TM 5-820-3
AFM 88-5, CHAP. 3

TECHNICAL MANUAL

DRAINAGE AND EROSION-CONTROL STRUCTURES FOR AIRFIELDS AND HELIPORTS

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DEPARTMENTS OF THE ARMY, AND THE AIR FORCE

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HEADQUARTERS
DEPARTMENTS OF THE
ARMY AND THE AIR FORCE
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DRAINAGE AND EROSION-CONTROL STRUCTURES FOR AIRFIELDS AND HELIPORTS

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*This manual supersedes TM 5-820-3, dated 30 January 1978.

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CHAPTER 1
INTRODUCTION

1-1. Purpose.

This manual 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.

1-2. Scope.

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

1-3. References.

Appendix A contains a list of references used in this document.

1-4. Problem areas.

a. 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.

b. 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.

c. 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.

1-5. Design.

a. 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.

b. 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.

c. 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.

1-6. 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.

1-7. Drainage law.

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

b. 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.

c. 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.

d. Both Roman civil law 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.

CHAPTER 2

DRAINAGE PIPE

2-1. 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.

2-2. Selection of type of pipe.

a. 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.

b. 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.

2-3. 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 2-1 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.

Table 2-1. 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 inch	0.024	0.021
3 by 1 inch	0.027	0.023
6 by 2 inch	0.028-0.033	0.024-0.028
9 by 2 + 1/2 inch	0.033	0.028

n values for helical corrugated metal (2 + 2/3 by 1/2 inch corrugations)

Pipe diameter	Unpaved	25% Paved
12-18 inches	0.011-0.014	X
24-30 inches	0.016-0.018	0.015-0.016
36-96 inches	0.019-0.024	0.017-0.021

* Includes asbestos cement, plastic, cast iron, clay, concrete (precast or cast-in-place) or fully paved corrugated metal pipe.

2-4. 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.

2-5. Minimum and maximum cover.

a. 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.

b. 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.

c. 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 and associated storm drainage facilities installed from the early 1940's until the mid-1960's 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, is 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.

d. Since 1940 gross aircraft weight has increased twenty-fold, from 35,000 pounds to approximately 700,000 pounds. 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 2 decades mentioned include among other items increased strength of buried pipes and conduits, increased compaction requirements, and revised minimum and maximum cover tables.

e. For minimum and maximum cover design, H-20, 15-K, F-15, C-5A, C-141, C-130, B-I and B-52 live loads and 120 pounds per cubic foot 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 reinforced concrete pipe. Nonreinforced concrete and vitrified clay pipe design are based on the American Concrete Pipe Association's D-load design procedure based on a 0.01-inch crack.

f. The tables (B-I through B-23) in appendix B 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 2-2 shows cover height reductions for standard riveted and bolted seams which do not develop a strength equivalent to f_y = 33,000 pounds per square inch. 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 2-2 can be determined by multiplying the maximum cover height for a continuously-welded or lock seam pipe (app B) by the reduction factors shown in table 2-2.

Table 2-2. Maximum cover height reduction factors for riveted and bolted seams.

Thickness in.	Gage	5/16 in. Rivets 2-2/3 × 1/2 in. Single	5/16 in. Rivets 2-2/3 × 1/2 in. Double	3/8 in. Rivets 2-2/3 × 1/2 in. Single	3/8 in. Rivets 2-2/3 × 1/2 in. Double	3/8 in. Rivets 3 × 1 in. Double	7/16 in. Rivets 3 × 1 in. Double	3/4 in. Bolts 6 × 2 in. 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	

g. Figures 2-1, 2-2, 2-3, and 2-4 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 2-5 is a schematic representation of the subdivision of classes of conduit installation which influences loads on underground conduits.

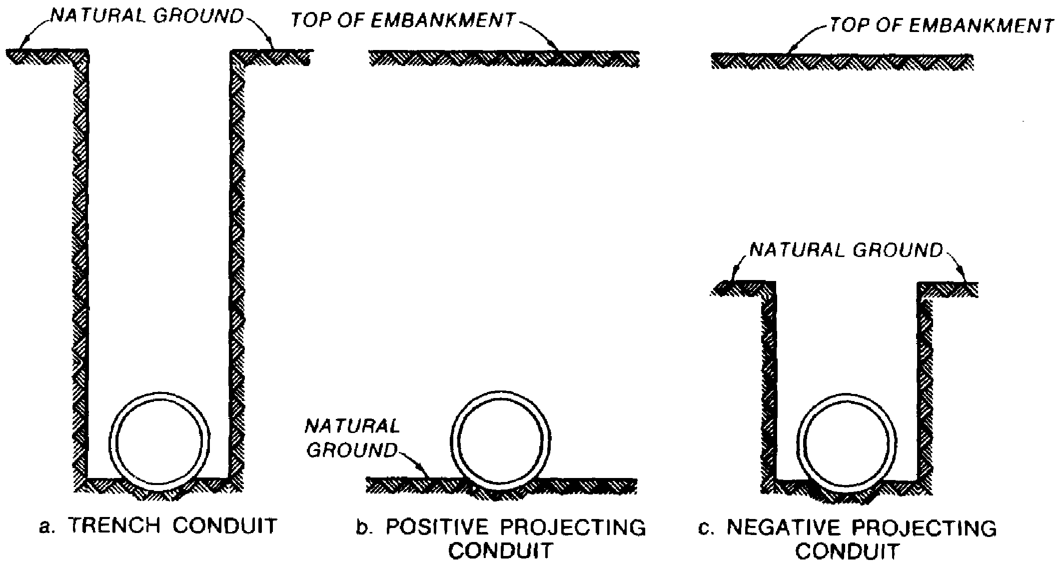


Figure 2-1. Three main classes of conduits.

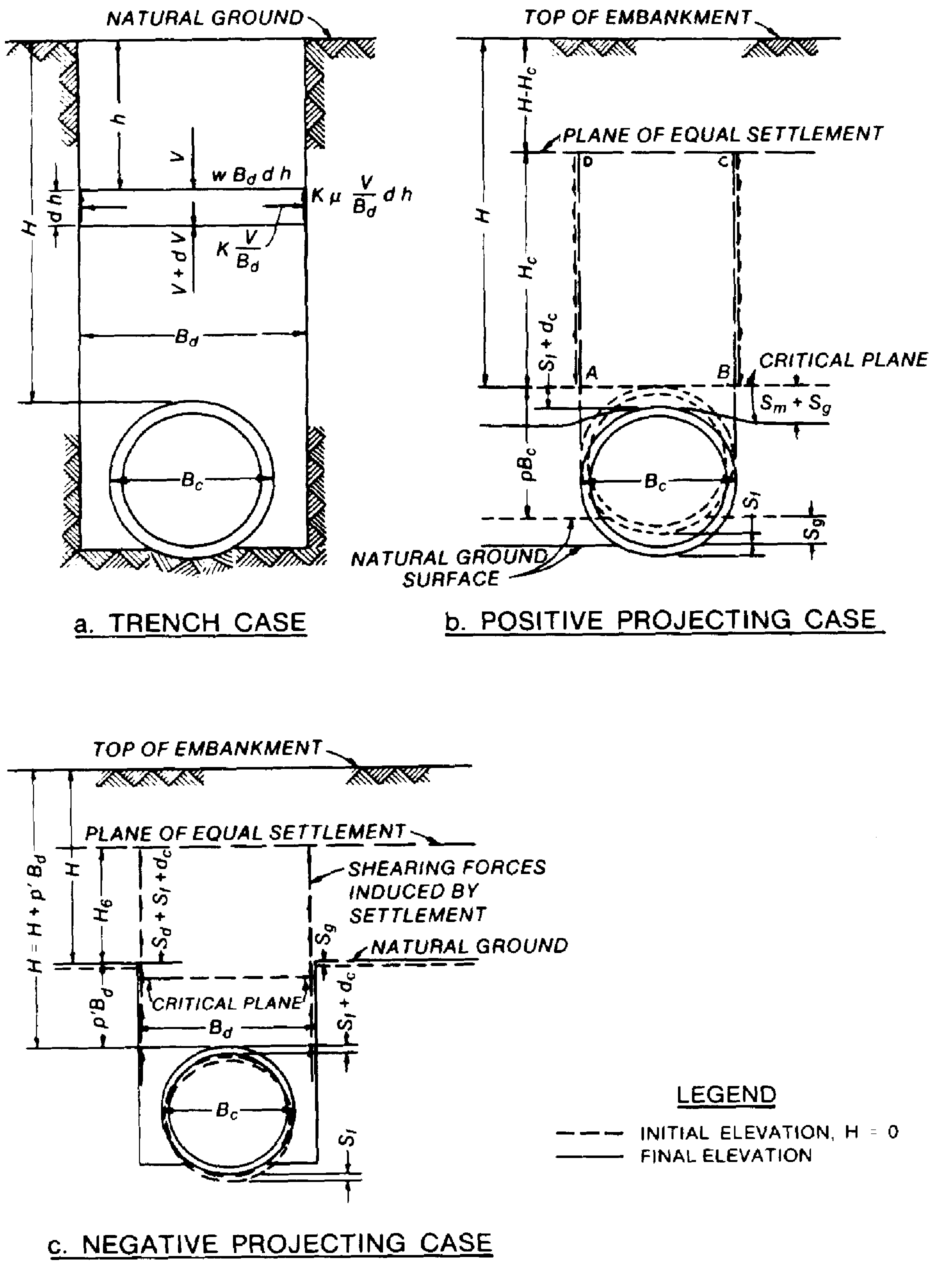
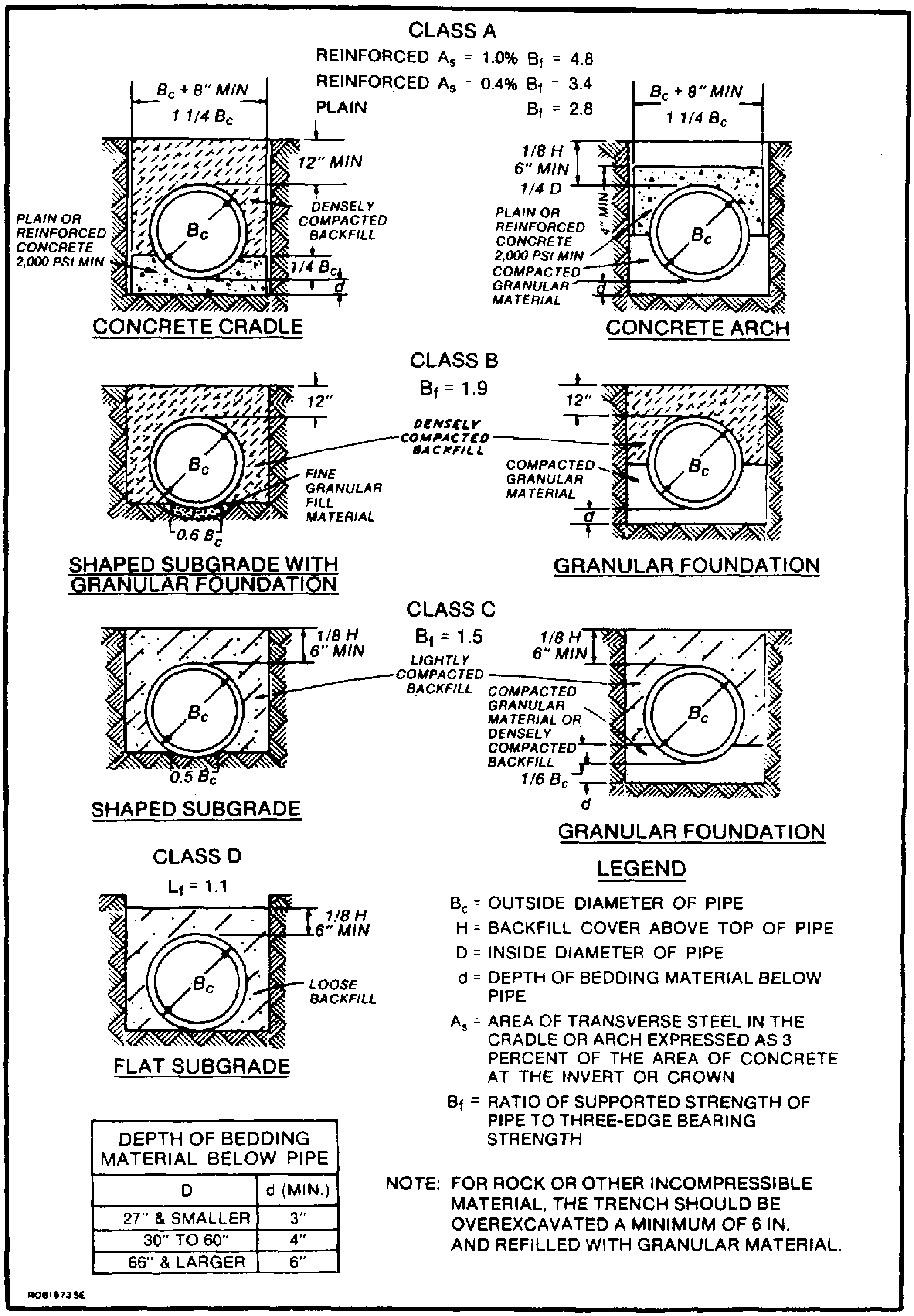


Figure 2-2. Free body conduit diagrams.



DEPTH OF BEDDING MATERIAL BELOW PIPE	
D	d (MIN.)
27" & SMALLER	3"
30" TO 60"	4"
66" & LARGER	6"

Figure 2-3. Trench beddings for circular pipe.

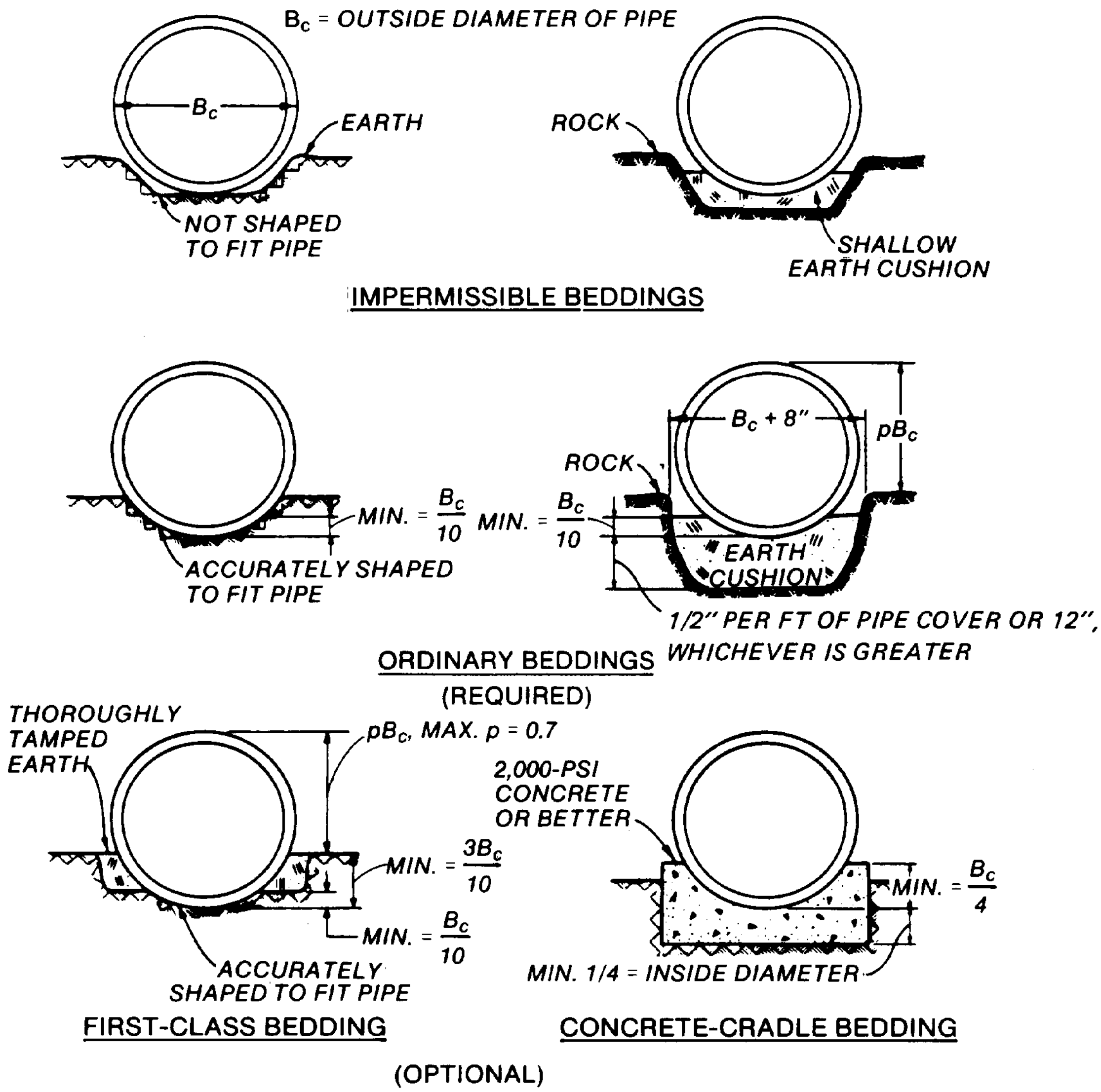


Figure 2-4. Beddings for positive projecting conduits.

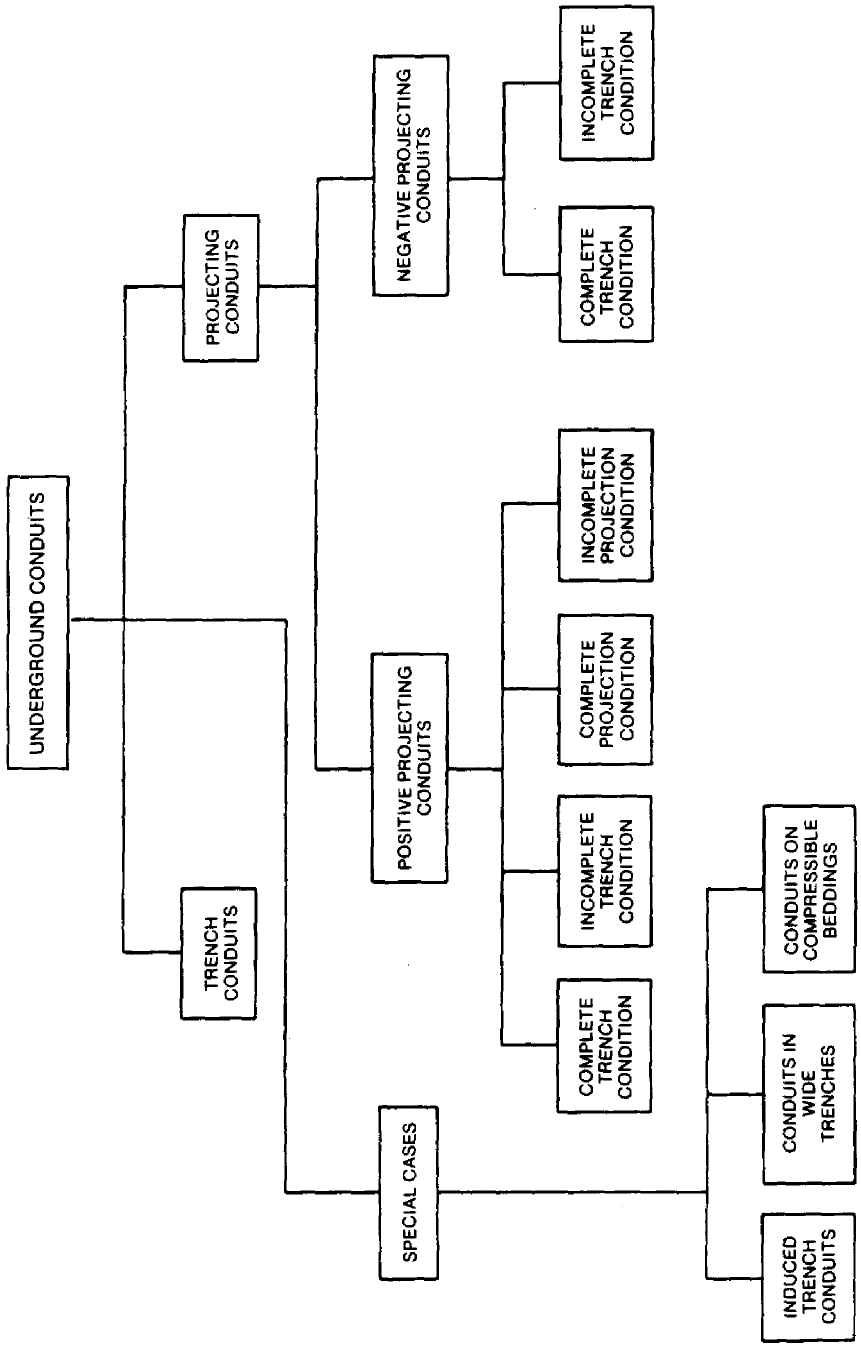


Figure 2-5. Installation conditions which influence loads on underground conduits.

2-6. 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.

a. 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.

b. No special measures are required to prevent heave in nonfrost-susceptible subgrades. In frost-susceptible subgrades where the highest ground-water table is 5 feet 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 feet below the depth of maximum frost penetration and the pipe diameter is 18 inches or more, one of the following measures should be taken:

(1) 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.

(2) Place the centerline of the pipe one-third diameter below the depth of maximum frost penetration.

c. 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 inches 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 TM 5-852-7/AFM 88-19, chapter 7.

d. The following design notes should be considered for installations located in seasonal frost areas.

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

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

(3) *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.

(4) *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-year period, or the air-freezing index for the coldest winter in the past 10-year 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.

(5) *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.

2-7. Infiltration of fine soils through drainage pipe joints.

a. 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.

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

c. 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 and 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.

d. 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.

e. 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 inches with provisions for drainage should be provided beneath pavements constructed on fine-grained subgrades and subject to perched water table conditions. Base courses containing more than 10 percent fines cannot be drained and remain saturated continuously.

CHAPTER 3

INLETS AND BOX DRAINS

3-1. General.

a. 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.

b. 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.

c. Local practices and requirements governing field inlets greatly influence design and construction details. Experience has indicated that the features described in paragraph 3-2 should be considered by the designer.

3-2. 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.

3-3. Design features.

a. 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.

b. 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 feet 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.

c. 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.

d. 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 (fig 3-1) indicate that vortex action influences the discharge characteristics when the head exceeds 0.4 foot. Hydraulically acceptable grates will result if the design criteria in the above figure are applied. For the entire area, the system of grates and their individual capacity will depend on the quantity of runoff to be handled and the allowable head at the grates. Head limitations should not exceed 0.5 foot.

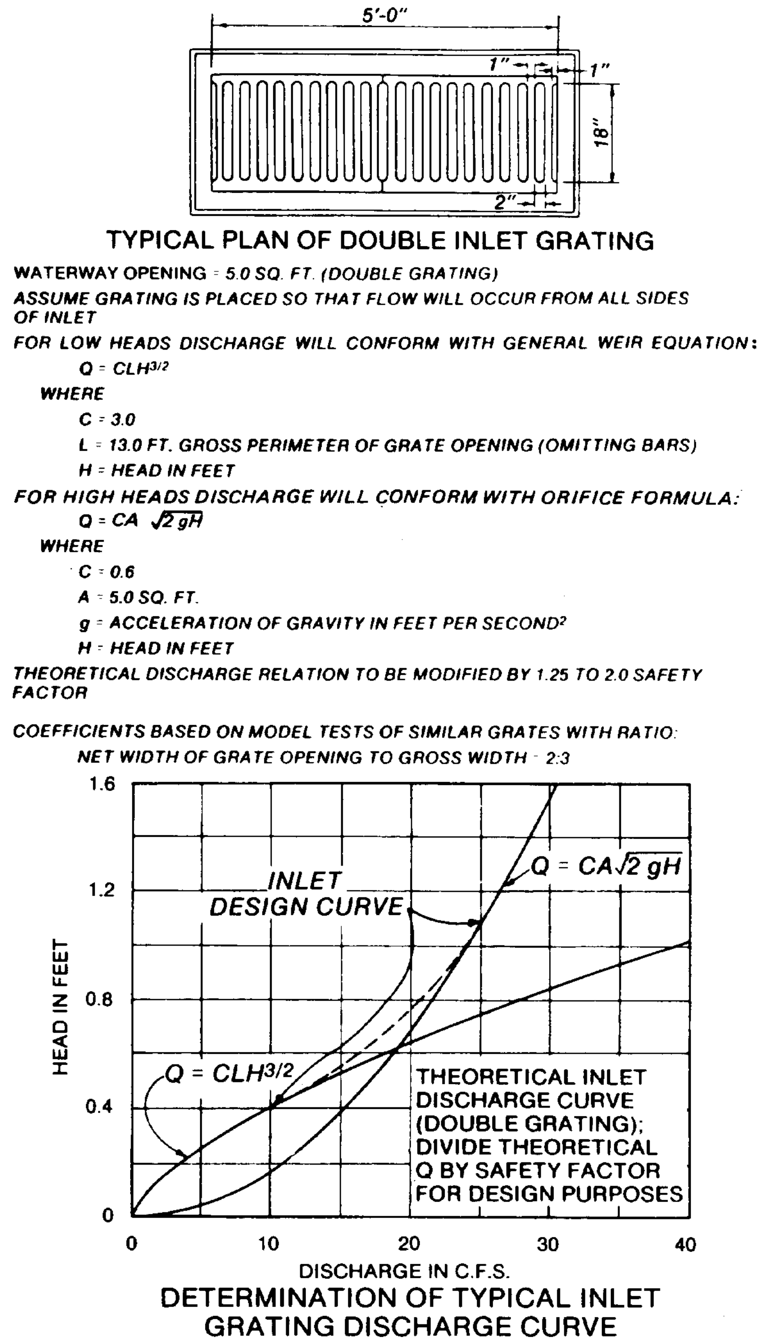


Figure 3-1. Determination of typical inlet grating discharge curve.

e. A grating in a sloping gutter will intercept all water approaching the gross width of grate opening if the length of grate is greater than the upper trajectory 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.

f. 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.

g. 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 3-2 and 3-3.

