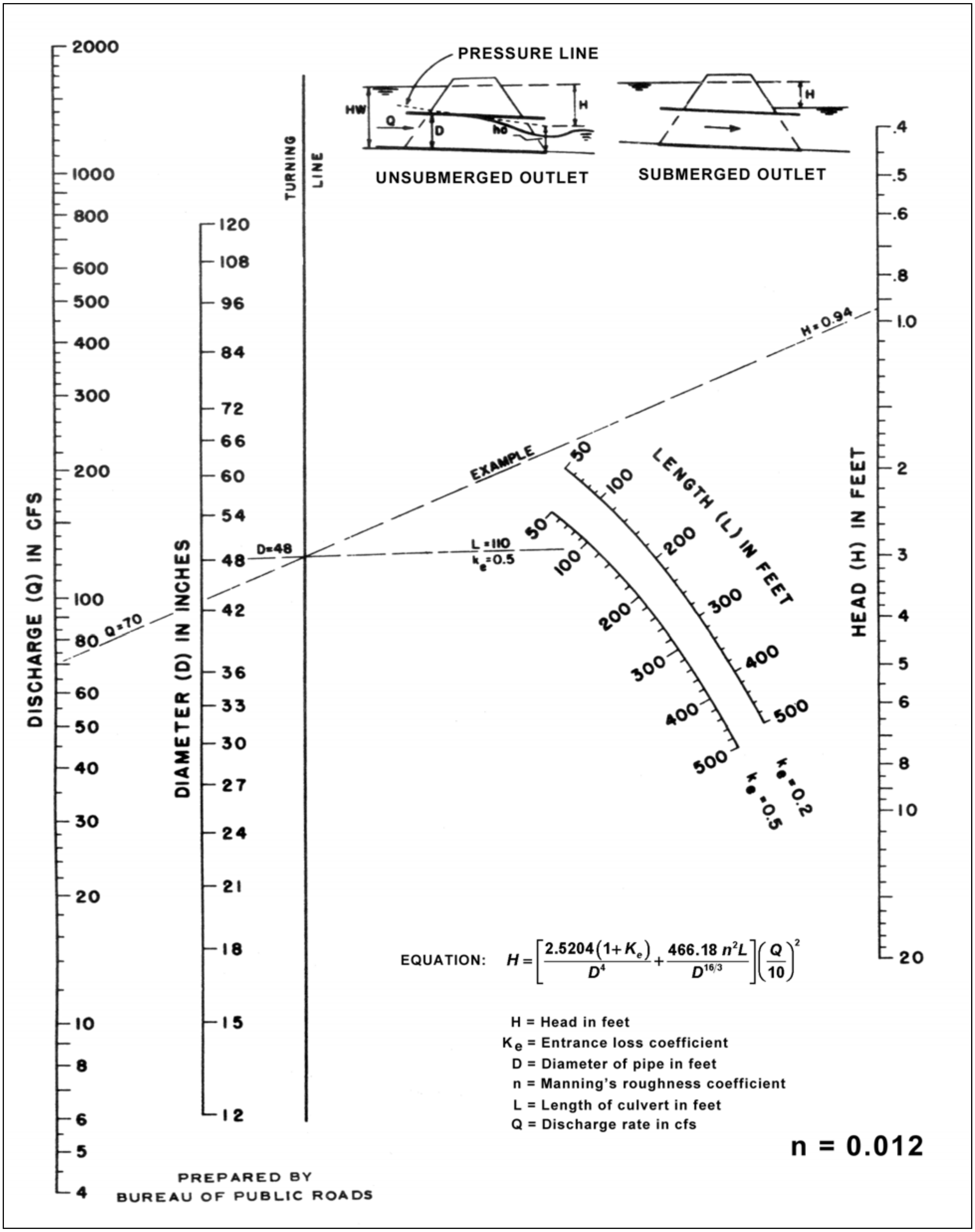


Figure 4-53. Head for Oval Circular Pipe Culverts Long Axis Horizontal or Vertical Flowing Full, n = 0.012

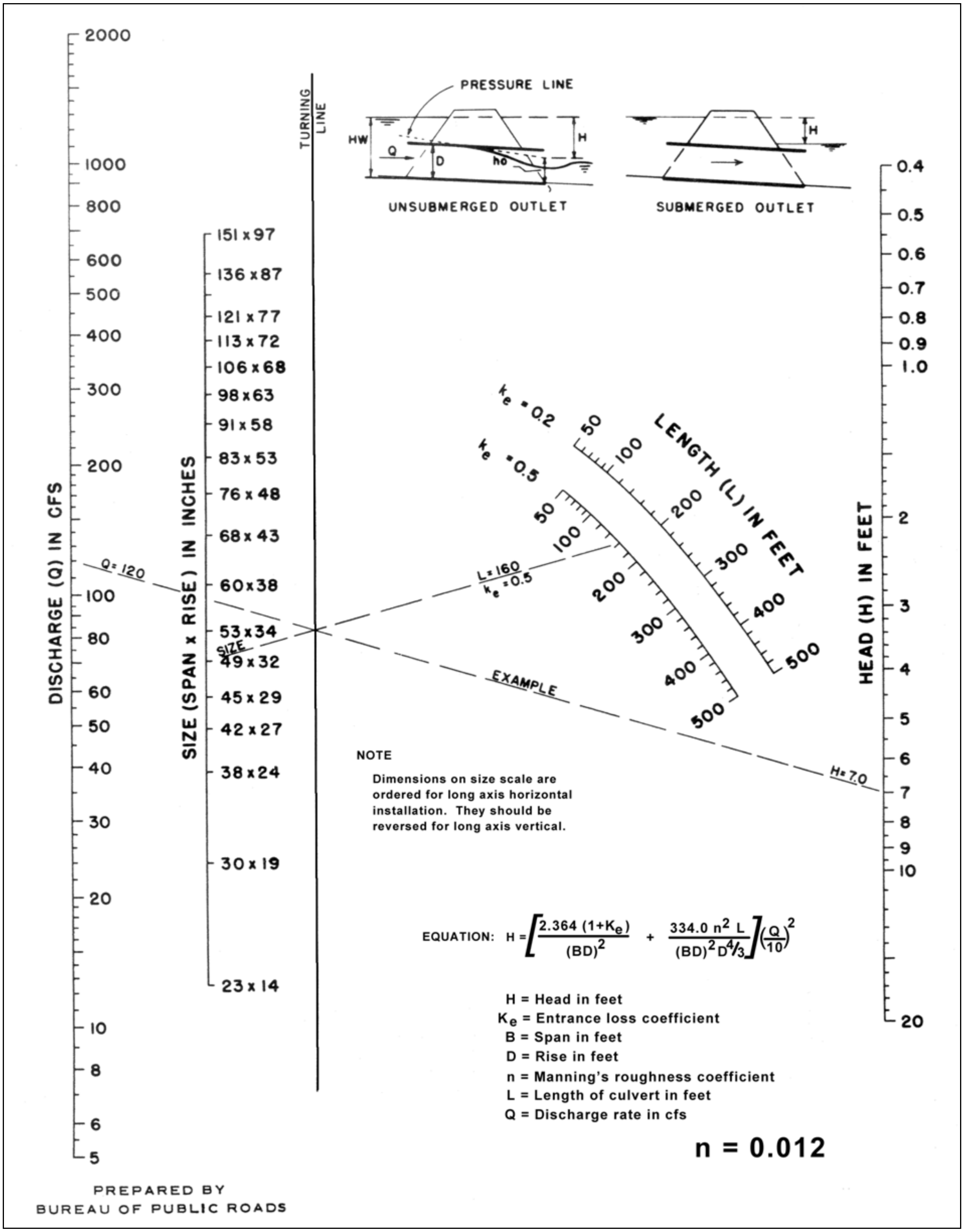


Figure 4-54. Head for Circular Pipe Culverts Flowing Full, n = 0.024

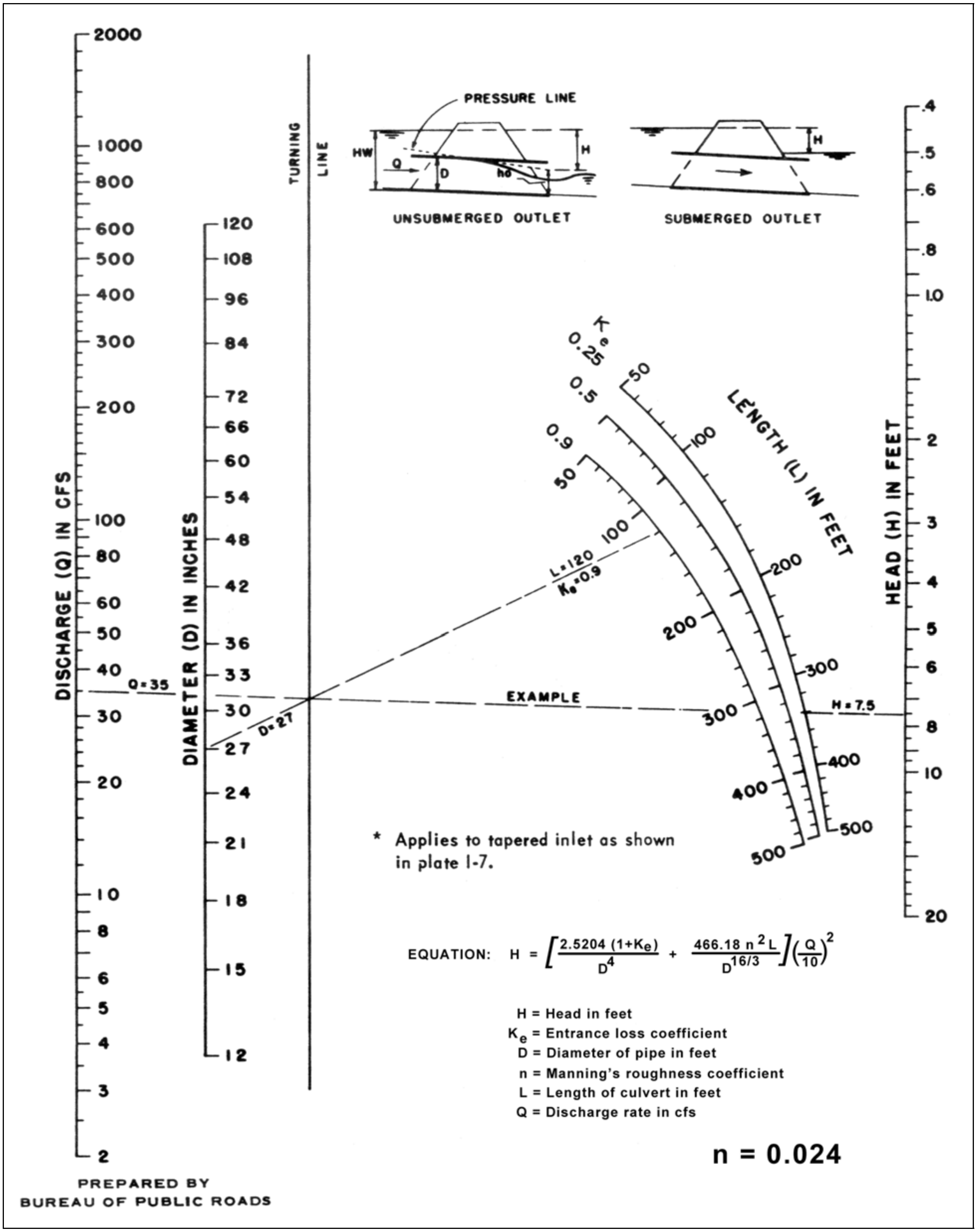
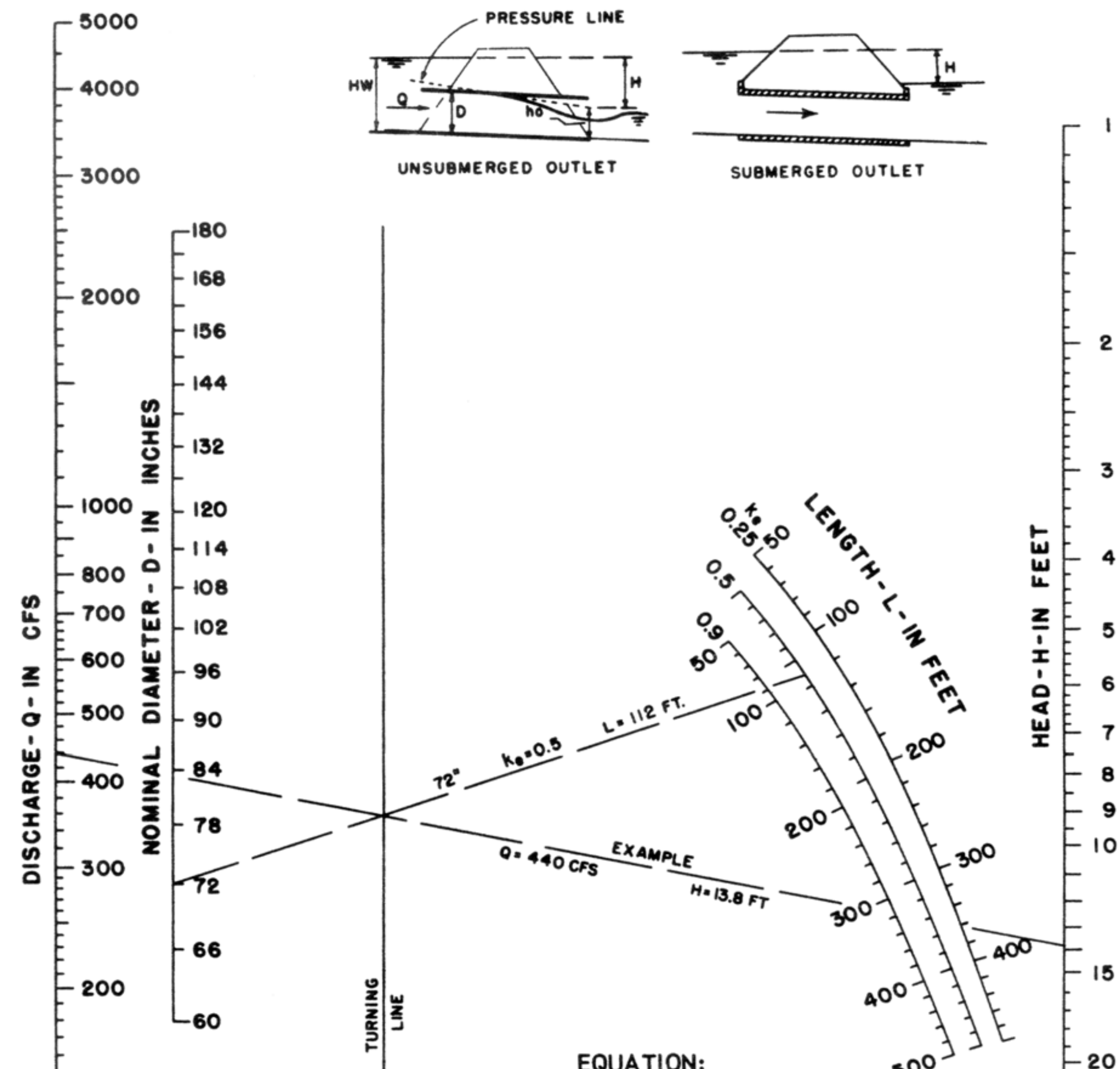


Figure 4-55. Head for Circular Pipe Culverts Flowing Full, n = 0.0328 to 0.0302



NOTE:

Computed for actual diameters given in fabricators' catalogs.

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

EQUATION:

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

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

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n = 0.0328 TO 0.0302

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

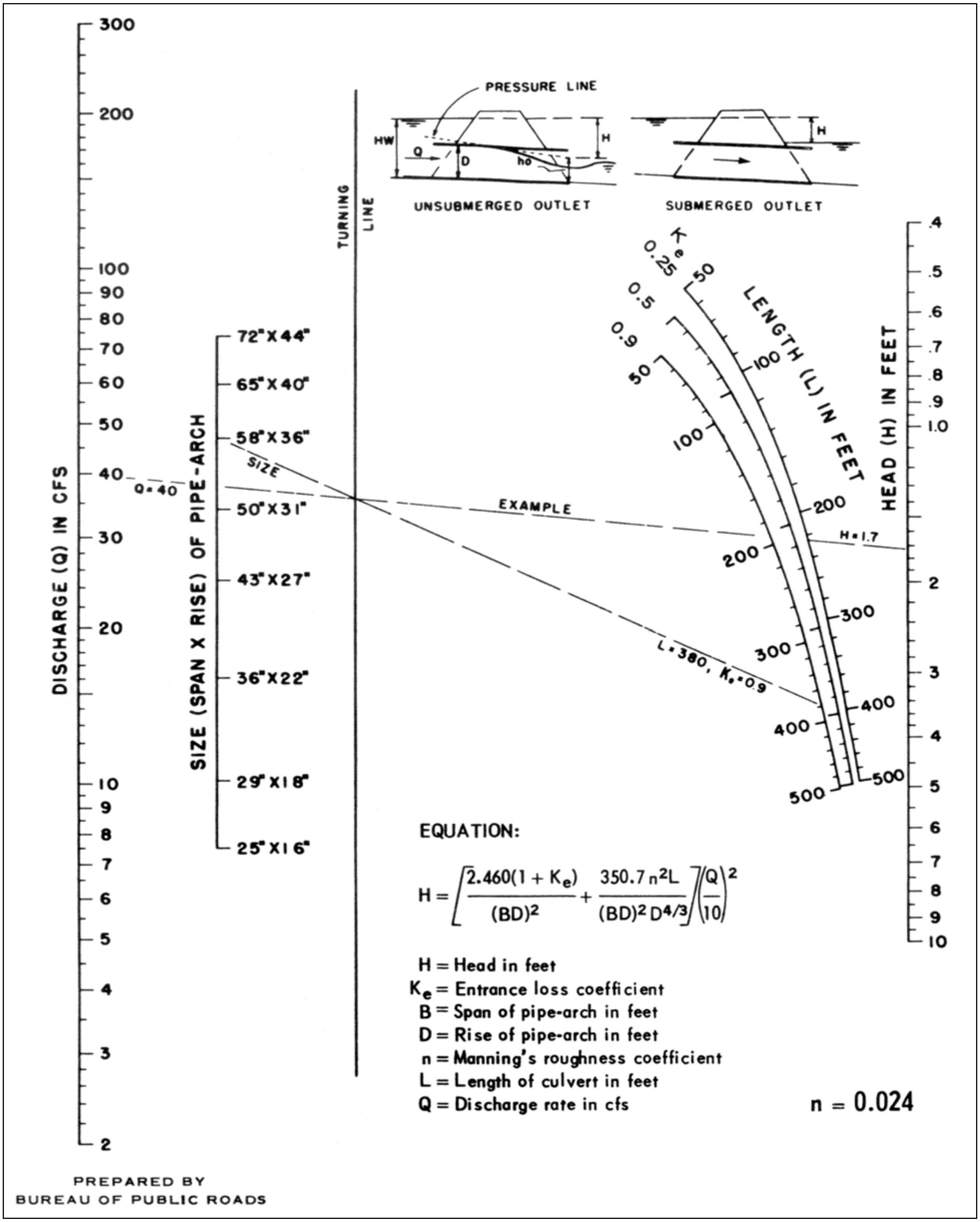


Figure 4-57. Head for Field-Bolted Structural Plate Pipe-Arch Culverts 18 in. Corner Radius Flowing Full, n = 0.0327 to 0.0306

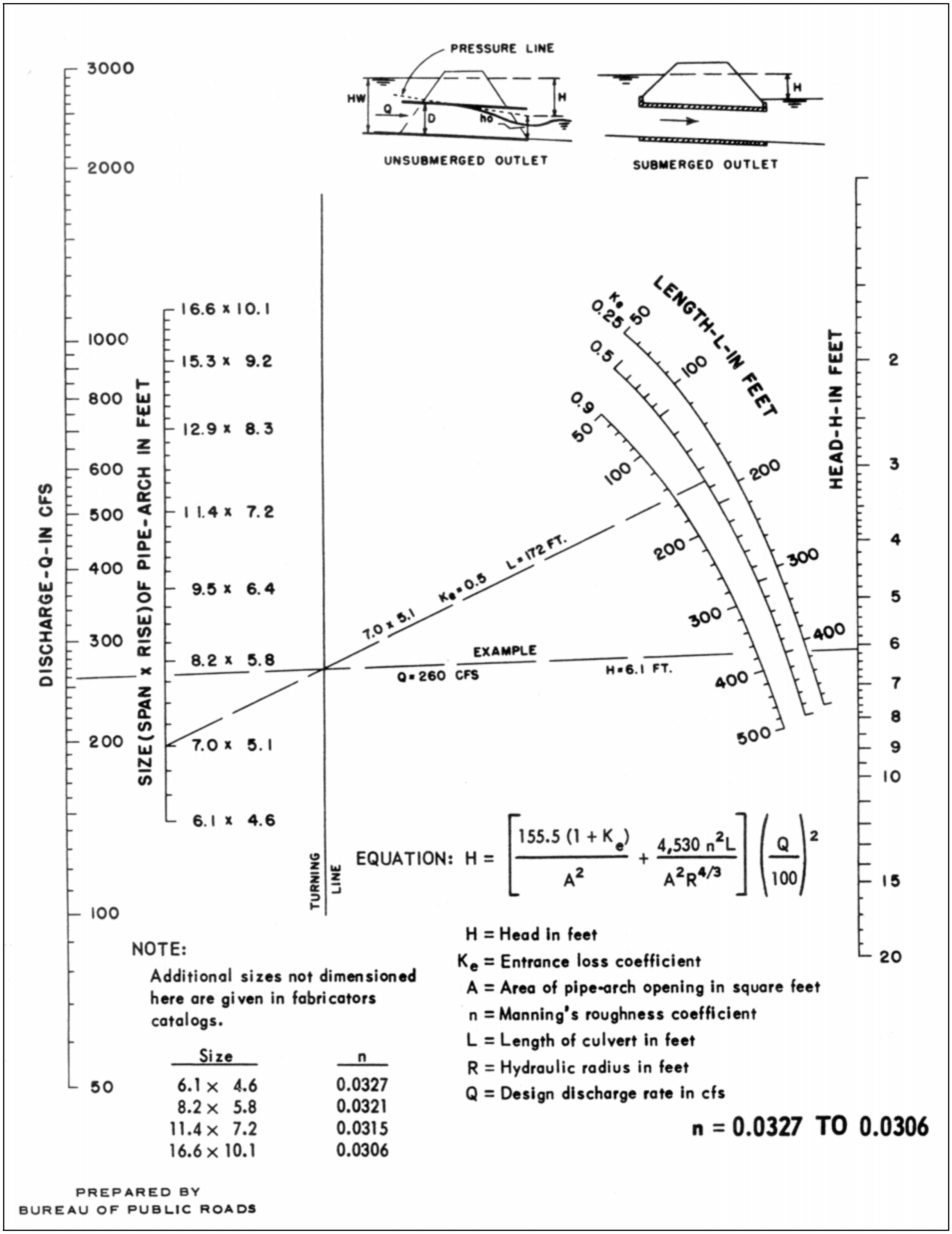


Figure 4-58. Head for Concrete Box Culverts Flowing Full, n = 0.012

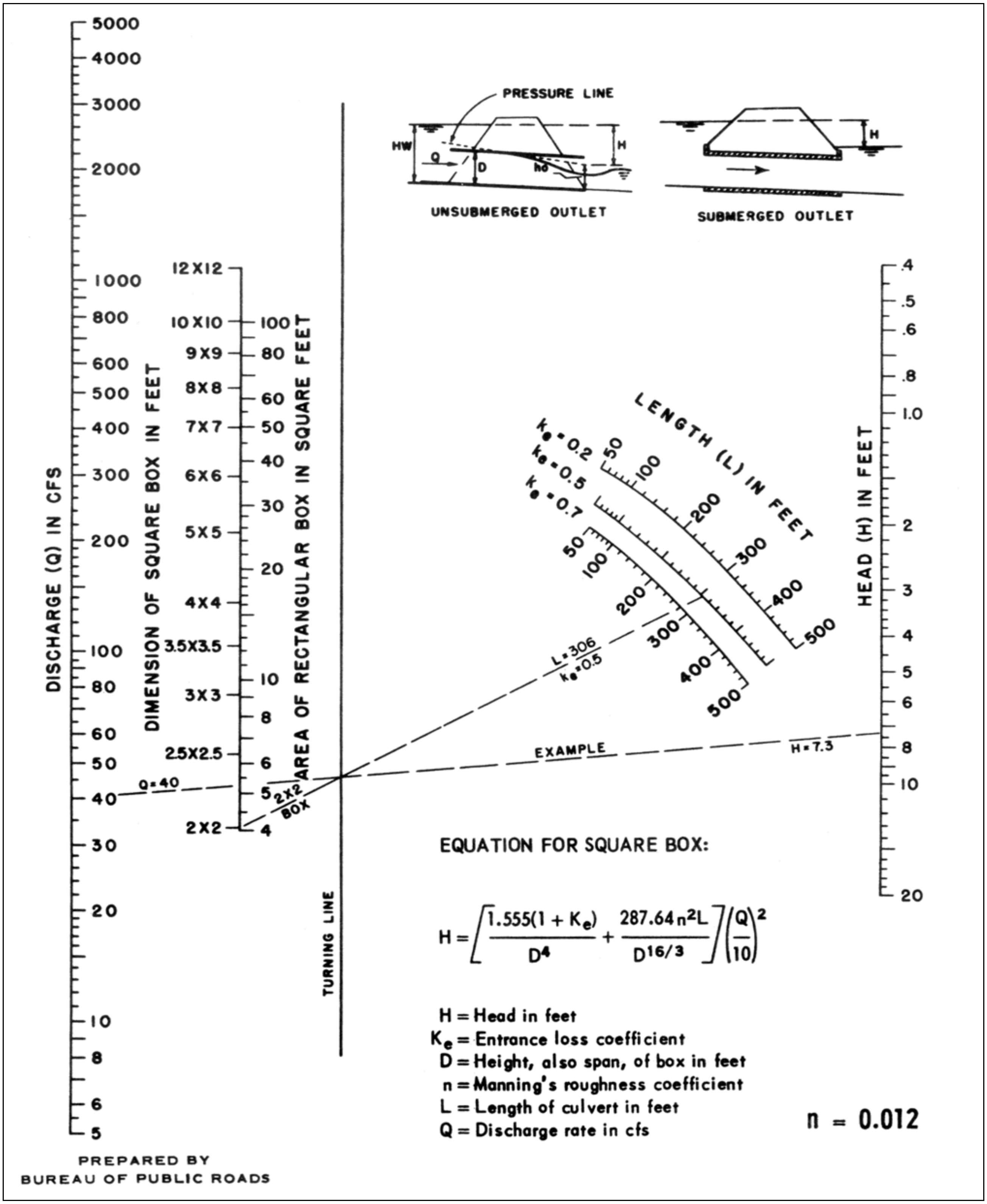
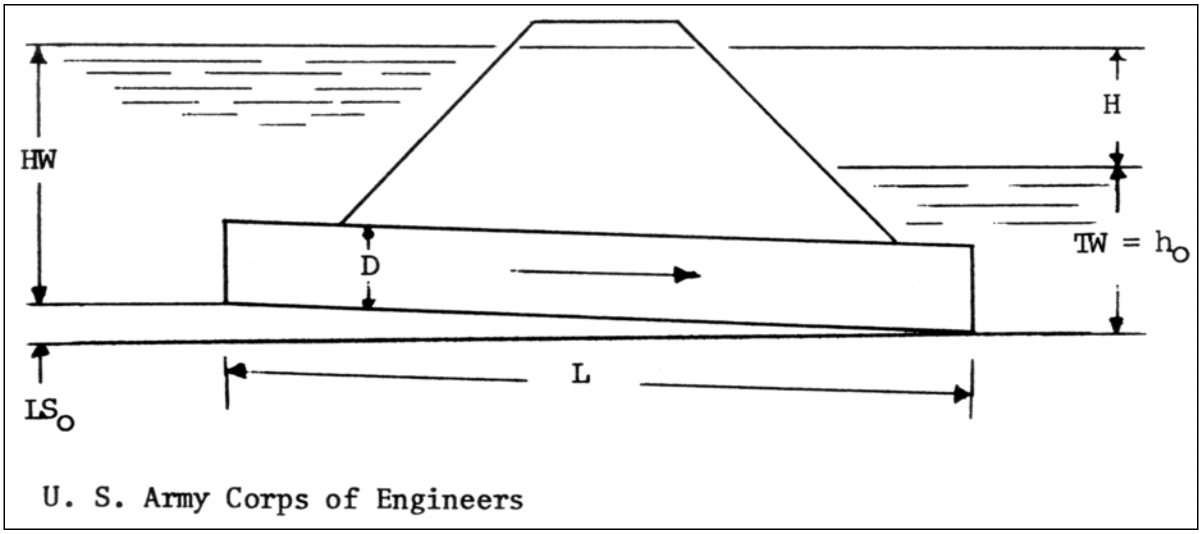


Figure 4-59. Tailwater Elevation at or Above Top of Culvert



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

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

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

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

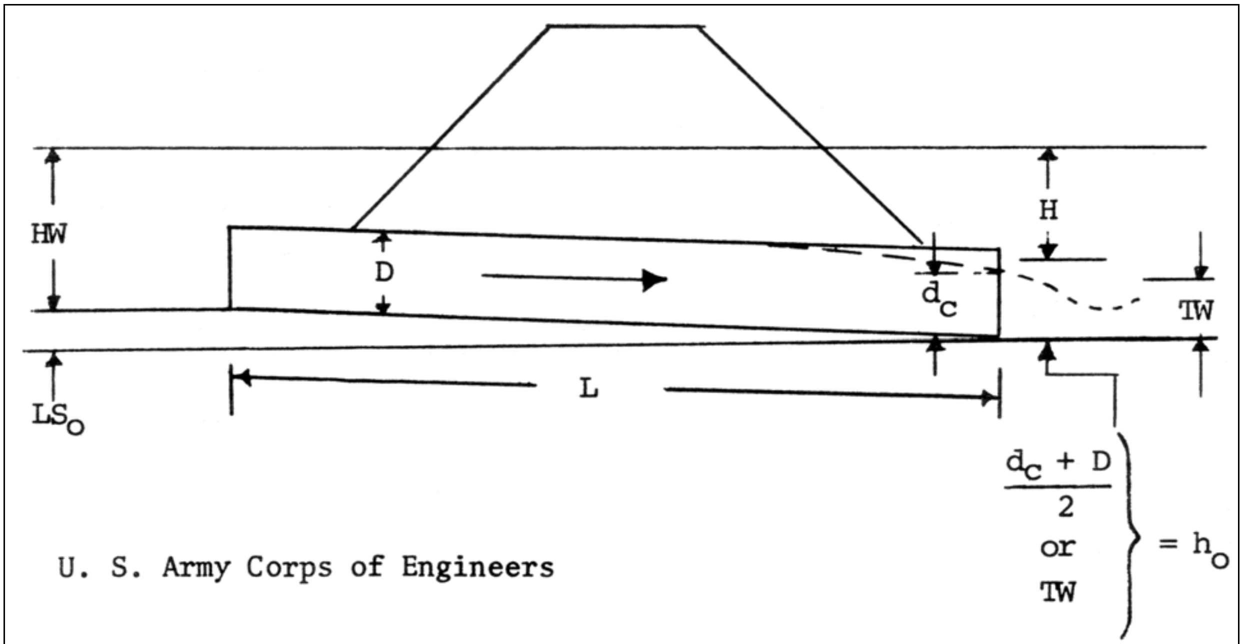


Figure 4-61. Circular Pipe—Critical Depth

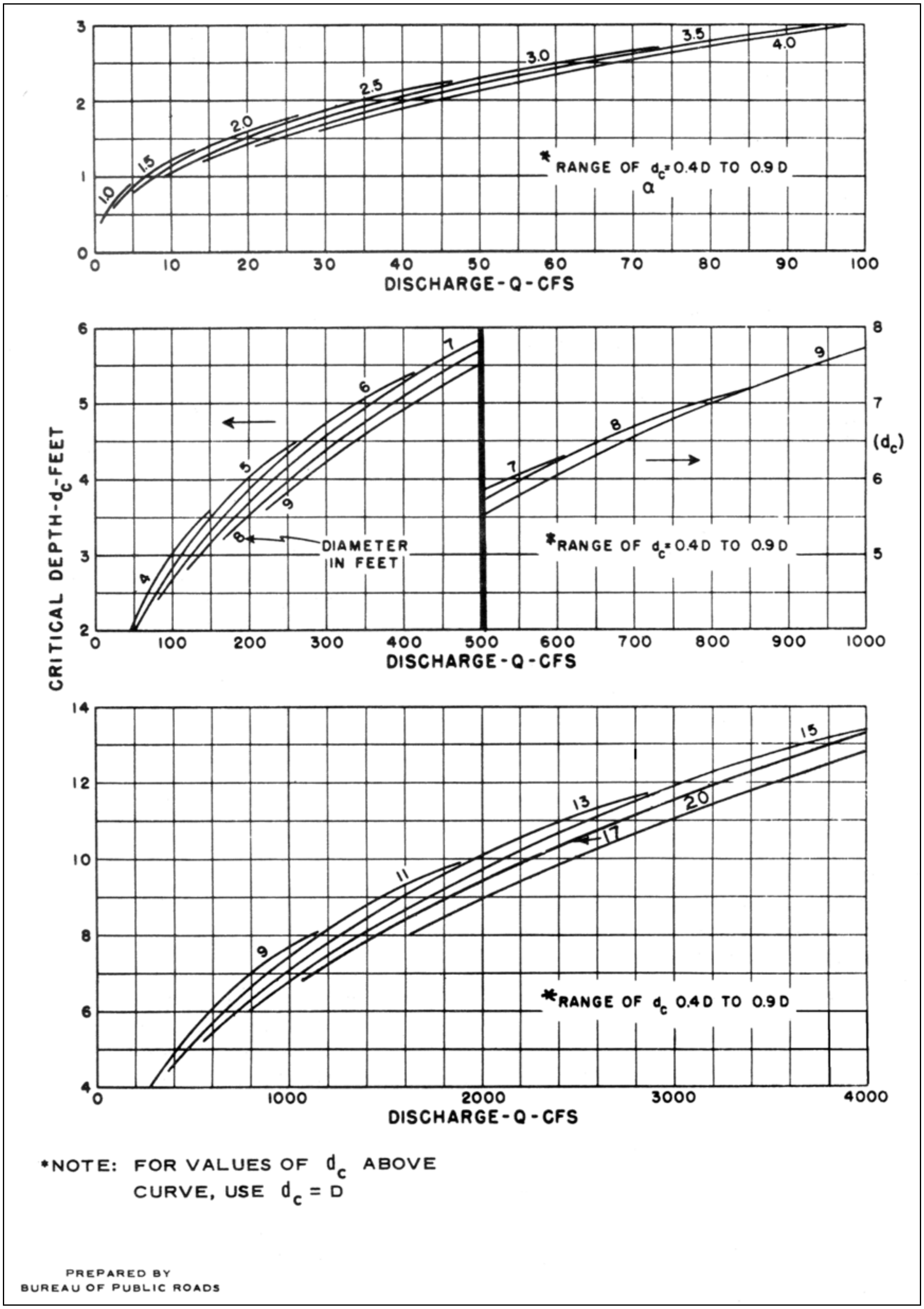


Figure 4-62. Oval Concrete Pipe Long Axis Horizontal Critical Depth

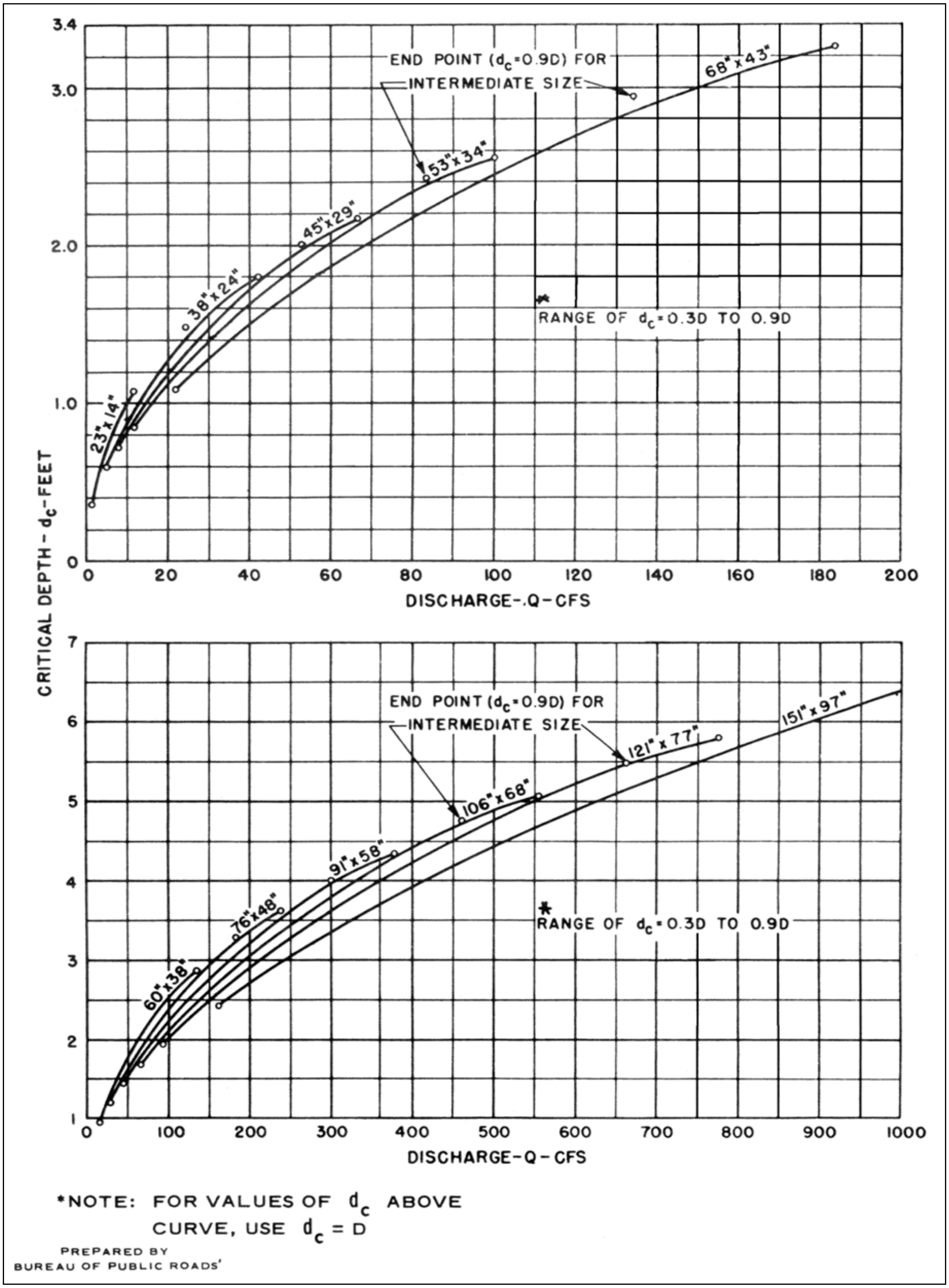


Figure 4-63. Oval Concrete Pipe Long Axis Vertical Critical Depth

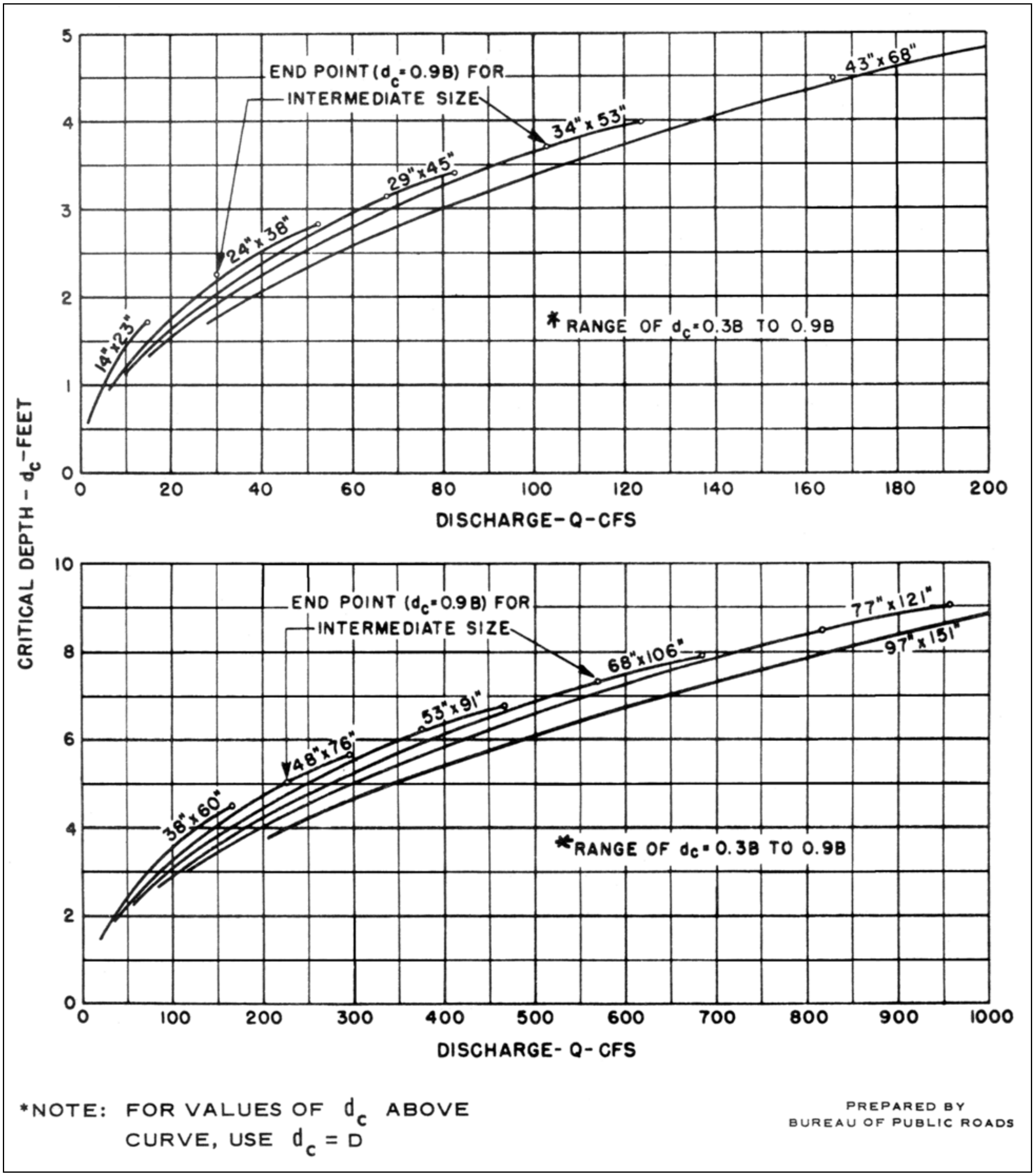
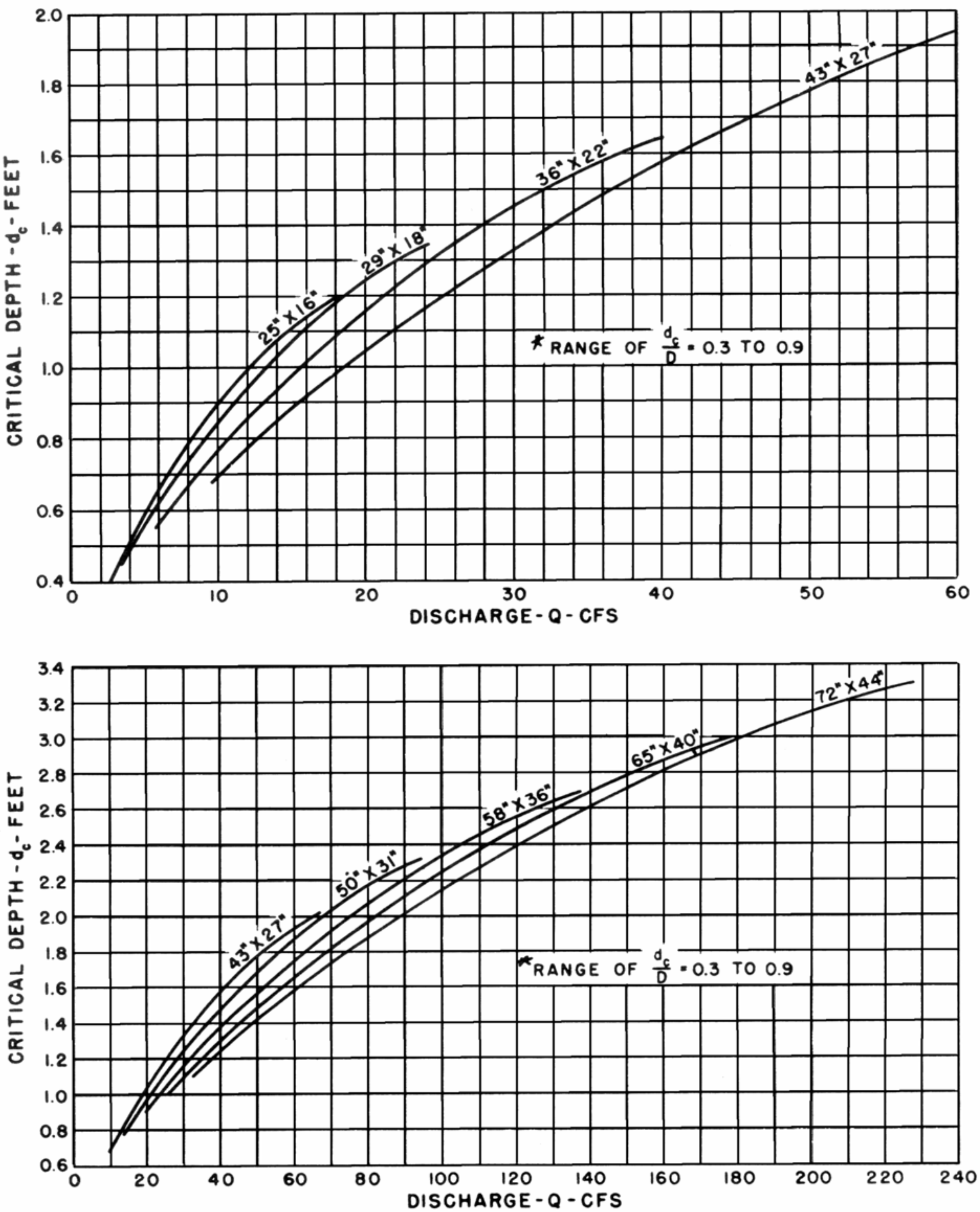


Figure 4-64. Standard Corrugated Metal Pipe-Arch Critical Depth



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

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

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

a. Step 1: List given data.