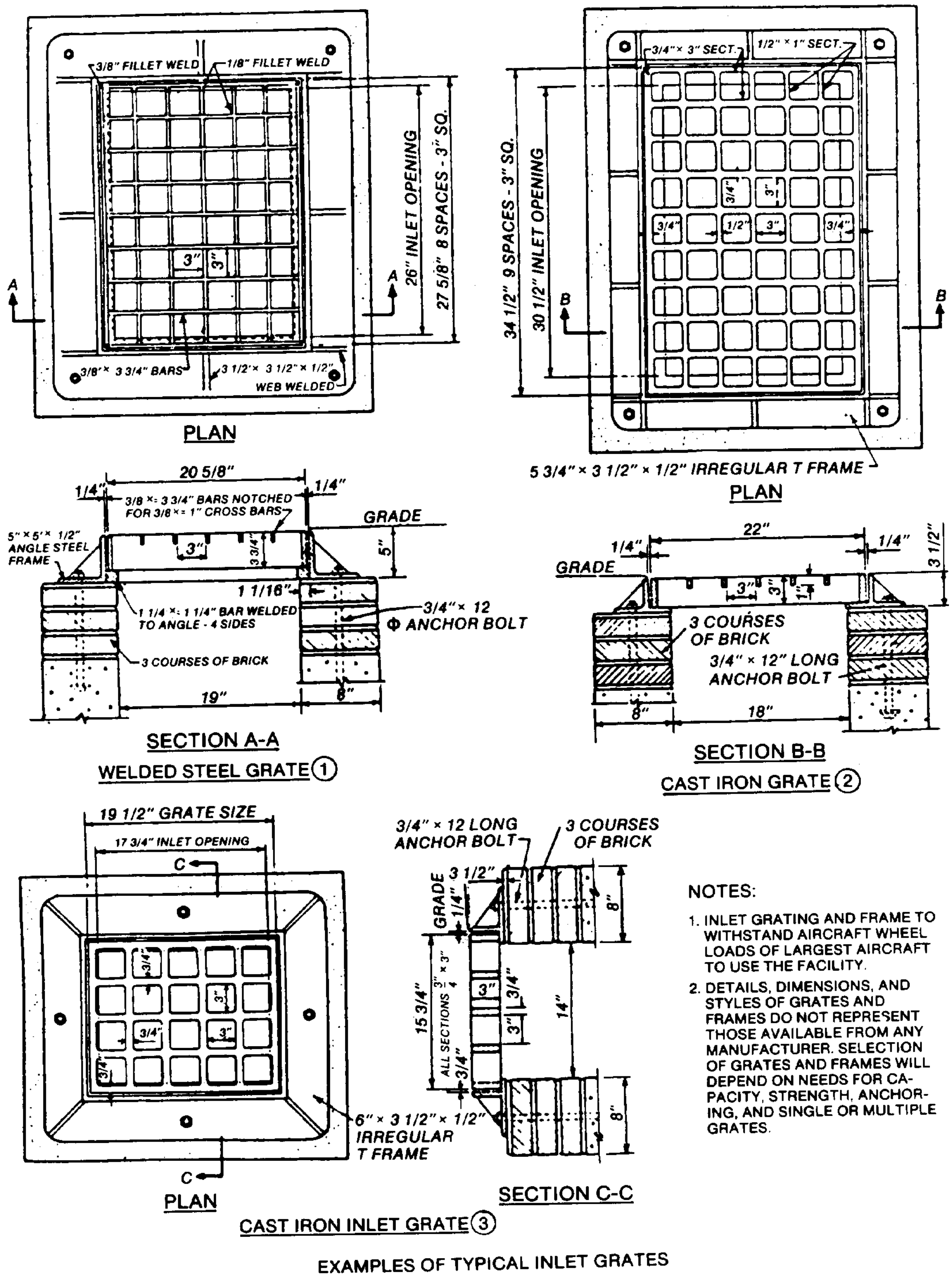
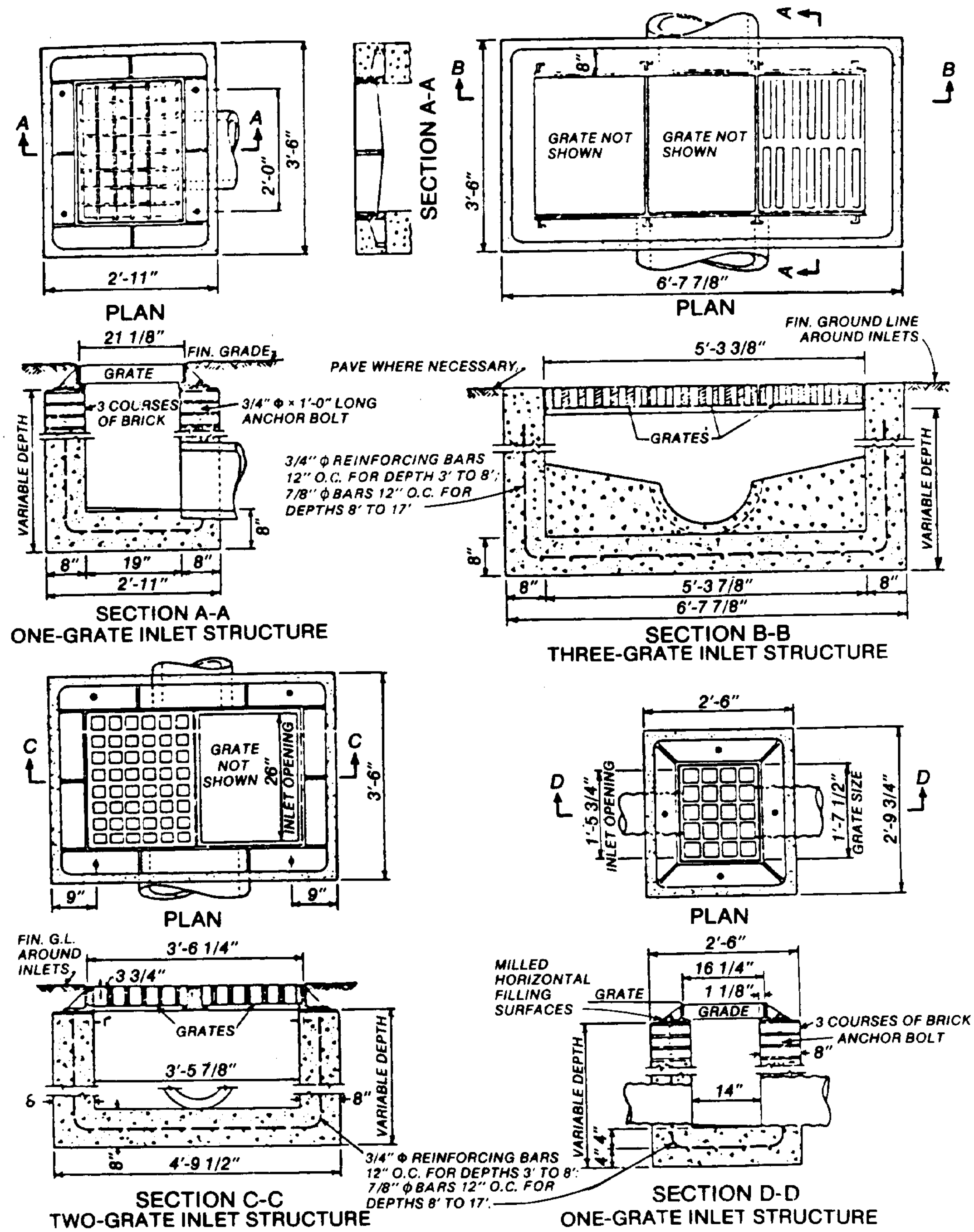


Figure 3-2. Examples of typical inlet grates.



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

Figure 3-3. Examples of inlet design.

h. Commercially manufactured grates and frames for airport loadings have been designed specifically for airport loadings from 50 to 250 pounds per square inch. 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.

i. 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.

j. 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.

3-4. Box drains.

a. 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 3-4. The walls of the box drain will extend to the surface of the pavement. The pavement will have a free thickened edge at the drain. An 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 ¾-inch-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.

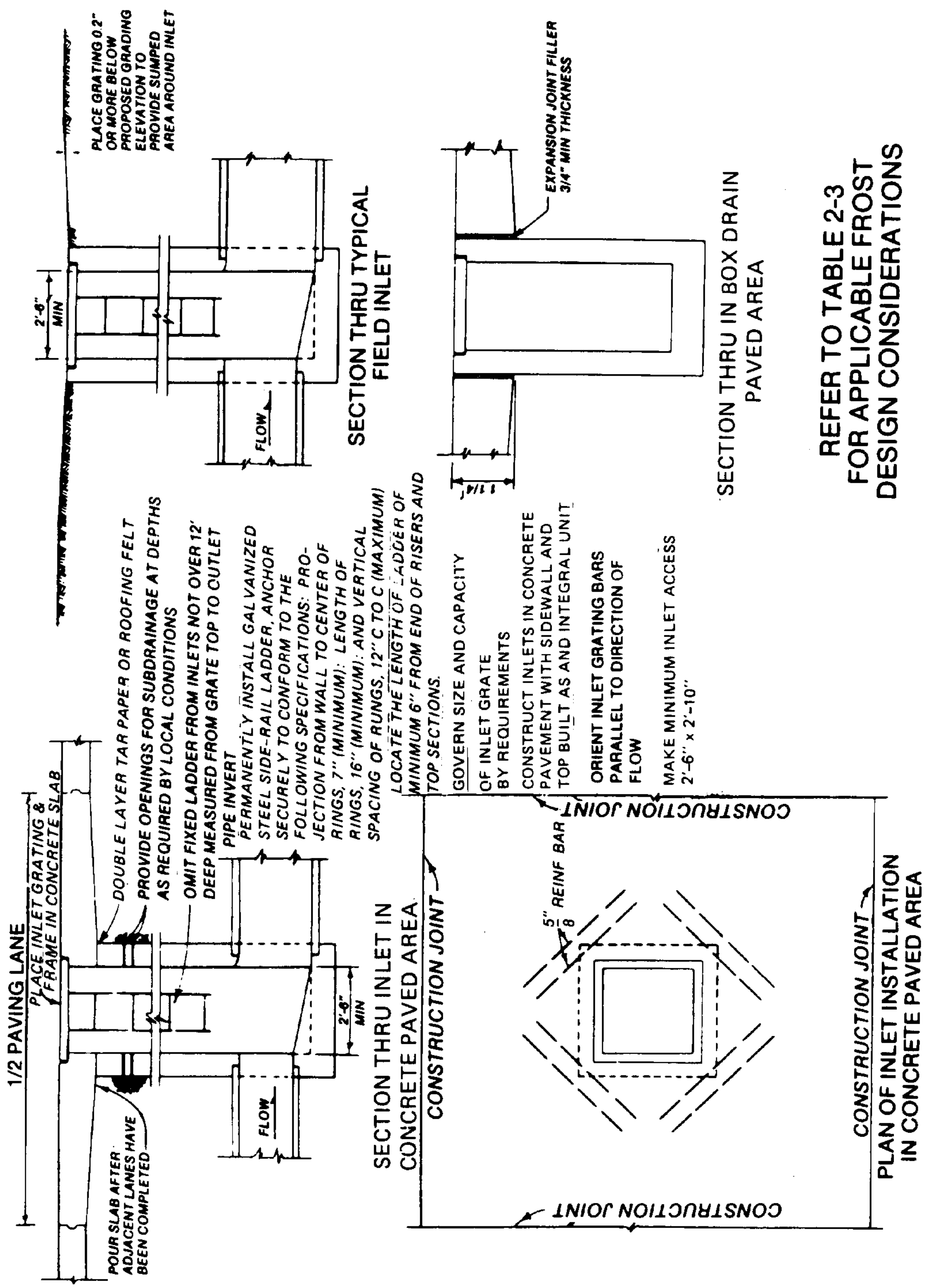


Figure 3-4. Typical inlet and box drain designs for airfield and heliport storm drainage systems.

b. A number of box drains similar to those shown in figure 3-4 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 3-5 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-inch-thick reinforced concrete slab with inserted lightweight circular pipes used for the grating openings. While only 4-inch-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.

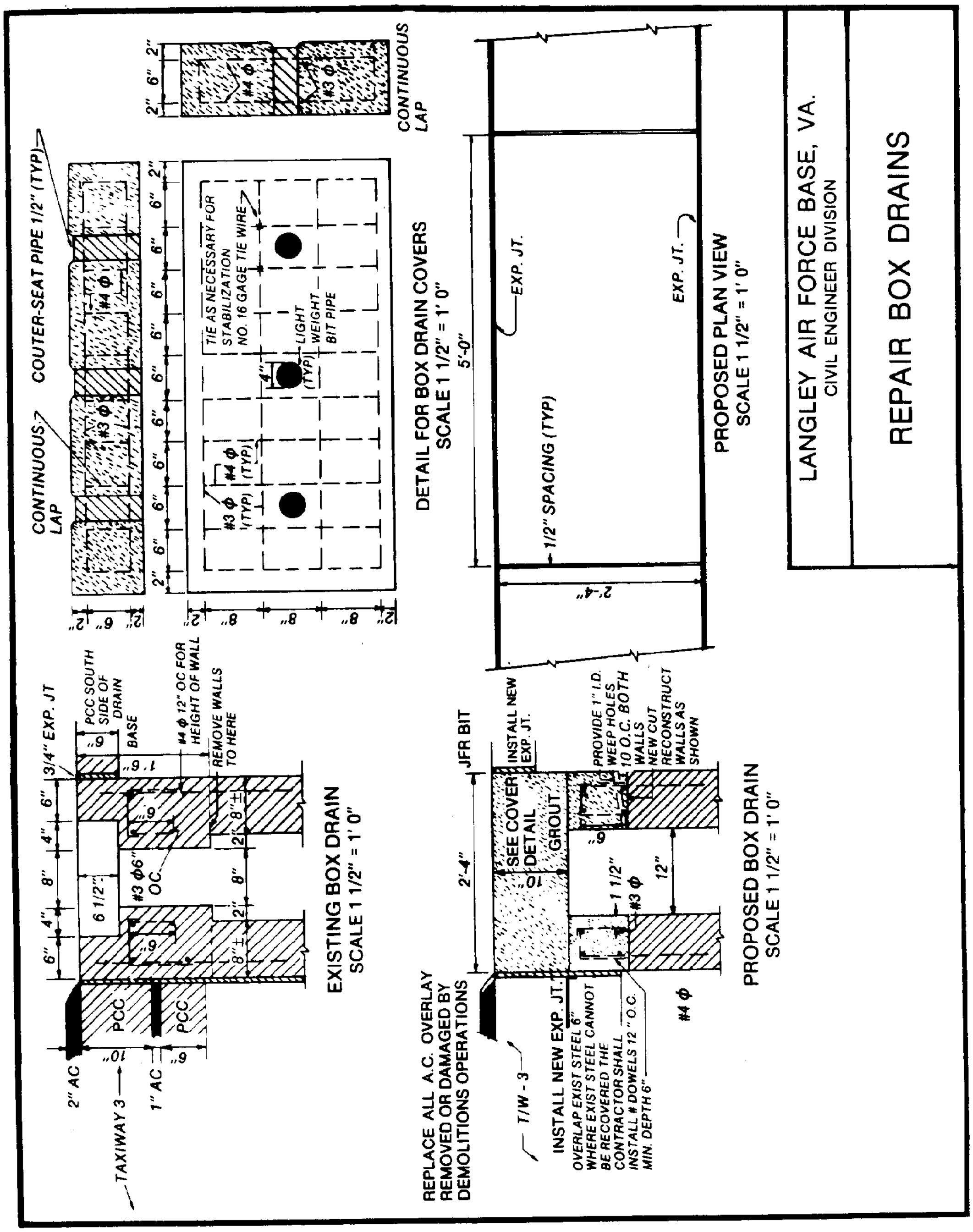


Figure 3-5. Repair box drains.

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

3-5. 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.

3-6. 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 chapter 4.

3-7. 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.

3-8. Clogging.

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

a. 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.

b. 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.

3-9. 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.

CHAPTER 4

GUTTERS

4-1. 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. 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.

4-3. Design charts.

A cross section of a typical runway gutter and the design charts are shown in figure 4-1. Safety and operational requirements for fast-landing speeds make it desirable to provide a continuous longitudinal grade in the gutter conforming closely to the runway gradient thereby minimizing the use of sumped inlets. A sufficient number of inlets will be provided in the gutter to prevent the depth of flow from exceeding about 21/2 inches.

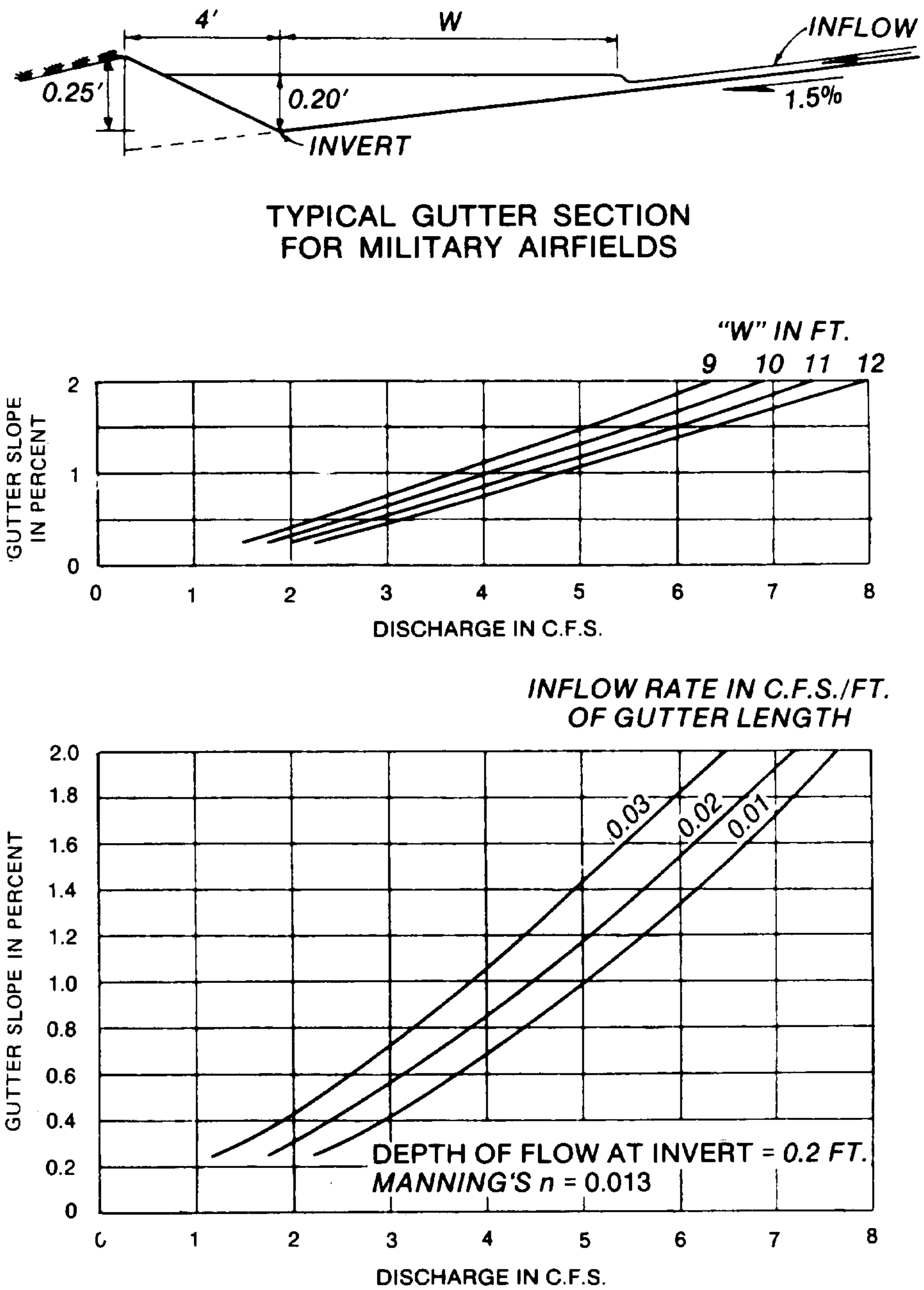


Figure 4-1. Drainage gutters for runways and aprons.

CHAPTER 5

STORM DRAINS AND CULVERTS

5-1. 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 in TM 5-820-4/AFM 88-5 chapter 4. 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 in TM 5-820-4/AFM 88-5 chapter 4.

5-2. Headwalls and endwalls.

a. 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.

b. 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.

c. 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.

d. 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.

e. 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 5-1. Straight headwalls (fig 5-la) 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 (fig 5-lb) is used if an abrupt change in flow direction is necessary with low to moderate velocities. Winged headwalls (fig 5-1c) 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 (fig 5-id) 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.

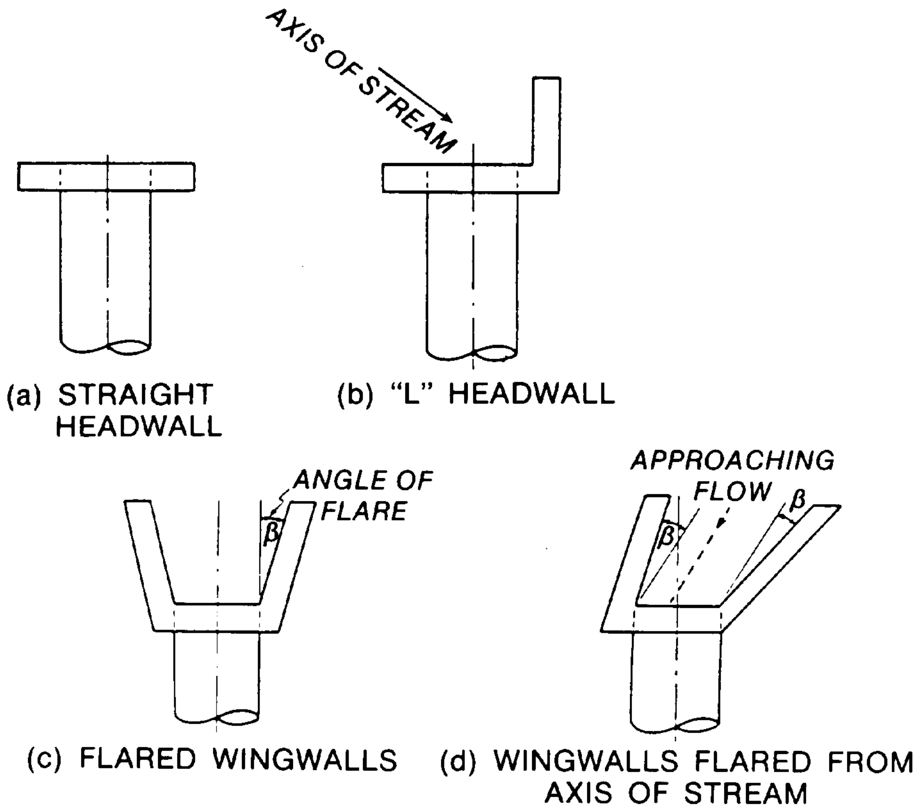


Figure 5-1. Culvert headwalls and wingwalls.

f. 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.

(1) 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.

(2) 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.

g. 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.

h. 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 note 4 of paragraph 2-6*d*. Criteria for determining the depth of backfill behind walls are given in TM 5-818-1.

i. 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.

5-3. 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 5-2.

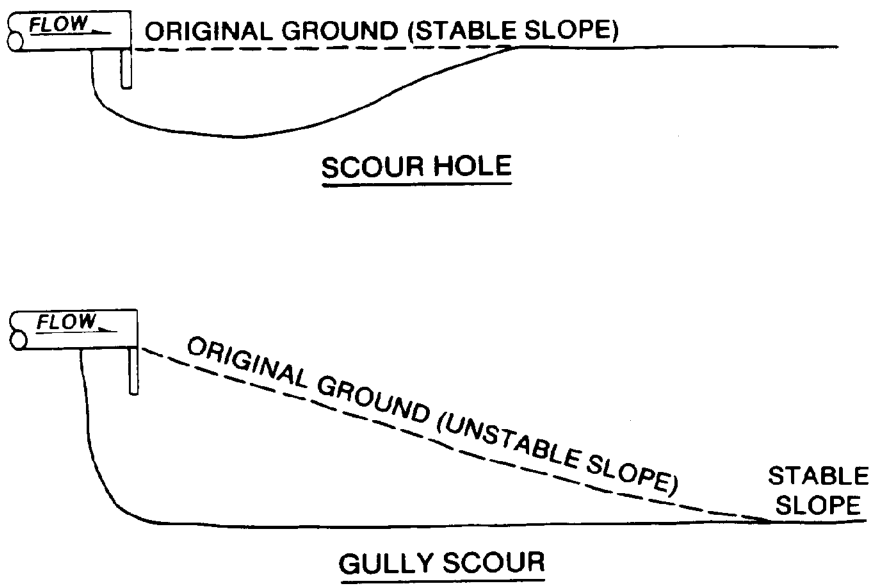


Figure 5-2. Types of scour at storm-drain and culvert outlets.

a. 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.

b. 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.

c. 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 5-3. Terms are defined in appendix E. 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 5-4 through 5-7. Minimum and maximum tailwater depths are defined as those less than $0.5D_o$ and equal to or great than $O.5D_o$, respectively. Dimensionless profiles along the center lines of the scour holes to be anticipated with minimum and maximum tailwaters are presented in figures 5-8 and 5-9. 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 5-8 and 5-9.

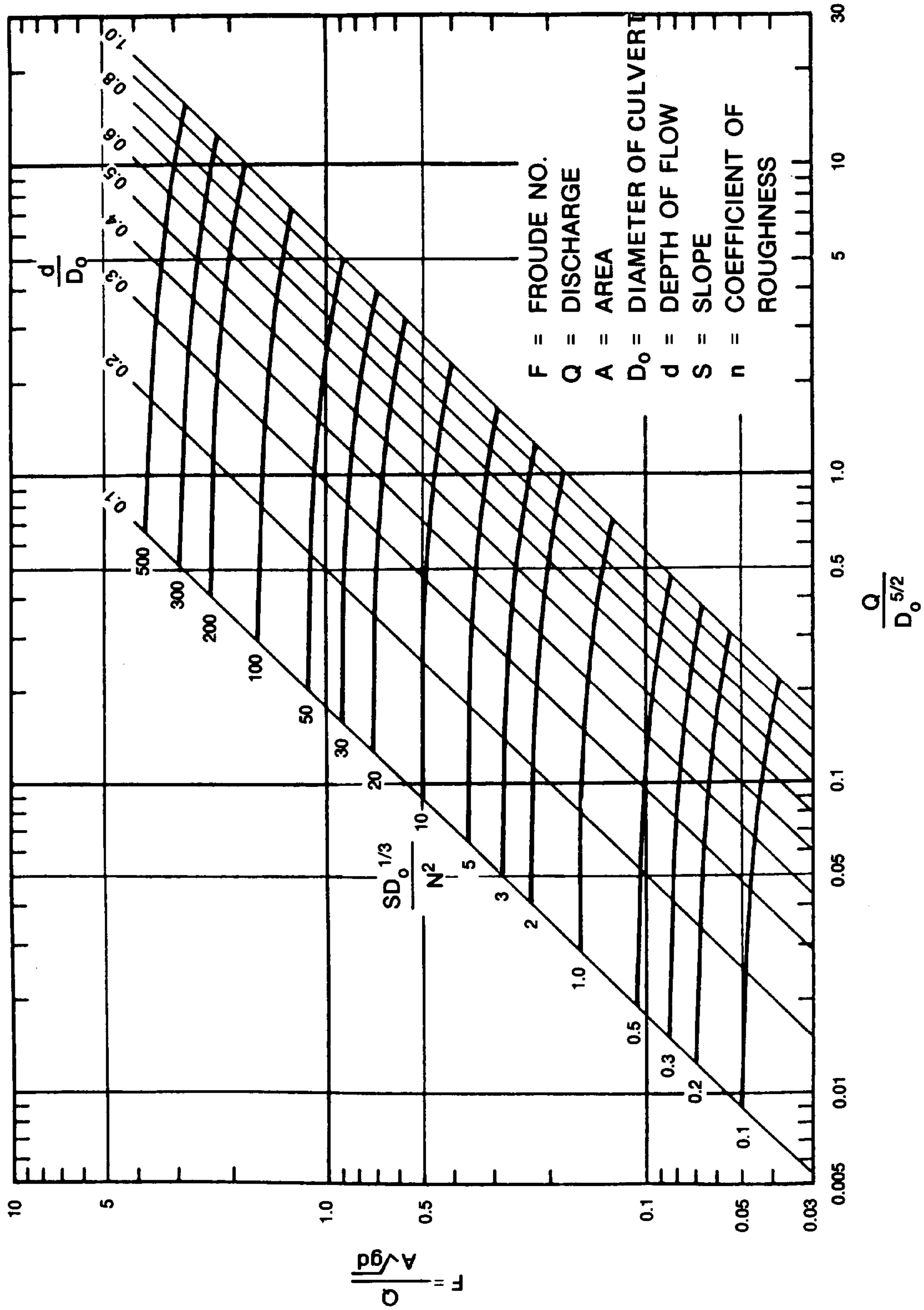


Figure 5-3. Square culvert-Froude number.

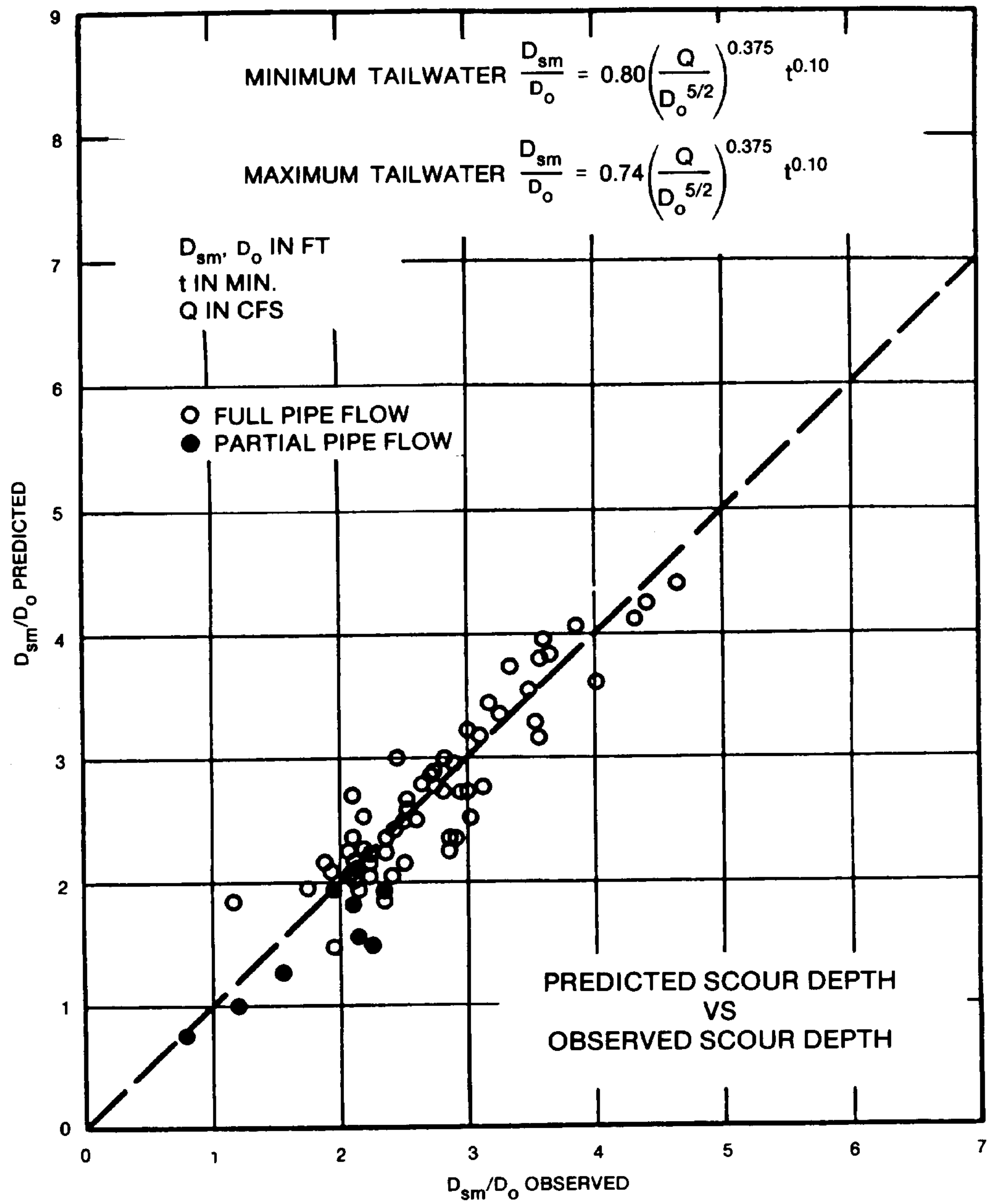


Figure 5-4. Predicted scour depth versus observed scour depth.

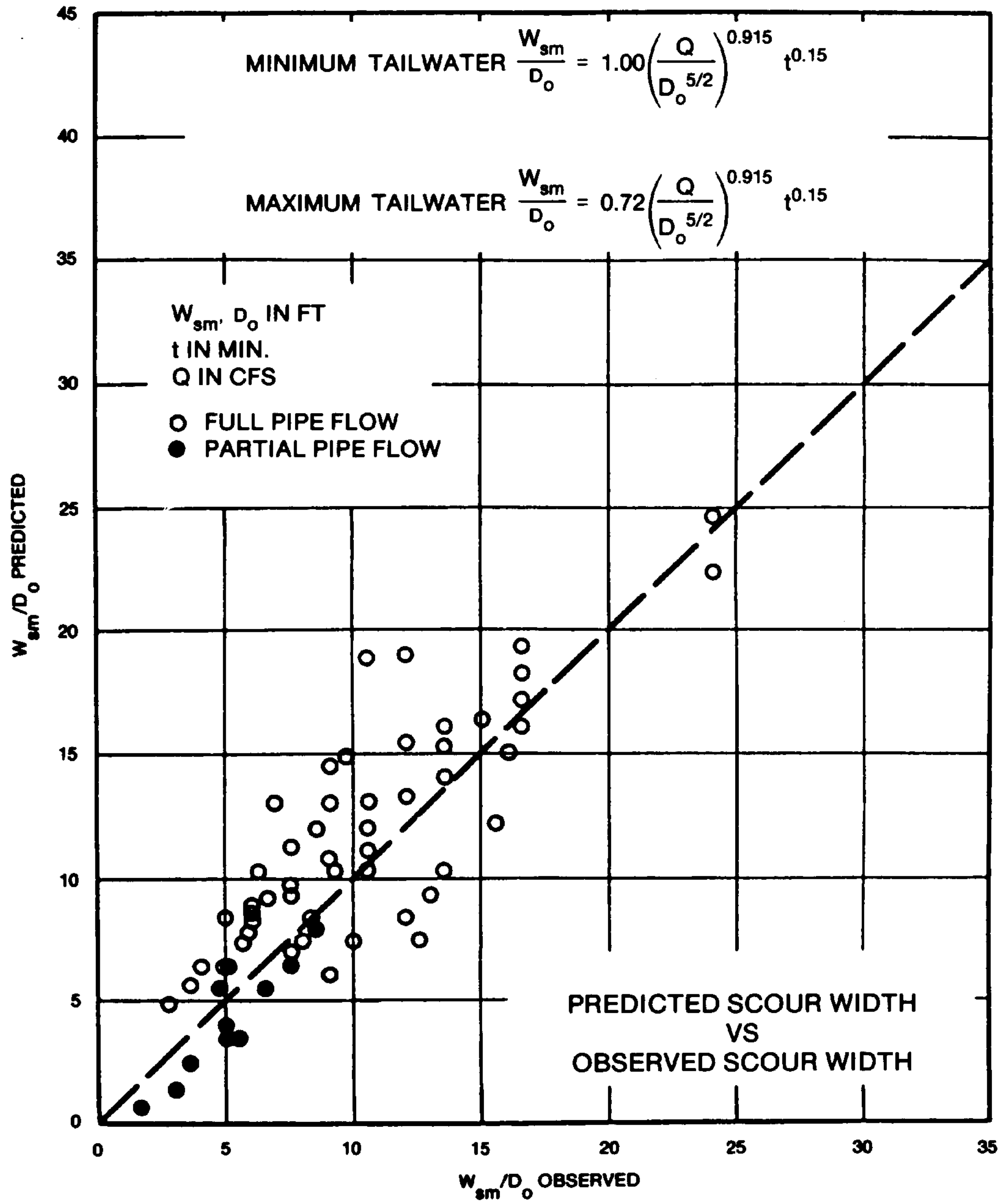


Figure 5-5. Predicted scour width versus observed scour width.

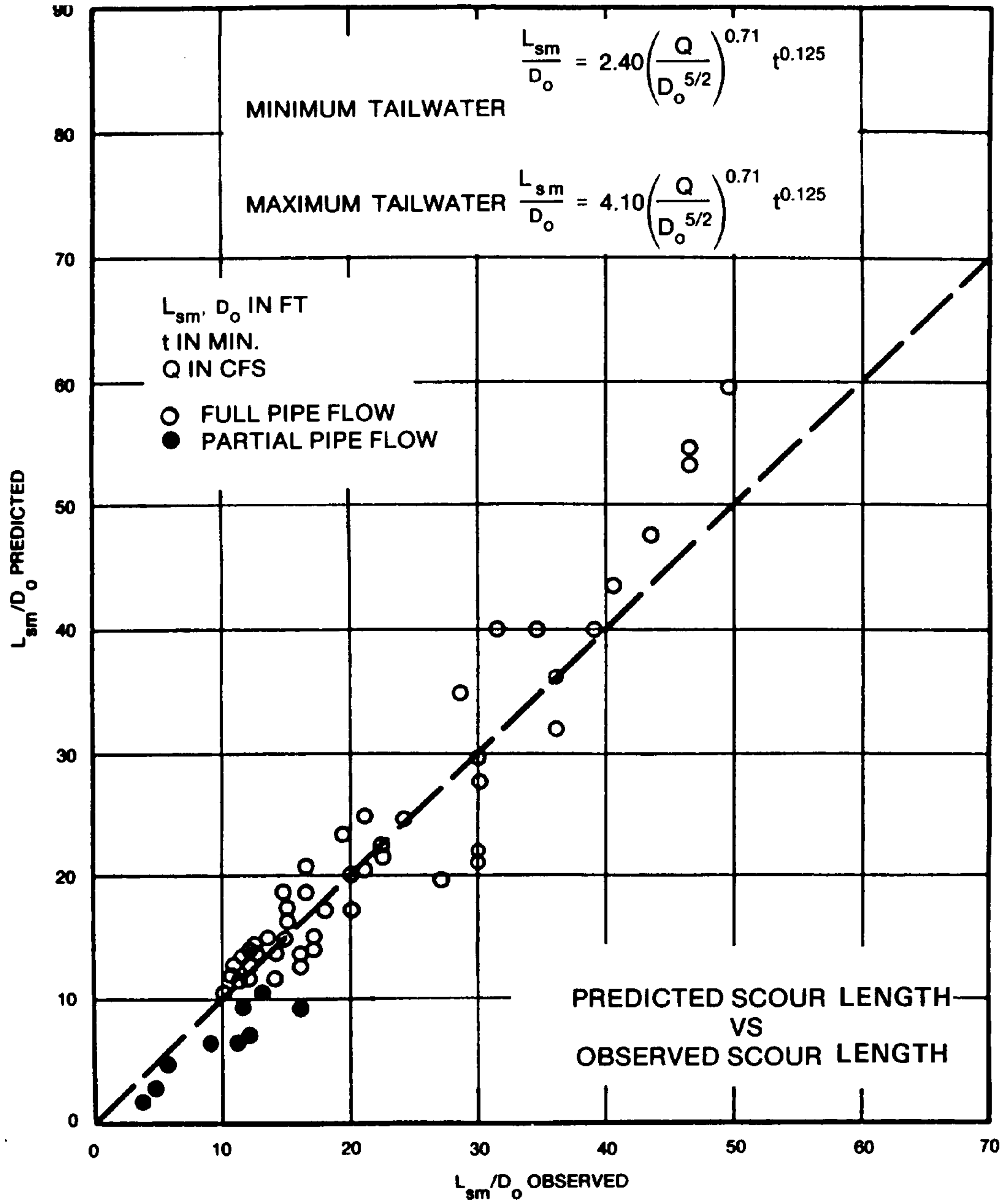


Figure 5-6. Predicted scour length versus observed scour length.

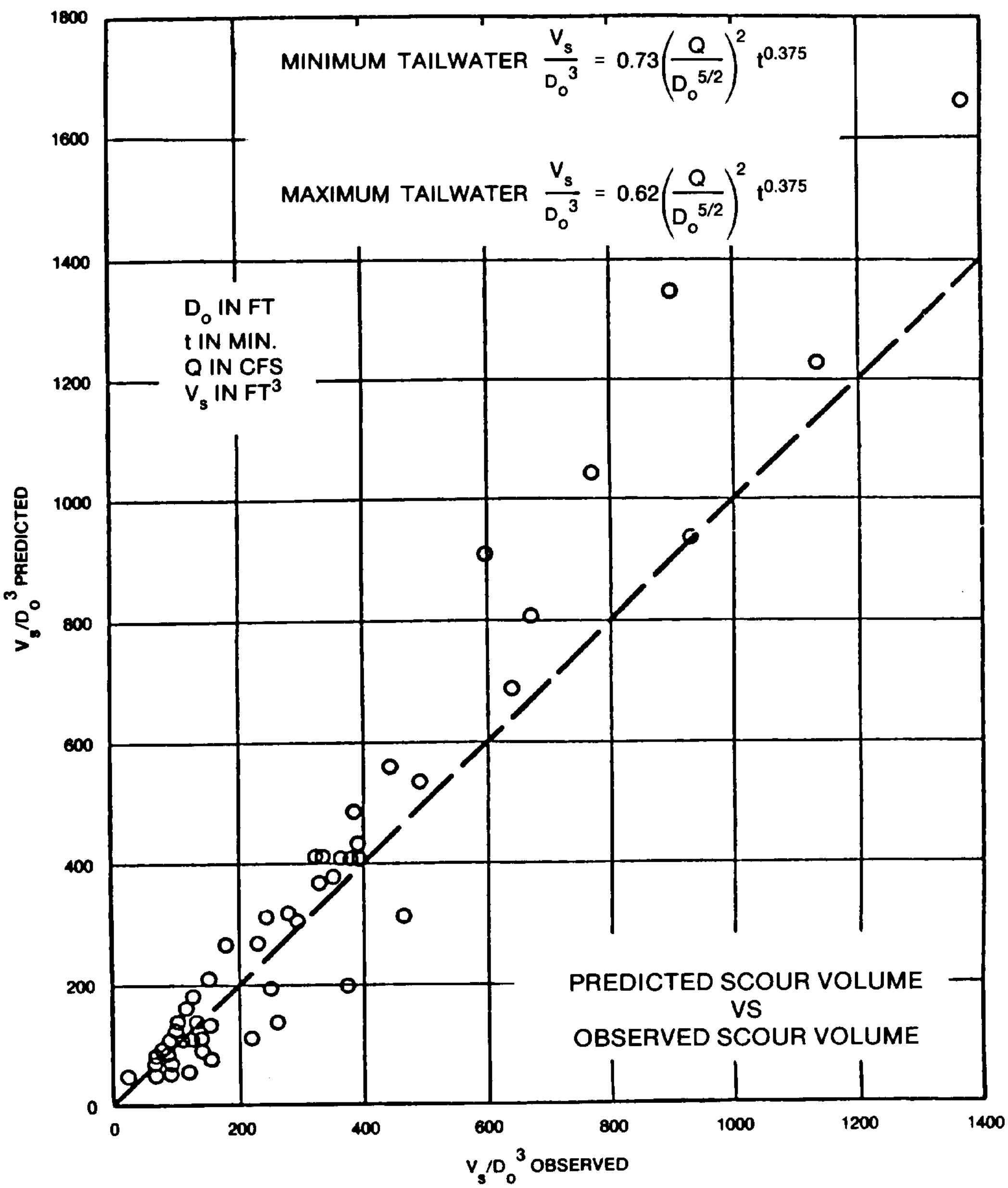


Figure 5-7. Predicted scour volume versus observed scour volume.

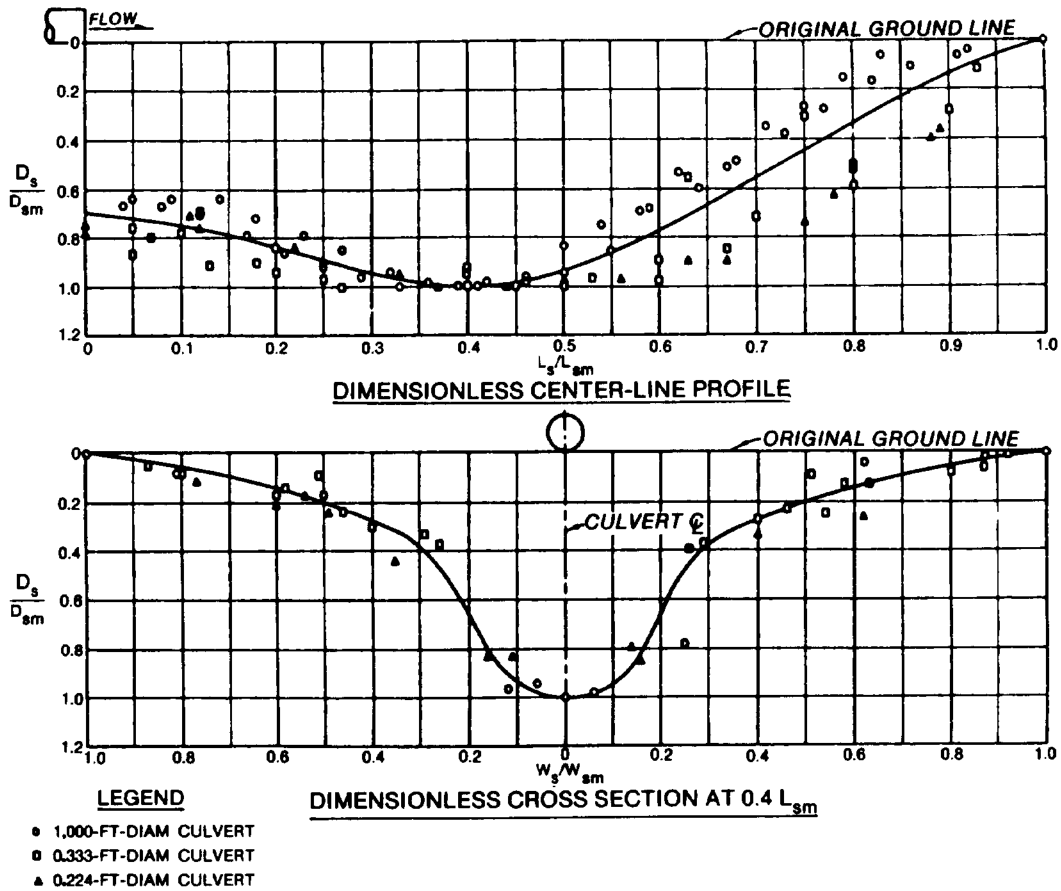


Figure 5-8. Dimensionless scour hole geometry for minimum tailwater.

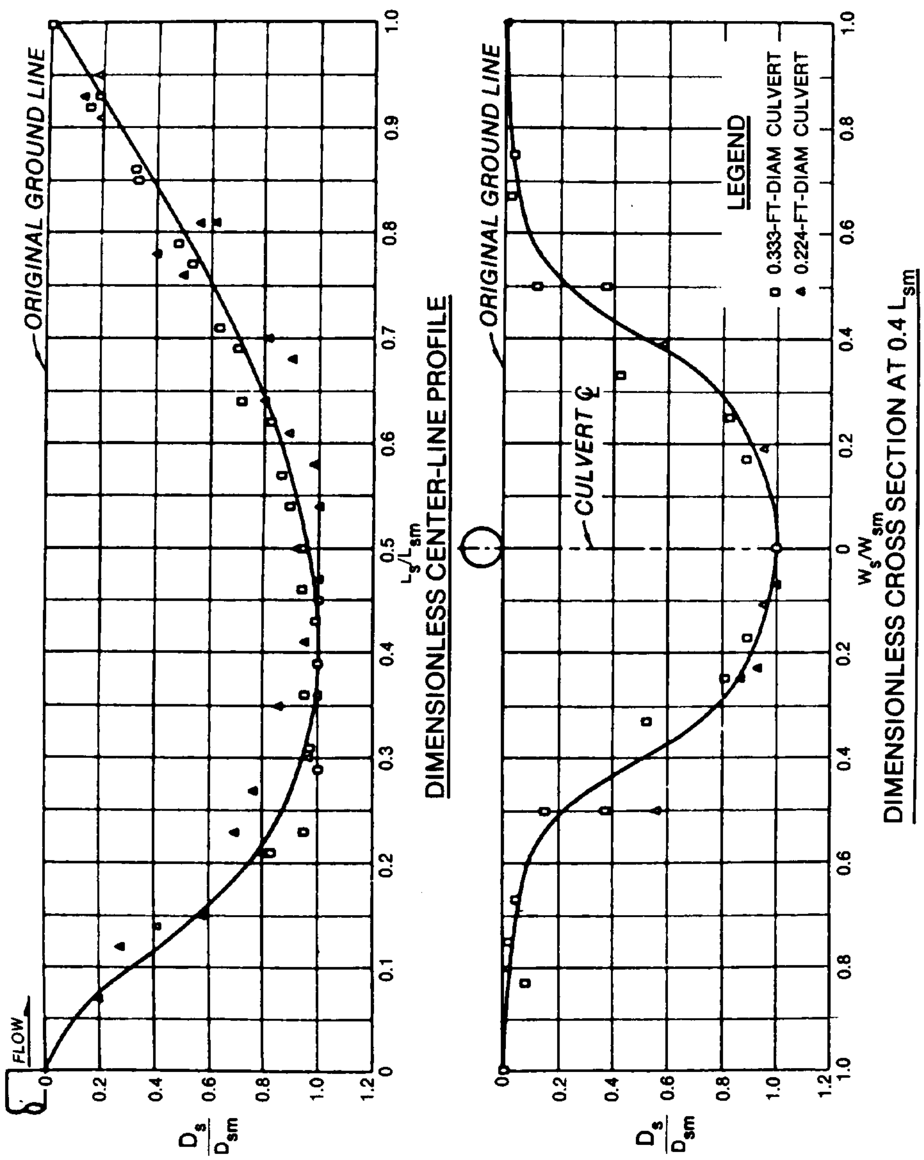


Figure 5-9. Dimensionless scour hole geometry for maximum tailwater.

5-4. 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.

a. 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 5-10 to 5-12. 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 5-10. The length of stone protection (LSP) can be determined by the relations shown in figure 5-11. The variables are defined in appendix E, and the recommended configuration of the blanket is shown in figure 5-12.

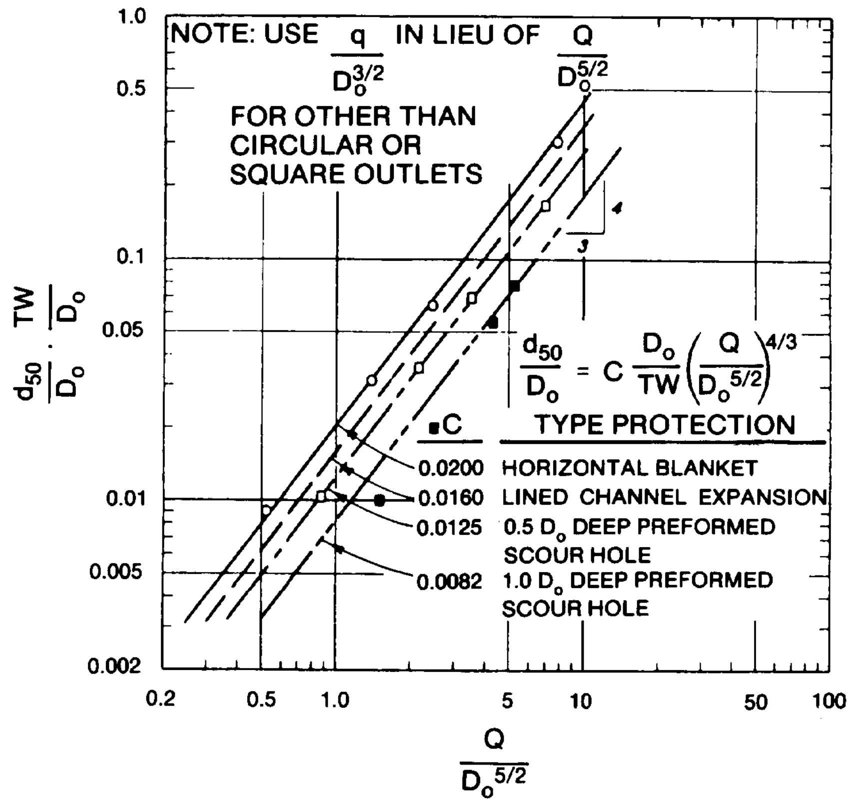


Figure 5-10. Recommended size of protective stone.