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

(2) Approximate length of culvert, in feet.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

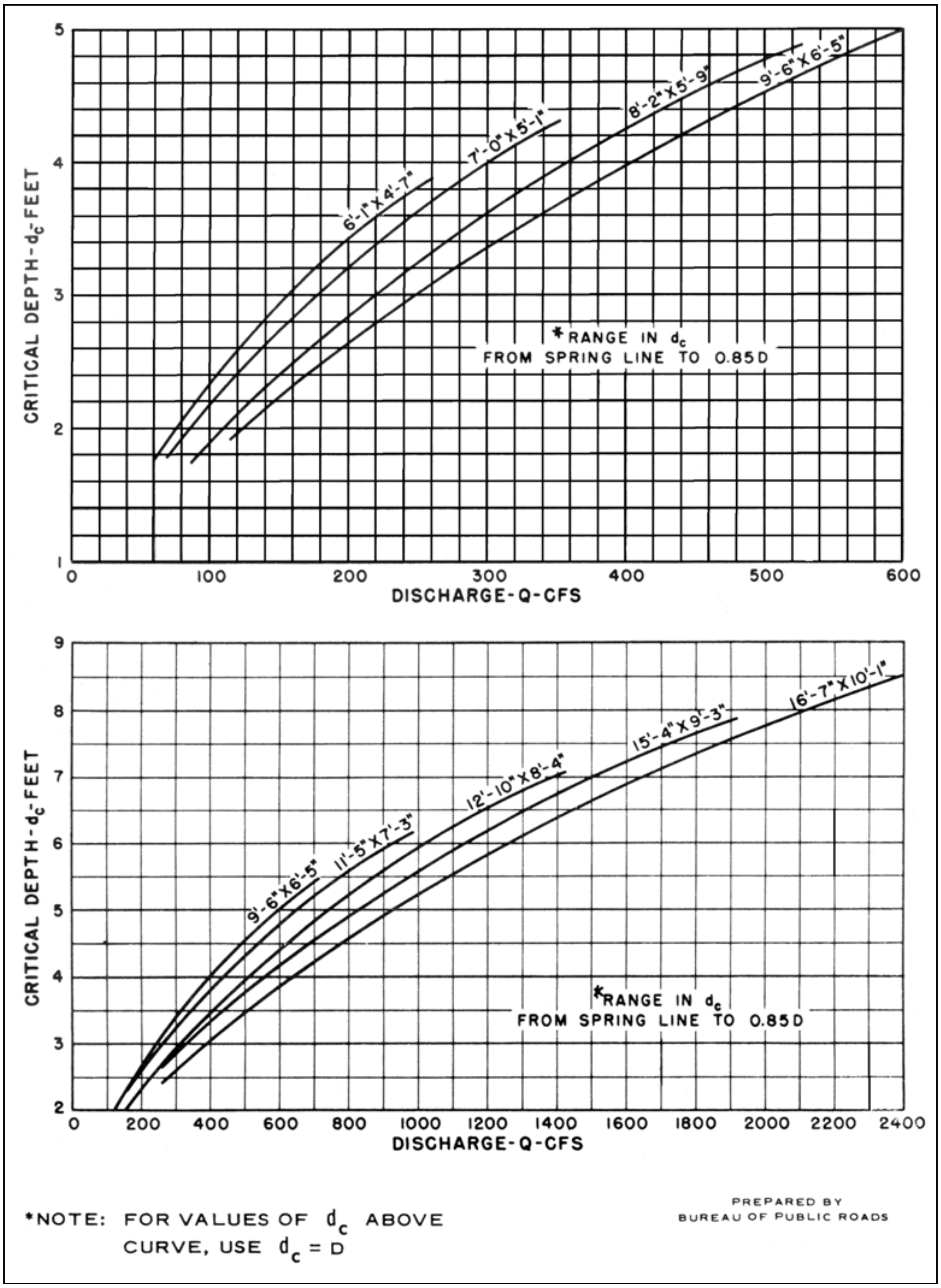
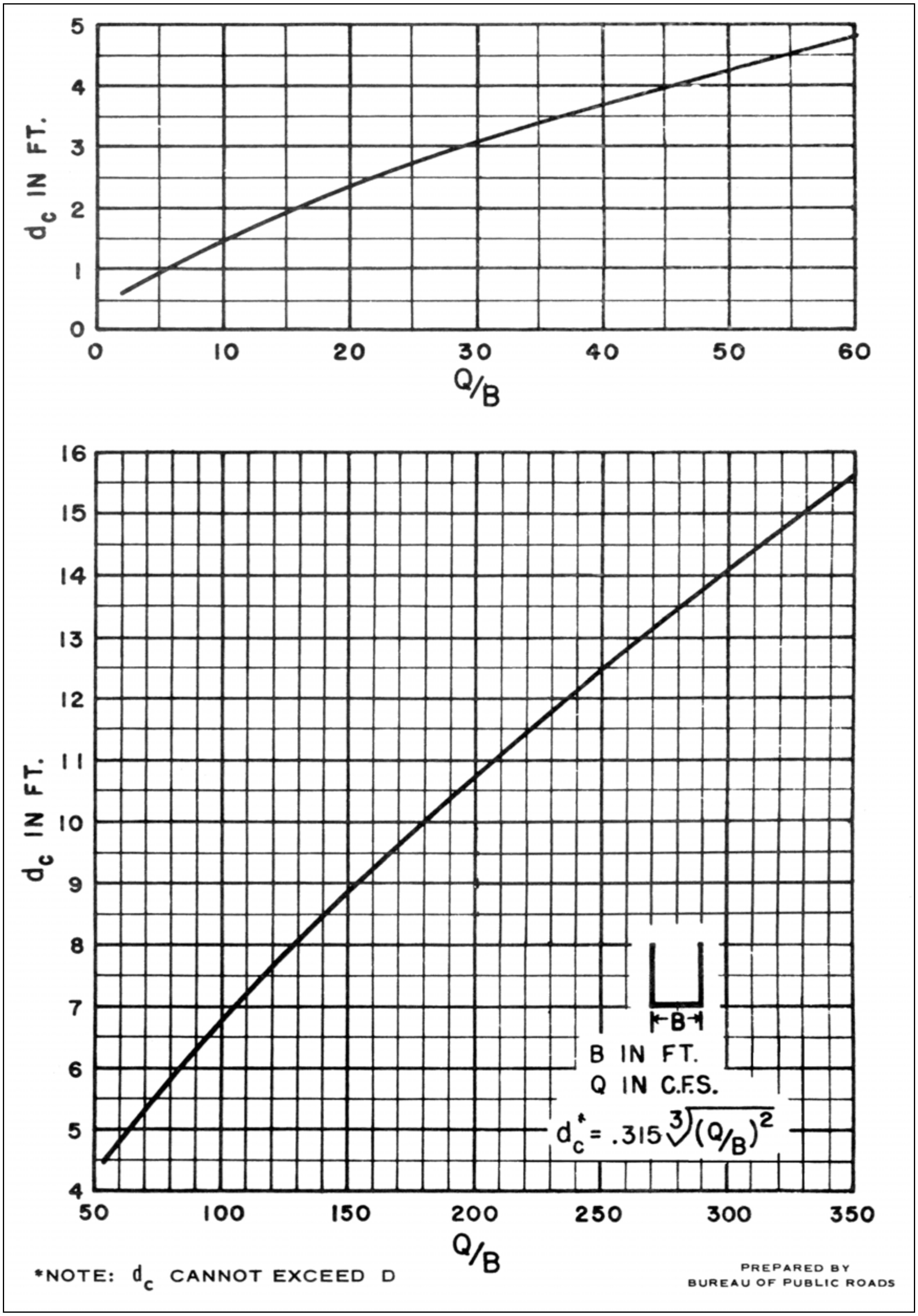


Figure 4-66. Critical Depth Rectangular Section



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

4-4.5.13.1 To determine headwater.

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

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

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

4-4.5.13.2 To determine culvert size.

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

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

4-4.5.13.3 To determine discharge.

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

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

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

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

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

a. Locate appropriate nomograph for type of culvert selected.

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

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

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

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

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

(See Section 4-4.5.14.2 for n values.)

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

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

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

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

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

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

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

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

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

4-4.6 Underground Hydraulic Design

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

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

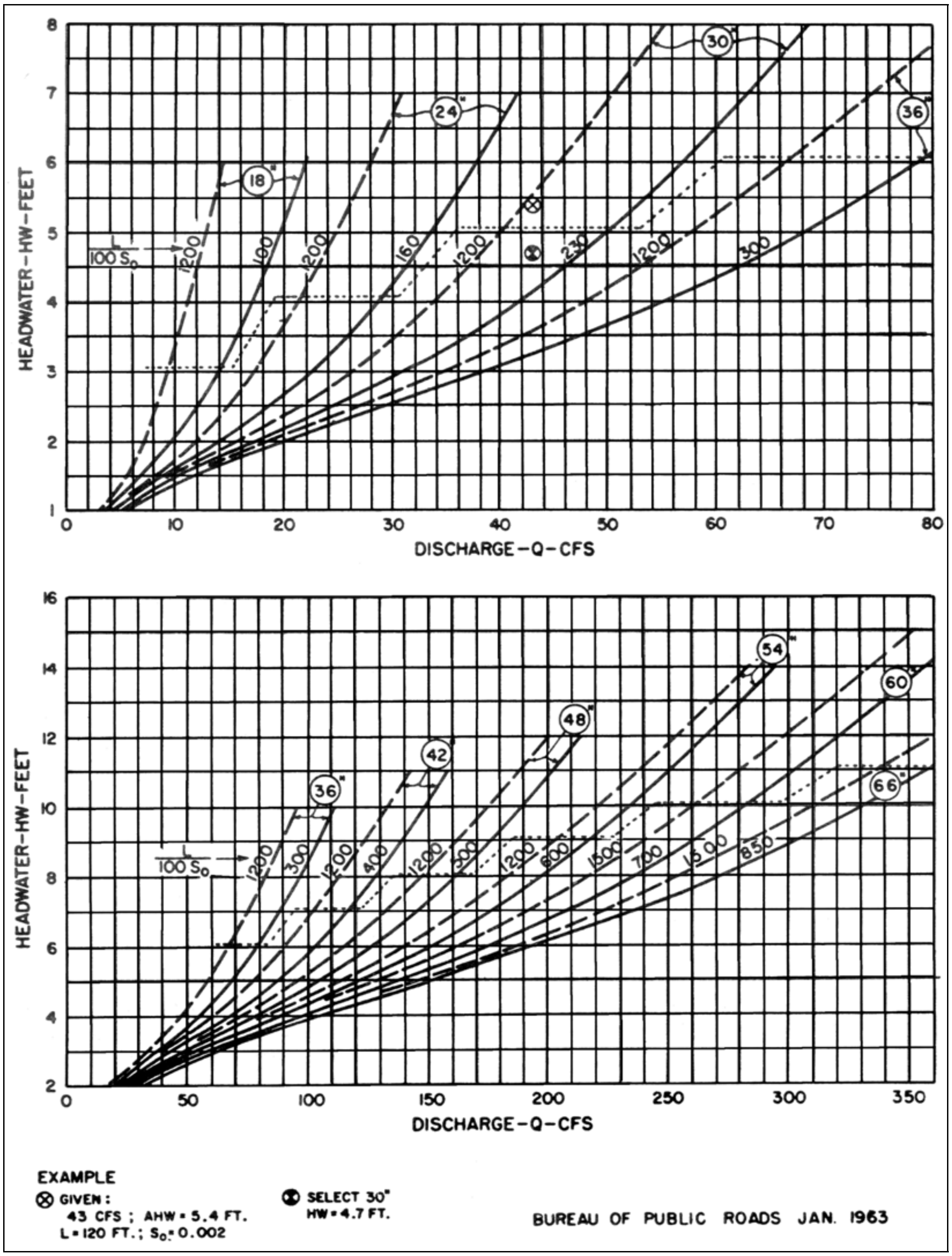


Figure 4-68. Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 60 in. to 180 in.

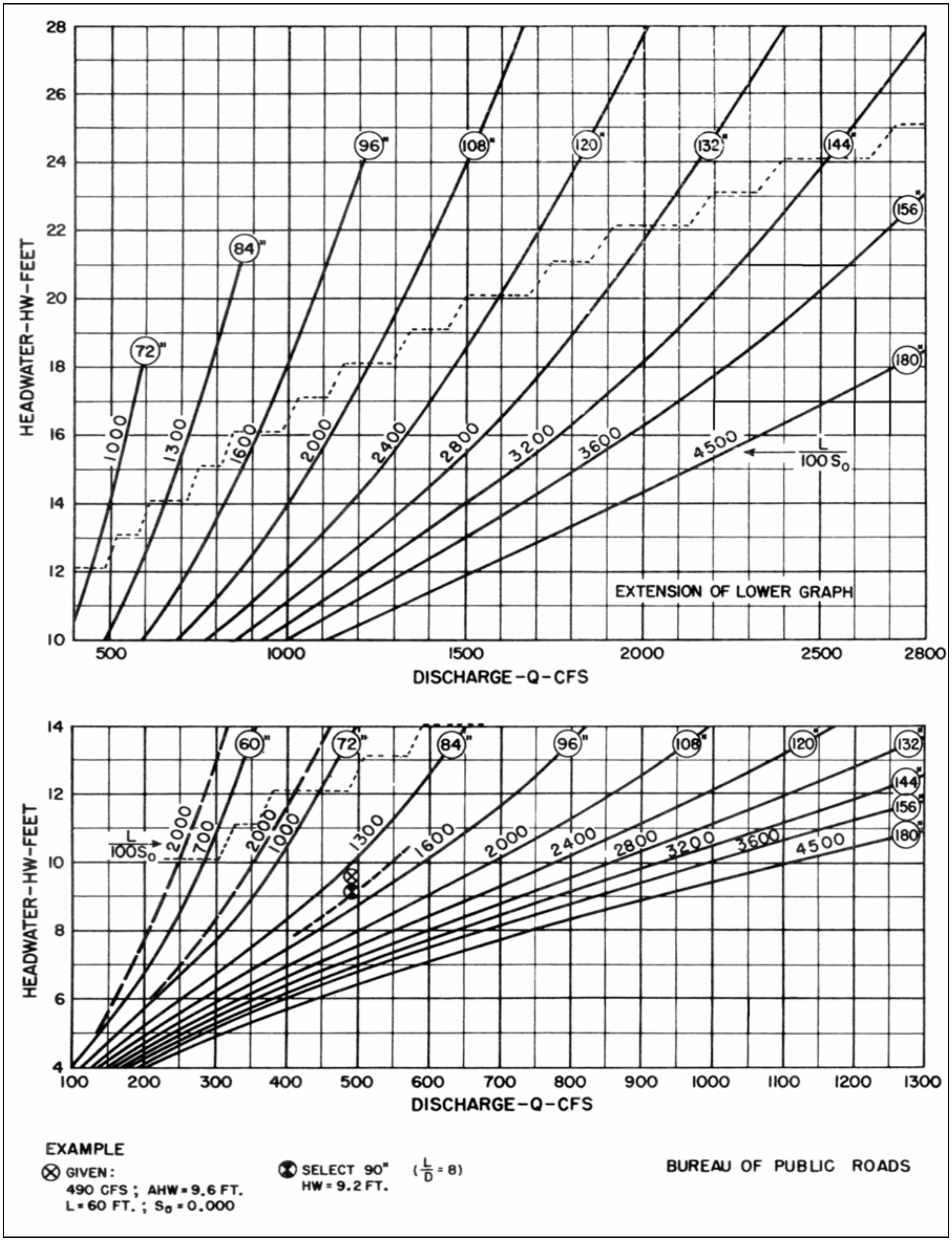


Figure 4-69. Culvert Capacity Standard Circular Corrugations Metal Pipe Projecting Entrance 18 in. to 36 in.

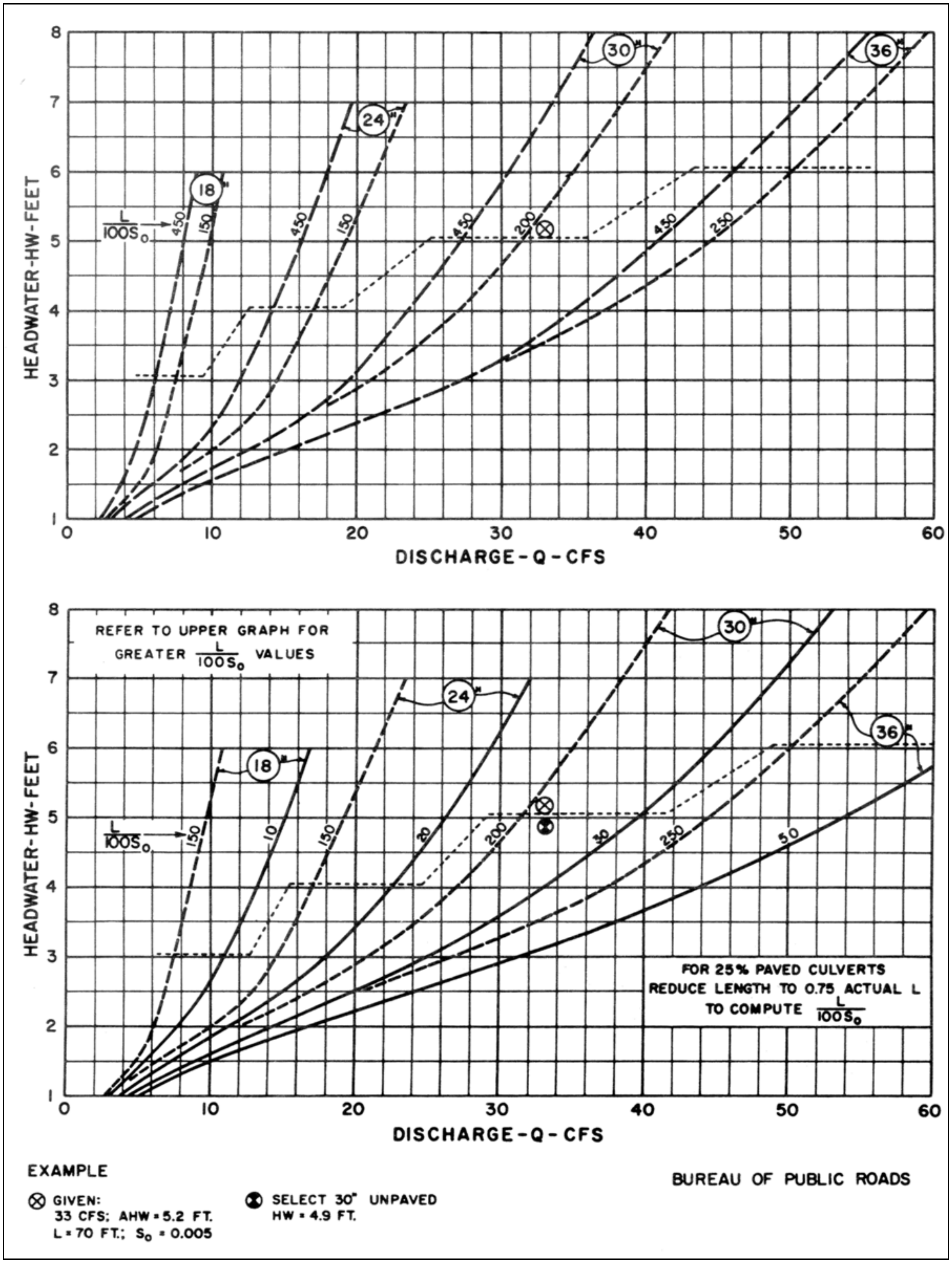


Figure 4-70. Culvert Capacity Standard Circular Corrugations Metal Projecting Entrance 36 in. to 66 in.

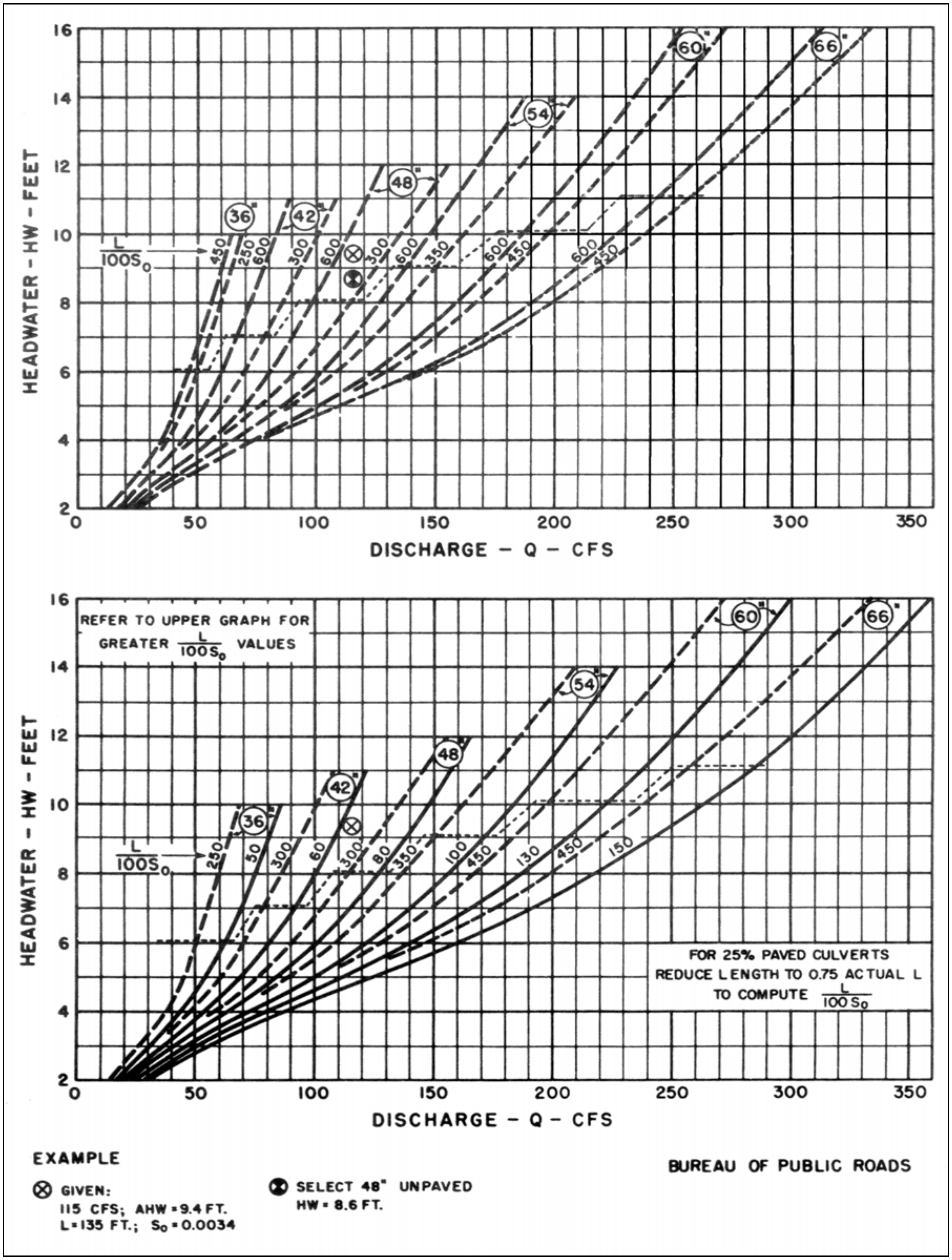


Figure 4-71. Culvert Capacity Standard Circular Corrugations Metal Headwall Entrance 18 in. to 36 in.

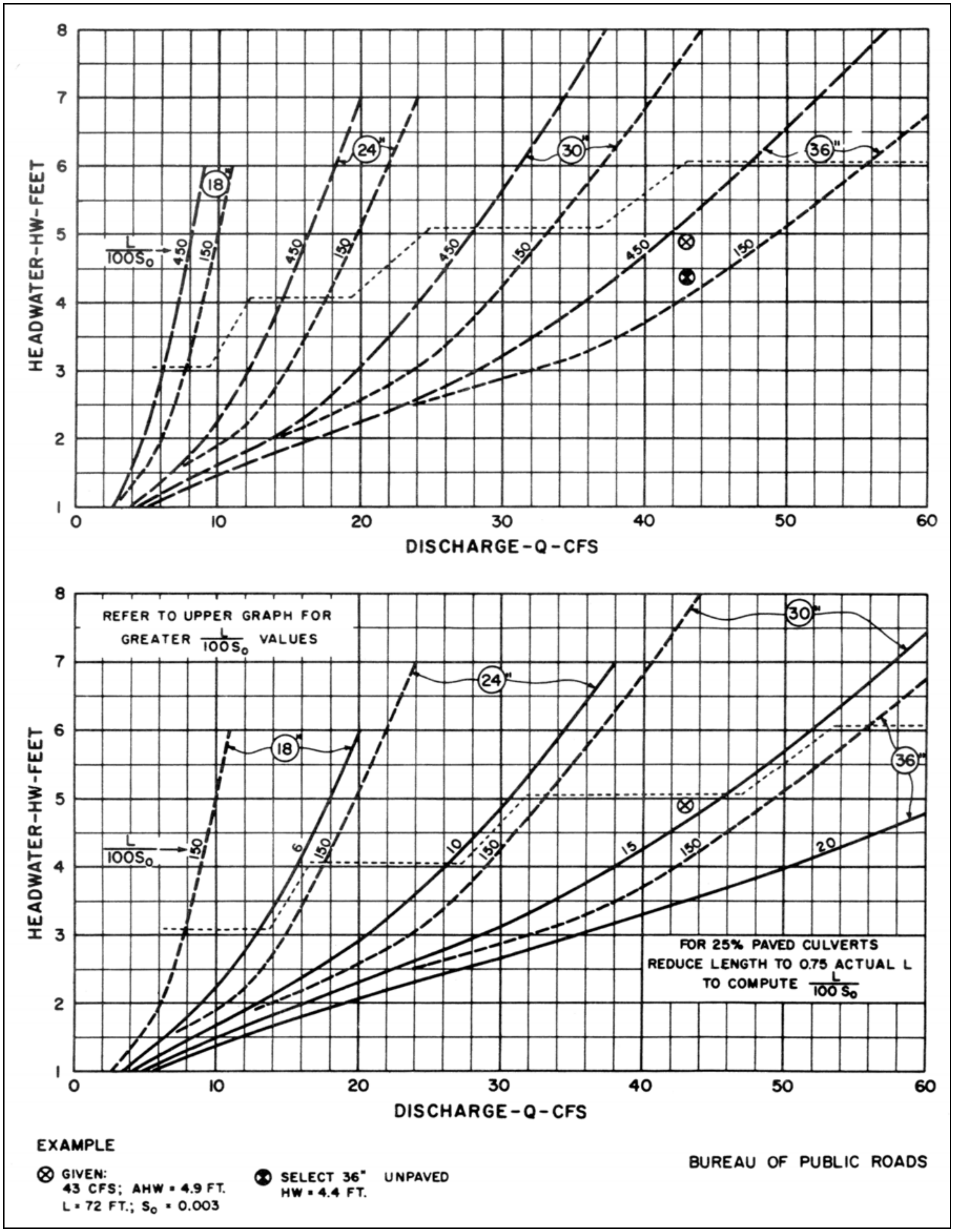


Figure 4-72. Culvert Capacity Standard Circular Corrugations Metal Headwall Entrance 36 in. to 66 in.