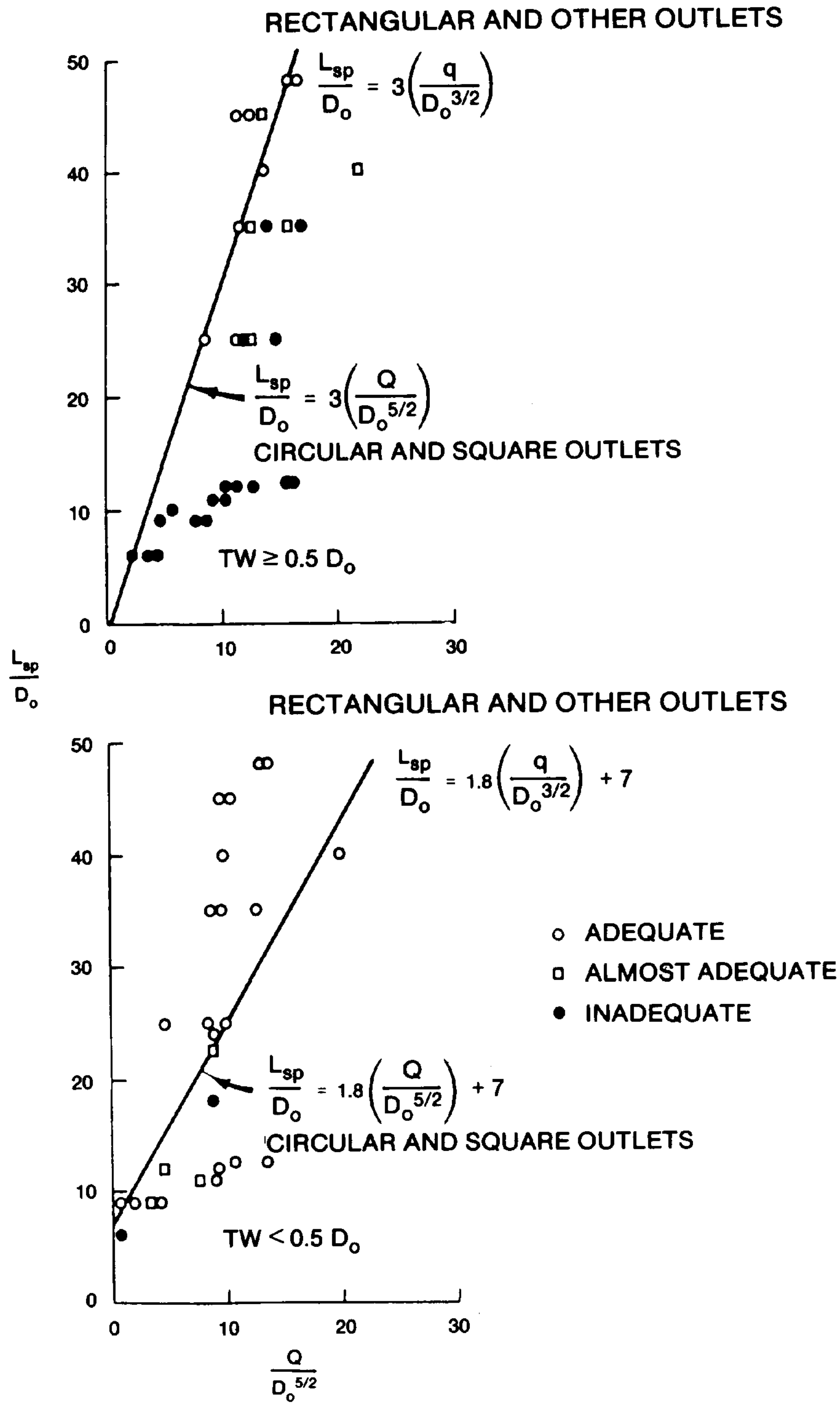


Figure 5-11. Length of stone protection, horizontal blanket.

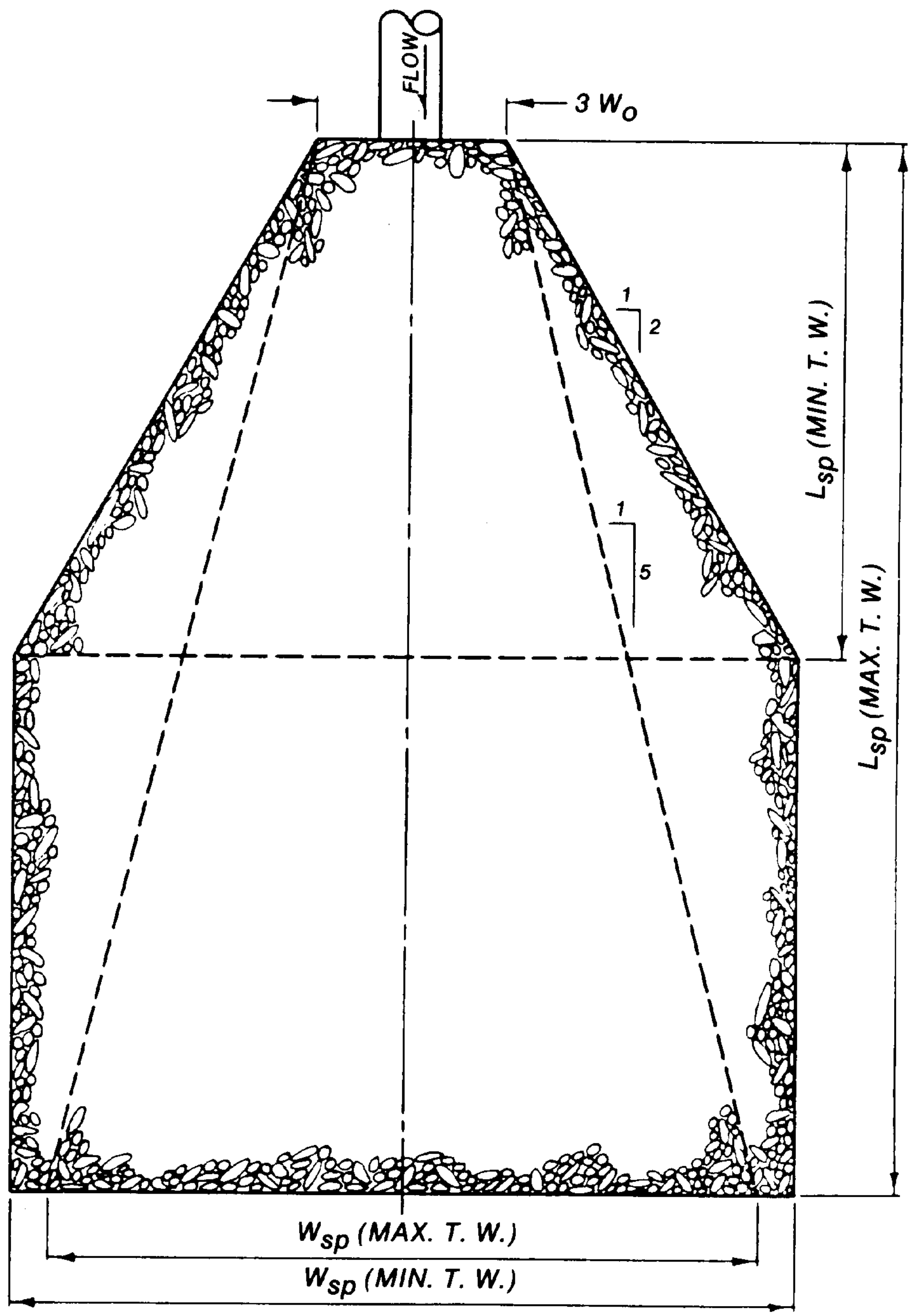


Figure 5-12. Recommended configuration of riprap blanket subject to minimum and maximum tailwaters.

b. 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 5-10. Figure 5-10 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 5-13.

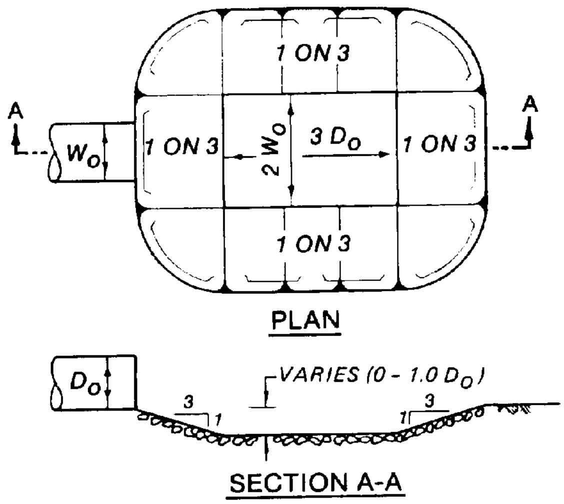


Figure 5-13. Preformed scour hole.

c. 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.

d. 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 5-14. The conditions of discharge and tailwater required to displace sack revetment with length, width, and thickness of 2, 1.5, and 0.33 feet, respectively (weight 120 pounds); cellular blocks, 0.66 by 0.66 foot and 0.33 foot thick (weight 14 pounds); or riprap with a given thickness are shown in figure 5-15. The effectiveness of the lined channel expansion relative to the other schemes of riprap protection described previously is shown in figure 5-10.

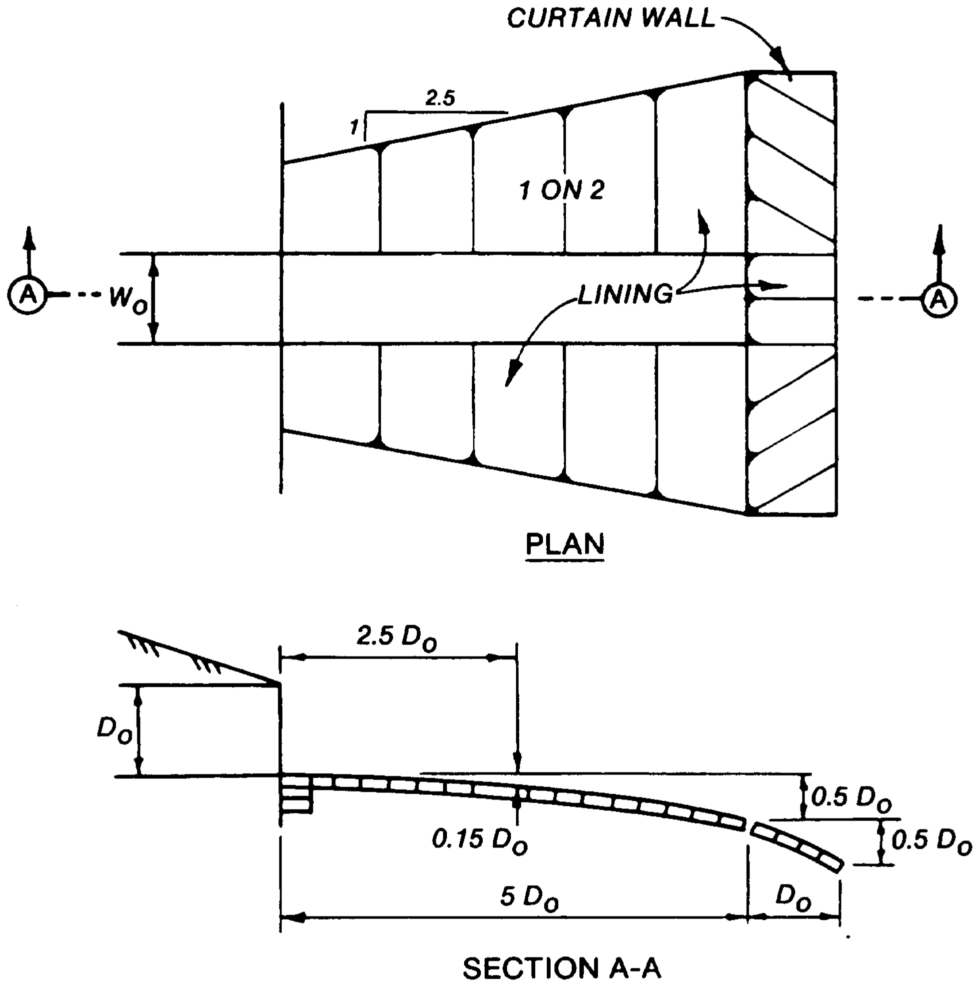


Figure 5-14. Culvert outlet erosion protection, lined channel expansions.

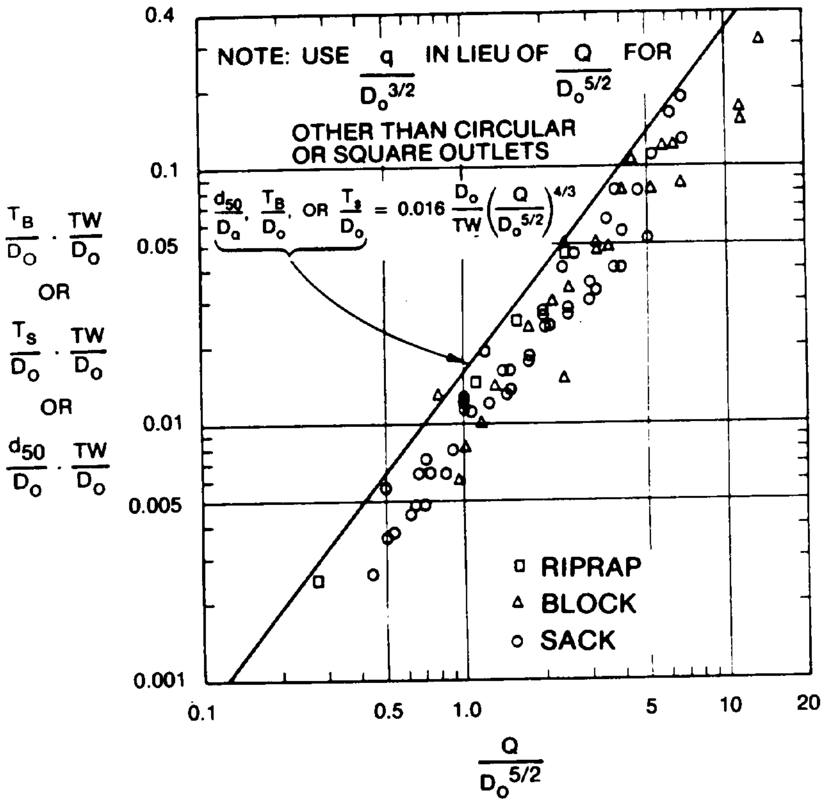


Figure 5-15. Maximum permissible discharge for lined channel expansions.

e. 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.

f. 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 5-16. 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.

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

Circular and square outlets (eq 5-1)

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

Rectangular and other shaped outlets (eq 5-2)

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

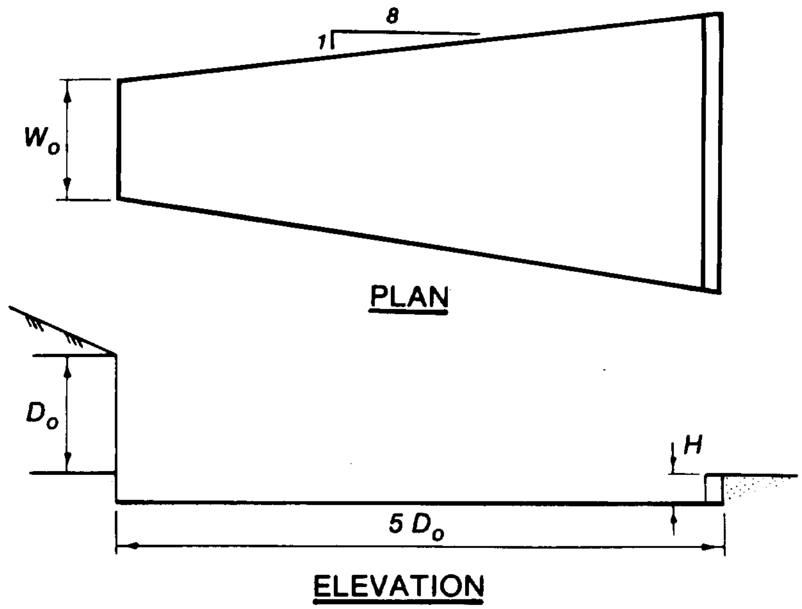
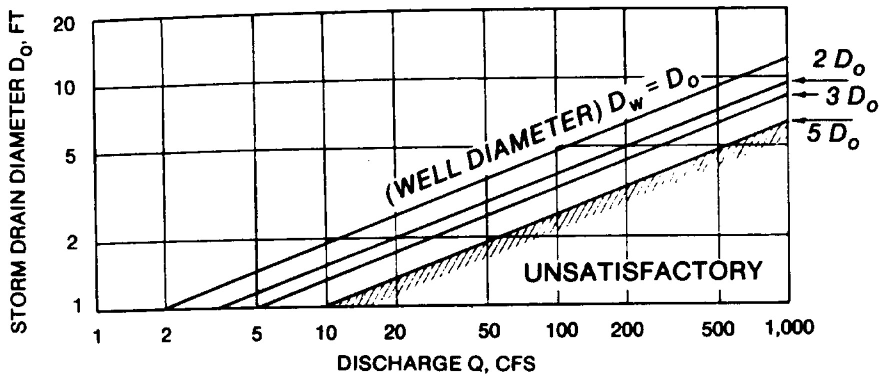


Figure 5-16. Flared outlet transition.

g. 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 5-17 to 5-19. 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.

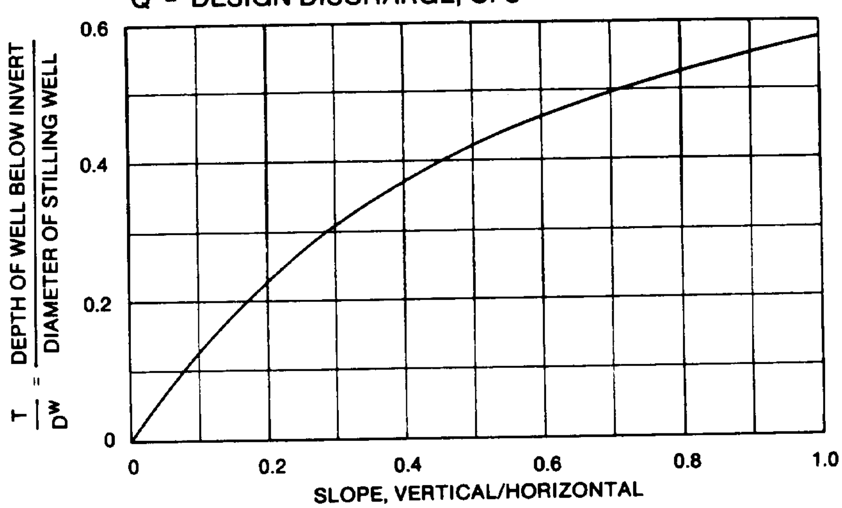


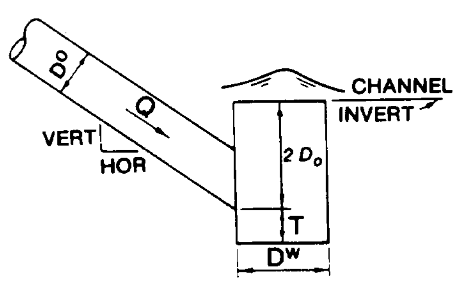
BASIC EQUATION

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

WHERE:

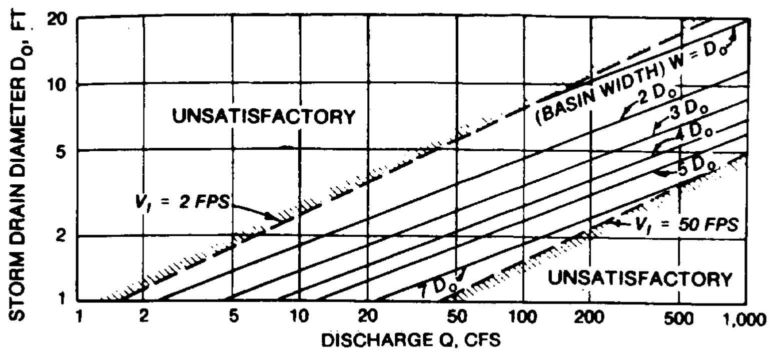
D^w = STILLING WELL DIAMETER, FT
D^o = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS





ELEVATION

Figure 5-17. Stilling well.

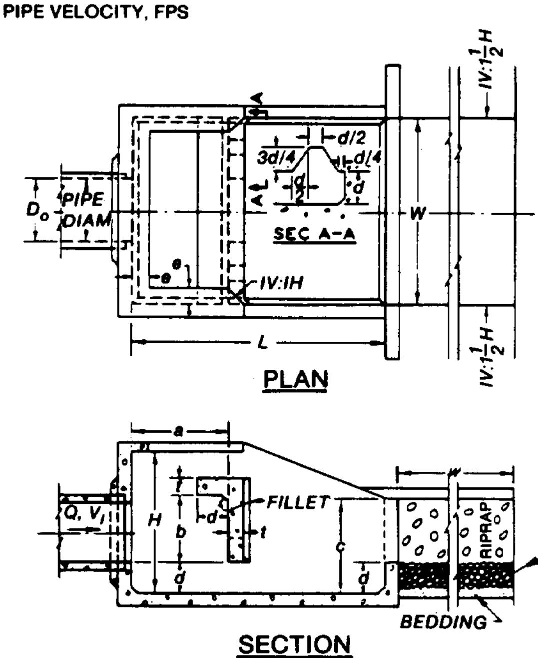


BASIC EQUATION

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

WHERE:

W = BASIN WIDTH, FT
D° = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS
V_1 = PIPE VELOCITY, FPS



$$d_{50} = 0.35\ D\left(\frac{V}{\sqrt{gD}}\right)^3$$

STILLING BASIN DESIGN

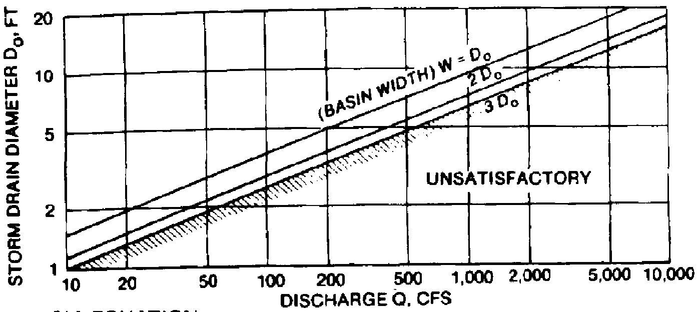
$H = \frac{3}{4}(W)$ $c = \frac{1}{2}(W)$

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

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

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

Figure 5-18. US Bureau of Reclamation impact basin.

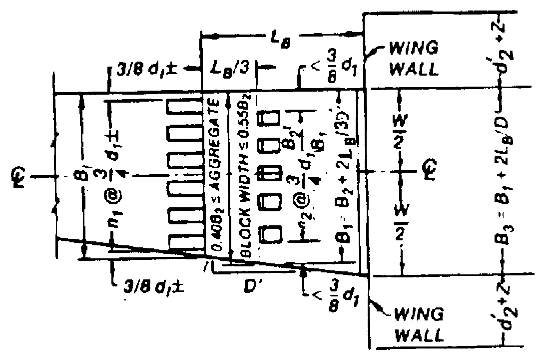


BASIC EQUATION

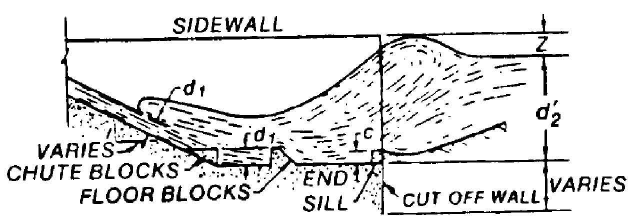
$$\frac{W}{D_o} = 0.3\frac{Q}{D_o^{5/2}} \quad \text{FOR} \quad \frac{Q}{D_o^{5/2}} \leq 9.5$$

WHERE:
W = END SILL LENGTH, FT
D° = DRAIN DIAMETER, FT
Q = DESIGN DISCHARGE, CFS

STILLING BASIN HALF-PLAN



TRAPEZOIDAL STILLING BASIN HALF-PLAN



CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{g d_1} \quad (1) \qquad L_B = \frac{4.5\, d_2}{F^{0.38}} \quad (4)$$

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

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

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

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

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

Figure 5-19. Saint Anthony Falls stilling basin.

h. The stilling well shown in figure 5-17 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 5-17. 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 5-17. 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.

i. The US Bureau of Reclamation (USBR) impact energy dissipator shown in figure 5-18 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 5-18. The other dimensions of energy dissipator are a function of the basin width as shown in figure 5-18. This basin can be used with $Q/D_o^{5/2}$ ratios up to 21.

j. The Saint Anthony Falls (SAF) stilling basin shown in figure 5-19 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 5-20 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 5-19. 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 5-19. However, outlet portal velocities exceeding 60 feet per second 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.

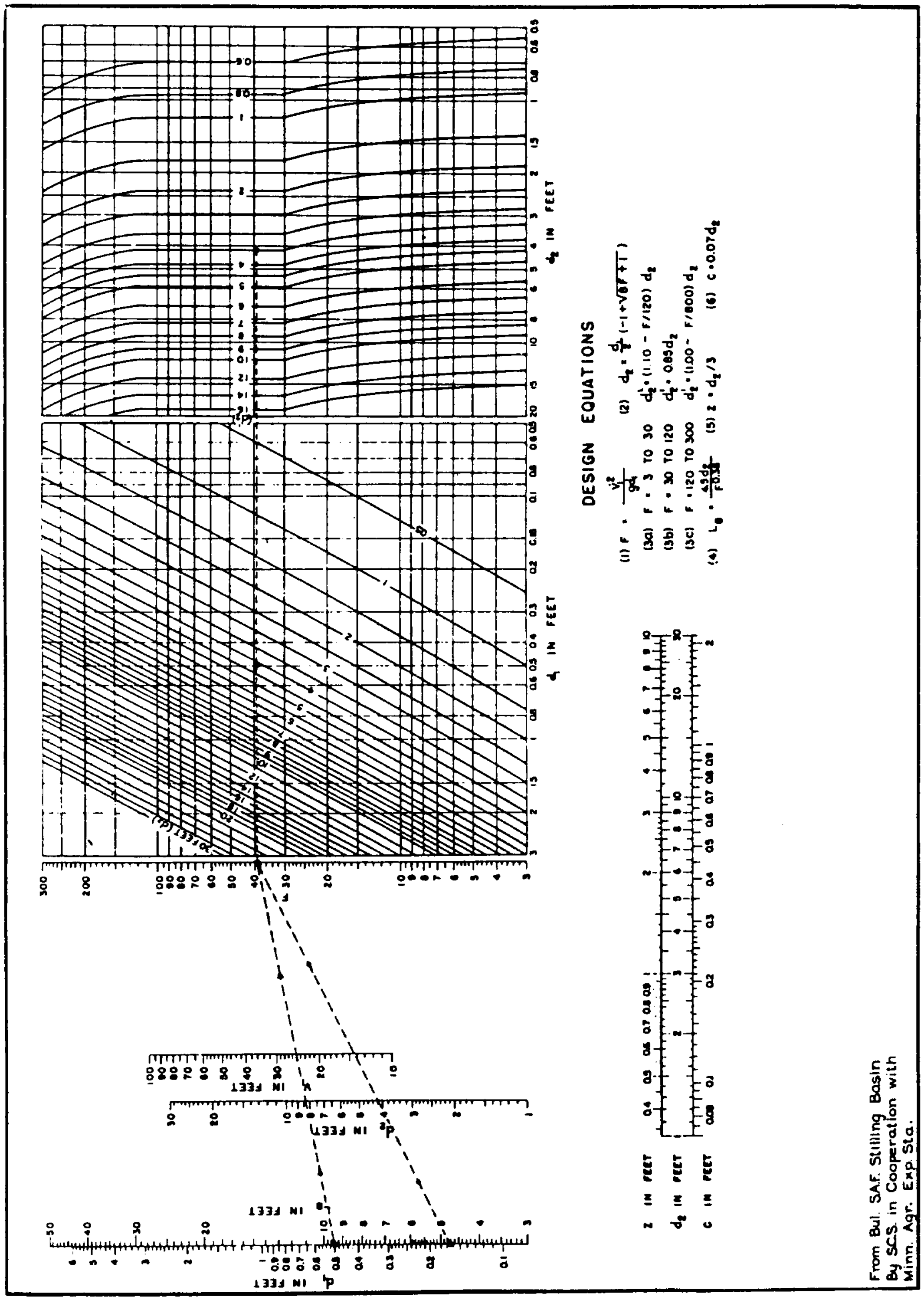


Figure 5-20. Design chart for SAF stilling basin.

k. Riprap Will be required downstream from the above energy dissipators. The size of the stone can be estimated by the following equation.