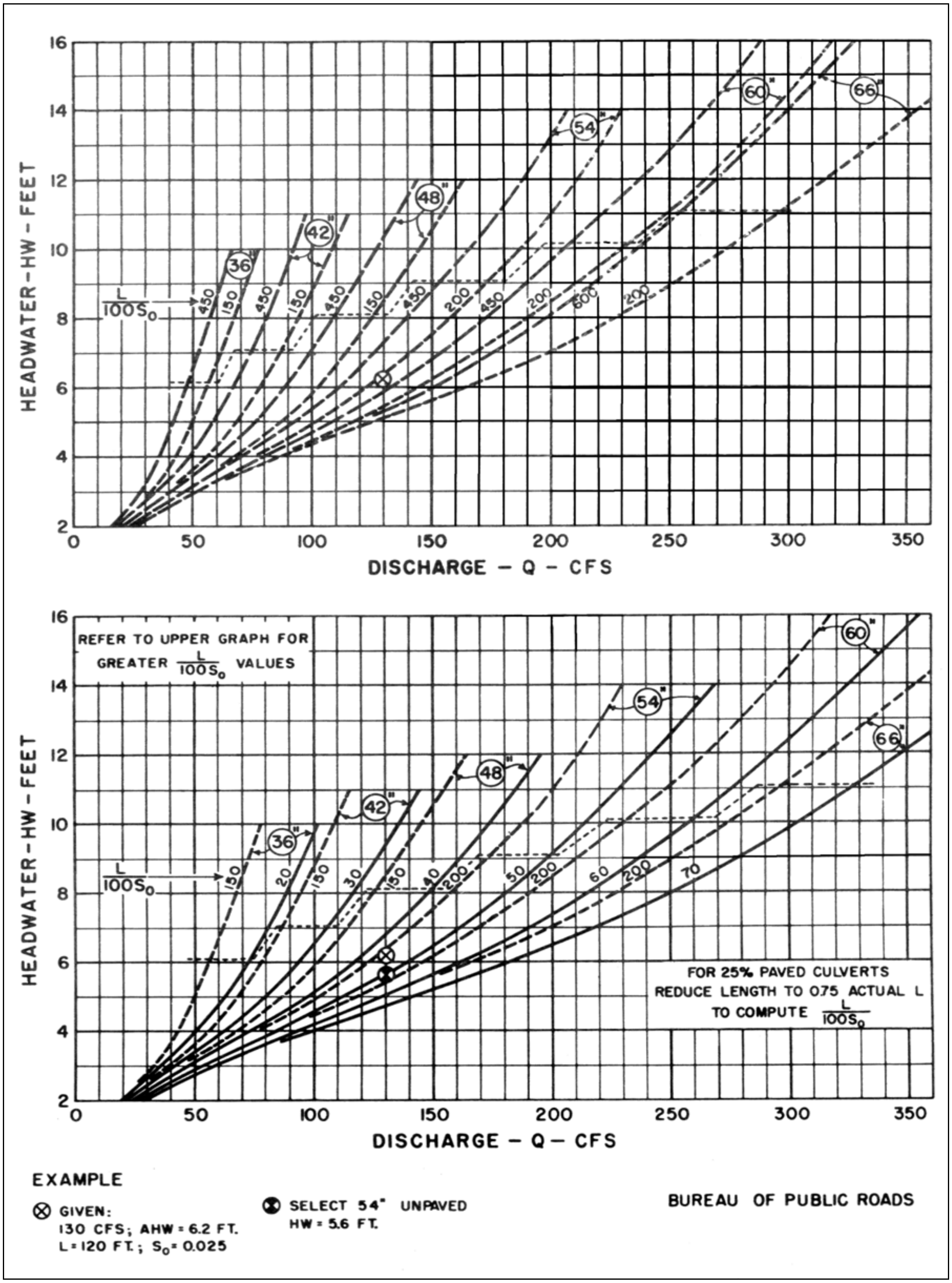


Figure 4-73. Culvert Capacity Standard Corrugations Metal Pipe-Arch Projecting Entrance 25 in. by 16 in. to 43 in. by 27 in.

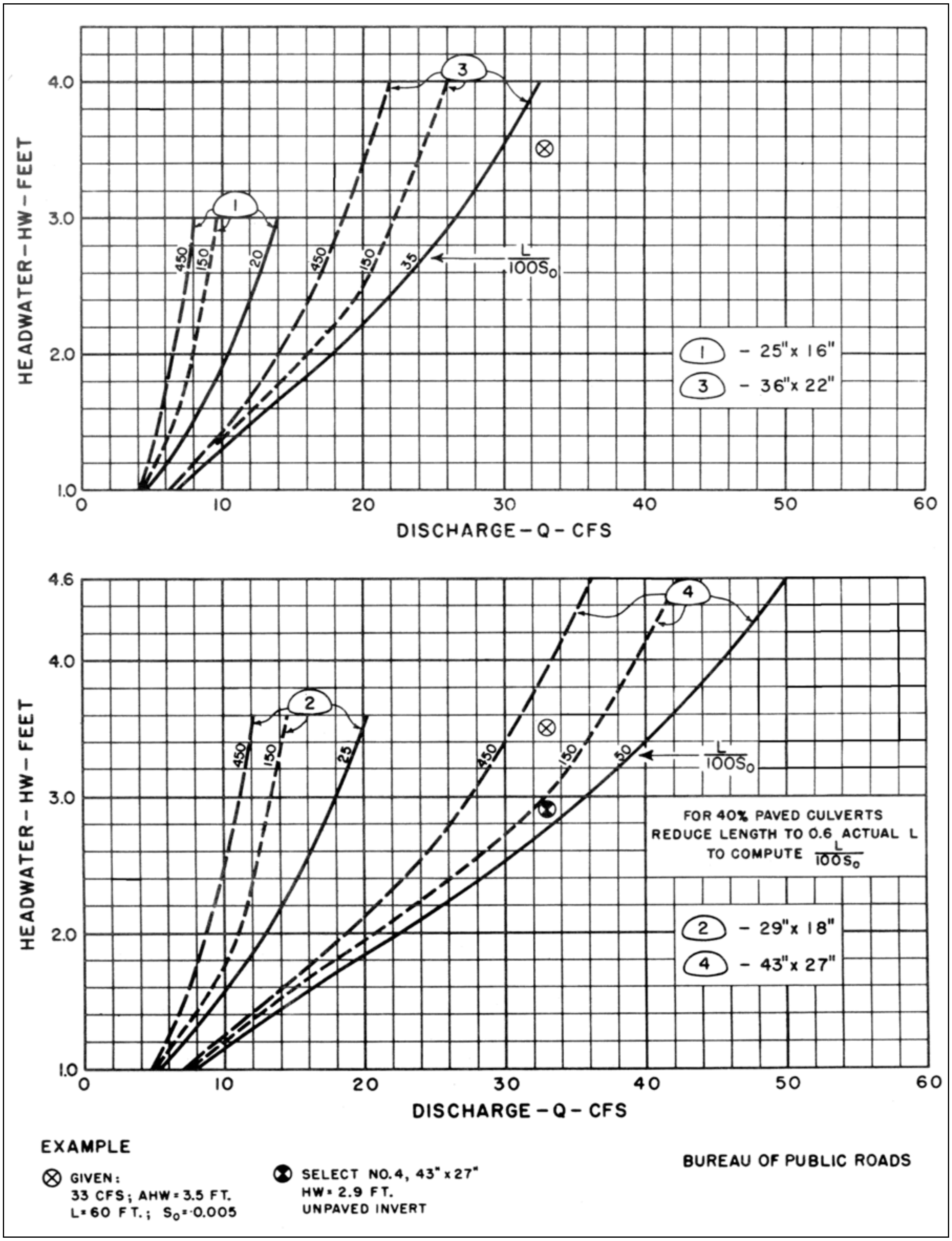


Figure 4-74. Culvert Capacity Standard Corrugations Metal Pipe-Arch Projecting Entrance 50 in. by 31 in. to 72 in. by 44 in.

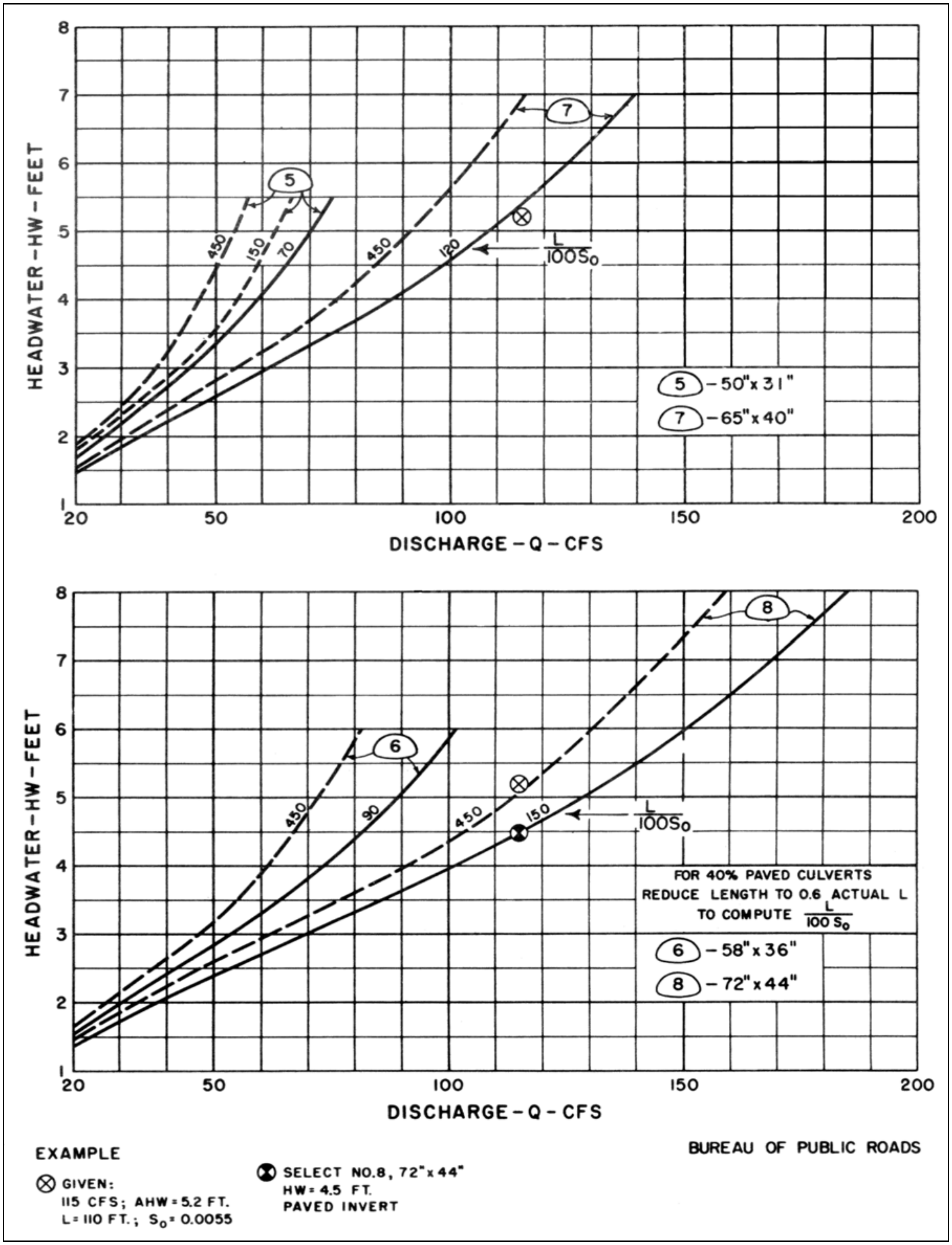


Figure 4-75. Culvert Capacity Standard Corrugations Metal Pipe-Arch Headwall Entrance 25 in. by 16 in. to 43 in. by 27 in.

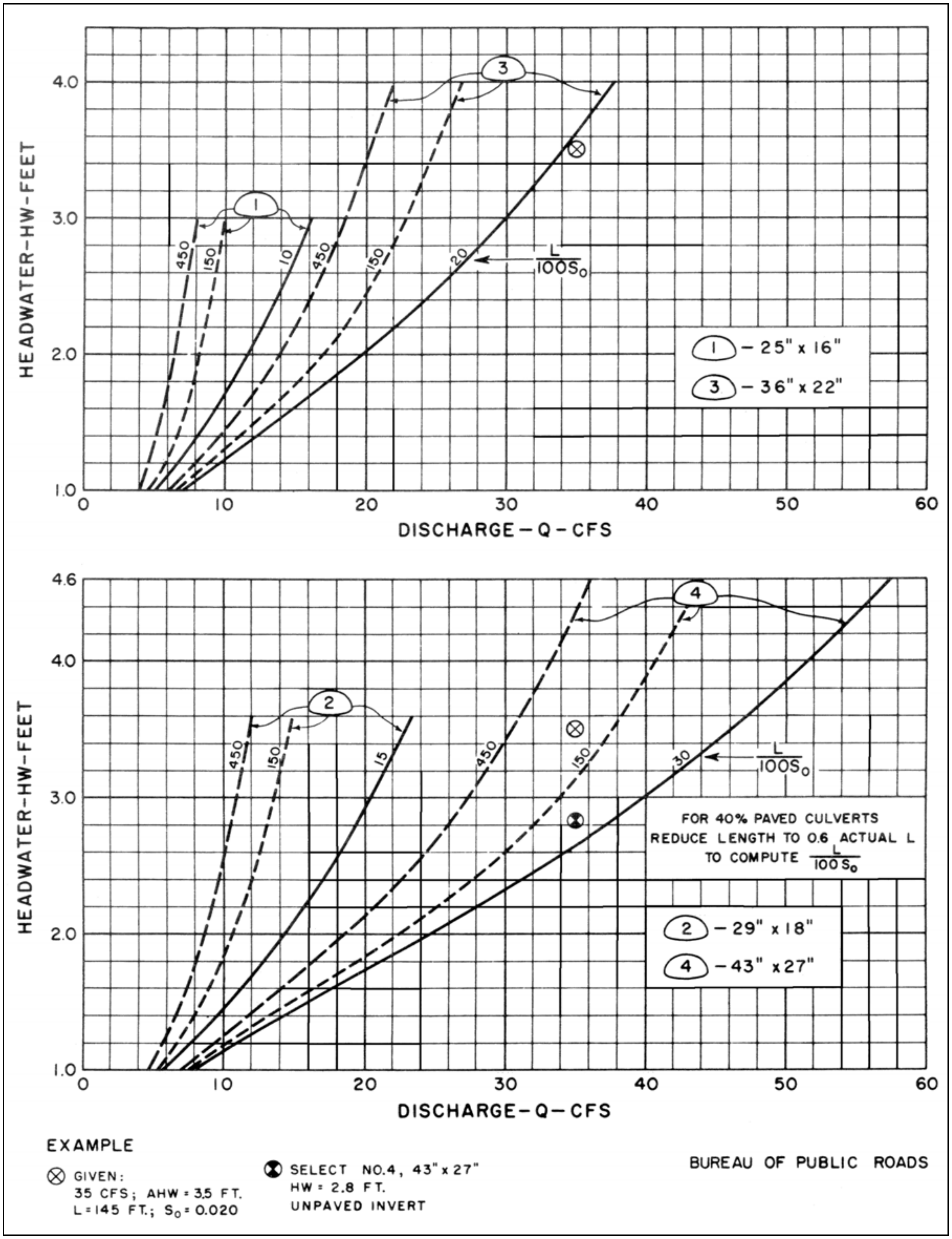


Figure 4-76. Culvert Capacity Standard Corrugations Metal Pipe-Arch Headwall Entrance 50 in. by 31 in. to 72 in. by 44 in.

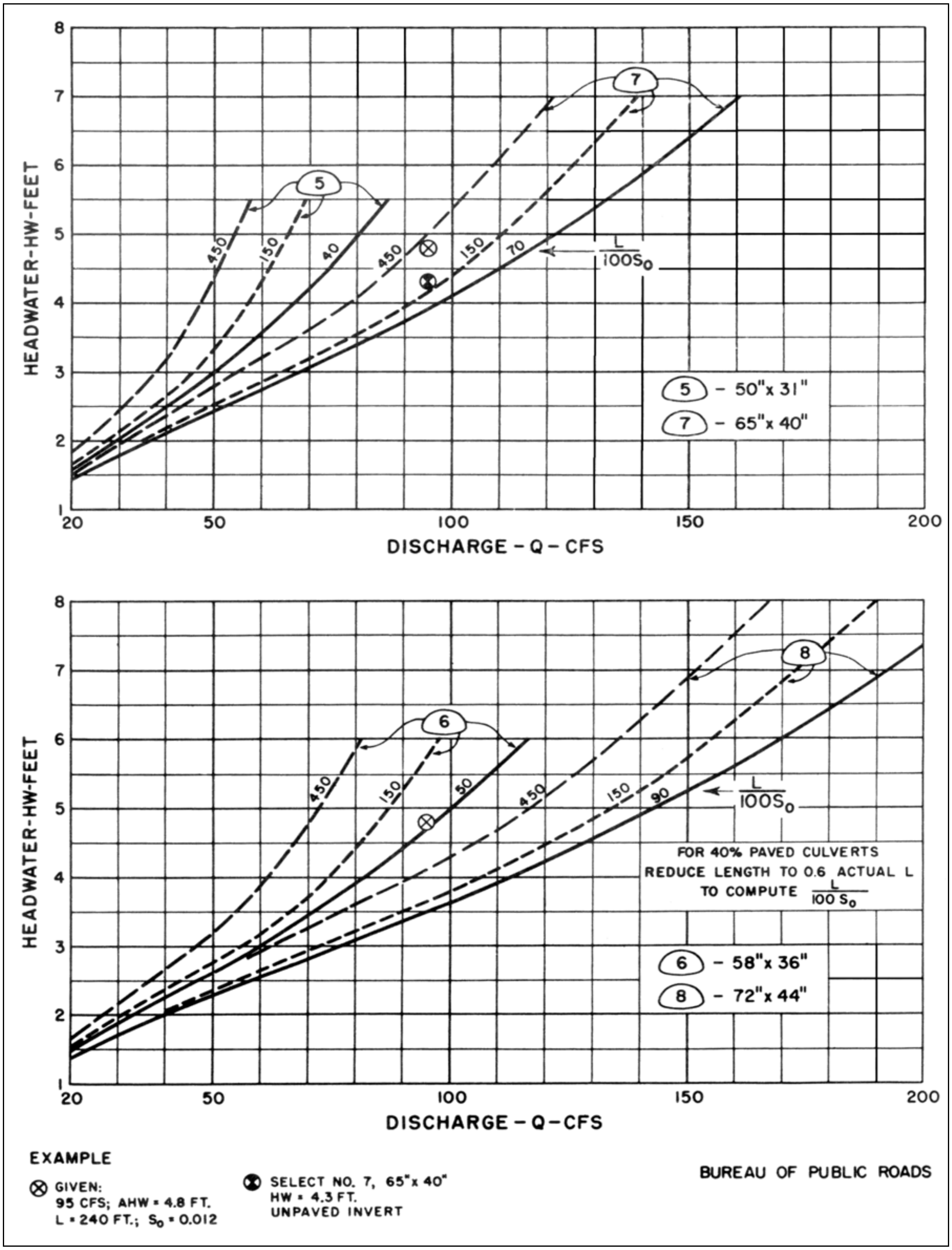


Figure 4-77. Culvert Capacity Square Concrete Box 90 Degree and 15 Degree Wingwall Flare 1.5 ft by 1.5 ft to 7 ft by 7 ft

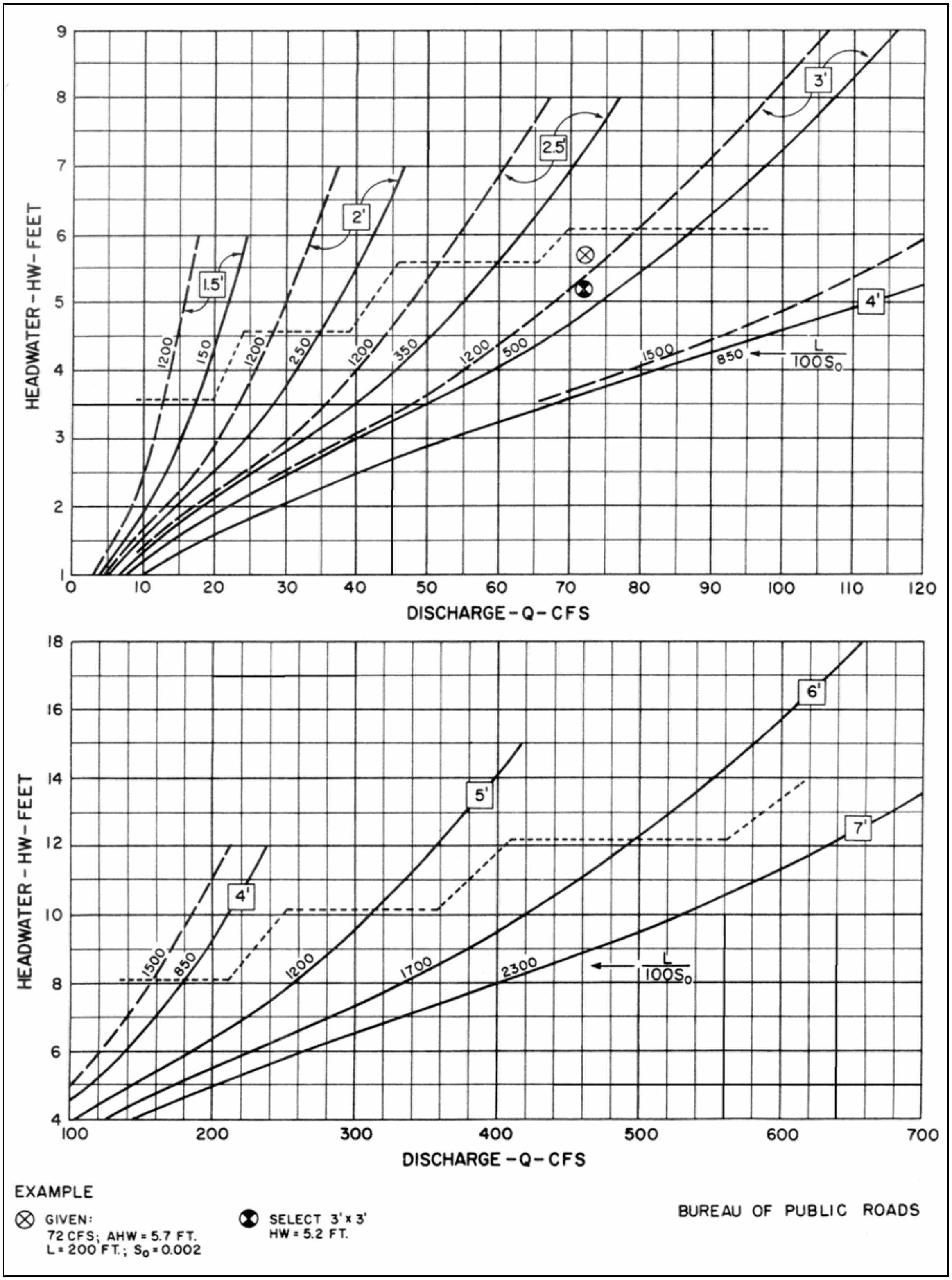


Figure 4-78. Culvert Capacity Square Concrete Box 30 Degree and 75 Degree Wingwall Flare 1.5 ft by 1.5 ft to 7 ft by 7 ft