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

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 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.

l. Smaller riprap sizes can be used to control channel erosion. Equation 5-4 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

$$d_{50} = 0.35\,D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{or } F = 1.42\left(d_{50}/D\right)^{1/3} \qquad \text{(eq 5-4)}$$

Equation 5-5 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

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

½ D deep scour hole

$$d_{50} = 0.09\,D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{or } F = 2.23\left(d_{50}/D\right)^{1/3} \qquad \text{(eq 5-6)}$$

D deep scour hole

$$d_{50} = 0.055\,D\left(\frac{V}{\sqrt{gD}}\right)^3 \quad \text{or } F = 2.63\left(d_{50}/D\right)^{1/3} \qquad \text{(eq 5-7)}$$

These relationships are shown in figures 5-21 and 5-22.

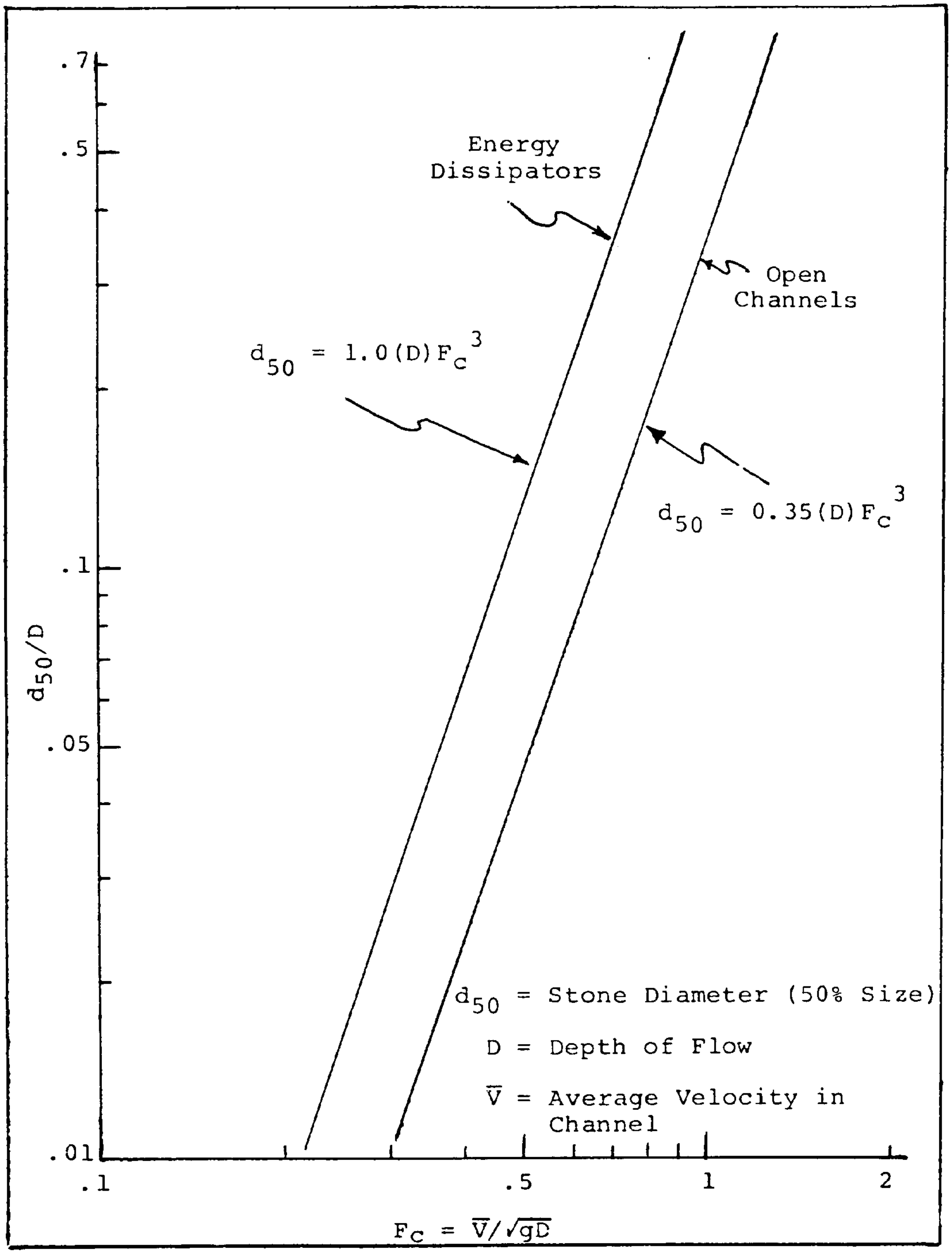


Figure 5–21. Recommended riprap sizes.

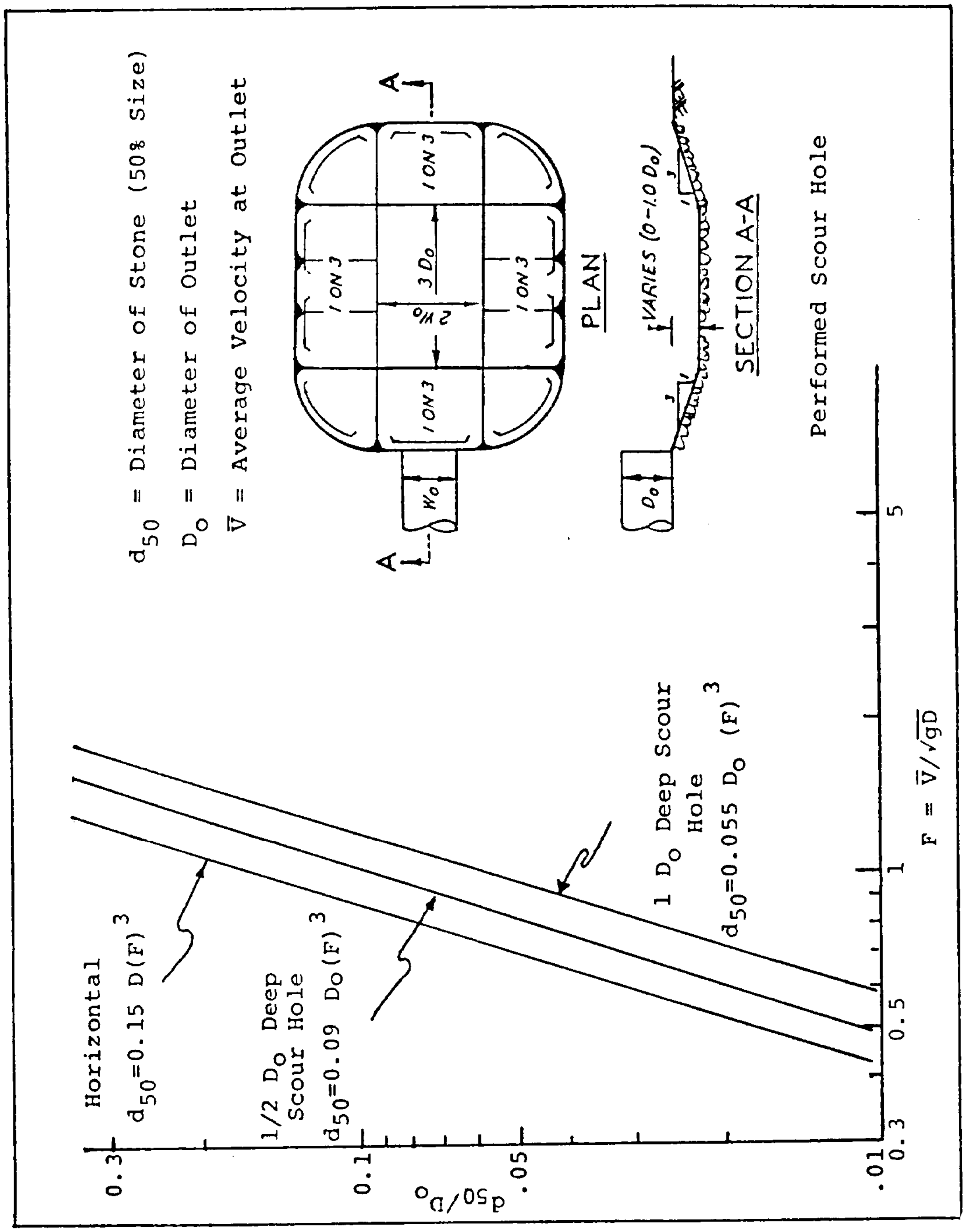


Figure 5-22. Scour hole riprap sizes.

m. 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.

n. 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, P0 Box 631, Vicksburg, MS 39180-0631.

CHAPTER 6

OPEN CHANNELS

6-1. 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.

6-2. Channel design.

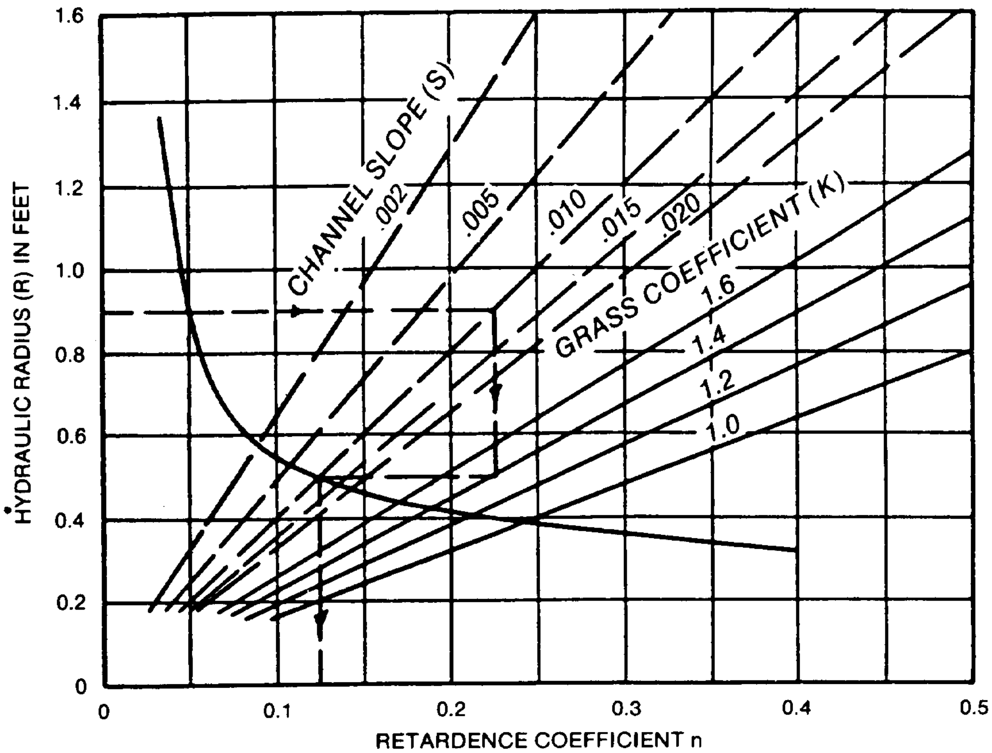
The following items merit special consideration in designing channels.

a. 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 6-1. Retardance coefficients for turf-lined channels are a function of both the turf characteristics and the depth and velocity of flow and can be estimated by the graphical relations shown in figure 6-1. 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 6-1. 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 6-1.



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.

Figure 6-1. Retardance coefficients for flow in turfed channels.

b. 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.

c. 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.

d. 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.

e. Channel design in appendix D 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 feet) 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 D.

f. 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 appendix E.

6-3. Design procedure.

a. 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.

b. Appendix D 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.

6-4. Drop structures and check dams.

a. 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 feet and over embankments higher than 5 feet 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.

b. 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.

c. Pertinent features of a typical drop structure are shown in figure 6-2. 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.

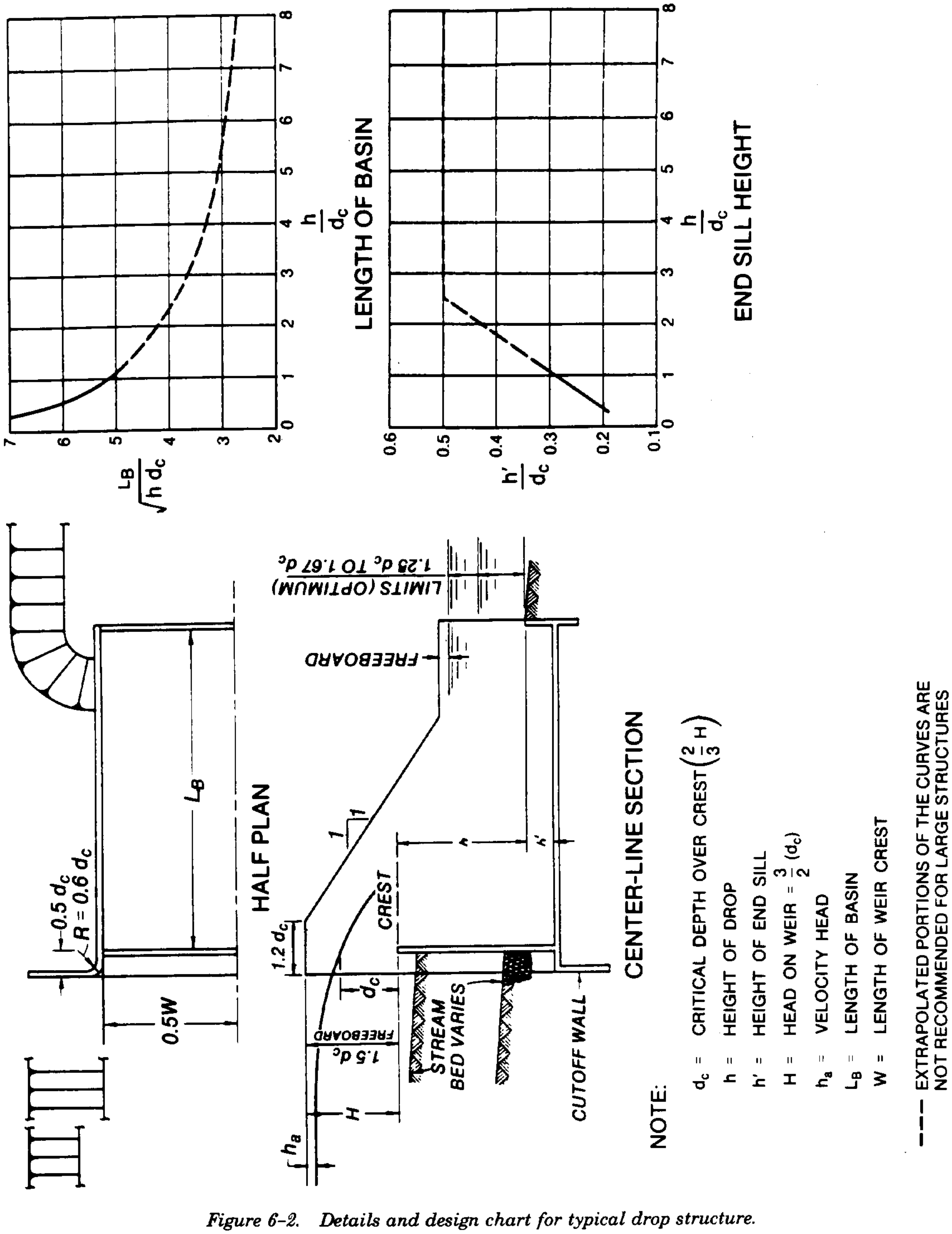


Figure 6-2. Details and design chart for typical drop structure.

d. 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.

(1) 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.

(2) 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

$$L_B = C_L\sqrt{hd_c} \qquad \text{(eq 6-1)}$$

where C_L is an empirical coefficient between 2 and 7, as shown in figure 6-2. The stilling basin length and end sill height can be determined from the design curves in figure 6-2. 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 paragraph 5-4*k* can be used for design of the riprap.

e. A design illustrating the use of the above information and figure 6-2 is shown in the following example. Design a drop structure for a discharge of 250 cubic feet per second in a trapezoidal channel with a 10-foot base width and side slopes of IV on 3H, and a depth of flow of 5 feet. The amount of drop required is 4 feet. 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 feet.

Width of Crest, W:

$$Q=CWH^{3/2} \qquad \text{(eq 6-2)}$$

$$W = \frac{250}{3\times(5)^{3/2}}\ 7.5 \text{ feet} \qquad \text{(eq 6-3)}$$

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

f. With width of crest equal to 10 feet determine head on the crest:

$$Q=CWH^{3/2} \qquad \text{(eq 6-4)}$$

$$H=(250/3\times10)^{2/3}=4.1 \text{ feet} \qquad \text{(eq 6-5)}$$

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

Critical depth, d_c:

$$d_c = \frac{2}{3}H = \frac{2}{3}(4.1)=2.73 \text{ feet} \qquad \text{(eq 6-6)}$$

$$\frac{h}{d_c} = \frac{4.9}{2.73} = 1.8 \qquad \text{(eq 6-7)}$$

From figure 6-2:

$$\frac{L_B}{\sqrt{hd_c}} = 4.4 \qquad \text{(eq 6-8)}$$

$$L_B=16.09 \text{ feet (use 16.1 feet)} \qquad \text{(eq 6-9)}$$

$$\frac{h'}{d_c} = 0.4 \qquad \text{(eq 6-10)}$$

$$h'=0.4\times2.73=1.09 \text{ feet (use 1.1 feet)} \qquad \text{(eq 6-11)}$$

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 tail-water will be 5 feet. Thus, the tailwater/dc 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- .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:

$$d_{50} = D\left(\frac{V}{\sqrt{gD}}\right)^3 \qquad \text{(eq 6-12)}$$

$$d_{50} = 5\left(\frac{5}{\sqrt{32.2 \times 5}}\right)^3 = 0.306 \text{ feet (use 4 inches)} \qquad \text{(eq 6-13)}$$

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

V = Discharge/area at end of basin = 250/ 10 × 5 = 5 feet per second

CHAPTER 7
CHUTES

7-1. 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 7-1. Frost penetration beneath the structure will be restricted to nonfrost-susceptible materials using procedures outlined in paragraph 2-6*b* and note 4 of paragraph 2-6*d*, 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.

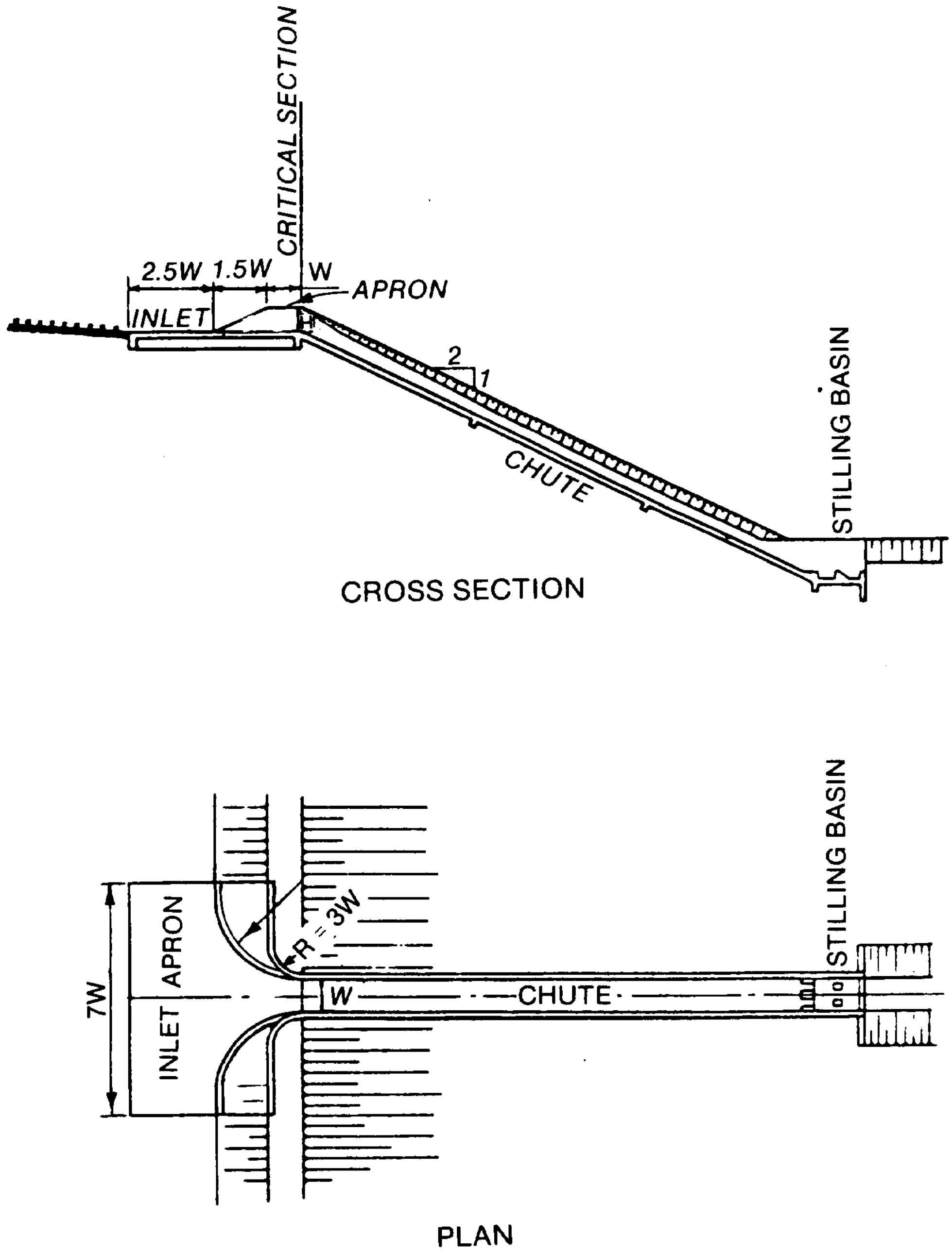


Figure 7-1. Details of typical drainage chute.

a. 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.

b. 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.

7-2. Design.

a. The entrance to the chute can be level or a drop can be provided as shown in figure 7-2. 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 7-3. The curve with D=O 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.

$$Q = 3.1\ W\ H^{1.5} \qquad \text{(eq 7-1)}$$

All of the curves shown in figure 7-3 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.

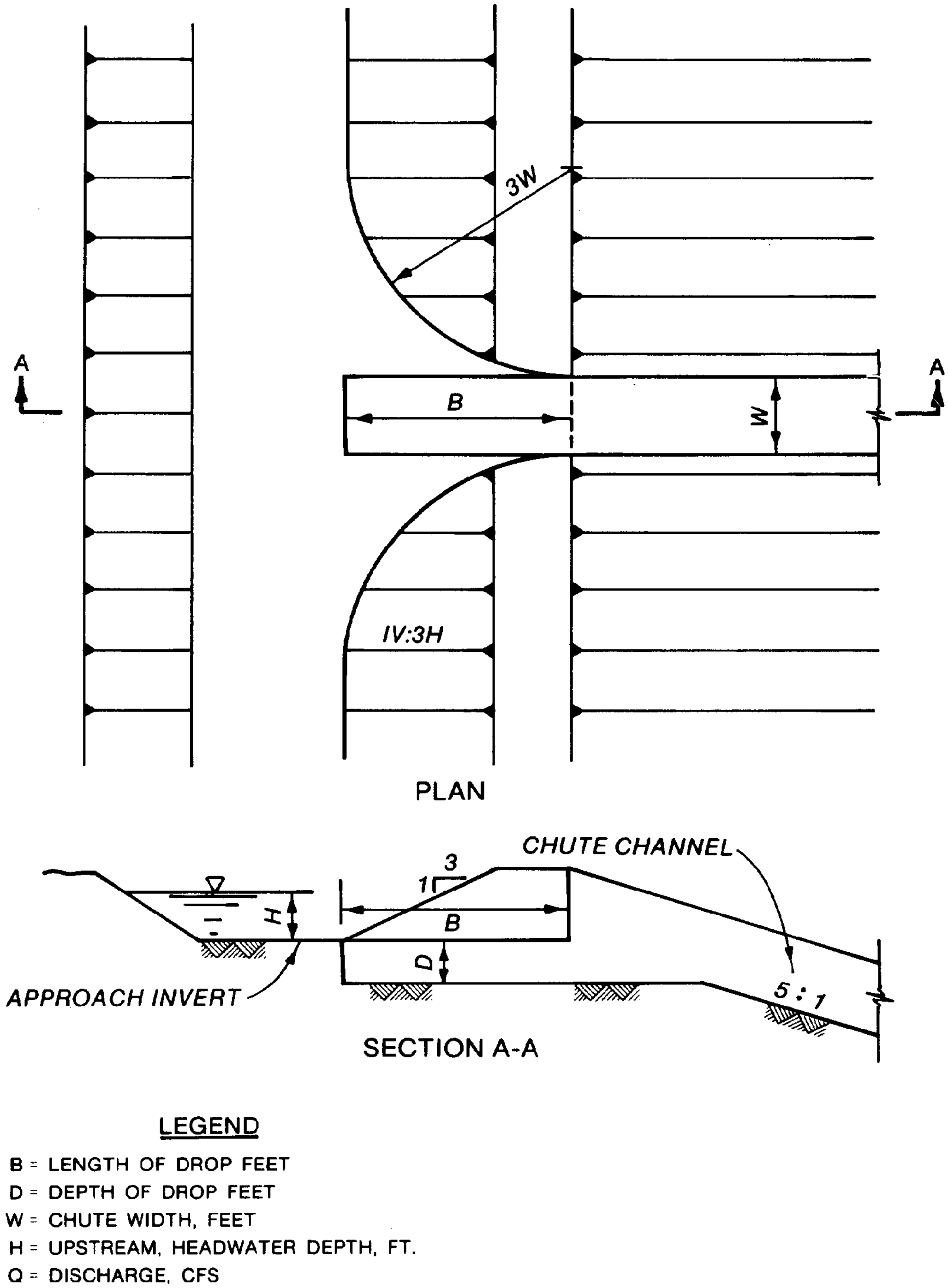


Figure 7-2. Details of typical drop intake.

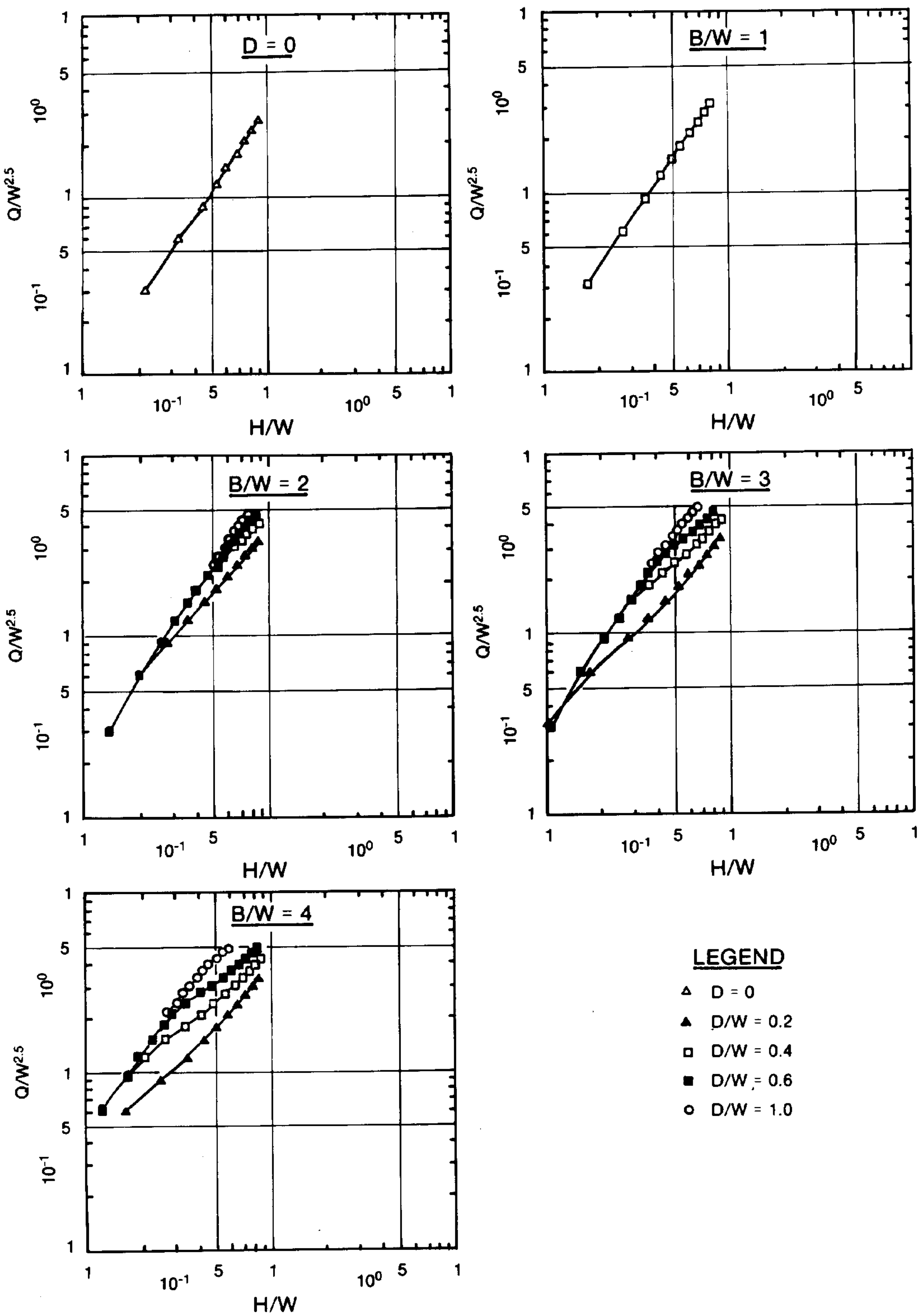


Figure 7-3. Drop structure calibration curves.

b. The depth of flow in the chute can be computed using Manning's equation

$$Q = \frac{1.486}{n} A S^{1/2} R^{2/3} \qquad \text{(eq 7-2)}$$

where:

Q=Discharge, cubic feet per second
n = Roughness factor
A=Area, square feet
S = Slope, feet per feet
R=Hydraulic radius, feet

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 7-4 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.

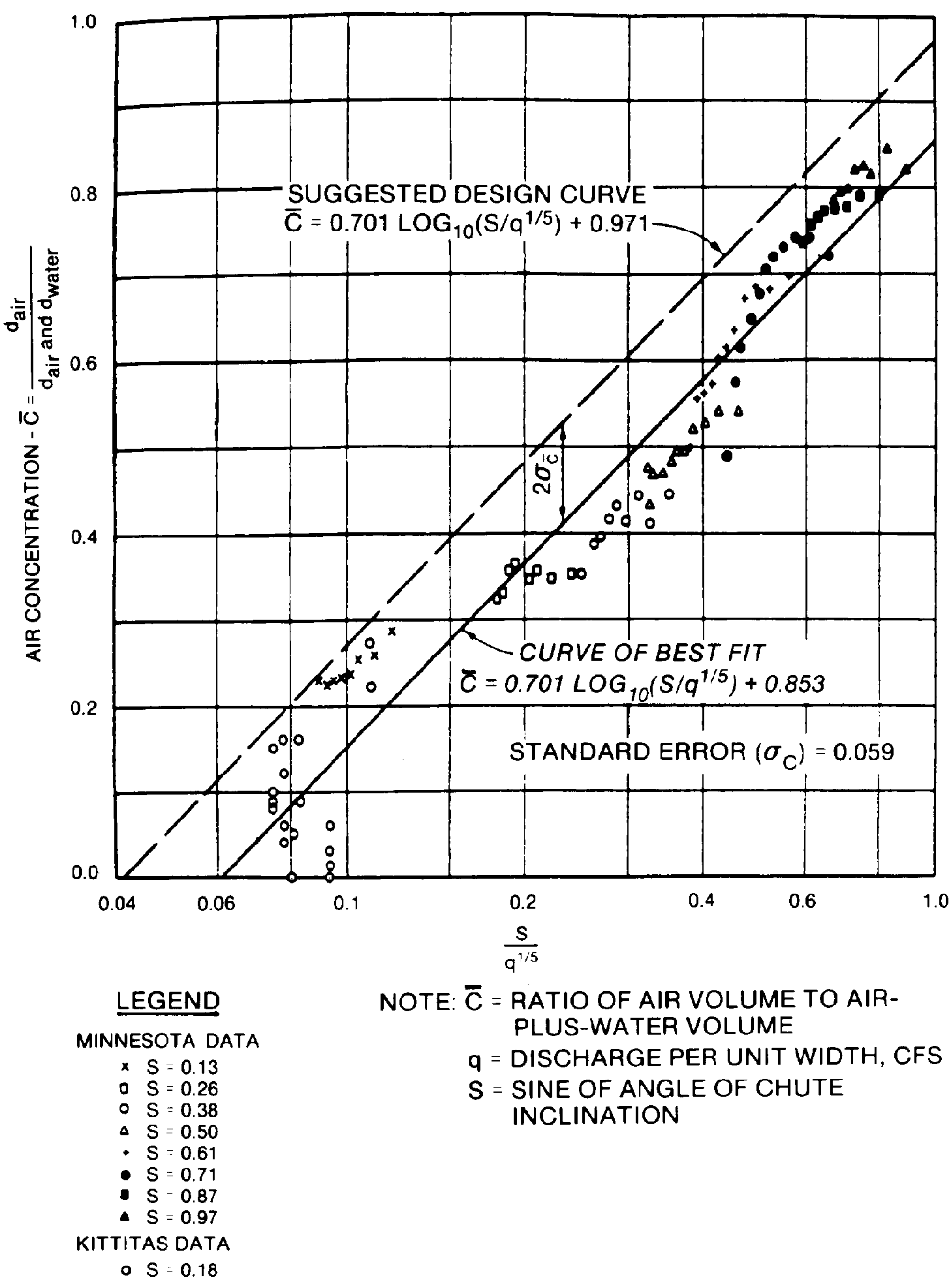


Figure 7-4. Air entrainment in chute flow.

c. 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 inches 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.

7-3. Design problem.

a. Design a concrete chute to carry 25 cubic feet per second down a slope with a 25 percent grade. The allowable head is 1 foot and Manning's n is 0.014.

b. Solution one. Using equation 7-1 with no drop at the entrance, $Q=3.1W(H)^{1/5}$, with Q=25 cubic feet per second and H = 1 foot.

$$25=3.1W(1)^{1/5} \text{ or } W=8.06 \text{ feet} \qquad \text{(eq 7-3)}$$

Use W = 8 feet

Now

$$A=Wd=8d \qquad \text{(eq 7-4)}$$

and

$$R = \frac{\text{area}}{\text{wetted perimeter}} = \frac{8d}{W+2d} = \frac{8d}{8+2d} \qquad \text{(eq 7-5)}$$

Use Manning's equation (7-2) to determine depth of water:

$$Q = \frac{1.486}{n} A S^{1/2} R^{2/3} = \frac{1.486}{0.014} A(0.25)^{1/2} R^{2/3} = 25 \qquad \text{(eq 7-6)}$$

$$25 = \frac{1.486}{0.014} \times 8d \times (0.25)^{1/2} \times \left(\frac{8d}{8+2d}\right)^{2/3} \qquad \text{(eq 7-7)}$$

Solving for d by trial and error, the depth of water is d=0.186 foot. For use in figure 7-4, the size of the angle of the chute is equal to 0.243 and q=Q/W=25/8=3.125. Thus, $S/q^{1/5}$ equals 0.1935, which corresponds to a design air concentration $T = d_{air}/(d_{air}+d)=0.471$. Solving for d_{air} gives 0.166 foot. Then, the total depth of flow is depth of water plus depth of air, 0.352 foot. Wall height should be 1.5 times the total depth of flow or 0.528 foot. One should use 0.5 foot. This design is shown in figure 7-5.

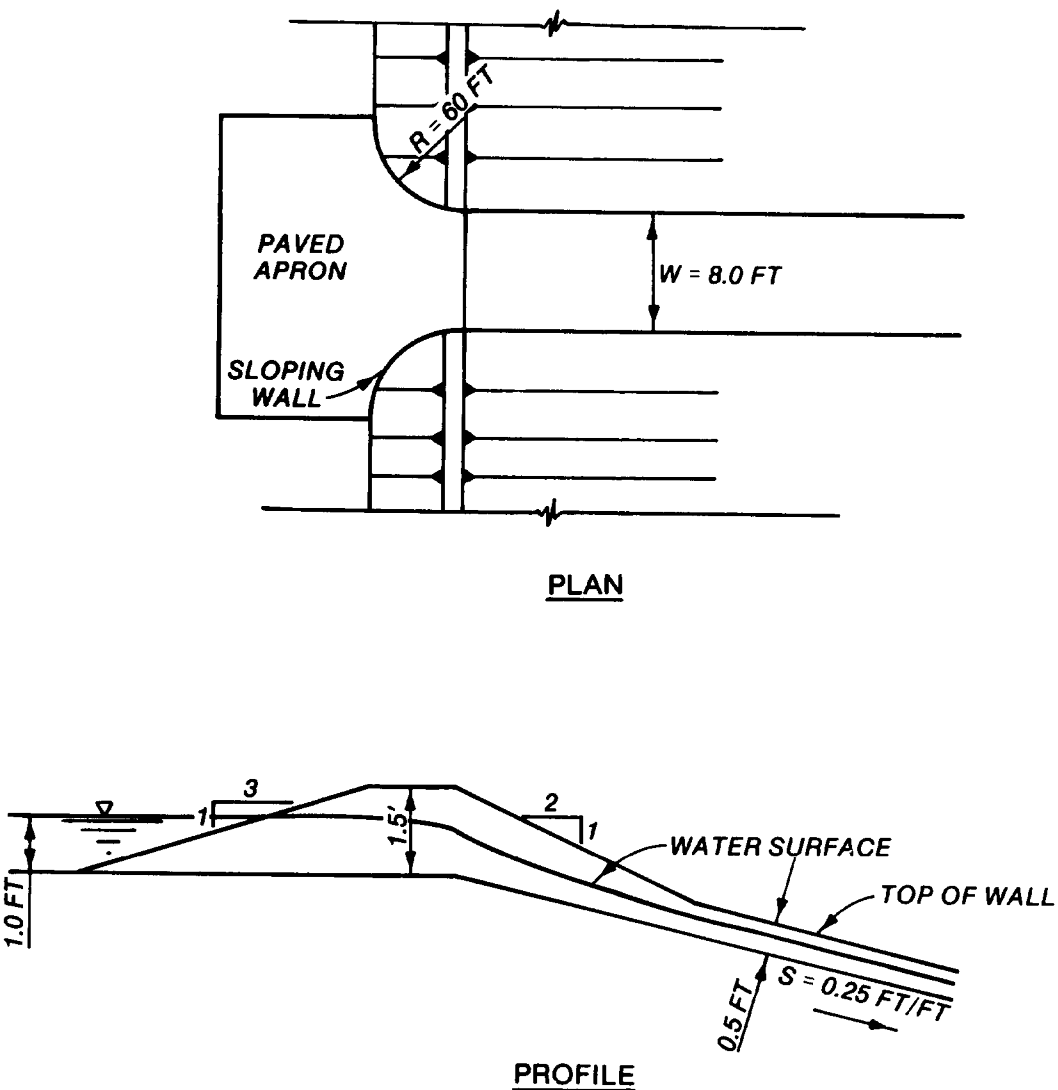


Figure 7-5. Design problem—solution one.

c. Solution two. A drop will he provided at the entrance. Therefore, a width of chute can he selected and the appropriate length and depth of drop determined from the curves in figure 7-3. For this design select a width of 2 feet. Then H/W = ½ = 0.5 and $Q/W^{5/2} = 25/(2)^{5/2} = 4.42$. From figure 7-3 find a curve that matches these values. This is found on the curve for D/w 1.0, on the chart for B/W=4. Therefore, B=8 feet and D=2.0 feet. Using Manning's equation (7-2) to determine depth of water as in the first solution, find d_w=0.493 foot. From figure 7-4, with q equals 12.5, sine of angle of slope equals 0.243 and d_w equals 0.493 foot, determine the depth of air to be 0.311 foot. Thus, total depth is 0.804 foot. Use 0.80 foot. Wall height is 1.5 times 0.80 foot, or 1.20 feet. This design is shown in figure 7-6.

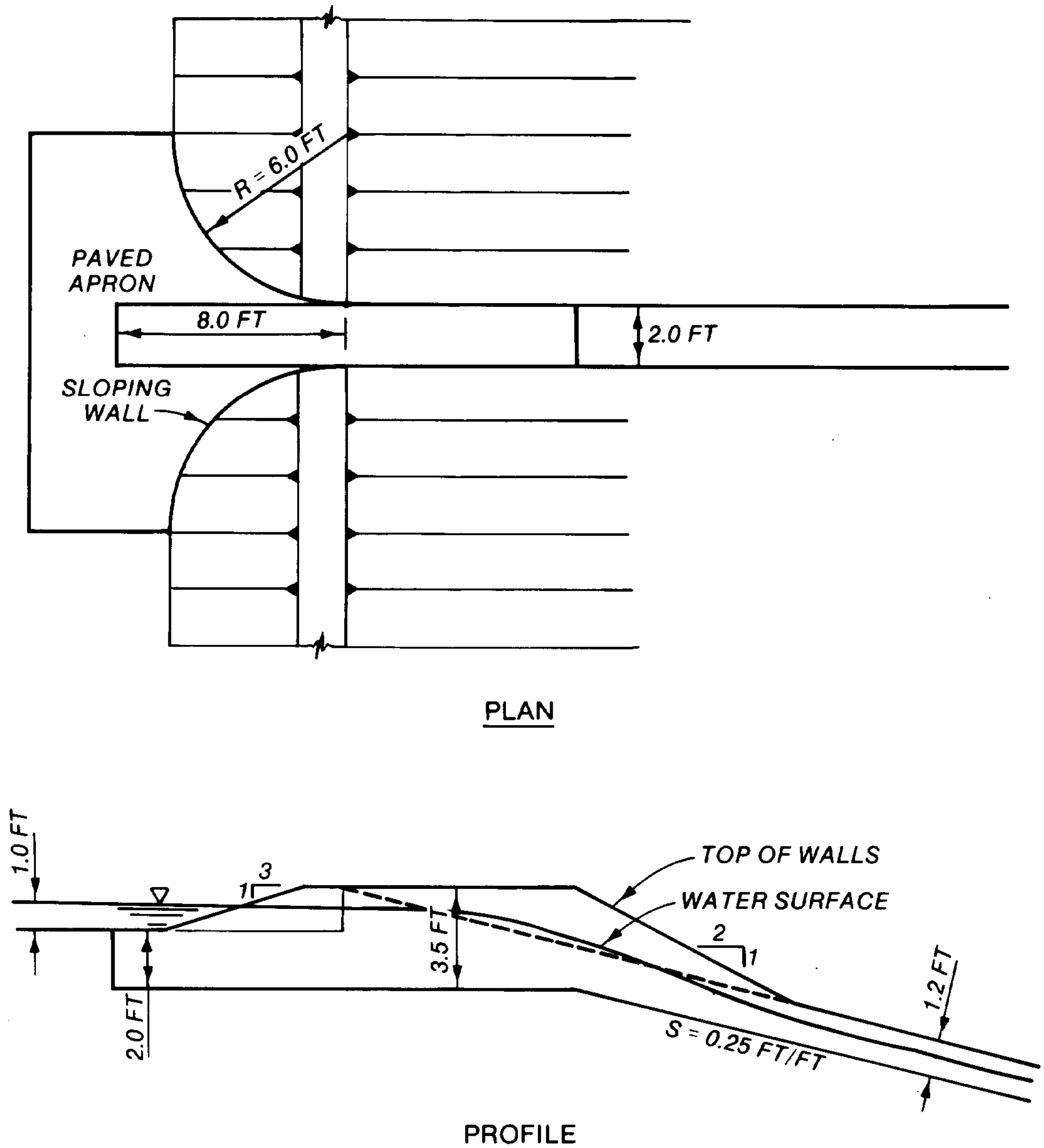


Figure 7-6. Design problem—solution two.

CHAPTER 8

CONSTRUCTION DRAINAGE

8-1. 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.

8-2. 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.

8-3. 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.

8-4. 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.

a. 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.

b. 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.

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

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

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

APPENDIX A

REFERENCES

Government Publications.

Departments of the Army and the Air Force

AFR 86-5	Planning Criteria and Waivers for Airfield Support Facilities
TM 5-803-4	Planning of Army Aviation Facilities
TM 5-818-1	Procedures for Foundation Design of Buildings and Other Structures
TM 5-818-2/AFM 88-6, Chap. 4	Pavement Design for Seasonal Frost Conditions
TM 5-820-4/AFM 88-5, Chap. 4	Drainage for Areas Other Than Airfields
TM 5-803-13/AFM 126-8	Landscape Design and Planting
TM 5-852-6/AFM 88-19, Vol.6	Calculation Methods for Determination of Depths of Freeze and Thaw in Soils
TM 5-852-7/AFM 88-19, Chap. 7	Surface Drainage Design for Airfields and Heliports in Arctic and Subarctic Regions

Department of Transportation

FHWA-RD-77-5	CANDE-A Modern Approach for Structural Design and Analysis of Buried Culverts
FHWA-RD-77-6	CANDE User Manual
FHWA-RD-80-172	CANDE-1980: Box Culverts and Soil Modes

Nongovernment Publications.

Juang, C.H. And Lee, W.J	Development of Cover Requirements for Buried Rigid Pipes Under Highway and Aircraft Loads

APPENDIX B
COVER TABLES

Notes

(1) Except where individual pipe installation designs are made, cover for pipe beneath roads, streets, runways, taxiways, aprons, parking lots or similar areas will be provided in accordance with tables B-1 through B-23.

(2) Cover depths are measured from the top of the pavement to the top of the pipe.

(3) Dashes indicate allowable load is less than load on pipe; blanks indicate that pipe is not specified by the applicable standards.

(4) Calculations are based on 120 pounds per cubic foot backfill compacted to 90 percent of CE 55 (MIL-STD-621) or AASHTO-T99 density (100 percent for cohesionless sands and gravels).

(5) Pipe provided by certain manufacturers exceeds strength requirements established by indicated standards. When additional strength is proved, the allowable cover limits may be reduced accordingly.

(6) Regardless of minimum cover requirements, the distance from the top of the pipe to the bottom of the slab for rigid pavements must exceed the values below to prevent cracking of the slab.

Minimum Cover

Pipe Size in.	Gear-Load	
	less than 100 kips	100 kips or greater
6–60	0.5	1.0
66–120	1.0	1.5

(7) Reinforced concrete pipe Classes I through V refer to ASTM size designations (Classes I through V).

Table B-1. Suggested maximum cover requirements (feet) for 1½- by ¼-inch corrugated steel pipe.

Pipe Diameter in.	18-Gage 0.052 in.	16-Gage 0.064 in.
H-20, 15-K, F-15, C-130, C-141, C-5A, B-1, and B-52 Loads		
4	555	695
6	370	463
8	278	347
10	221	278
12	184	231

Table B-2. Suggested maximum cover requirements (feet) for 2⅔- by ½-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
H-20, 15-K, F-15, C-130, C-5A, C-141, B-1, and B-52 Loads					
12	236	295			
15	188	236			
18	156	196			
21	135	168	236		
24	117	147	206		
30	94	117	164		
36	78	97	137	176	
42		83	117	151	185
48		72	102	131	162
54			89	115	142
60			75	98	121
66			64	83	103
72				70	87
78					74
84					62
90					50
96					42

Table B-3. Suggested maximum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
		H-20, 15-K, F-15, and C-130 Loads			
54	59	74	105	135	166
60	53	67	94	121	149
66	48	61	86	110	135
72	44	55	78	101	124
78	41	51	72	93	114
84	38	47	66	86	105
90	35	43	63	81	99
96		41	58	75	92
102		39	55	71	86
108		35	50	65	80
114		32	46	60	74
120			42	55	68
		C-5A Load			
54	59	74	105	135	166
60	53	67	94	121	149
66	48	61	86	110	135
72	44	55	78	101	124
78	41	51	72	93	114
84	37	47	66	86	105
90	33	43	63	81	99
96		41	58	75	92
102		38	55	71	86
108		33	50	65	80
114		30	46	60	74
120			42	55	68

(Continued)

Table B-3. Suggested maximum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
		C-141, B-1, and B-52 Loads			
54	59	74	105	135	166
60	53	67	94	121	149
66	48	61	86	110	135
72	44	55	78	101	124
78	41	51	72	93	114
84	37	47	66	86	105
90	34	43	63	81	99
96		41	58	75	92
102		38	55	71	86
108		34	50	65	80
114		31	46	60	74
120			42	55	68
		C-141, B-1, and B-57 Loads			
54	59	74	105	135	166
60	53	67	94	121	149
66	48	61	86	110	135
72	44	55	78	101	124
78	41	51	72	93	114
84	37	47	66	86	105
90	34	43	63	81	99
96		41	58	75	92
102		38	55	71	86
108		34	50	65	80
114		31	46	60	74
120			42	55	68

Table B-4. Suggested maximum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			H-20, 15-K, F-15, and C-130 Loads		
54	52	66	93	120	148
60	48	59	84	108	132
66	43	54	76	98	121
72	39	49	69	89	110
78	36	46	64	83	102
84	33	42	59	77	94
90	21	39	55	71	88
96		37	52	66	83
102		34	48	62	78
108		32	45	60	73
114			42	55	67
120			38	50	62
			C-5A Load		
54	52	66	93	120	148
60	48	59	84	108	132
66	43	54	76	98	121
72	39	49	69	89	110
78	35	46	64	83	102
84	31	42	59	77	94
90	28	38	55	71	88
96		36	52	66	83
102		32	48	62	78
108		30	45	60	73
114			42	55	67
120			38	50	62

(Continued)

Table B-4. Suggested maximum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			C-141, B-1, and B-52 Loads		
54	52	66	93	120	148
60	48	59	84	108	132
66	43	54	76	98	121
72	39	49	69	89	110
78	35	46	64	83	102
84	32	42	59	77	94
90	30	39	55	71	88
96		36	52	66	83
102		33	48	62	78
108		31	45	60	73
114			42	55	67
120			38	50	62

Table B-5. Suggested maximum cover requirements (feet) for 6- by 2-inch corrugated steel pipe.

Pipe Diameter in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.	7-Gage 0.188 in.	5-Gage 0.218 in.	3-Gage 0.249 in.	1-Gage 0.280 in.
	H-20, 15-K, F-15, C-130, C-5A, C-141, B-1, and B-52 Loads						
60	94	121	149	166	194	222	249
66	85	109	134	150	176	202	227
72	78	101	124	138	161	185	208
78	72	93	113	127	148	170	192
84	66	85	105	118	138	158	178
90	62	80	98	110	128	147	166
96	58	75	92	102	121	138	156
102	55	70	86	97	113	130	146
108	51	66	81	91	107	122	138
114	48	62	77	86	101	116	131
120	46	59	73	82	96	110	124

Table B-6. Suggested minimum cover requirements (feet) for 1½- by ¼-inch corrugated steel pipe.