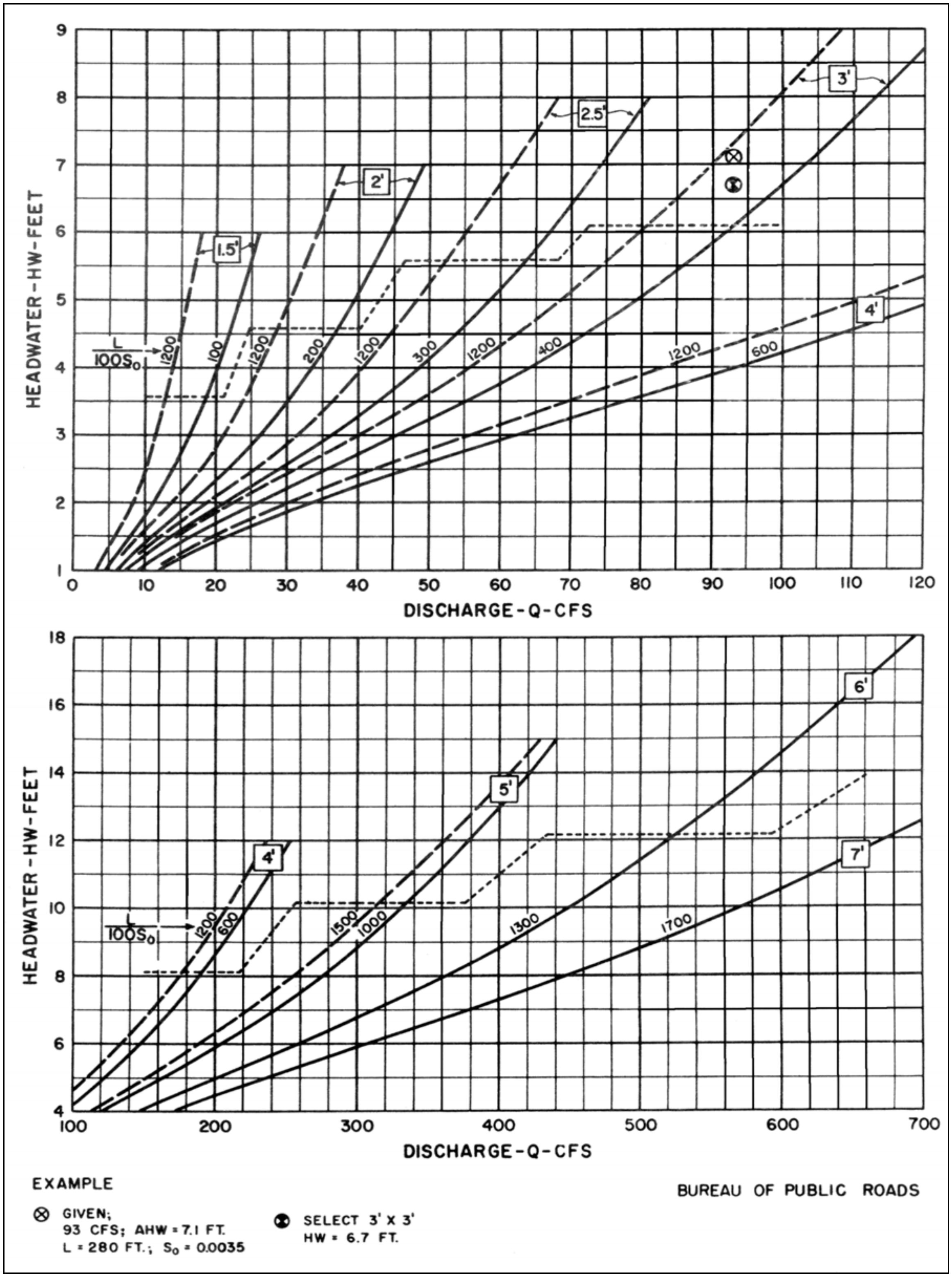


Figure 4-79. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 1.5 ft, 2.0 ft, and 2.5 ft Heights

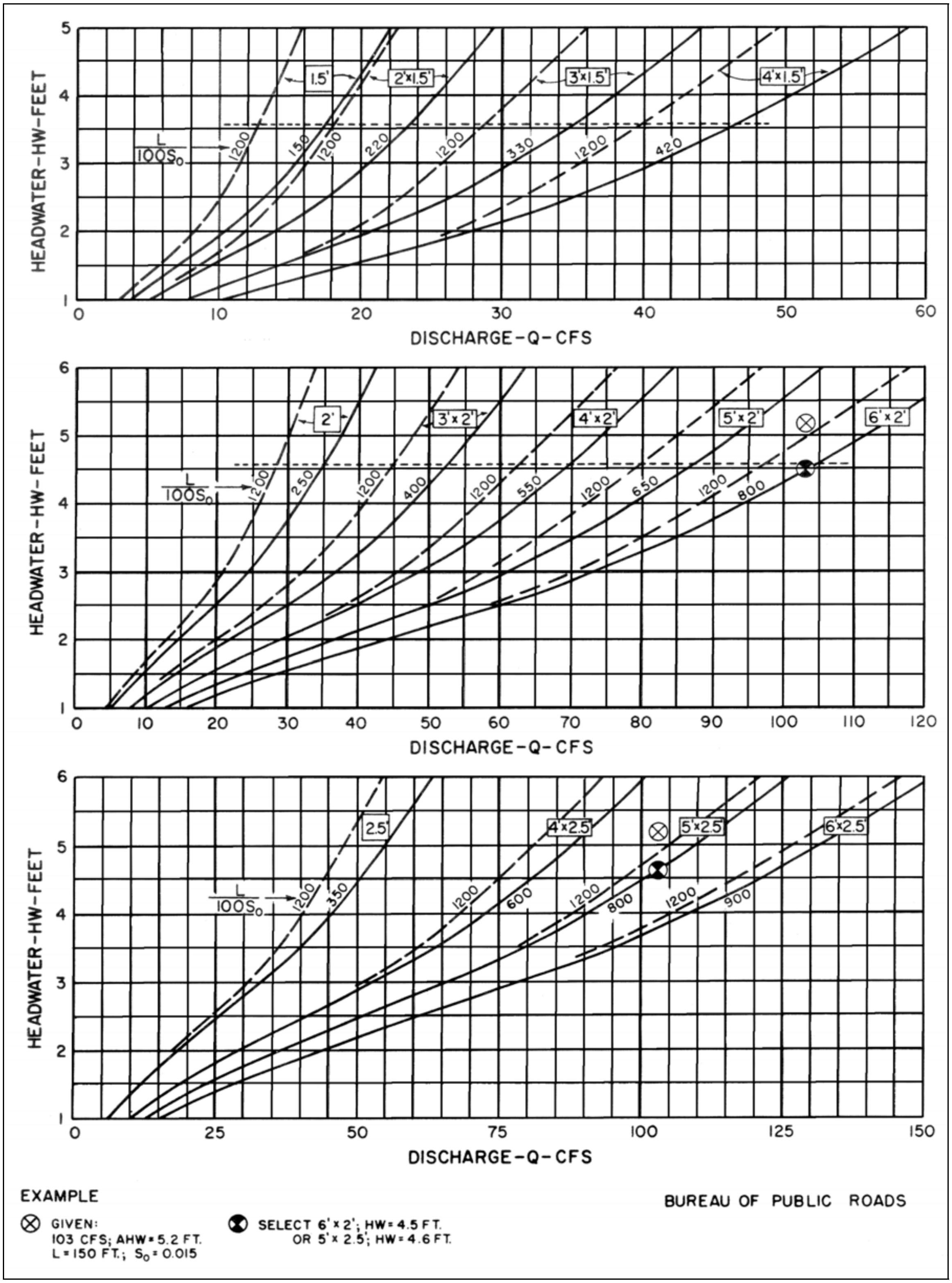


Figure 4-80. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 3 ft and 4 ft Heights

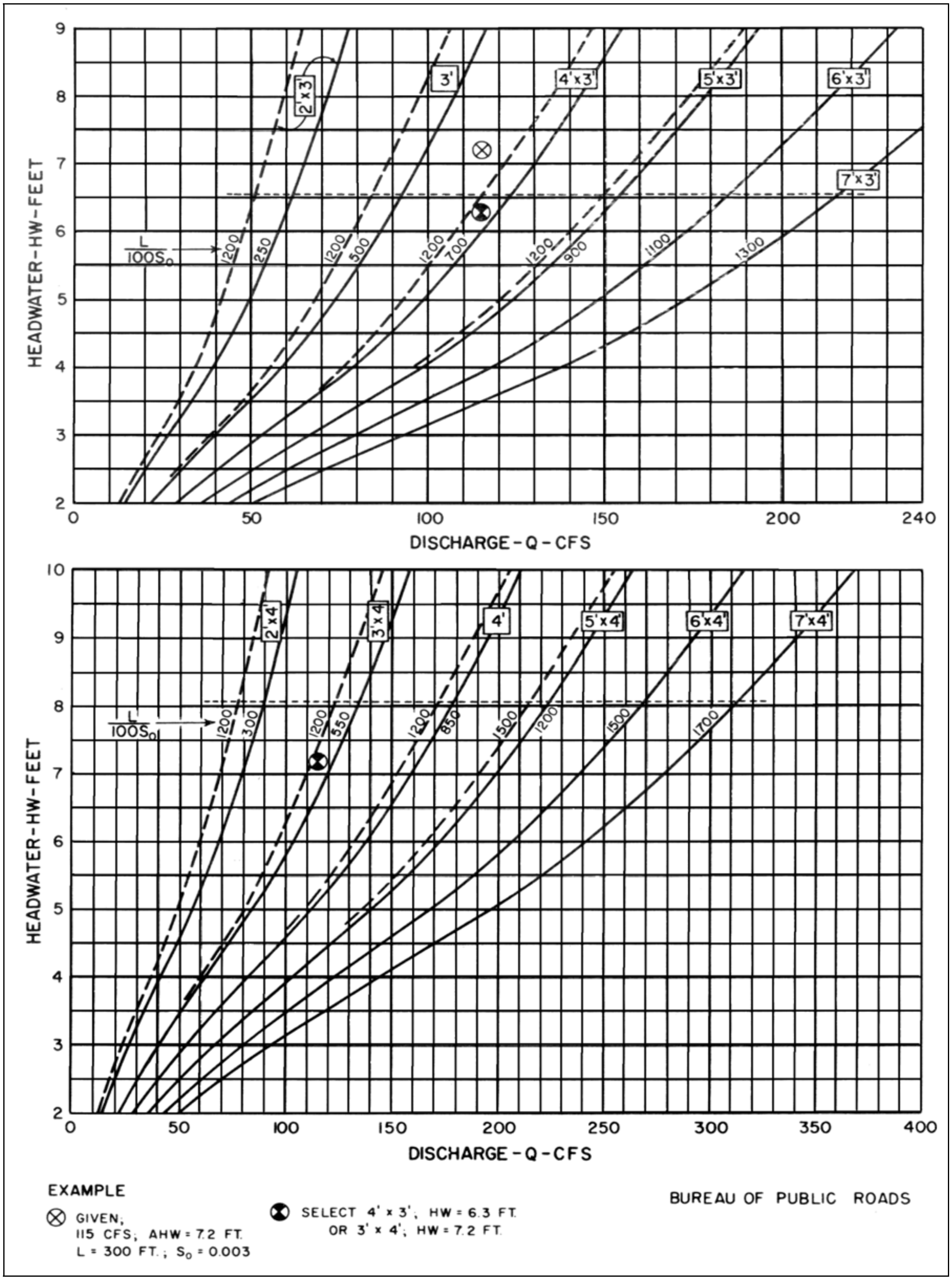


Figure 4-81. Culvert Capacity Rectangular Concrete Box 90 Degree and 15 Degree Wingwall Flare 5 ft and 6 ft Heights

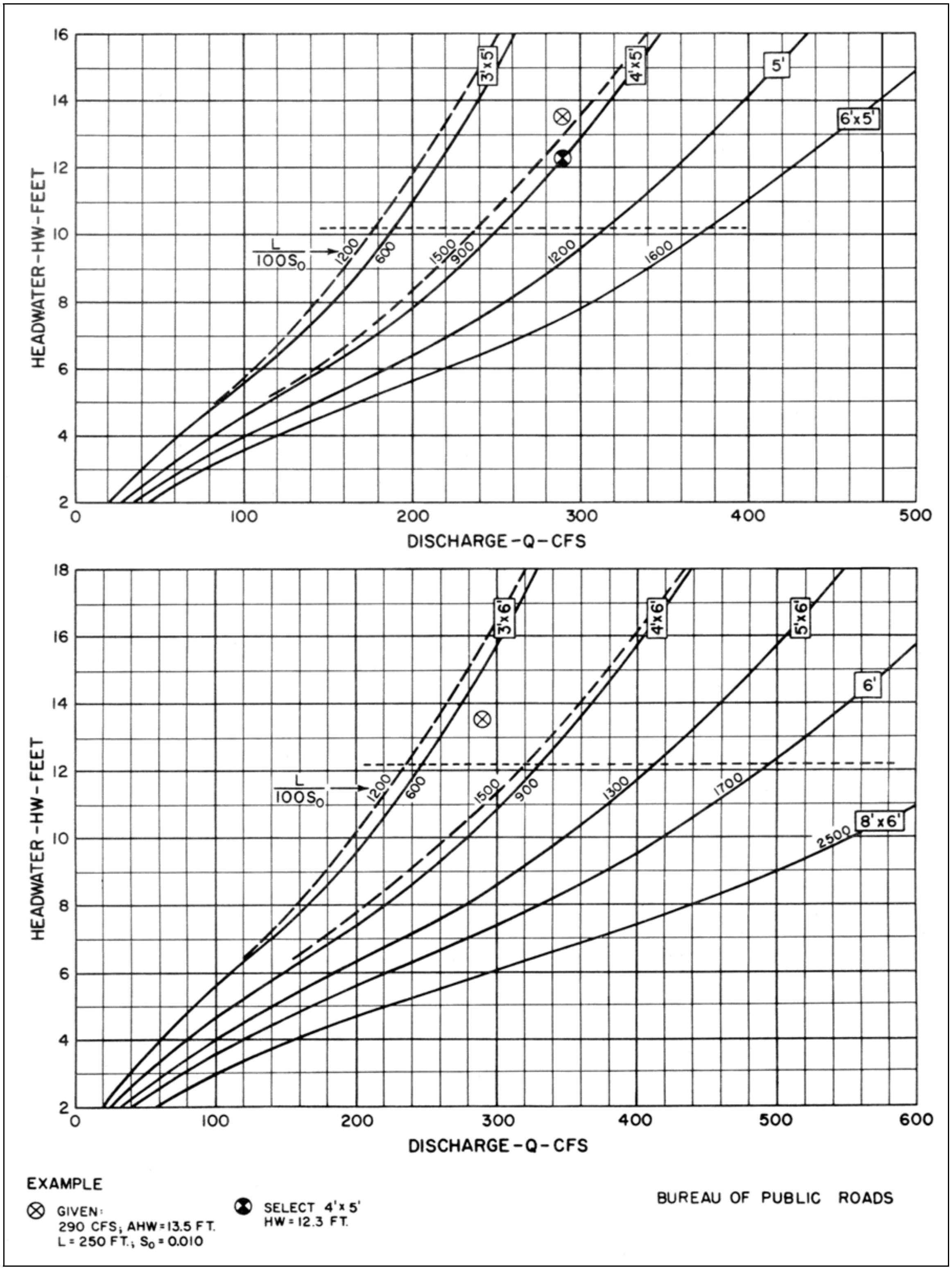


Figure 4-82. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 1.5 ft, 2.0 ft, and 2.5 ft Heights

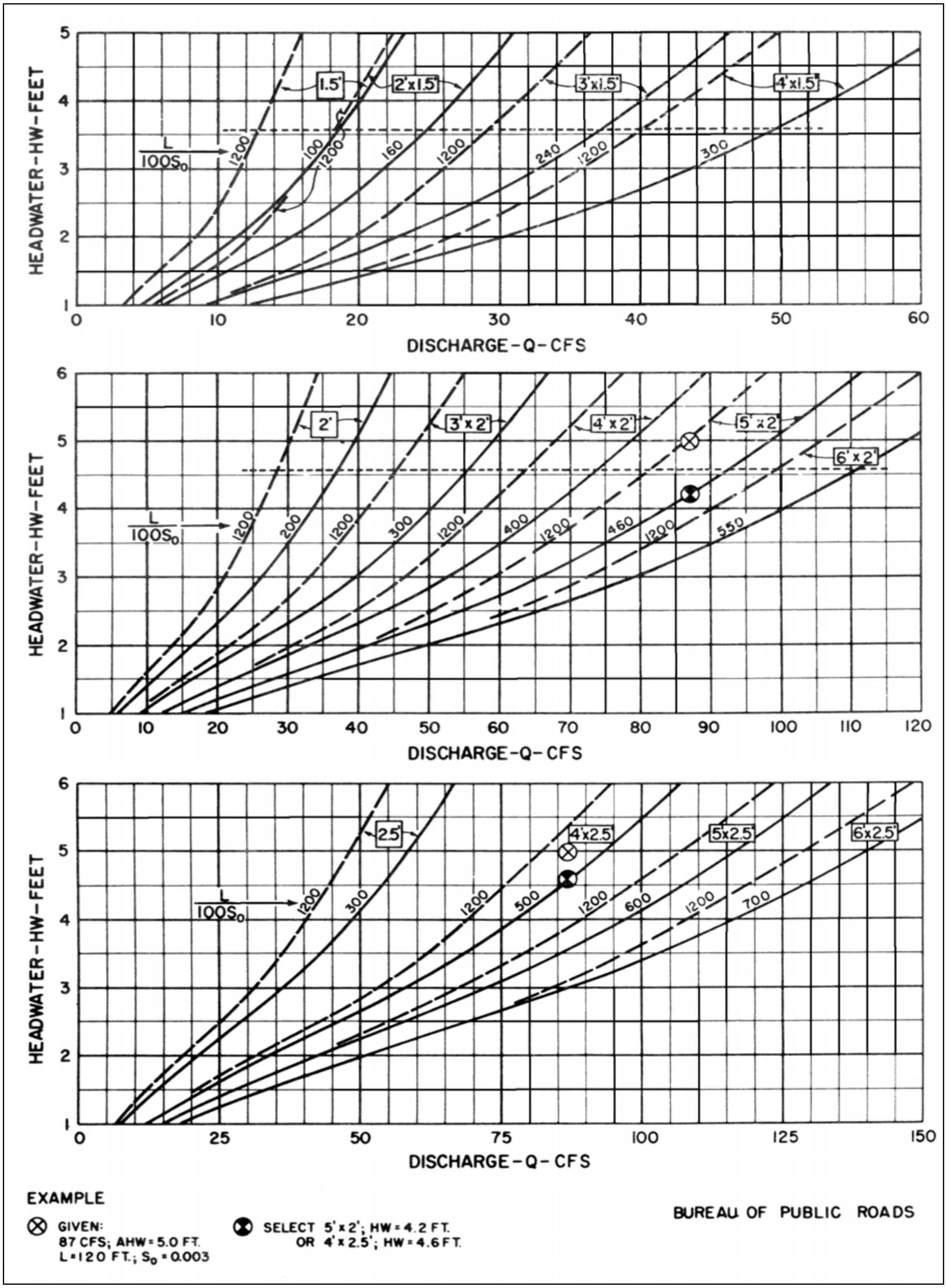


Figure 4-83. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 3 ft and 4 ft Heights

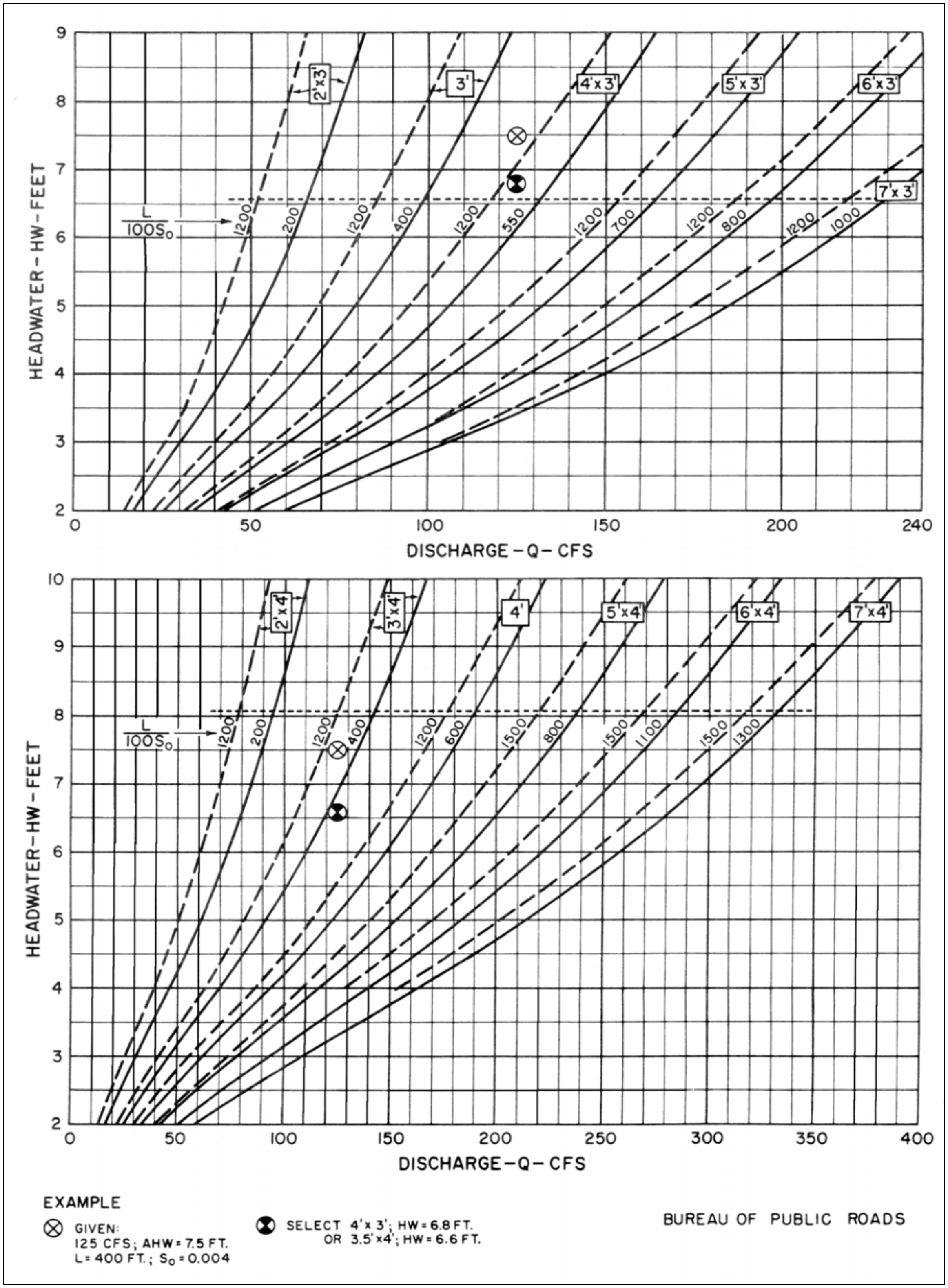
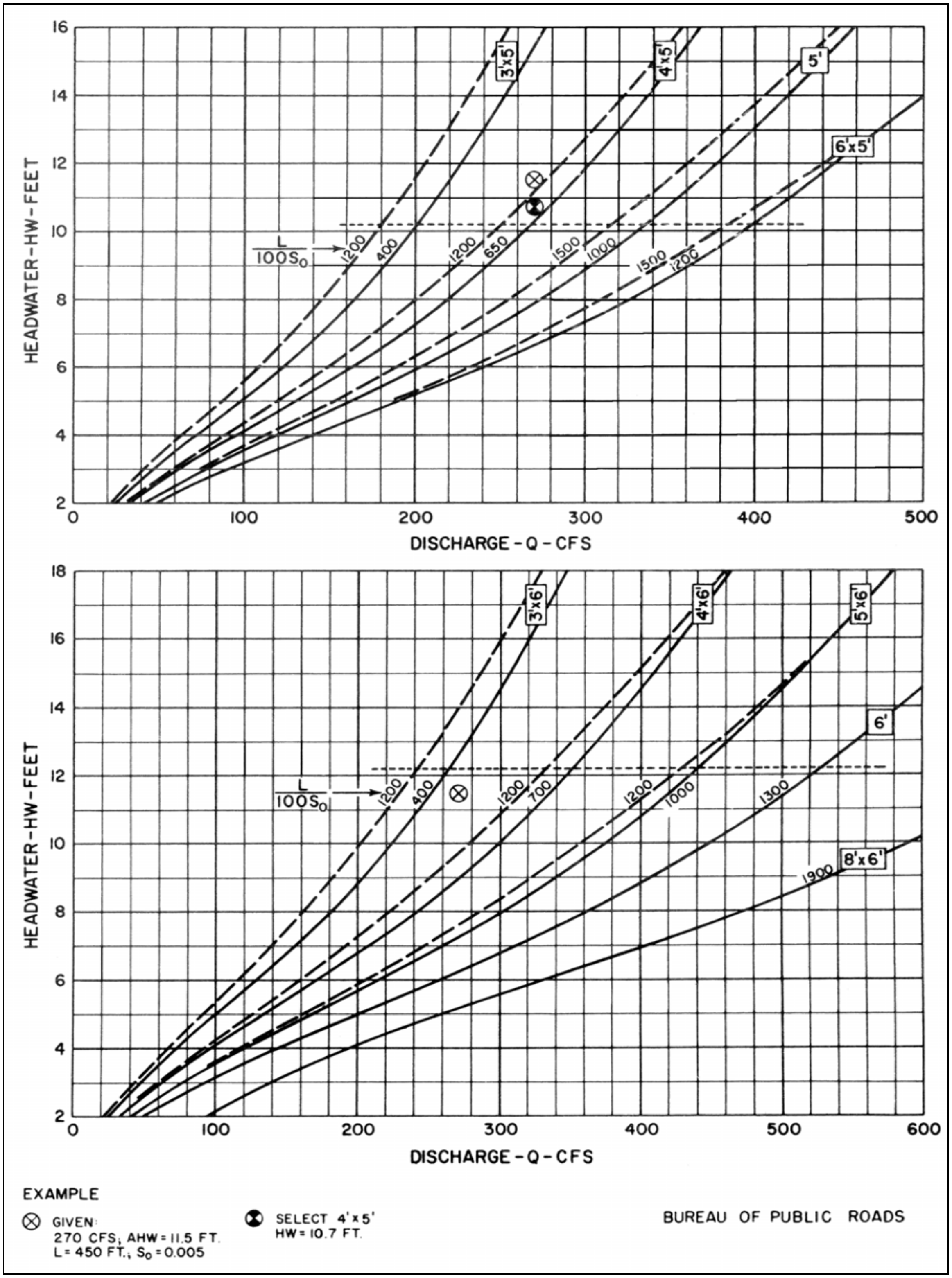