Pipe Diameter in.	18-Gage 0.052 in.	16-Gage 0.064 in.
	H-20, 15-K, and C-5A Loads	
4	1.0	1.0
6	1.0	1.0
8	1.0	1.0
10	1.0	1.0
12	1.0	1.0
	F-15, C-141 Loads	
4	1.0	1.0
6	1.5	1.5
8	1.5	1.5
10	1.5	1.5
12	1.5	1.5
	C-130 Load	
4	1.0	1.0
6	1.0	1.0
8	1.0	1.0
10	1.5	1.5
12	1.5	1.5
	B-1 Load	
4	1.5	1.5
6	1.5	1.5
8	1.5	1.5
10	2.0	1.5
12	2.0	2.0
	B-52 Load	
4	1.5	1.5
6	1.5	1.5
8	2.0	1.5
10	2.0	2.0
12	2.0	2.0

Table B-7. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			H-20 Load		
12	1.0	1.0			
15	1.0	1.0			
18	1.0	1.0			
21	1.0	1.0	1.0		
24	1.0	1.0	1.0		
30	1.0	1.0	1.0		
36	1.0	1.0	1.0	1.0	
42		1.5	1.0	1.0	1.0
48		1.5	1.5	1.5	1.0
54			1.5	1.5	1.5
60			1.5	1.5	1.5
66			1.5	1.5	1.5
72				1.5	1.5
78					1.5
84					1.5
90					1.5
96					1.5
			15-K Load		
12	1.0	1.0			
15	1.0	1.0			
18	1.0	1.0			
21	1.0	1.5	1.0		
24	1.0	1.5	1.0		
30	1.5	1.5	1.5		
36	1.5	1.5	1.5	1.5	
42	1.5	1.5	1.5	1.5	1.5
48			1.5	1.5	1.5
54			1.5	1.5	1.5
60			1.5	1.5	1.5
66			1.5	1.5	1.5
72				1.5	1.5
78					1.5
84					1.5
90					1.5
96					1.5

(Continued)

(Sheet 1 of 4)

Table B-7. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
		F-15 Load			
12	1.5	1.5			
15	1.5	1.5			
18	1.5	1.5			
21	2.0	2.0	1.5		
24	2.0	2.0	2.0		
30	2.0	2.0	2.0		
36	2.0	2.0	2.0	2.0	
42		2.0	2.0	2.0	2.0
48		2.0	2.0	2.0	2.0
54			2.0	2.0	2.0
60			2.0	2.0	2.0
66			2.0	2.0	2.0
72				2.0	2.0
78					2.0
84					2.5
90					2.5
96					2.5
		C-130 Load			
12	1.5	1.5			
15	1.5	1.5			
18	1.5	1.5			
21	1.5	1.5	1.5		
24	2.0	2.0	1.5		
30	2.0	2.0	2.0		
36	2.0	2.0	2.0	2.0	
42		2.0	2.0	2.0	2.0
48		2.0	2.0	2.0	2.0
54			2.0	2.0	2.0
60			2.0	2.0	2.0
66			2.5	2.0	2.0
72				2.5	2.0
78					2.5
84					2.5
90					2.5
96					2.5

(Continued)

(Sheet 2 of 4)

Table B-7. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
		C-5A Load			
12	1.5	1.5			
15	1.5	1.5			
18	1.5	1.5			
21	1.5	1.5	1.5		
24	1.5	1.5	1.5		
30	1.5	1.5	1.5		
36	2.0	2.0	2.0	1.5	
42		2.0	2.0	2.0	2.0
48		2.0	2.0	2.0	2.0
54			2.0	2.0	2.0
60			2.0	2.0	2.0
66			2.0	2.0	2.0
72				2.0	2.0
78					2.0
84					2.5
90					2.5
96					2.5
		C-141 Load			
12	1.5	1.5			
15	2.0	2.0			
18	2.0	2.0			
21	2.0	2.0	2.0		
24	2.0	2.0	2.0		
30	2.0	2.0	2.0		
36	2.5	2.5	2.0	2.0	
42		2.5	2.5	2.5	2.0
48		2.5	2.5	2.5	2.5
54			2.5	2.5	2.5
60			2.5	2.5	2.5
66			3.0	2.5	2.5
72				3.0	2.5
78					3.0
84					3.0
90					3.0
96					3.0

(Continued)

(Sheet 3 of 4)

Table B-7. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
		B-1 Load			
12	2.0	2.0			
15	2.0	2.0			
18	2.0	2.0			
21	2.0	2.0	2.0		
24	2.5	2.5	2.0		
30	2.5	2.5	2.5		
36	2.5	2.5	2.5	2.5	
42		3.0	2.5	2.5	2.5
48		3.0	3.0	2.5	2.5
54			3.0	3.0	3.0
60			3.0	3.0	3.0
66			3.0	3.0	3.0
72				3.0	3.0
78					3.0
84					3.0
90					3.0
96					3.0
		B-52 Load			
12	2.0	2.0			
15	2.0	2.0			
18	2.5	2.5			
21	2.5	2.5	2.5		
24	2.5	2.5	2.5		
30	3.0	3.0	2.5		
36	3.0	3.0	3.0	3.0	
42		3.0	3.0	3.0	3.0
48		3.0	3.0	3.0	3.0
54			3.0	3.0	3.0
60			3.0	3.0	3.0
66			3.0	3.0	3.0
72				3.0	3.0
78					3.0
84					3.5
90					3.5
96					3.5

(Sheet 4 of 4)

Table B-8. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			H-20 Load		
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5
78	1.5	1.5	1.5	1.5	1.5
84	1.5	1.5	1.5	1.5	1.5
90	1.5	1.5	1.5	1.5	1.5
96		1.5	1.5	1.5	1.5
102		1.5	1.5	1.5	1.5
108		1.5	1.5	1.5	1.5
114		2.0	1.5	1.5	1.5
120			2.0	1.5	1.5
			15-K Load		
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5
78	1.5	1.5	1.5	1.5	1.5
84	2.0	1.5	1.5	1.5	1.5
90	2.0	2.0	1.5	1.5	1.5
96		2.0	2.0	1.5	1.5
102		2.0	2.0	2.0	2.0
108		2.0	2.0	2.0	2.0
114		2.0	2.0	2.0	2.0
120			2.0	2.0	2.0

(Continued)

(Sheet 1 of 4)

Table B-8. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			F-15 Load		
54	2.0	2.0	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
66	2.5	2.5	2.5	2.0	2.0
72	2.5	2.5	2.5	2.5	2.5
78	2.5	2.5	2.5	2.5	2.5
84	2.5	2.5	2.5	2.5	2.5
90	2.5	2.5	2.5	2.5	2.5
96		2.5	2.5	2.5	2.5
102		2.5	2.5	2.5	2.5
108		2.5	2.5	2.5	2.5
114		2.5	2.5	2.5	2.5
120			2.5	2.5	2.5
			C-130 Load		
54	2.5	2.5	2.0	2.0	2.0
60	2.5	2.5	2.5	2.5	2.0
66	2.5	2.5	2.5	2.5	2.5
72	2.5	2.5	2.5	2.5	2.5
78	2.5	2.5	2.5	2.5	2.5
84	2.5	2.5	2.5	2.5	2.5
90	3.0	2.5	2.5	2.5	2.5
96		3.0	2.5	2.5	2.5
102		3.0	2.5	2.5	2.5
108		3.0	3.0	2.5	2.5
114		3.0	3.0	3.0	2.5
120			3.0	3.0	3.0

(Continued)

(Sheet 2 of 4)

Table B-8. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage .064 in.	14-Gage .079 in.	12-Gage .109 in.	10-Gage .138 in.	8-Gage .168 in.
			C-5A Load		
54	2.0	2.0	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
66	2.5	2.5	2.5	2.5	2.0
72	2.5	2.5	2.5	2.5	2.5
78	2.5	2.5	2.5	2.5	2.5
84	3.0	2.5	2.5	2.5	2.5
90	3.0	3.0	2.5	2.5	2.5
96		3.0	3.0	2.5	2.5
102		3.0	3.0	3.0	2.5
108		3.0	3.0	3.0	3.0
114		3.0	3.0	3.0	3.0
120			3.0	3.0	3.0
			C-141 Load		
54	3.0	3.0	2.5	2.5	2.5
60	3.0	3.0	3.0	3.0	2.5
66	3.0	3.0	3.0	3.0	3.0
72	3.0	3.0	3.0	3.0	3.0
78	3.0	3.0	3.0	3.0	3.0
84	3.0	3.0	3.0	3.0	3.0
90	3.5	3.0	3.0	3.0	3.0
96		3.5	3.0	3.0	3.0
102		3.5	3.5	3.0	3.0
108		3.5	3.5	3.5	3.0
114		3.5	3.5	3.5	3.5
120			3.5	3.5	3.5

(Continued)

(Sheet 3 of 4)

Table B-8. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			B-1 Load		
54	3.0	3.0	3.0	3.0	3.0
60	3.0	3.0	3.0	3.0	3.0
66	3.0	3.0	3.0	3.0	3.0
72	3.5	3.0	3.0	3.0	3.0
78	3.5	3.5	3.0	3.0	3.0
84	3.5	3.5	3.5	3.0	3.0
90	3.5	3.5	3.5	3.5	3.0
96		3.5	3.5	3.5	3.5
102		4.0	3.5	3.5	3.5
108		4.0	3.5	3.5	3.5
114		4.0	4.0	3.5	3.5
120			4.0	4.0	3.5
			B-52 Load		
54	3.5	3.0	3.0	3.0	3.0
60	3.5	3.5	3.5	3.0	3.0
66	3.5	3.5	3.5	3.5	3.0
72	3.5	3.5	3.5	3.5	3.5
78	4.0	3.5	3.5	3.5	3.5
84	4.0	4.0	3.5	3.5	3.5
90	4.0	4.0	4.0	3.5	3.5
96		4.0	4.0	4.0	3.5
102		4.0	4.0	4.0	4.0
108		4.0	4.0	4.0	4.0
114		4.5	4.0	4.0	4.0
120			4.0	4.0	4.0

(Sheet 4 of 4)

Table B-9. Suggested minimum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			H-20 Load		
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5
78	1.5	1.5	1.5	1.5	1.5
84	1.5	1.5	1.5	1.5	1.5
90	1.5	1.5	1.5	1.5	1.5
96		1.5	1.5	1.5	1.5
102		2.0	1.5	1.5	1.5
108		2.0	2.0	1.5	1.5
114			2.0	1.5	1.5
120			2.0	2.0	1.5
			15-K Load		
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5
78	2.0	2.0	1.5	1.5	1.5
84	2.0	2.0	2.0	1.5	1.5
90	2.0	2.0	2.0	2.0	1.5
96		2.0	2.0	2.0	2.0
102		2.0	2.0	2.0	2.0
108		2.0	2.0	2.0	2.0
114			2.0	2.0	2.0
120			2.0	2.0	2.0

(Continued)

(Sheet 1 of 4)

Table B-9. Suggested minimum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			F-15 Load		
54	2.5	2.5	2.0	2.0	2.0
60	2.5	2.5	2.5	2.0	2.0
66	2.5	2.5	2.5	2.5	2.5
72	2.5	2.5	2.5	2.5	2.5
78	2.5	2.5	2.5	2.5	2.5
84	2.5	2.5	2.5	2.5	2.5
90	2.5	2.5	2.5	2.5	2.5
96		2.5	2.5	2.5	2.5
102		2.5	2.5	2.5	2.5
108		3.0	2.5	2.5	2.5
114			2.5	2.5	2.5
120			3.0	2.5	2.5
			C-130 Load		
54	2.5	2.5	2.5	2.5	2.0
60	2.5	2.5	2.5	2.5	2.5
66	2.5	2.5	2.5	2.5	2.5
72	2.5	2.5	2.5	2.5	2.5
78	2.5	2.5	2.5	2.5	2.5
84	3.0	2.5	2.5	2.5	2.5
90	3.0	3.0	2.5	2.5	2.5
96		3.0	3.0	2.5	2.5
102		3.0	3.0	3.0	2.5
108		3.0	3.0	3.0	3.0
114			3.0	3.0	3.0
120			3.0	3.0	3.0

(Continued)

(Sheet 2 of 4)

Table B-9. Suggested minimum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			C-5A Load		
54	2.5	2.5	2.0	2.0	2.0
60	2.5	2.5	2.5	2.0	2.0
66	2.5	2.5	2.5	2.5	2.5
72	2.5	2.5	2.5	2.5	2.5
78	3.0	3.0	2.5	2.5	2.5
84	3.0	3.0	2.5	2.5	2.5
90	3.0	3.0	3.0	2.5	2.5
96		3.0	3.0	3.0	2.5
102		3.0	3.0	3.0	3.0
108		3.0	3.0	3.0	3.0
114			3.0	3.0	3.0
120			3.0	3.0	3.0
			C-141 Load		
54	3.0	3.0	3.0	3.0	2.5
60	3.0	3.0	3.0	3.0	3.0
66	3.0	3.0	3.0	3.0	3.0
72	3.0	3.0	3.0	3.0	3.0
78	3.5	3.0	3.0	3.0	3.0
84	3.5	3.5	3.0	3.0	3.0
90	3.5	3.5	3.0	3.0	3.0
96		3.5	3.5	3.0	3.0
102		3.5	3.5	3.5	3.0
108		4.0	3.5	3.5	3.5
114			3.5	3.5	3.5
120			3.5	3.5	3.5

(Continued)

(Sheet 3 of 4)

Table B-9. Suggested minimum cover requirements (feet) for 5- by 1-inch corrugated steel pipe.—Continued

Pipe Diameter in.	16-Gage 0.064 in.	14-Gage 0.079 in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.
			B-1 Load		
54	3.0	3.0	3.0	3.0	3.0
60	3.0	3.0	3.0	3.0	3.0
66	3.5	3.0	3.0	3.0	3.0
72	3.5	3.5	3.0	3.0	3.0
78	3.5	3.5	3.5	3.0	3.0
84	4.0	3.5	3.5	3.5	3.0
90	4.0	3.5	3.5	3.5	3.5
96		4.0	3.5	3.5	3.5
102		4.0	4.0	3.5	3.5
108		4.0	4.0	3.5	3.5
114			4.0	4.0	3.5
120			4.0	4.0	4.0
			B-52 Load		
54	3.5	3.5	3.0	3.0	3.0
60	3.5	3.5	3.5	3.5	3.0
66	4.0	3.5	3.5	3.5	3.5
72	4.0	4.0	3.5	3.5	3.5
78	4.0	4.0	3.5	3.5	3.5
84	4.0	4.0	4.0	4.0	3.5
90	4.0	4.0	4.0	4.0	4.0
96		4.0	4.0	4.0	4.0
102		4.5	4.0	4.0	4.0
108		4.5	4.0	4.0	4.0
114			4.0	4.0	4.0
120			4.5	4.0	4.0

(Sheet 4 of 4)

Table B-10. Suggested minimum cover requirements (feet) for 6- by 2-inch corrugated steel pipe.

Pipe Diameter in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.	7-Gage 0.188 in.	5-Gage 0.218 in.	3-Gage 0.249 in.	1-Gage 0.280 in.
			H-20 Load				
60	1.5	1.5	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5	1.5	1.5
78	1.5	1.5	1.5	1.5	1.5	1.5	1.5
84	1.5	1.5	1.5	1.5	1.5	1.5	1.5
90	1.5	1.5	1.5	1.5	1.5	1.5	1.5
96	1.5	1.5	1.5	1.5	1.5	1.5	1.5
102	1.5	1.5	1.5	1.5	1.5	1.5	1.5
108	1.5	1.5	1.5	1.5	1.5	1.5	1.5
114	1.5	1.5	1.5	1.5	1.5	1.5	1.5
120	2.0	1.5	1.5	1.5	1.5	1.5	1.5
			15-K Load				
60	1.5	1.5	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5	1.5	1.5
72	1.5	1.5	1.5	1.5	1.5	1.5	1.5
78	1.5	1.5	1.5	1.5	1.5	1.5	1.5
84	1.5	1.5	1.5	1.5	1.5	1.5	1.5
90	2.0	1.5	1.5	1.5	1.5	1.5	1.5
96	2.0	1.5	1.5	1.5	1.5	1.5	1.5
102	2.0	2.0	1.5	1.5	1.5	1.5	1.5
108	2.0	2.0	2.0	1.5	1.5	1.5	1.5
114	2.0	2.0	2.0	1.5	1.5	1.5	1.5
120	2.0	2.0	2.0	2.0	1.5	1.5	1.5

(Continued)

(Sheet 1 of 4)

Table B-10. Suggested minimum cover requirements (feet) for 6- by 2-inch corrugated steel pipe.—Continued

Pipe Diameter in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.	7-Gage 0.188 in.	5-Gage 0.218 in.	3-Gage 0.249 in.	1-Gage 0.280 in.
			F-15 Load				
60	2.0	2.0	2.0	2.0	2.0	2.0	2.0
66	2.5	2.5	2.0	2.0	2.0	2.0	2.0
72	2.5	2.5	2.5	2.5	2.0	2.0	2.0
78	2.5	2.5	2.5	2.5	2.5	2.0	2.0
84	2.5	2.5	2.5	2.5	2.5	2.5	2.0
90	2.5	2.5	2.5	2.5	2.5	2.5	2.0
96	2.5	2.5	2.5	2.5	2.5	2.5	2.5
102	2.5	2.5	2.5	2.5	2.5	2.5	2.5
108	2.5	2.5	2.5	2.5	2.5	2.5	2.5
114	2.5	2.5	2.5	2.5	2.5	2.5	2.5
120	2.5	2.5	2.5	2.5	2.5	2.5	2.5
			C-130 Load				
60	2.5	2.5	2.5	2.5	2.0	2.0	2.0
66	2.5	2.5	2.5	2.5	2.5	2.0	2.0
72	2.5	2.5	2.5	2.5	2.5	2.5	2.0
78	2.5	2.5	2.5	2.5	2.5	2.5	2.5
84	2.5	2.5	2.5	2.5	2.5	2.5	2.5
90	2.5	2.5	2.5	2.5	2.5	2.5	2.5
96	2.5	2.5	2.5	2.5	2.5	2.5	2.5
102	3.0	2.5	2.5	2.5	2.5	2.5	2.5
108	3.0	3.0	2.5	2.5	2.5	2.5	2.5
114	3.0	3.0	2.5	2.5	2.5	2.5	2.5
120	3.0	3.0	3.0	2.5	2.5	2.5	2.5

(Continued)

(Sheet 2 of 4)

Table B-10. Suggested minimum cover requirements (feet) for 6- by 2-inch corrugated steel pipe.-Continued

Pipe Diameter in.	12-Gage 0.109 in.	10-Gage 0.138 in.	8-Gage 0.168 in.	7-Gage 0.188 in.	5-Gage 0.218 in.	3-Gage 0.249 in.	1-Gage 0.280 in.
			C-5A Load				
60	2.5	2.0	2.0	2.0	2.0	2.0	2.0
66	2.5	2.5	2.0	2.0	2.0	2.0	2.0
72	2.5	2.5	2.5	2.5	2.0	2.0	2.0
78	2.5	2.5	2.5	2.5	2.5	2.0	2.0
84	2.5	2.5	2.5	2.5	2.5	2.5	2.0
90	3.0	2.5	2.5	2.5	2.5	2.5	2.0
96	3.0	3.0	2.5	2.5	2.5	2.5	2.5
102	3.0	3.0	2.5	2.5	2.5	2.5	2.5
108	3.0	3.0	3.0	3.0	2.5	2.5	2.5
114	3.0	3.0	3.0	3.0	2.5	2.5	2.5
120	3.0	3.0	3.0	3.0	3.0	2.5	2.5
			C-141 Load				
60	3.0	3.0	3.0	2.5	2.5	2.5	2.5
66	3.0	3.0	3.0	3.0	3.0	2.5	2.5
72	3.0	3.0	3.0	3.0	3.0	3.0	2.5
78	3.0	3.0	3.0	3.0	3.0	3.0	3.0
84	3.0	3.0	3.0	3.0	3.0	3.0	3.0
90	3.0	3.0	3.0	3.0	3.0	3.0	3.0
96	3.5	3.0	3.0	3.0	3.0	3.0	3.0
102	3.5	3.0	3.0	3.0	3.0	3.0	3.0
108	3.5	3.5	3.0	3.0	3.0	3.0	3.0
114	3.5	3.5	3.5	3.0	3.0	3.0	3.0
120	3.5	3.5	3.5	3.5	3.0	3.0	3.0

(Continued)

(Sheet 3 of 4)

Table B–10. Suggested minimum cover requirements (feet) for 6- by 2-inch corrugated steel pipe.- Continued

Pipe Diameter in.	12–Gage 0.109 in.	10–Gage 0.138 in.	8–Gage 0.168 in.	7–Gage 0.188 in.	5–Gage 0.218 in.	3–Gage 0.249 in.	1–Gage 0.280 in.
			B–1 Load				
60	3.0	3.0	3.0	3.0	3.0	3.0	3.0
66	3.0	3.0	3.0	3.0	3.0	3.0	3.0
72	3.0	3.0	3.0	3.0	3.0	3.0	3.0
78	3.5	3.0	3.0	3.0	3.0	3.0	3.0
84	3.5	3.5	3.0	3.0	3.0	3.0	3.0
90	3.5	3.5	3.5	3.0	3.0	3.0	3.0
96	3.5	3.5	3.5	3.5	3.0	3.0	3.0
102	3.5	3.5	3.5	3.5	3.5	3.0	3.0
108	4.0	3.5	3.5	3.5	3.5	3.5	3.0
114	4.0	3.5	3.5	3.5	3.5	3.5	3.0
120	4.0	4.0	3.5	3.5	3.5	3.5	3.5
			B–52 Load				
60	3.5	3.5	3.0	3.0	3.0	3.0	3.0
66	3.5	3.5	3.5	3.5	3.0	3.0	3.0
72	3.5	3.5	3.5	3.5	3.5	3.0	3.0
78	3.5	3.5	3.5	3.5	3.5	3.5	3.5
84	4.0	3.5	3.5	3.5	3.5	3.5	3.5
90	4.0	4.0	3.5	3.5	3.5	3.5	3.5
96	4.0	4.0	4.0	4.0	3.5	3.5	3.5
102	4.0	4.0	4.0	4.0	3.5	3.5	3.5
108	4.0	4.0	4.0	4.0	4.0	3.5	3.5
114	4.0	4.0	4.0	4.0	4.0	4.0	3.5
120	4.0	4.0	4.0	4.0	4.0	4.0	4.0

(Sheet 4 of 4)

Table B-11. Suggested maximum cover requirements (feet) for 2⅔- by ½-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
		H-20, 15-K, F-15, C-130, C-141, B-1, B-52 Loads			
12	117	147	206	265	324
15	94	117	165	212	259
18	78	97	137	176	216
24	58	73	103	133	162
30	47	58	82	106	129
36	37	49	68	88	108
42		41	58	75	92
48			51	66	80
54			45	58	71
60				52	64
66					58
72					53
78					48
84					45
			C-5A Load		
12	117	147	206	265	324
15	94	117	165	212	259
18	78	97	137	176	216
24	58	73	103	133	162
30	47	58	82	106	129
36	37	49	68	88	108
42		41	58	75	92
48			51	66	80
54			45	58	71
60				52	64
66					58
72					53
78					48
84					45

Table B–12. Suggested maximum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
		H-20, 15-K, F-15 Loads			
30	54	67	94	122	
36	44	55	78	101	
42	37	48	67	87	
48	30	42	58	75	92
54	25	35	51	67	82
60	21	30	47	60	74
66	19	26	42	54	68
72		24	38	50	62
78		20	33	45	57
84		18	30	43	52
90			27	40	48
96			25	36	45
102			23	33	42
108			21	30	41
114			19	28	37
120			18	26	34
		C-130 Load			
30	54	67	94	122	
36	44	55	78	101	
42	38	48	67	87	
48	32	42	58	75	92
54	28	36	51	67	82
60	24	32	47	60	74
66	21	28	42	54	68
72		26	39	50	62
78		22	34	45	57
84		20	32	43	52
90			29	40	48
96			27	36	45
102			25	34	42
108			24	32	41
114			21	30	37
120			20	28	35

(Continued)

(Sheet 1 of 3)

Table B-12. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
		C-51 Load			
30	54	67	94	122	
36	44	55	78	101	
42	37	48	67	87	
48	30	42	58	75	92
54	25	35	51	67	82
60	21	30	47	60	74
66	19	26	42	54	68
72		24	38	50	62
78		20	33	45	57
84		18	30	43	52
90			27	40	48
96			25	36	45
102			23	33	42
108			21	30	41
114			19	28	37
120			18	26	34
		C-141 Load			
30	54	67	94	122	
36	44	55	78	101	
42	38	48	67	87	
48	32	42	58	75	92
54	28	36	51	67	82
60	24	32	47	60	74
66	21	28	42	54	68
72		26	39	50	62
78		22	34	45	57
84		20	32	43	52
90			29	40	48
96			27	36	45
102			25	34	42
108			24	32	41
114			21	30	37
120			20	28	35

(Continued)

(Sheet 2 of 3)

Table B-12. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.– Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			B-1 Load		
30	54	67	94	122	
36	44	55	78	101	
42	38	48	67	87	
48	32	42	58	75	92
54	27	36	51	67	82
60	23	32	47	60	74
66	20	27	42	54	68
72		25	39	50	62
78		22	34	45	57
84		19	32	43	52
90			28	40	48
96			26	36	45
102			24	34	42
108			23	32	41
114			20	29	37
120			19	27	35
			B-52 Load		
30	54	67	94	122	
36	44	55	78	101	
42	37	48	67	87	
48	31	42	58	75	92
54	27	36	51	67	82
60	23	31	47	60	74
66	20	27	42	54	68
72		25	39	50	62
78		22	34	45	57
84		19	32	43	52
90			28	40	48
96			26	36	45
102			24	34	42
108			23	31	41
114			20	29	37
120			19	27	35

(Sheet 3 of 3)

Table B-13. Suggested maximum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			H-20, 15-K, F-15 Loads		
48	28	36	51	66	80
54	25	31	45	58	71
60	22	28	40	52	64
72		23	33	44	53
78		21	30	40	48
84			28	37	45
90			26	34	42
96			24	32	40
102			23	30	37
108			20	28	35
114			19	26	33
120			18	25	31
			C-130 Load		
48	28	36	51	66	80
54	25	31	45	58	71
60	21	28	40	52	64
72		22	33	44	53
78		20	30	40	48
84			28	37	45
90			26	34	42
96			23	31	40
102			22	29	36
108			20	27	34
114			19	25	32
120			17	24	30

(Continued)

(Sheet 1 of 3)

Table B-13. Suggested maximum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			C-5A Load		
48	24	34	51	66	80
54	21	28	45	58	71
60	18	24	40	52	64
72		19	30	44	53
78		17	27	40	48
84			25	35	45
90			23	32	42
96			20	29	40
102			19	27	35
108			17	25	33
114			16	23	31
120			15	22	28
			C-141 Load		
48	27	35	51	66	80
54	23	30	45	58	71
60	20	27	40	52	64
72		21	32	44	53
78		18	29	40	48
84			27	36	45
90			25	33	42
96			22	31	40
102			21	29	36
108			19	27	34
114			17	25	32
120			16	24	30