Figure 4-84. Culvert Capacity Rectangular Concrete Box 30 Degree and 75 Degree Wingwall Flare 5 ft and 6 ft Heights



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

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

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

4-4.7 **Inlets**

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

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

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

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

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

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

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

4-4.7.2.6 Where a road pavement on a continuous grade is warped in transitions between superelevated and normal sections, surface water should normally be intercepted upstream of the point where the pavement cross slope begins to change, especially in areas where icing occurs.

4-4.7.2.7 On roads where curbs are used, runoff from cut slopes and from off-site areas should, wherever possible, be intercepted by ditches at the tops of slopes or in swales along the shoulders and not be allowed to flow onto the roadway. This practice minimizes the amount of water to be intercepted by gutter inlets and helps to prevent mud and debris from being carried onto the pavement.

4-4.7.3 Inlets placed in sumps have a greater potential capacity than inlets on a slope because of the possible submergence in the sump. Capacities of grated, curb, and combination inlets in sumps will be computed as outlined below. To allow for blockage by trash, the size of inlet opening selected for construction will be increased above the computed size by 100 percent for grated inlets and 25 to 75 percent, depending on trash conditions, for curb inlets and combination inlets.

4-4.7.3.1 **Grated type (in sump)**

a. For depths of water up to 0.4 ft use the weir formula:

$$Q = 3.0LH^{3/2} \tag{eq. 4-30}$$

If one side of a rectangular grate is against a curb, this side must be omitted in computing the perimeter.

b. For depths of water above 1.4 ft use the orifice formula:

$$Q = 0.6A\sqrt{2gH} \tag{eq. 4-31}$$

c. For depths between 0.4 and 1.4 ft, operation is indefinite due to vortices and other disturbances. Capacity will be somewhere between those given by the preceding formulas.

d. Problems involving the above criteria may be solved graphically by use of Figure 4-85.

4-4.7.3.2 **Curb type (in sump)**. For a curb inlet in a sump, the above listed general concepts for weir and orifice flow apply, the latter being in effect for depths greater than about 1.4 h (where h is the height of curb opening entrance). Figure 4-86 presents a graphic method for estimating capacity.

4-4.7.3.3 **Combination type (in sump)**. For a combination inlet in a sump no specific formulas are given. Some increase in capacity over that provided singly by either a grated opening or a curb opening may be expected, and the curb opening will operate as a relief opening if the grate becomes clogged by debris. In estimating the capacity, the inlet will be treated as a simple grated inlet, but a safety factor of 25 to 75 percent will be applied.

4-4.7.3.4 **Slotted drain type**. For a slotted drain inlet in a sump, the flow will enter the slot as either all orifice type or all weir type, depending on the depth of water at the edge of the slot. If the depth is less than .18 ft, the length of slot required to intercept total flow is equal to:

$$\frac{Q}{3.125\, d^{3}/2} \tag{eq. 4-32}$$

If the depth is greater than .18 feet, the length of slot required to intercept total flow is equal to:

$$\frac{Q}{.5\, w\, \sqrt{2gd}} \tag{eq. 4-33}$$

d = depth of flow-in.

w = width of slot-.146 ft

Figure 4-85. Capacity of Grate Inlet in Sump Water Pond on Grate

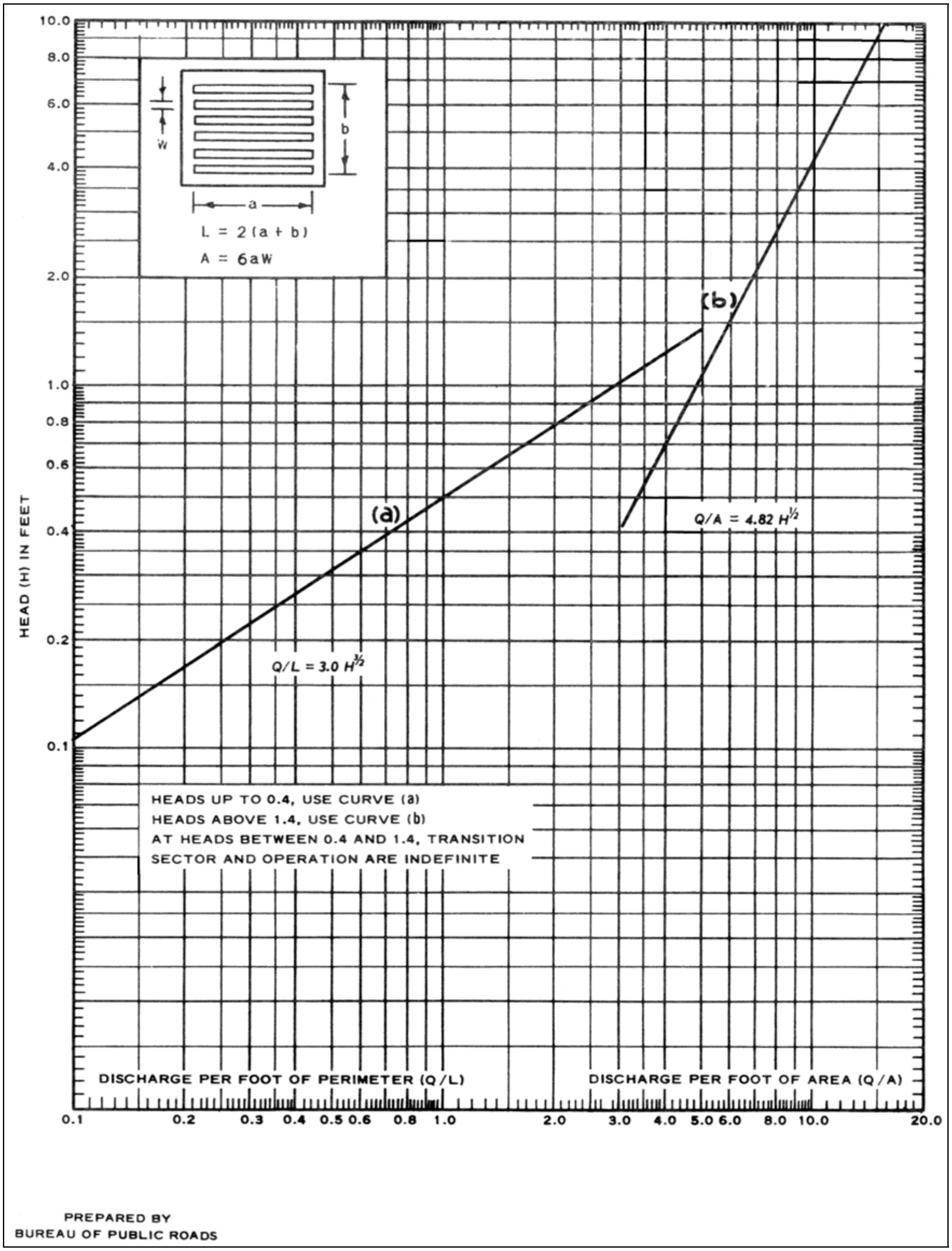
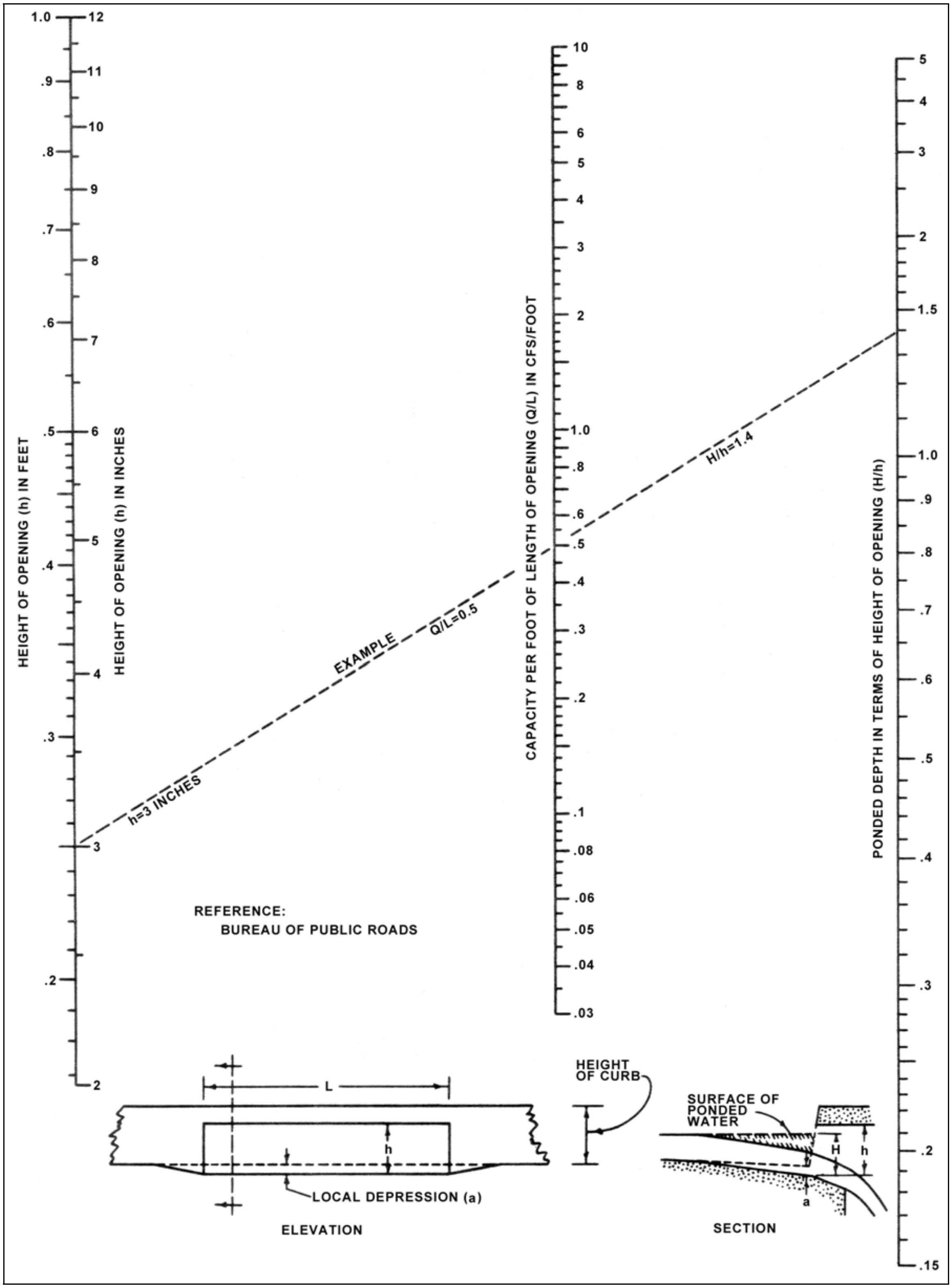


Figure 4-86. Capacity of Curb Opening Inlet at Low Point in Grade



4-4.7.4 Each of a series of inlets placed on a slope is usually, for optimum efficiency, designed to intercept somewhat less than the design gutter flow, the remainder being passed to downstream inlets. The amount that must be intercepted is governed by whatever width and depth of bypassed flow can be tolerated from a traffic and safety viewpoint. Such toleration levels will nearly always be influenced by costs of drainage construction. With the flat street crowns prevalent in modern construction, many gutter flows are relatively wide and in built-up areas some inconveniences are inevitable, especially in regions of high rainfall, unless an elaborate inlet system is provided. The achievement of a satisfactory system at reasonable cost requires careful consideration of use factors and careful design of the inlets themselves. However, it must also be remembered that a limitation on types and sizes for a given project is also desirable, for standardization will lead to lower construction costs. Design of grated, curb, and combination inlets on slopes will be based on principles outlined below.

4-4.7.4.1 **Grated type (on slope)**. A grated inlet placed in a sloping gutter will provide optimum interception of flow if the bars are placed parallel to the direction of flow, if the openings total at least 50 percent of the width of the grate (i.e., normal to the direction of flow), and if the unobstructed opening is long enough (parallel to the direction of flow) that the water falling through will clear the downstream end of the opening. The minimum length of clear opening required depends on the depth and velocity of flow in the approach gutter and the thickness of the grate at the end of the slot. This minimum length may be estimated by the partly empirical formula:

$$L = \frac{V}{2}\sqrt{y+d} \qquad \text{(eq. 4-34)}$$

A rectangular grated inlet in a gutter on a continuous grade can be expected to intercept all the water flowing in that part of the gutter cross section that is occupied by the grating plus an amount that will flow in along the exposed sides. However, unless the grate is over 3 ft long or greatly depressed (extreme warping of the pavement is seldom permissible), any water flowing outside the grate width can be considered to bypass the inlet. The quantity of flow in the prism intercepted by such a grate can be computed by following instruction 3 in Figure 4-41. For a long grate the inflow along the side can be estimated by considering the edge of the grate as a curb opening whose effective length is the total grate length (ignoring crossbars) reduced by the length of the jet directly intercepted at the upstream end of the grate. To attain the optimum capacity of an inlet consisting of two grates separated by a short length of paved gutter, the grates should be so spaced that the carryover from the upstream grate will move sufficiently toward the curb to be intercepted by the downstream grate.

4-4.7.4.2 **Curb type (on slope)**. In general, a curb inlet placed on a grade is a hydraulically inefficient structure for flow interception. A relatively long opening is required for complete interception because the heads are normally low and the direction of oncoming flows is not favorable. The cost of a long curb inlet must be weighed against that of a drop type with potentially costly grate. The capacity of a curb inlet intercepting all the flow can be calculated by an empirical equation. The equation is a function of length of clear opening of the inlet, depth of depression of flow line at inlet in feet, and the depth of flow in approach gutter in feet. Depression of the inlet flow line is an essential part of good design, for a curb inlet with no depression is very inefficient. The flow intercepted may be markedly increased without changing the opening length if the flow line can be depressed by one times the depth of flow in the approach gutter. The use of long curb openings with intermediate supports should generally be avoided because of the tendency for the supports to accumulate trash. If supports are essential, they should be set back several inches from the gutter line.

4-4.7.4.2 **Combination type (on slope)**. The capacity of a combination inlet on a continuous grade is not much greater than that of the grated portion itself, and should be computed as a separate grated inlet except in the following situations. If the curb opening is placed upstream from the grate, the combination inlet can be considered to operate as two separate inlets and the capacities can be computed accordingly. Such an arrangement is sometimes desirable, for in addition to the increased capacity the curb opening will tend to intercept debris and thereby reduce clogging of the grate. If the curb opening is placed downstream from the grate, effective operation as two separate inlets requires that the curb opening be sufficiently downstream to allow flow bypassing the grate to move into the curb opening. The minimum separation will vary with both the cross slope and the longitudinal slope.

4-4.7.5 Structural aspects of inlet construction should generally be as indicated in Figures 4-87, 4-88, and 4-89 which show respectively, standard circular grate inlets, types A and B; typical rectangular grate combination inlet, type C; and curb inlet, type D. It will be noted that the type D inlet provides for extension of the opening by the addition of a collecting trough whose backwall is cantilevered to the curb face. Availability of gratings and standards of municipalities in a given region may limit the choice of inlet types. Grated inlets subject to heavy wheel loads will require grates of precast steel or of built-up, welded steel. Steel grates will be galvanized or bituminous coated. Unusual inlet conditions will require special design.

4-4.8 Vehicular Safety and Hydraulically Efficient Drainage Practice

4-4.8.1 Some drainage structures are potentially hazardous and, if located in the path of an errant vehicle, can substantially increase the probability of an accident. Inlets should be flush with the ground, or should present no obstacle to a vehicle that is out of control. End structures or culverts should be placed outside the designated recovery area wherever possible. If grates are necessary to cover culvert inlets, care must be taken to design the grate so that the inlet will not clog during periods of high water. Where curb inlet systems are used, setbacks should be minimal, and grates should be designed for hydraulic efficiency and safe passage of vehicles. Hazardous channels or energy dissipating devices should be located outside the designated recovery area or adequate guardrail protection should be provided.

4-4.8.2 It is necessary to emphasize that liberties should not be taken with the hydraulic design of drainage structures to make them safer unless it is clear that their function and efficiency will not be impaired by the contemplated changes. Even minor changes at culvert inlets can seriously disrupt hydraulic performance.

Figure 4-87. Standard Type “A” and Type “B” Inlets

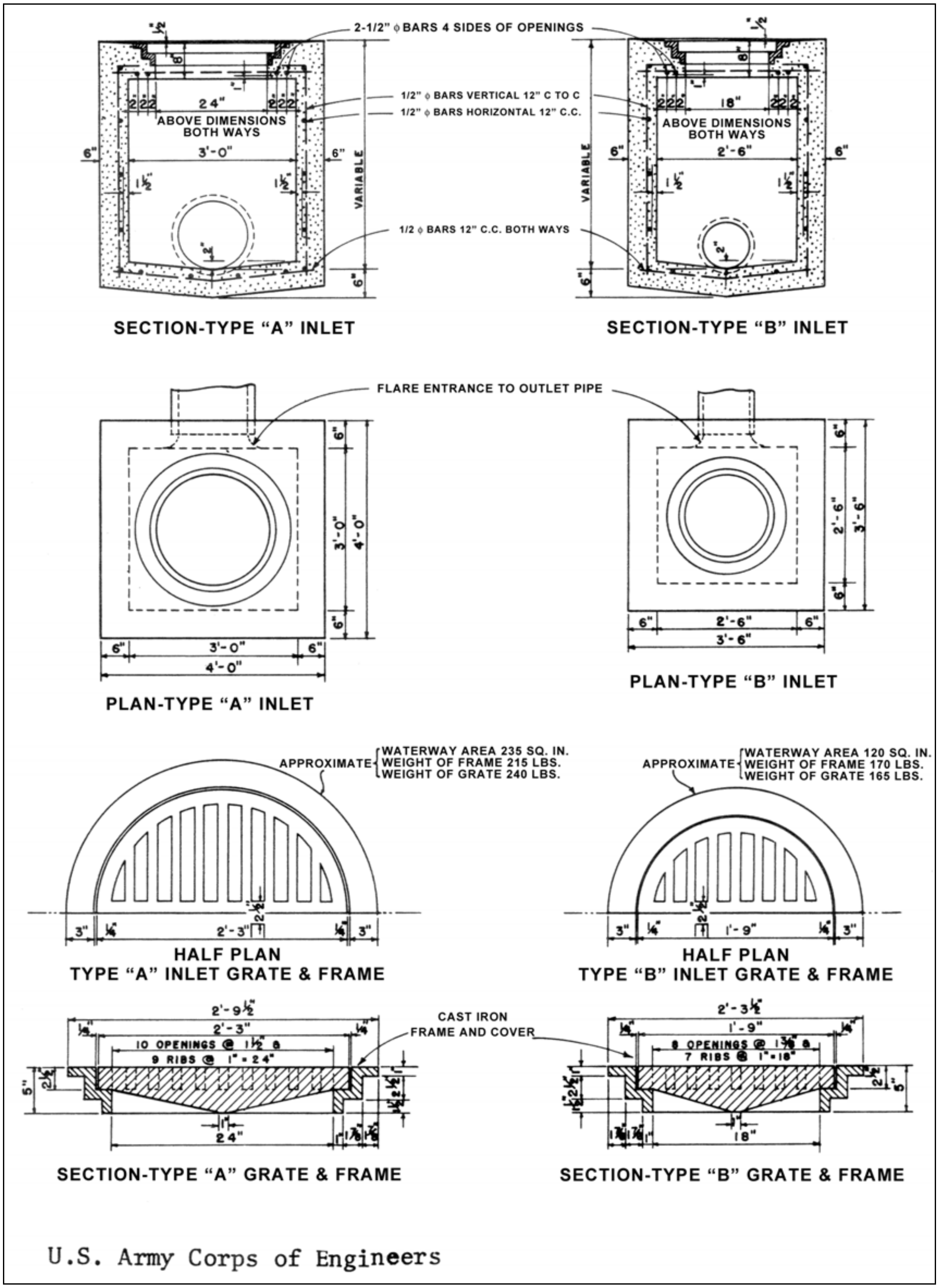
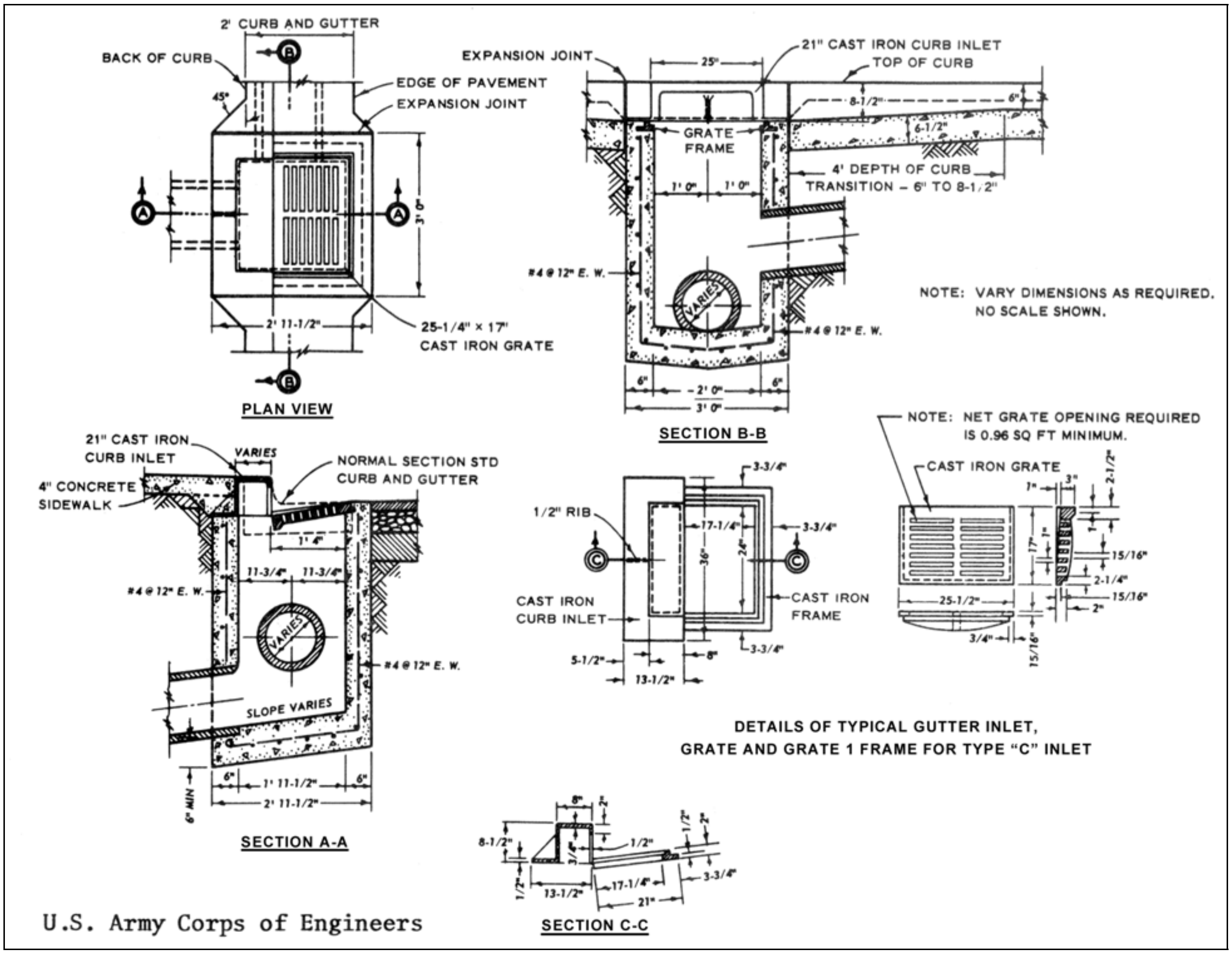


Figure 4-88. Type "C" Inlet—Square Grating

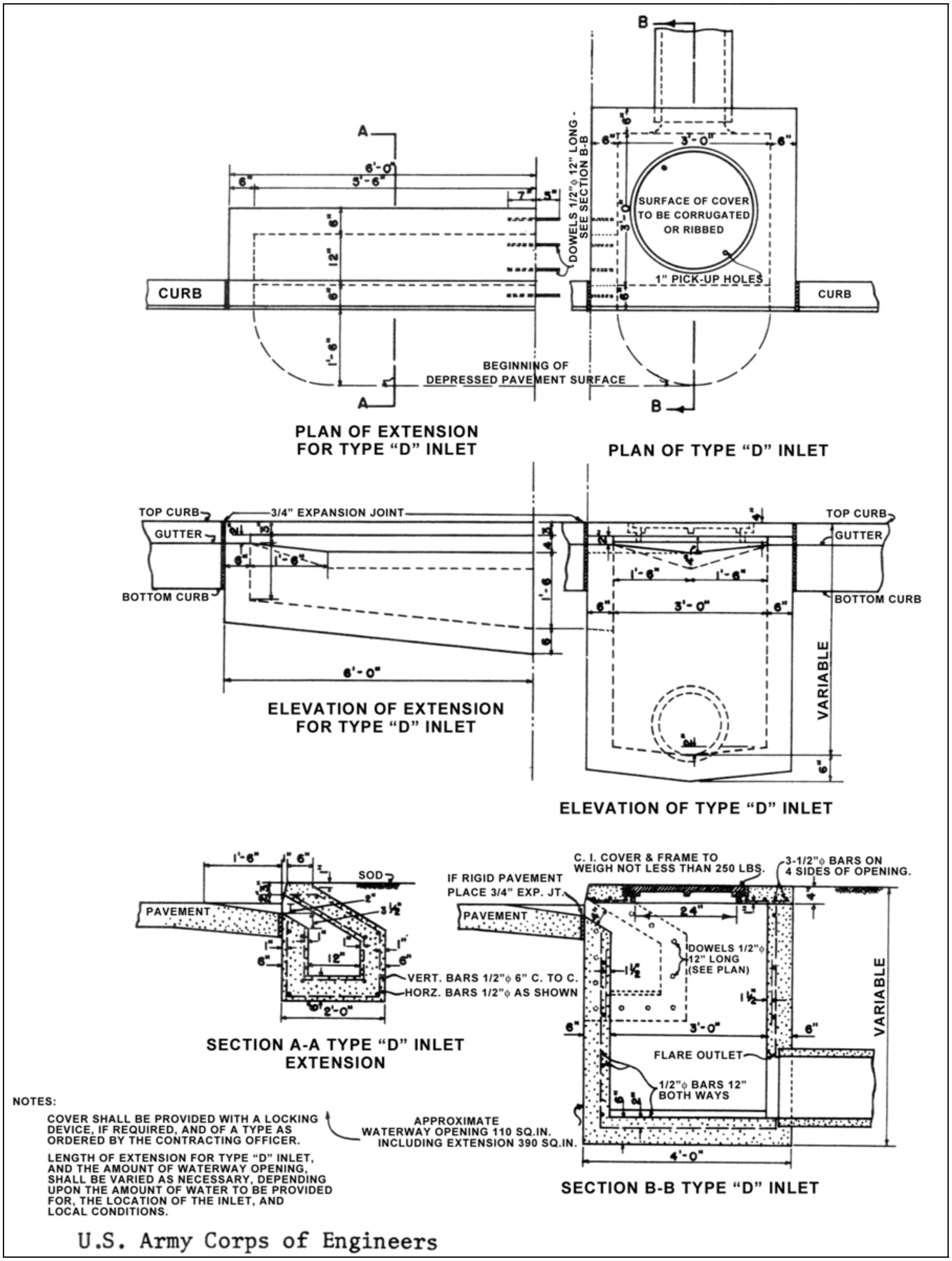


4-4.9 **Pipe Strength, Cover, and Bedding**. A drainage pipe is defined as a structure (other than a bridge) to convey water through a trench or under a fill or some other obstruction. Materials for permanent-type installations include non-reinforced concrete, reinforced concrete, corrugated steel, asbestos-cement, clay, corrugated aluminum alloy, and structural plate steel pipe.

4-4.9.1 **Selection of type of pipe**

4-4.9.1.1 The selection of a suitable construction conduit will be governed by the availability and suitability of pipe materials for local conditions with due consideration of economic factors. It is desirable to permit alternates so that bids can be received with contractor's options for the different types of pipe suitable for a specific installation. Allowing alternates serves as a means of securing bidding competition. When alternate designs are advantageous, each system will be economically designed, taking advantage of full capacity, best slope, least depth, and proper strength and installation provisions for each material involved. Where field conditions dictate the use of one pipe material in preference to others, the reasons will be clearly presented in the design analysis.

Figure 4-89. Standard Type "D" Inlet



4-4.9.1.2 Several factors should be considered in selecting the type of pipe to be used in construction. The factors include strength under either maximum or minimum cover being provided, pipe bedding and backfill conditions, anticipated loadings, length of pipe sections, ease of installation, resistance to corrosive action by liquids carried or surrounding soil materials, suitability of jointing methods, provisions for expected deflection without adverse effect on the pipe structure or on the joints or overlying materials, and cost of maintenance. Although it is possible to obtain an acceptable pipe installation to meet design requirements by establishing special provisions for several possible materials, ordinarily only one or two alternates will economically meet the individual requirements for a proposed drainage system.

4-4.9.2 **Selection of n values.** A designer is continually confronted with what coefficient of roughness n to use in a given situation. The question of whether n should be based on the new and ideal condition of a pipe or on anticipated condition at a later date is difficult to answer. Sedimentation or paved pipe can affect the coefficient of roughness. Table 4-1 gives the n values for smooth interior pipe of any size, shape, or type and for annular and helical corrugated metal pipe both unpaved and 25 percent paved. When n values other than those listed are selected, such values will be amply justified in the design analysis.

4-4.9.3 **Restricted use of bituminous-coated pipe.** Corrugated-metal pipe with any percentage of bituminous coating will not be installed where solvents can be expected to enter the pipe. Polymeric coated corrugated steel pipe is recommended where solvents might be expected.

4-4.9.4 **Minimum cover**

4-4.9.4.1 In the design and construction of the drainage system it will be necessary to consider both minimum and maximum earth cover allowable on the underground conduits to be placed under both flexible and rigid pavements. Underground conduits are subject to two principal types of loads: dead loads (DL) caused by embankment or trench backfill plus superimposed stationary surface loads, uniform or concentrated; and live or moving loads (LL), including impact. Live loads assume increasing importance with decreasing fill height.

4-4.9.4.2 AASHTO Standard Specifications for Highway Bridges should be used for all H-20 Highway Loading Analyses. AREA Manual for Railway Engineering should be used for all Cooper's E 80 Railway Loadings. Appropriate pipe manufacturer design manuals should be used for maximum cover analyses.

4-4.9.4.3 Drainage systems should be designed in order to provide an ultimate capacity sufficient to serve the planned installation, Addition to, or replacement of, drainage lines following initial construction is costly.

4-4.9.4.4 Investigations of in-place drainage and erosion control facilities at 50 military installations were made during the period 1966 to 1972. The facilities observed varied from one to more than 30 years of age. The study revealed that buried conduits and associated storm drainage facilities installed from the early 1940s until the mid-1960s appeared to be in good to excellent structural condition. However, many reported failures of buried conduits occurred during construction. Therefore, it should be noted that minimum conduit cover requirements are not always adequate during construction. When construction equipment, which may be heavier than live loads for which the conduit has been designed, is operated over or near an already in-place underground conduit, it is the responsibility of the contractor to provide any additional cover during construction to avoid damage to the conduit. Major improvements in the design and construction of buried conduits in the two decades mentioned include, among other items, increased strength of buried pipes and conduits, increased compaction requirements, and revised minimum cover tables.

4-4.9.4.5 The necessary minimum cover in certain instances may determine pipe grades. A safe minimum cover design requires consideration of a number of factors including selection of conduit material, construction conditions and specifications, selection of pavement design, selection of backfill material and compaction, and the method of bedding underground conduits. Emphasis on these factors must be carried from the design stage through the development of final plans and specifications.

4-4.9.4.6 Tables 4-6 through 4-11 identify certain suggested cover requirements for storm drains and culverts which should be considered as guidelines only. Cover requirements have been formulated for asbestos-cement pipe, reinforced and non-reinforced concrete pipe, corrugated-aluminum-alloy pipe, corrugated-steel pipe, structural-plate-aluminum- alloy pipe, and structural-plate-steel pipe. The different sizes and materials of conduit and pipe have been selected to allow the reader an appreciation for the many and varied items which are commercially available for construction purposes. The cover depths listed are suggested only for average bedding and backfill conditions. Deviations from average conditions may result in significant minimum cover requirements and separate cover analyses must be made in each instance of a deviation from average conditions. Specific bedding, backfill, and trench widths may be required in certain locations; each condition deviating from the average condition should be analyzed separately. Where warranted by design analysis the suggested maximum cover may be exceeded.

4-4.9.5 **Classes of bedding and installation**. Figures 4-1 through 4-4 indicate the classes of bedding for conduits. Figure 4-5 is a schematic representation of the subdivision of classes of conduit installation which influences loads on underground conduits.

4-4.9.6 **Strength of pipe**. Pipe shall be considered of ample strength when it meets the conditions specified for the loads indicated in Tables 4-6 through 4-13. When railway or vehicular wheel loads or loads due to heavy construction equipment (live loads, LL) impose heavier loads, or when the earth (or dead loads, DL) vary materially from those normally encountered, these tables cannot be used for pipe installation design and separate analyses must be made. The suggested minimum and maximum cover shown in the tables pertain to pipe installations in which the back fill material is compacted to at least 90 percent of CE55 (MIL-STD-621) or AASHTO-T99 density (100 percent for cohesionless sands and gravels). This does not modify requirements for any greater degree of compaction specified for other reasons. It is emphasized that proper bedding, backfilling, compaction, and prevention of infiltration of backfill material into pipe are important not only to the pipe, but also to protect overlying and nearby structures. When in doubt about minimum and maximum cover for local conditions, a separate cover analysis must be performed.

4-4.9.7 **Rigid pipe**. Tables 4-6 and 4-7 indicate maximum and minimum cover for trench conduits employing asbestos-cement pipe and concrete pipe. If positive projecting conduits are employed, they are those which are installed in shallow bedding with a part of the conduit projecting above the surface of the natural ground and then covered with an embankment. Due allowance will be made in amounts of minimum and maximum cover for positive projecting conduits. Table 4-14 suggests guidelines for minimum cover to protect the pipe during construction and the minimum finished height of cover.

4-4.9.8 **Flexible pipe**. Suggested maximum cover for trench and positive projecting conduits are indicated in Tables 4-8 through 4-11 for corrugated-aluminum-alloy pipe, corrugated-steel pipe, structural-plate-aluminum-alloy pipe, and structural-plate-steel pipe. Conditions other than those stated in the tables, particularly other loading conditions will be compensated for as necessary. For unusual installation conditions, a detailed analysis will be made so that ample safeguards for the pipe will be provided with regard to strength and resistance to deflection due to loads. Determinations for deflections of flexible pipe should be made if necessary. For heavy live loads and heavy loads due to considerable depth of cover, it is desirable that a selected material, preferably bank-run gravel or crushed stone where economically available, be used for backfill adjacent to the pipe. Table 4-14 suggests guidelines for minimum cover to protect the pipe during construction and the minimum finished height of cover.

4-4.9.9 **Bedding of pipe (culverts and storm drains)**. The contact between a pipe and the foundation on which it rests is the pipe bedding. It has an important influence on the supporting strength of the pipe. For drainpipes at military installations, the method of bedding shown in Figure 4-3 is generally satisfactory for both trench and positive projecting (embankment) installations. Some designs standardize and classify various types of bedding in regard to the shaping of the foundation, use of granular material, use of concrete, and similar special requirements. Although such refinement is not considered necessary, at least for standardized cover requirements, select, fine granular material can be used as an aid in shaping the bedding, particularly where foundation conditions are difficult. Also, where economically available, granular materials can be used to good advantage for backfill adjacent to the pipe. When culverts or storm drains are to be installed in unstable or yielding soils, under great heights of fill, or where pipe will be subjected to very heavy live loads, a method of bedding can be used in which the pipe is set in plain or reinforced concrete of suitable thickness extending upward on each side of the pipe. In some instances, the pipe may be totally encased in concrete or concrete may be placed along the side and over the

Table 4-6. Suggested Maximum Cover Requirements for Asbestos-Cement Pipe H-20 Highway Loading

Diameter in.	Suggested Maximum Cover Above Top of Pipe, ft				
	Circular Section				
	Class				
	1500	2000	2500	3000	3750
12	9	13	16	19	24
15	10	13	17	19	24
18	10	13	17	20	25
21	10	13	17	20	25
24	10	14	17	20	25
27	10	14	17	20	25
30	11	14	17	21	24
33	11	14	17	21	26
36	11	14	17	21	26
42	11	14	17	21	26

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Notes:
1. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. The number in the class designation for asbestos-cement pipe is the minimum 3-edge test load to produce failure in pounds per linear foot. It is independent of pipe diameter. An equivalent to the D-load can be obtained by dividing the number in the class designation by the internal pipe diameter in feet.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then coyer depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-7. Suggested Maximum Cover Requirements for Concrete Pipe, Reinforced Concrete, H-20 Highway Loading

Diameter in.	Suggested Maximum Cover Above Top of Pipe, ft — Circular Section — Class				
	1500	2000	2500	3000	3750
12	9	13	16	19	24
24	10	13	17	19	24
36	10	13	17	20	25
48	10	13	17	20	25
60	10	14	17	20	25
72	10	14	17	20	25
84	11	14	17	21	24
108	11	14	17	21	26

Non-Reinforced Concrete

Diameter in.	Suggested Maximum Cover Above Top of Pipe, ft — Circular Section		
	I	II	III
12	14	14	17
24	13	13	14
36	9	12	12

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Notes:
1. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. “D” loads listed for the various classes of reinforced-concrete pipe are the minimum required 3-edge test loads to produce ultimate failure in pounds per linear foot of interval pipe diameter.
7. Each diameter pipe in each class designation of non-reinforced concrete has a different D-load value which increases with wall thickness.
8. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
9. See Table 4-14 for suggested minimum cover requirements.

Table 4-8. Suggested Maximum Cover Requirements for Corrugated-Aluminum-Alloy Pipe, Riveted, Helical, or Welded Fabrication 2-2/3-in. Spacing, 1/2-in.-Deep Corrugations, H-20 Highway Loading

Diameter in.	Suggested Maximum Cover Above Top of Pipe, ft									
	Circular Section					Vertically Elongated Section				
	Thickness, in.					Thickness, in.				
	.060	.075	.105	.135	.164	.060	0.075	.105	.135	.164
12	50	50	86	90	93					
15	40	40	69	72	74					
18	33	33	57	60	62					
24	25	25	43	45	46					
30	20	20	34	36	37					
36	16	16	28	30	31					
42	16	16	28	30	31			50	52	53
48			28	30	31			43	45	47
54			28	30	31					
60				30	31					
66					31					
72					31					

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Notes:
1. Corrugated-aluminum-alloy pipe will conform to the requirements of Federal Specification WW-P-402.
2. The suggested values shown are for average conditions and are to be considered as guidelines only for deal load plus H-20 live load. Cooper E-80 railway loadings should be independently made.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. Vertical elongation will be accomplished by shop fabrication and will generally be 5 percent of the pipe diameter.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-9. Suggested Maximum Cover Requirements for Corrugated-Steel-Pipe, 2-2/3-in. Spacing, 1/2-in.-Deep Corrugations

H-20 Highway Loading												
Maximum Cover Above Top of Pipe, feet												
Diameter, in.	Riveted – Thickness, in.						Helical – Thickness, in.					
	.052	.064	.079	.109	.138	.168	.052	.064	.079	.109	.138	.168
12	92	92	101	130			170	213	266	372		
15	74	74	80	104			136	170	212	298		
18	61	61	67	86			113	142	173	212		
21	53	53	57	74			97	121	139	164		
24	46	46	50	65	68		85	106	120	137	155	
27	41	41	44	57	60		75	94	109	120	133	
30	37	37	40	52	54		68	85	101	110	119	
36	30	30	33	43	45		56	71	88	98	103	
42	34	34	47	74	77	81	48	60	76	92	95	99
48		30	41	65	68	71		53	66	88	91	93
54			36	57	60	63			59	82	88	90
60				52	54	57				74	86	87
66					49	51					85	86
72					45	47					79	85
78						43						84
84						40						75

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the tables are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-10. Suggested Maximum Cover Requirements for Structural-Plate-Aluminum-Alloy Pipe, 9-in. Spacing, 2-1/2-in. Corrugations

H-20 Highway Loading							
Diameter, in.	Suggested Maximum Cover Above Top of Pipe, ft						
	Circular Section						
	Thickness, in.						
	0.10	0.125	0.15	0.175	0.20	0.225	0.250
72	24	32	41	48	55	61	64
84	20	27	35	41	47	52	55
96	18	24	30	36	41	45	50
108	16	21	27	32	37	40	44
120	14	19	24	29	33	36	40
132	13	17	22	26	30	33	36
144	12	16	20	24	27	30	33
156		14	18	22	25	28	30
168		13	17	20	23	26	28
180			16	19	22	24	26

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Notes:
1. Structural-plate-aluminum-alloy pipe will conform to the requirements of Federal Specification WW-P-402.
2. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
3. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
4. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
5. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
6. The number in the class designation for asbestos-cement pipe is the minimum 3-edge test load to produce failure in pounds per linear foot. It is independent of pipe diameter. An equivalent to the D-load can be obtained by dividing the number in the class designation by the internal pipe diameter in feet.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-11. Suggested Maximum Cover Requirements for Corrugated Steel Pipe, 125-mm Span, 25-mm-Deep Corrugations

H-20 Highway Loading					
Diameter, in.	Maximum cover above top of pipe, ft				
	Helical—thickness, in.				
	.064	.079	.109	.138	.168
48	54	68	95	122	132
54	48	60	84	109	117
60	43	54	76	98	107
66	39	49	69	89	101
72	36	45	63	81	96
78	33	41	58	75	92
84	31	38	54	70	85
90	29	36	50	65	80
96		34	47	61	75
102		32	44	57	70
108			42	54	66
114			40	51	63
120			38	49	60

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-12. Suggested Maximum Cover Requirements for Structural Plate Steel Pipe, 6-in. Span, 2-in.-Deep Corrugations

H-20 Highway Loading							
	Maximum Cover Above Top of Pipe, ft						
	Thickness, in.						
Diameter, ft	.109	.138	.168	.188	.218	.249	.280
5.0	46	68	90	103	124	146	160
5.5	42	62	81	93	113	133	145
6.0	38	57	75	86	103	122	133
6.5	35	52	69	79	95	112	123
7.0	33	49	64	73	88	104	114
7.5	31	45	60	68	82	97	106
8.0	29	43	56	64	77	91	100
8.5	27	40	52	60	73	86	94
9.0	25	38	50	57	69	81	88
9.5	24	36	47	54	65	77	84
10.0	23	34	45	51	62	73	80
10.5	22	32	42	49	59	69	76
11.0	21	31	40	46	56	66	72
11.5	20	29	39	44	54	63	69
12.0	19	28	37	43	51	61	66
12.5	18	27	36	41	49	58	64
13.0	17	26	34	39	47	56	61
13.5	17	25	33	38	46	54	59
14.0	16	24	32	36	44	52	57
14.5	16	23	31	35	42	50	55
15.0	15	22	30	34	41	48	53
15.5	15	22	29	33	40	47	51
16.0		21	28	32	38	45	50
16.5		20	27	31	37	44	48
17.0		20	26	30	36	43	47
17.5		19	25	29	35	41	45
18.0			25	28	34	40	44
18.5			24	27	33	39	43
19.0			23	27	32	38	42
19.5			23	26	31	37	41
20.0				25	31	36	40
20.5				25	30	35	39
21.0					29	34	38
21.5					28	34	37
22.0					28	33	36
22.5					27	32	35
23.0						31	34
23.5						31	34
24.0						30	33
24.5							32
25.0							32
25.5							31

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-13. Suggested Maximum Cover Requirements for Corrugated Steel Pipe, 3-in. Span, 1-in. Corrugations

H-20 Highway Loading										
Diameter, in.	Maximum Cover Above Top of Pipe, ft									
	Riveted - Thickness, in.					Helical – Thickness, in.				
	.064	.079	.109	.138	.168	.064	.079	.109	.138	.168
36	53	66	98	117	130	81	101	142	178	201
42	45	56	84	101	112	69	87	122	142	157
48	39	49	73	88	98	61	76	107	122	132
54	35	44	65	78	87	54	67	95	110	117
60	31	39	58	70	78	48	61	85	102	107
66	28	36	53	64	71	44	55	77	97	101
72	26	33	49	58	65	40	50	71	92	96
78	24	30	45	54	60	37	47	65	84	93
84	22	28	42	50	56	34	43	61	78	91
90	21	26	39	47	52	32	40	57	73	89
96		24	36	44	49		38	53	69	84
102		23	34	41	46		35	50	64	79
108			32	39	43			47	61	75
114			30	37	41			45	58	71
120			29	35	39			42	55	67

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Notes:
1. Corrugated steel pipe will conform to the requirements of Federal Specification WW-P-405.
2. The suggested maximum heights of cover shown in the table are calculated on the basis of the current AASHTO Standard Specifications for Highway Bridges and are based on circular pipe.
3. Soil conditions, trench width and bedding conditions vary widely throughout varying climatic and geographical areas.
4. Calculations to determine maximum cover should be made for all individual pipe and culvert installations underlying roads, streets and open storage areas subject to H-20 live loads. Cooper E-80 railway loadings should be independently made.
5. Cover depths are measured from the bottom of the subbase of pavements, or the top of unsurfaced areas, to top of pipe.
6. Calculations to determine maximum cover for Cooper E-80 railway loadings are measured from the bottom of the tie to the top of the pipe.
7. If pipe produced by a manufacturer exceeds the strength requirements established by indicated standards then cover depths may be adjusted accordingly.
8. See Table 4-14 for suggested minimum cover requirements.

Table 4-14. Suggested Guidelines for Minimum Cover

H-20 Highway Loading			
	Minimum Cover to Protect Pipe		Minimum Finished Height of Cover (From Bottom of Subbase, toTop of Pipe)
	Pipe Diameter, in.	Height of Cover During Construction, ft	
Asbestos-Cement Pipe	12 to 42	Diameter/2 or 3.0 ft whichever is greater	Diameter/2 or 2.0 ft whichever is greater
Concrete Pipe Reinforced	12 to 108	Diameter/2 or 3.0 ft whichever is greater	Diameter/2 or 2.0 ft whichever is greater
Non-Reinforced	12 to 36	Diameter/2 or 3.0 ft whichever is greater	Diameter/2 or 2.0 ft whichever is greater
Corrugated Aluminum Pipe 2-2/3 in. by 1/2 in.	12 to 24 30 and over	1.5 ft Diameter	Diameter/2 or 1.0 ft whichever is greater Diameter/2
Corrugated Steel Pipe 3 in. by 1 in.	12 to 30 36 and over	1.5 ft Diameter	Diameter/2 or 1.0 ft whichever is greater Diameter/2
Structural Plate Aluminum Alloy Pipe 9 in. by 2-1/2 in.	72 and over	Diameter/2	Diameter/4
Structural Plate Steel 6 in. by 2 in.	60 and over	Diameter/2	Diameter/4

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Notes:
1. All values shown above are for average conditions and are to be considered as guidelines only.
2. Calculations should be made for minimum cover for all individual pipe installation for pipe underlying roads, streets and open storage areas subject to H-20 live loads.
3. Calculations for minimum cover for all pipe installations should be separately made for all Cooper E-80 railroad live loading.
4. In seasonal frost areas, minimum pipe cover must meet requirements of Table 2-3 of TM 5-820-3 for protection of storm drains.
5. Pipe placed under rigid pavement will have minimum cover from the bottom of the subbase to the top of pipe of 1.0 ft for pipe up to 60 in. and greater than 1.0 ft for sizes above 60 in. if calculations so indicate.
6. Trench widths depend upon varying conditions of construction but may be as wide as is consistent with space required to install the pipe and as deep as can be managed from practical construction methods.
7. Non-reinforced concrete pipe is available in sizes up to 36 in.
8. See Tables 4-6 through 4-13 for suggested minimum cover requirements.

top of the pipe (top or arch encasement) after proper bedding and partial backfilling. Pipe manufacturers will be helpful in recommending type and specific requirements for encased, partially encased, or specially reinforced pipe in connection with design for complex conditions.

4-4.10 **Manholes and Junction Boxes**. Drainage systems require a variety of appurtenances to assure proper operations. Most numerous appurtenances are manholes and junction boxes. Manholes and junction boxes are generally constructed of any suitable materials such as brick, concrete block, reinforced concrete, precast reinforced-concrete sections, or preformed corrugated metal sections. Manholes are located at intersections, changes in alignment or grade, and at intermediate joints in the system up to every 500 ft. Junction boxes for large pipes are located as necessary to assure proper operation of the drainage system. Inside dimensions of manholes will not be less than 2.5 ft. Inside dimensions of junction boxes will provide for not less than 3 in. of wall on either side of the outside diameter of the largest pipes involved. Manhole frames and cover will be provided as required; rounded manhole and box covers are preferred to square covers. Slab top covers will be provided for large manholes and junction boxes too shallow to permit corbeling of the upper part of the structure. A typical large box drain cover is shown in Figure 4-10. Fixed ladders will be provided depending on the depth of the structures. Access to manhole and junction boxes without fixed ladders will be by portable ladders. Manhole and junction box design will insure minimum hydraulic losses through them. Typical manhole and junction box construction is shown in Figures 4-90 through 4-92.

4-4.11 **Detention Pond Storage**. Hydrologic studies of the drainage area will reveal if detention ponds are required. Temporary storage or ponding may be required if the outflow from a drainage area is limited by the capacity of the drainage system serving a given area. A full discussion of temporary storage or ponding design will be found in Section 3-11. Ponding areas should be designed to avoid creation of a facility that would be unsightly, difficult to maintain, or a menace to health or safety.

4-4.12 **Outlet Energy Dissipators**

4-4.12.1 Most drainage systems are designed to operate under normal free outfall conditions. Tailwater conditions are generally absent. However, it is possible for a discharge resulting from a drainage system to possess kinetic energy in excess of that which normally occurs in waterways. To reduce the kinetic energy, and thereby reduce downstream scour, outfalls may sometimes be required to reduce streambed scour. Scour may occur in the streambed if discharge velocities exceed the values listed in Table 4-15. These values are to be used only as guides; studies of local materials must be made prior to a decision to install energy dissipation devices. Protection against scour may be provided by plain outlets, transitions and stilling basins. Plain outlets provide no protective works and depend on natural material to resist erosion. Transitions provide little or no dissipation of energy themselves, but by spreading the effluent jet to approximately the flow cross-section of the natural channel, the energy is greatly reduced prior to releasing the effluent into the outlet channel. Stilling basins dissipate the high kinetic energy of flow by a hydraulic jump or other means. Riprap may be required at any of the three types of outfalls.

Table 4-15. Maximum Permissible Mean Velocities to Prevent Scour

Material	Maximum Permissible Mean Velocity
Uniform graded sand and cohesionless silts	1.5 fps
Well-graded sand	2.5 fps
Silty sand	3.0 fps
Clay	4.0 fps
Gravel	6.0 fps

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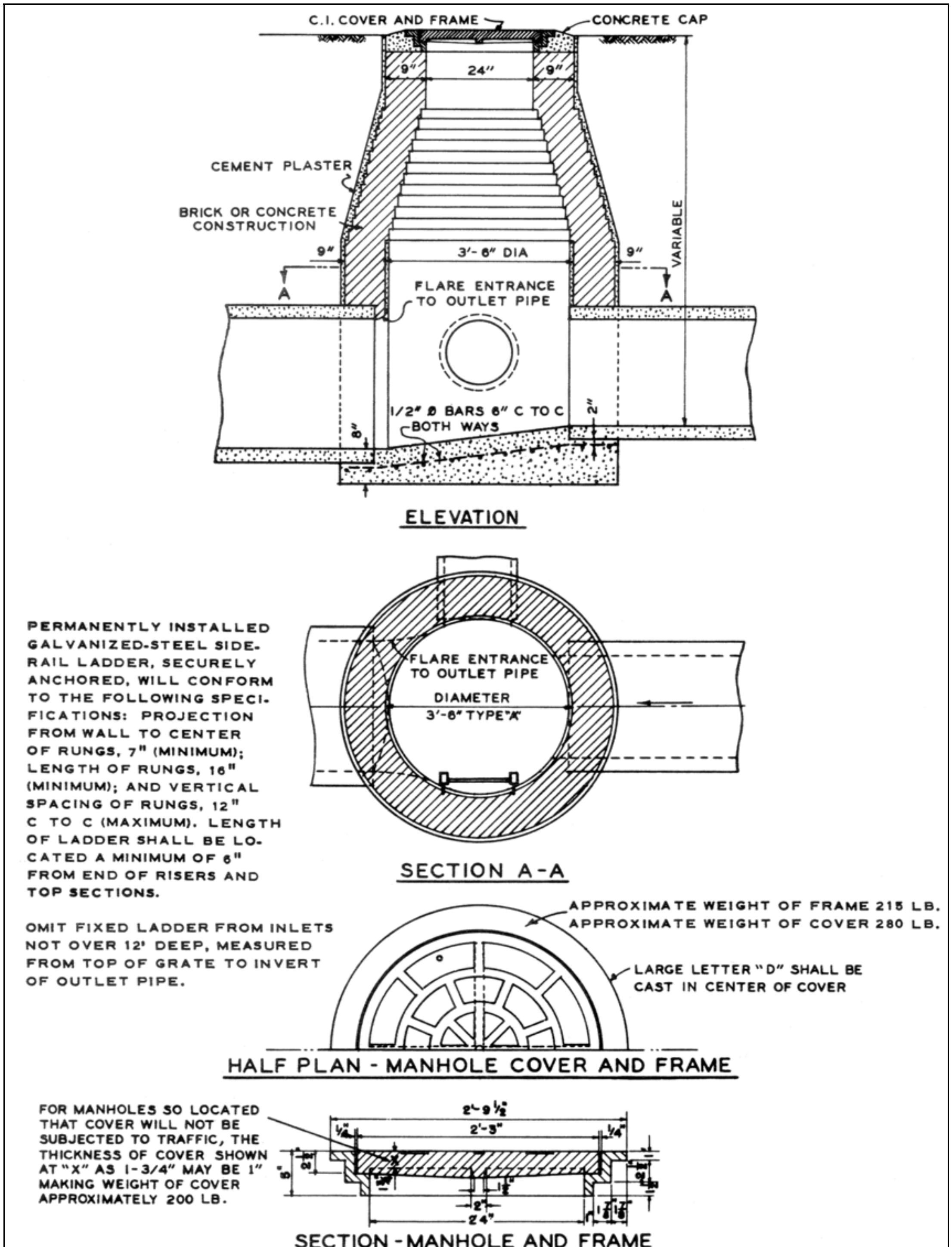
4-4.12.1.1 **Plain type**

a. If the discharge channel is in rock or a material highly resistant to erosion, no special erosion protection is required. However, since flow from the culvert will spread with a resultant drop in water surface and increase in velocity, this type of outlet should be used without riprap only if the material in the outlet channel can withstand velocities about 1.5 times the velocity in the culvert. At such an outlet, side erosion due to eddy action or turbulence is more likely to prove troublesome than is bottom scour.

b. Cantilevered culvert outlets may be used to discharge a free-falling jet onto the bed of the outlet channel. A plunge pool will be developed, the depth and size of which will depend on the energy of the falling jet at the tailwater and the erodibility of the bed material.

4-4.12.1.2 **Transition type.** Endwalls (outfall headwalls) serve the dual purpose of retaining the embankment and limiting the outlet transition boundary. Erosion of embankment toes usually can be traced to eddy attack at the ends of such walls. A flared transition is very effective, if proportioned so that eddies induced by the effluent jet do not continue beyond the end of the wall or overtop a sloped wall. As a guide, it is suggested that the product of velocity and flare angle should not exceed 150. That is, if effluent velocity is 5 ft/sec each wingwall may flare 30 degrees; but if velocity is 15 ft/sec, the flare should not exceed 10 degrees. Unless wingwalls can be anchored on a stable foundation, a paved apron between the wingwalls is required. Special care must be taken in design of the structure to preclude undermining. A newly excavated channel may be expected to degrade, and proper allowance for this action should be included in establishing the apron elevation and depth of cutoff wall. Warped endwalls provide excellent transitions in that they result in the release of flow in a trapezoidal section, which generally approximates the cross section of the outlet channel. If a warped transition is placed at the end of a curved section below a culvert, the transition is made at the end of the curved section to minimize the possibility of overtopping due to superelevation of the water surface. A paved apron is required with warped endwalls. Riprap usually is required at the end of a transition-type outlet.

Figure 4-90. Standard Storm Drain Manhole



U.S. Army Corps of Engineers

Figure 4-91. Standard Precast Manholes

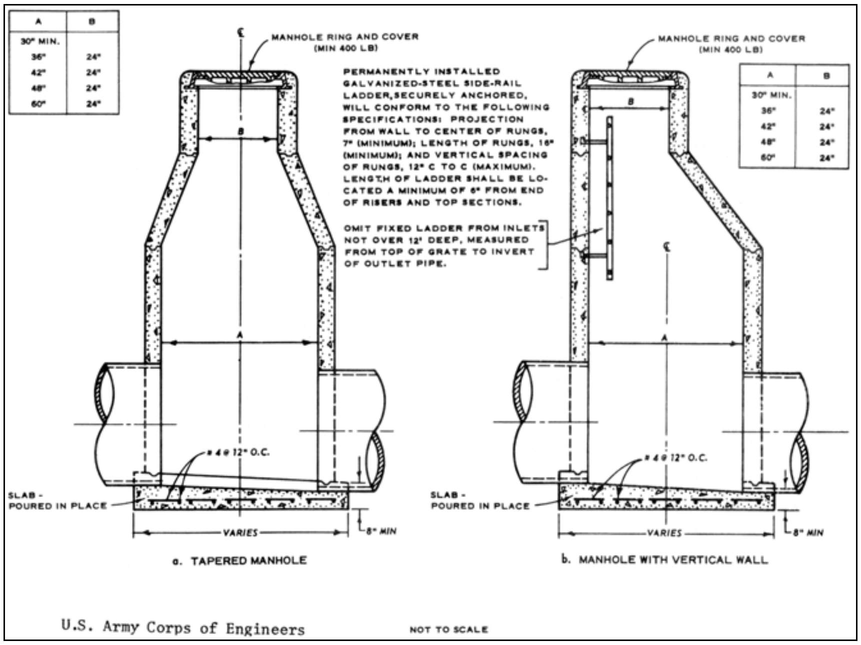
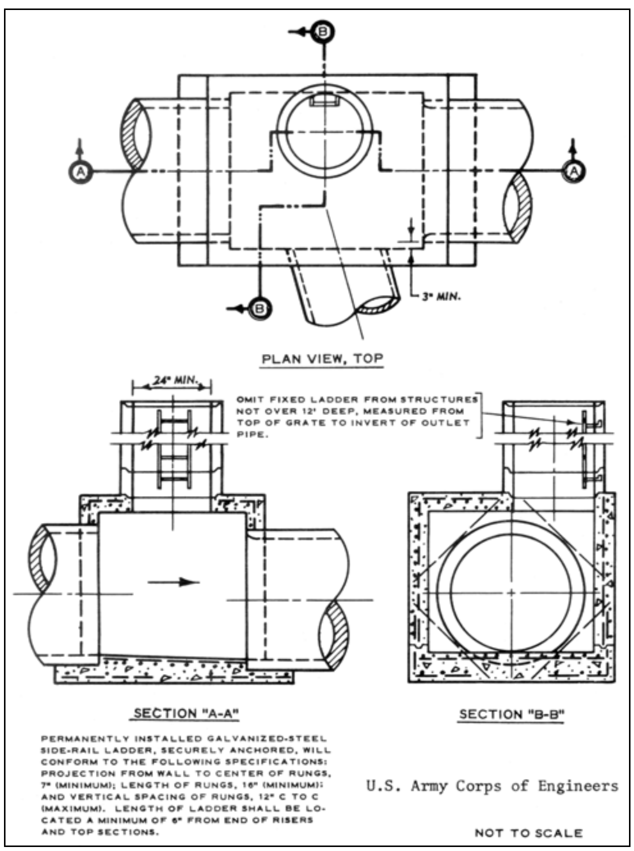


Figure 4-92. Junction Details for Large Pipes



4-4.12.1.3 **Stilling basins.** A detailed discussion of stilling basins for circular storm drain outlets can be found in Section 4-2.6.

4-4.12.2 Improved channels, especially the paved ones, commonly carry water at velocities higher than those prevailing in the natural channels into which they discharge. Often riprap will suffice for dissipation of excess energy. A cutoff wall may be required at the end of a paved channel to preclude undermining. In extreme cases a flared transition, stilling basin, or impact device may be required.

4–4.13 **Drop Structures and Check Dams.** Drop structures and check dams are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. The structures also provide satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 ft and over embankments higher than 5 ft provided the end sill of the drop structure extends beyond the toe of the embankment. The check dam is a modification of the drop structure used for erosion control in small channels where a less elaborate structure is permissible. Pertinent design features are covered in Section 4-2.4.

4-4.14 **Miscellaneous Structures**

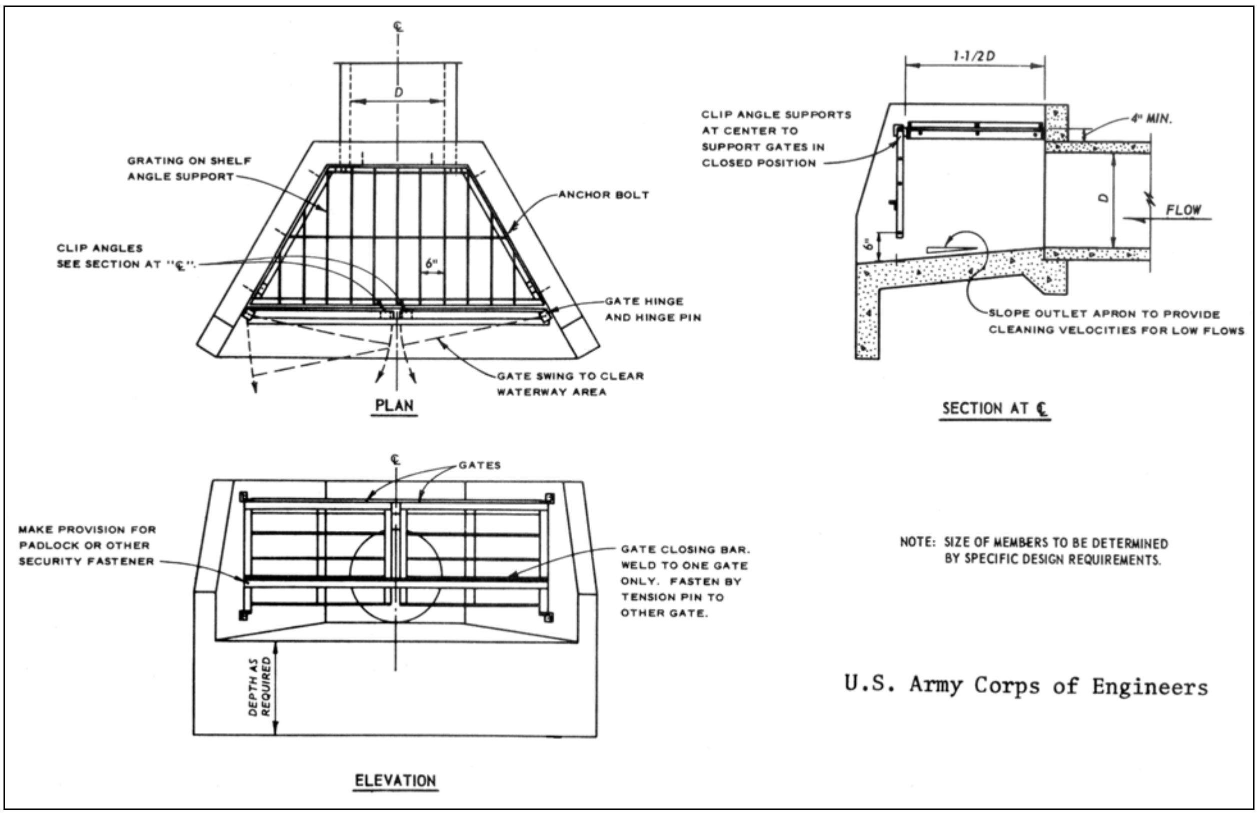
4-4.14.1 A chute is a steep open channel which provides a method of discharging accumulated surface runoff over fills and embankments. A typical design is included in Section 4-2.5.

4-4.14.2 When a conduit or channel passes through or beneath a security fence and forms an opening greater than 96 in.^{2} in area a security barrier must be installed. Barriers are usually of bars, grillwork, or chain-link screens, parallel bars used to prevent access will be spaced not more than 6 in. apart, and will be of sufficient strength to preclude bending by hand after assembly.

4-4.14.2.1 Where fences enclose maximum security areas such as exclusion and restricted areas, drainage channels, ditches, and equalizers will, wherever possible, be carried under the fence in one or more pipes having an internal diameter of not more than 10 in. Where the volume of flow is such that the multipipe arrangement is not feasible, the conduit or culvert will be protected by a security grill composed of 3/4-in.-diameter rods or 1/2-in. bars spaced not more than 6 in. on center, set and welded in an internal frame. Where rods or bars exceed 18 in. in length, suitable spacer bars will be provided at not more than 18 in. on center, welded at all intersections. Security grills will be located inside the protected area. Where the grill is on the downstream end of the culvert, the grill will be hinged to facilitate cleaning and provided with a latch and padlock, and a debris catcher will be installed in the upstream end of the conduit or culvert. Elsewhere the grill will be permanently attached to the culvert. Security regulations normally require the guard to inspect such grills at least once every shift. For culverts in rough terrain, steps will be provided to the grill to facilitate inspection and cleaning.

4-4.14.2.2 For culverts and storm drains, barriers at the intakes would be preferable to barriers at the outlets because of the relative ease of debris removal. However, barriers at the outfalls are usually essential; in these cases consideration should be given to placing debris interceptors at the inlets. Bars constituting a barrier should be placed in a horizontal position, and the number of vertical members should be limited in order to minimize clogging; the total clear area should be at least twice the area of the conduit or larger under severe debris conditions. For large conduits an elaborate cagelike structure may be required. Provisions to facilitate cleaning during or immediately after heavy runoff should be made. Figure 4-93 shows a typical barrier for the outlet of a pipe drain. It will be noted that a 6-in. underclearance is provided to permit passage of normal bedload material, and that the apron between the conduit outlet and the barrier is placed on a slope to minimize deposition of sediment on the apron during ordinary flow. Erosion protection, where required, is placed immediately downstream from the barrier.

Figure 4-93. Outlet Security Barrier



4-4.14.2.3 If manholes must be located in the immediate vicinity of a security fence their covers must be so fastened as to prevent unauthorized opening.

4-4.14.2.4 Open channels may present special problems due to the relatively large size of the waterway and the possible requirements for passage of large floating debris. For such channels, a barrier should be provided that can be unfastened and opened or lifted during periods of heavy runoff or when clogged. The barrier is hinged at the top and an empty tank is welded to it at the bottom to serve as a float. Open channels or swales which drain relatively small areas and whose flows carry only minor quantities of debris may be secured merely by extending the fence down to a concrete sill set into the sides and across the bottom of the channel.

4-4.15 **Notation**

A Drainage area, acres, total area of clear opening, or cross-sectional area of flow, ft^2

AHW Allowable Headwater depth, ft

B Width, ft

C Coefficient

D Height of culvert barrel, ft

d Depth of thickness of grate, ft

$\mathbf{d_c}$ Critical depth, ft

F Infiltration rate, in/hr

g Acceleration due to gravity, ft/sec^2

H Depth of water, ft

$\mathbf{H_f}$ Headloss due to friction, ft

HW Headwater, ft

$\mathbf{h_0}$ Distance from culvert invert at the outlet to the control elevation, ft

I Rainfall intensity, in./hr

i Hydraulic gradient

K Constant

$\mathbf{K_e}$ Coefficient

k Coefficient of permeability

L Length of slot or gross perimeter of grate opening, or length, ft

$\mathbf{L_1}$ Adjusted length, ft

L_s Length of spiral, ft (nonsuperelevated channel)

L_t Length of spiral, ft (superelevated channel)

n Manning's roughness coefficient

Q Discharge or peak rate of runoff, cfs

R Hydraulic radius, ft

R_c Radius of curvature center line of channel, ft

S Slope of energy gradient, ft/ft

S_0 Slope of flow line, ft/ft

T Top width at water surface, ft

TW Tailwater, ft

V Mean velocity of flow, ft/sec

v Discharge velocity in Darcy's law, ft/sec

y Depth of water, ft