(Continued)

(Sheet 2 of 3)

Table B-13. Suggested maximum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
		B-1 Load			
48	26	35	51	66	80
54	23	30	45	58	71
60	19	26	40	52	64
72		20	32	44	53
78		18	28	40	48
84			26	36	45
90			24	33	42
96			22	31	40
102			20	28	36
108			18	26	34
114			16	24	32
120			15	23	29
		B-52 Load			
48	26	35	51	66	80
54	22	29	45	58	71
60	19	26	40	52	64
72		20	32	44	53
78		18	28	40	48
84			26	36	45
90			24	33	42
96			21	30	40
102			20	28	36
108			18	26	34
114			16	24	31
120			15	23	29

(Sheet 3 of 3)

Table B-14. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			H-20 Load		
12	1.0	1.0	1.0	1.0	1.0
15	1.0	1.0	1.0	1.0	1.0
18	1.0	1.0	1.0	1.0	1.0
24	1.0	1.0	1.0	1.0	1.0
30	1.0	1.0	1.0	1.0	1.0
36	1.0	1.0	1.0	1.0	1.0
42		1.0	1.0	1.0	1.0
48			1.0	1.0	1.0
54			1.0	1.0	1.0
60				1.0	1.0
66					1.0
72					1.0
78					1.0
84					1.5
			15-K Load		
12	1.0	1.0	1.0	1.0	1.0
15	1.0	1.0	1.0	1.0	1.0
18	1.0	1.0	1.0	1.0	1.0
24	1.5	1.5	1.0	1.0	1.0
30	1.5	1.5	1.5	1.0	1.0
36	1.5	1.5	1.5	1.5	1.5
42		1.5	1.5	1.5	1.5
48			1.5	1.5	1.5
54			1.5	1.5	1.5
60				1.5	1.5
66					1.5
72					1.5
78					1.5
84					1.5

(Continued)

(Sheet 1 of 4)

Table B-14. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
		F-15 Load			
12	1.5	1.5	1.5	1.5	1.5
15	1.5	1.5	1.5	1.5	1.5
18	1.5	1.5	1.5	1.5	1.5
24	2.0	2.0	1.5	1.5	1.5
30	2.0	2.0	2.0	1.5	1.5
36	2.0	2.0	2.0	2.0	2.0
42		2.0	2.0	2.0	2.0
48			2.0	2.0	2.0
54			2.0	2.0	2.0
60				2.0	2.0
66					2.0
72					2.0
78					2.0
84					2.0
		C-130 Load			
12	1.5	1.5	1.5	1.0	1.0
15	1.5	1.5	1.5	1.5	1.5
18	1.5	1.5	1.5	1.5	1.5
24	2.0	1.5	1.5	1.5	1.5
30	2.0	2.0	1.5	1.5	1.5
36	2.0	2.0	2.0	2.0	1.5
42		2.0	2.0	2.0	2.0
48			2.0	2.0	2.0
54			2.0	2.0	2.0
60				2.0	2.0
66					2.0
72					2.0
78					2.0
84					2.0

(Continued)

(Sheet 2 of 4)

Table B-14. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			C-5A Load		
12	1.5	1.0	1.0	1.0	1.0
15	1.5	1.5	1.5	1.0	1.0
18	1.5	1.5	1.5	1.5	1.5
24	1.5	1.5	1.5	1.5	1.5
30	1.5	1.5	1.5	1.5	1.5
36	2.0	2.0	1.5	1.5	1.5
42		2.0	1.5	1.5	1.5
48			2.0	2.0	1.5
54			2.0	2.0	2.0
60				2.0	2.0
66					2.0
72					2.0
78					2.0
84					2.0
			C-141 Load		
12	1.5	1.5	1.5	1.5	1.5
15	1.5	1.5	1.5	1.5	1.5
18	2.0	1.5	1.5	1.5	1.5
24	2.0	2.0	2.0	1.5	1.5
30	2.0	2.0	2.0	2.0	2.0
36	2.5	2.0	2.0	2.0	2.0
42		2.5	2.0	2.0	2.0
48			2.5	2.0	2.0
54			2.5	2.5	2.0
60				2.5	2.5
66					2.5
72					2.5
78					2.5
84					2.5

(Continued)

(Sheet 3 of 4)

Table B-14. Suggested minimum cover requirements (feet) for 2⅔- by ½-inch corrugated aluminum pipe.- Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			B-1 Load		
12	2.0	1.5	1.5	1.5	1.5
15	2.0	2.0	2.0	1.5	1.5
18	2.0	2.0	2.0	2.0	2.0
24	2.5	2.0	2.0	2.0	2.0
30	2.5	2.5	2.0	2.0	2.0
36	2.5	2.5	2.5	2.0	2.0
42		3.0	2.5	2.5	2.5
48			2.5	2.5	2.5
54			3.0	2.5	2.5
60				2.5	2.5
66					2.5
72					3.0
78					3.0
84					3.0
			B-52 Load		
12	2.0	2.0	1.5	1.5	1.5
15	2.0	2.0	2.0	2.0	1.5
18	2.0	2.0	2.0	2.0	2.0
24	2.5	2.5	2.0	2.0	2.0
30	3.0	2.5	2.5	2.5	2.0
36	3.0	3.0	2.5	2.5	2.5
42		3.0	3.0	2.5	2.5
48			3.5	3.0	2.5
54			3.0	3.0	3.0
60				3.0	3.0
66					3.0
72					3.0
78					3.0
84					3.0

(Sheet 4 of 4)

Table B-15. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			H-20 Load		
30	1.0	1.0	1.0	1.0	
36	1.0	1.0	1.0	1.0	
42	1.5	1.5	1.0	1.0	
48	1.5	1.5	1.5	1.0	1.0
54	1.5	1.5	1.5	1.0	1.0
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72		1.5	1.5	1.5	1.5
78		1.5	1.5	1.5	1.5
84		1.5	1.5	1.5	1.5
90			1.5	1.5	1.5
96			1.5	1.5	1.5
102			1.5	1.5	1.5
108			1.5	1.5	1.5
114			1.5	1.5	1.5
120			1.5	1.5	1.5
			15-K Load		
30	1.5	1.5	1.5	1.5	
36	1.5	1.5	1.5	1.5	
42	1.5	1.5	1.5	1.5	
48	1.5	1.5	1.5	1.5	1.5
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
66	1.5	1.5	1.5	1.5	1.5
72		1.5	1.5	1.5	1.5
78		1.5	1.5	1.5	1.5
84		1.5	1.5	1.5	1.5
90			1.5	1.5	1.5
96			1.5	1.5	1.5
102			1.5	1.5	1.5
108			1.5	1.5	1.5
114			1.5	1.5	1.5
120			1.5	1.5	1.5

(Continued) (Sheet 1 of 4)

Table B-15. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			F-15 Load		
30	2.0	2.0	2.0	2.0	
36	2.0	2.0	2.0	2.0	
42	2.0	2.0	2.0	2.0	
48	2.0	2.0	2.0	2.0	2.0
54	2.0	2.0	2.0	2.0	2.0
60	2.5	2.0	2.0	2.0	2.0
66	2.5	2.5	2.0	2.0	2.0
72		2.5	2.5	2.0	2.0
78		2.5	2.5	2.0	2.0
84		2.5	2.5	2.5	2.0
90			2.5	2.5	2.0
96			2.5	2.5	2.5
102			2.5	2.5	2.5
108			2.5	2.5	2.5
114			2.5	2.5	2.5
120			2.5	2.5	2.5
			C-130 Load		
30	2.0	2.0	2.0	2.0	
36	2.0	2.0	2.0	2.0	
42	2.0	2.0	2.0	2.0	
48	2.0	2.0	2.0	2.0	2.0
54	2.5	2.5	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
66	2.5	2.5	2.5	2.0	2.0
72		2.5	2.5	2.5	2.0
78		2.5	2.5	2.5	2.5
84		2.5	2.5	2.5	2.5
90			2.5	2.5	2.5
96			2.5	2.5	2.5
102			2.5	2.5	2.5
108			2.5	2.5	2.5
114			3.0	2.5	2.5
120			3.0	2.5	2.5

(Continued)

(Sheet 2 of 4)

Table B-15. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			C-5A Load		
30	2.0	1.5	1.5	1.5	
36	2.0	2.0	2.0	1.5	
42	2.0	2.0	2.0	2.0	
48	2.0	2.0	2.0	2.0	2.0
54	2.0	2.0	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
66	2.5	2.5	2.0	2.0	2.0
72		2.5	2.5	2.0	2.0
78		2.5	2.5	2.0	2.0
84		3.0	2.5	2.5	2.0
90			2.5	2.5	2.0
96			2.5	2.5	2.5
102			3.0	2.5	2.5
108			3.0	2.5	2.5
114			3.0	2.5	2.5
120			3.0	3.0	2.5
			C-141 Load		
30	2.0	2.0	2.0	2.0	
36	2.5	2.5	2.0	2.0	
42	2.5	2.5	2.5	2.0	
48	3.0	2.5	2.5	2.5	2.0
54	3.0	3.0	2.5	2.5	2.5
60	3.0	3.0	2.5	2.5	2.5
66	3.0	3.0	3.0	2.5	2.5
72		3.0	3.0	2.5	2.5
78		3.0	3.0	3.0	2.5
84		3.5	3.0	3.0	3.0
90			3.0	3.0	3.0
96			3.0	3.0	3.0
102			3.5	3.0	3.0
108			3.5	3.0	3.0
114			3.5	3.0	3.0
120			3.5	3.0	3.0

(Continued) (Sheet 3 of 4)

Table B-15. Suggested minimum cover requirements (feet) for 3- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			B-1 Load		
30	2.5	2.5	2.5	2.0	
36	2.5	2.5	2.5	2.5	
42	3.0	3.0	2.5	2.5	
48	3.0	3.0	3.0	2.5	2.5
54	3.0	3.0	3.0	2.5	2.5
60	3.0	3.0	3.0	3.0	3.0
66	3.5	3.0	3.0	3.0	3.0
72		3.5	3.0	3.0	3.0
78		3.5	3.0	3.0	3.0
84		4.0	3.0	3.0	3.0
90			3.5	3.0	3.0
96			3.5	3.0	3.0
102			3.5	3.5	3.0
108			4.0	3.5	3.0
114			4.0	3.5	3.5
120			4.0	3.5	3.5
			B-52 Load		
30	3.0	2.5	2.5	2.5	
36	3.0	3.0	2.5	2.5	
42	3.0	3.0	3.0	3.0	
48	3.0	3.0	3.0	3.0	3.0
54	3.5	3.0	3.0	3.0	3.0
60	4.0	3.5	3.0	3.0	3.0
66	4.0	3.5	3.0	3.0	3.0
72		4.0	3.5	3.0	3.0
78		4.0	3.5	3.0	3.0
84		4.0	3.5	3.5	3.0
90			4.0	3.5	3.5
96			4.0	3.5	3.5
102			4.0	3.5	3.5
108			4.0	4.0	3.5
114			4.5	4.0	3.5
120			4.5	4.0	4.0

(Sheet 4 of 4)

Table B-16. Suggested minimum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			H-20 Load		
48	1.5	1.5	1.5	1.0	1.0
54	1.5	1.5	1.5	1.5	1.0
60	1.5	1.5	1.5	1.5	1.5
72		1.5	1.5	1.5	1.5
78		1.5	1.5	1.5	1.5
84			1.5	1.5	1.5
90			1.5	1.5	1.5
96			1.5	1.5	1.5
102			1.5	1.5	1.5
108			1.5	1.5	1.5
114			1.5	1.5	1.5
120			1.5	1.5	1.5
			15-K Load		
48	1.5	1.5	1.5	1.5	1.5
54	1.5	1.5	1.5	1.5	1.5
60	1.5	1.5	1.5	1.5	1.5
72		1.5	1.5	1.5	1.5
78		1.5	1.5	1.5	1.5
84			1.5	1.5	1.5
90			1.5	1.5	1.5
96			1.5	1.5	1.5
102			1.5	1.5	1.5
108			1.5	1.5	1.5
114			1.5	1.5	1.5
120			1.5	1.5	1.5

(Continued)

(Sheet 1 of 4)

Table B-16. Suggested minimum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			F-15 Load		
48	2.0	2.0	2.0	2.0	2.0
54	2.5	2.0	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
72		2.5	2.5	2.0	2.0
78		2.5	2.5	2.0	2.0
84			2.5	2.5	2.0
90			2.5	2.5	2.5
96			2.5	2.5	2.5
102			2.5	2.5	2.5
108			2.5	2.5	2.5
114			2.5	2.5	2.5
120			2.5	2.5	2.5
			C-130 Load		
48	2.5	2.0	2.0	2.0	2.0
54	2.5	2.5	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
72		2.5	2.5	2.5	2.0
78		2.5	2.5	2.5	2.5
84			2.5	2.5	2.5
90			2.5	2.5	2.5
96			2.5	2.5	2.5
102			2.5	2.5	2.5
108			3.0	2.5	2.5
114			3.0	2.5	2.5
120			3.0	2.5	2.5

(Continued)

(Sheet 2 of 4)

Table B-16. Suggested minimum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.- Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			C-5A Load		
48	2.0	2.0	2.0	2.0	2.0
54	2.5	2.0	2.0	2.0	2.0
60	2.5	2.5	2.0	2.0	2.0
72		2.5	2.5	2.0	2.0
78		3.0	2.5	2.0	2.0
84			2.5	2.5	2.0
90			2.5	2.5	2.5
96			3.0	2.5	2.5
102			3.0	2.5	2.5
108			3.0	2.5	2.5
114			3.0	3.0	2.5
120			3.0	3.0	2.5
			C-141 Load		
48	3.0	2.5	2.5	2.5	2.5
54	3.0	3.0	2.5	2.5	2.5
60	3.0	3.0	2.5	2.5	2.5
72		3.0	3.0	3.0	2.5
78		3.5	3.0	3.0	3.0
84			3.0	3.0	3.0
90			3.0	3.0	3.0
96			3.5	3.0	3.0
102			3.5	3.0	3.0
108			3.5	3.0	3.0
114			4.0	3.5	3.0
120			4.0	3.5	3.0

(Continued)

(Sheet 3 of 4)

Table B-16. Suggested minimum cover requirements (feet) for 6- by 1-inch corrugated aluminum pipe.—Continued

Pipe Diameter in.	16-Gage 0.060 in.	14-Gage 0.075 in.	12-Gage 0.105 in.	10-Gage 0.135 in.	8-Gage 0.164 in.
			B-1 Load		
48	3.0	3.0	3.0	2.5	2.5
54	3.0	3.0	3.0	3.0	2.5
60	3.5	3.0	3.0	3.0	3.0
72		3.5	3.0	3.0	3.0
78		4.0	3.0	3.0	3.0
84			3.5	3.0	3.0
90			3.5	3.0	3.0
96			3.5	3.5	3.0
102			4.0	3.5	3.0
108			4.0	3.5	3.5
114			4.0	3.5	3.5
120			4.5	4.0	3.5
			B-52 Load		
48	3.5	3.0	3.0	3.0	3.0
54	3.5	3.5	3.0	3.0	3.0
60	4.0	3.5	3.0	3.0	3.0
72		4.0	3.5	3.0	3.0
78		4.0	3.5	3.5	3.0
84			4.0	3.5	3.5
90			4.0	3.5	3.5
96			4.0	3.5	3.5
102			4.0	4.0	3.5
108			4.5	4.0	3.5
114			4.5	4.0	4.0
120			5.0	4.0	4.0

(Sheet 4 of 4)

Table B-17. Suggested cover requirements for corrugated polyethylene (PE) pipe.

Pipe Diameter in.	Minimum ft	Maximum ft
	H-20 Load	
12	1.0	9.6
15	1.0	9.7
18	1.0	10.0
24	1.0	10.3

Table B–18. Maximum feet of cover for reinforced-concrete culverts.

Pipe Diameter in.	Loads: H-20 highway, 15-K single wheel, F-15 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		11	14	20	25
15		11	14	24	25
18		11	14	29	30
21		12	17	34	33
24		12	20	35	35
27		12	23	33	40
30		12	23	32	45
33		12	23	30	45
36		12	24	30	45
42		12	24	30	45
48		13	25	27	40
54		13	25	27	35
60	10	13	25	27	35
66	10	14	25	27	35
72	11	14	25	27	35
78	12	16	25	27	
84	12	16	25	27	
90	12	16	25		
96	12	16	25		
102	12	16	25		
108	12	16	25		

(Continued)

(Sheet 1 of 3)

Table B-18. Maximum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Loads: H-20 highway, 15-K single wheel, F-15 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		10	13	20	25
15		10	13	24	25
18		10	13	28	30
21		10	16	34	33
24		10	19	35	35
27		10	22	33	40
30		10	22	32	45
33		10	22	29	45
36		10	23	29	45
42		10	23	29	45
48		10	23	29	40
54		10	23	26	33
60	--	11	23	26	33
66	--	12	23	26	33
72	--	13	23	26	33
78	10	14	23	26	
84	10	15	23	26	
90	10	15	23		
96	10	15	23		
102	10	15	23		
108	10	16	23		

(Continued)

(Sheet 2 of 3)

Table B-18. Maximum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Loads: H-20 highway, 15-K single wheel, F-15 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		--	--	16	22
15		--	--	20	25
18		--	--	25	28
21		--	20	32	31
24		--	20	34	34
27		--	20	32	40
30		--	20	31	45
33		--	20	27	45
36		--	20	26	45
42		--	20	26	45
48		--	20	26	40
54		--	20	26	32
60	--	--	20	25	32
66	--	--	20	25	32
72	--	--	20	25	32
78	--	--	20	25	
84	--	--	20	25	
90	--	--	20		
96	--	--	20		
102	--	--	20		
108	--	--	20		

(Sheet 3 of 3)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.

Pipe Diameter in.	Load: H-20 highway				
	Class I	Class II	Class III	Class IV	Class V
12		2.5	2.5	2.0	2.0
15		2.5	2.5	2.0	2.0
18		2.5	2.5	2.0	2.0
21		2.5	2.0	2.0	2.0
24		2.5	2.0	1.5	1.5
27		2.5	2.0	1.5	1.0
30		2.5	2.0	1.5	1.0
33		2.5	2.0	1.5	1.0
36		2.5	2.0	1.5	1.0
42		2.5	2.0	1.5	1.0
48		2.5	1.5	1.0	1.0
54		2.0	1.5	1.0	1.0
60	2.0	2.0	1.5	1.0	1.0
66	2.0	1.5	1.0	1.0	1.0
72	1.5	1.5	1.0	1.0	1.0
78	1.5	1.0	1.0	1.0	
84	1.0	1.0	1.0	1.0	
90	1.0	1.0	1.0		
96	1.0	1.0	1.0		
102	1.0	1.0	1.0		
108	1.0	1.0	1.0		

(Continued)

(Sheet 1 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: 15-K single wheel				
	Class I	Class II	Class III	Class IV	Class V
12		2.5	2.5	2.0	2.0
15		2.5	2.5	2.0	2.0
18		2.5	2.5	2.0	2.0
21		2.5	2.0	2.0	2.0
24		2.5	2.0	1.5	1.5
27		2.5	2.0	1.5	1.0
30		2.5	2.0	1.5	1.0
33		2.5	2.0	1.5	1.0
36		2.5	2.0	1.5	1.0
42		2.5	2.0	1.5	1.0
48		2.5	1.5	1.0	1.0
54		2.0	1.5	1.0	1.0
60	2.0	2.0	1.5	1.0	1.0
66	2.0	1.5	1.0	1.0	1.0
72	1.5	1.5	1.0	1.0	1.0
78	1.5	1.0	1.0	1.0	
84	1.0	1.0	1.0	1.0	
90	1.0	1.0	1.0		
96	1.0	1.0	1.0		
102	1.0	1.0	1.0		
108	1.0	1.0	1.0		

(Continued)

(Sheet 2 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: F-15 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		4.0	4.0	3.0	2.5
15		4.0	4.0	3.0	2.5
18		4.0	4.0	3.0	2.5
21		4.0	3.0	2.5	2.5
24		4.0	3.0	2.5	2.5
27		4.0	3.0	2.5	2.0
30		4.0	3.0	2.5	2.0
33		4.0	3.0	2.5	2.0
36		4.0	3.0	2.5	2.0
42		4.0	3.0	2.5	2.0
48		4.0	3.0	2.0	2.0
54		3.5	2.5	2.0	2.0
60	4.0	3.5	2.5	2.0	1.5
66	3.5	3.0	2.5	2.0	1.5
72	3.5	3.0	2.0	2.0	1.5
78	3.0	2.5	2.0	2.0	
84	3.0	2.5	2.0	1.5	
90	2.5	2.5	2.0		
96	2.5	2.5	1.5		
102	2.5	2.0	1.5		
108	2.5	2.0	1.5		

(Continued)

(Sheet 3 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-130 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		5.0	4.5	3.5	3.0
15		5.0	4.5	3.0	3.0
18		5.0	4.5	3.0	3.0
21		5.0	3.5	3.0	3.0
24		5.0	3.5	2.5	2.5
27		5.0	3.5	2.5	2.0
30		5.0	3.5	2.5	2.0
33		5.0	3.5	2.5	2.0
36		5.0	3.5	2.5	2.0
42		5.0	3.5	2.5	2.0
48		5.0	3.0	2.5	2.0
54		5.0	3.0	2.0	2.0
60	--	4.5	3.0	2.0	1.5
66	--	4.0	2.5	2.0	1.0
72	--	3.5	2.5	1.5	1.0
78	4.0	3.0	2.5	1.5	
84	3.5	3.0	2.0	1.0	
90	3.5	2.5	2.0		
96	3.0	2.5	1.5		
102	3.0	2.0	1.0		
108	2.5	2.0	1.0		

(Continued)

(Sheet 4 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-5A aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		--	--	7.0	4.5
15		--	--	6.0	4.5
18		--	--	5.0	4.0
21		--	7.5	4.0	3.5
24		--	7.5	4.0	3.5
27		--	7.5	4.0	2.5
30		--	7.5	4.0	2.0
33		--	7.5	4.0	1.5
36		--	7.0	4.0	1.5
42		--	7.0	4.0	1.5
48		--	7.0	3.0	1.5
54		--	6.5	2.5	1.5
60	--	--	6.0	2.0	1.0
66	--	--	6.0	1.5	1.0
72	--	--	4.5	1.0	1.0
78	--	--	4.0	1.0	
84	--	--	3.5	1.0	
90	--	--	2.5		
96	--	--	1.5		
102	--	--	1.0		
108	--	--	1.0		

(Continued)

(Sheet 5 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-141 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		--	--	6.0	5.0
15		--	--	5.5	4.5
18		--	--	5.0	4.0
21		--	7.0	4.5	4.0
24		--	7.0	4.5	4.0
27		--	7.0	4.5	3.0
30		--	7.0	4.5	3.0
33		--	7.0	4.5	3.0
36		--	7.0	4.5	3.0
42		--	7.0	4.5	3.0
48		--	6.5	4.0	3.0
54		--	6.0	3.5	3.0
60	--	--	6.0	3.5	2.5
66	--	--	5.5	3.0	2.0
72	--	--	5.0	3.0	1.5
78	--	--	5.0	3.0	
84	--	--	4.5	2.0	
90	--	--	3.5		
96	--	--	2.5		
102	--	--	2.0		
108	--	--	1.5		

(Continued)

(Sheet 6 of 8)

Table B-19. Minimum feet of cover for reinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-1 aircraft Class I	Class II	Class III	Class IV	Class V
12		--	--	7.5	5.5
15		--	--	6.5	5.5
18		--	--	5.5	5.0
21		--	8.0	5.0	4.5
24		--	8.0	5.0	4.5
27		--	8.0	5.0	3.5
30		--	8.0	5.0	3.5
33		--	8.0	5.0	3.5
36		--	8.0	5.0	3.5
42		--	8.0	5.0	3.0
48		--	7.5	4.5	3.0
54		--	7.5	4.0	3.0
60	--	--	6.5	4.0	3.0
66	--	--	6.5	3.5	2.5
72	--	--	6.5	3.0	2.0
78	--	--	6.5	3.0	
84	--	--	5.5	3.0	
90	--	--	5.0		
96	--	--	3.5		
102	--	--	3.0		
108	--	--	2.0		

(Continued)

(Sheet 7 of 8)

Table B-19. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-52 aircraft				
	Class I	Class II	Class III	Class IV	Class V
12		--	--	6.5	5.5
15		--	--	6.0	5.0
18		--	--	5.5	5.0
21		--	7.0	5.0	4.5
24		--	7.0	5.0	4.5
27		--	7.0	5.0	4.0
30		--	7.0	5.0	4.0
33		--	7.0	5.0	4.0
36		--	7.0	5.0	4.0
42		--	7.0	5.0	3.5
48		--	7.0	5.0	3.5
54		--	6.5	4.5	3.5
60	--	--	6.0	4.5	3.5
66	--	--	6.0	4.0	3.0
72	--	--	6.0	4.0	2.5
78	--	--	6.0	4.0	
84	--	--	5.5	4.0	
90	--	--	5.0		
96	--	--	4.5		
102	--	--	4.0		
108	--	--	3.0		

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Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.

Pipe Diameter in.	Load: H-20 highway		
	Class I	Class II	Class III
4	28	36	43
6	20	26	31
8	15	20	23
10	13	16	19
12	12	15	16
15	11	14	15
18	10	13	14
21	9	12	14
24	8	12	14
27	8	11	13
30	7	11	12
33	7	10	11
36	7	9	11

(Continued)

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Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: 15-K single wheel Class I	Class II	Class III
4	28	36	43
6	20	26	31
8	15	20	23
10	13	16	19
12	12	15	16
15	11	14	15
18	10	13	14
21	9	13	14
24	9	12	14
27	8	11	13
30	8	11	12
33	7	10	12
36	7	10	11

(Continued)

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Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: F-15 aircraft		
	Class I	Class II	Class III
4	28	36	43
6	20	26	31
8	15	20	23
10	13	16	19
12	12	15	16
15	11	14	15
18	10	13	14
21	9	12	14
24	8	11	14
27	7	11	13
30	7	11	12
33	6	10	11
36	6	9	11

(Continued)

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Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-130 aircraft		
	Class I	Class II	Class III
4	28	36	43
6	20	26	30
8	15	20	23
10	12	16	18
12	11	14	16
15	10	13	14
18	8	12	14
21	7	12	14
24	6	11	13
27	--	10	12
30	--	10	11
33	--	9	10
36	--	8	10

(Continued)

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Table B–20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-5A aircraft		
	Class I	Class II	Class III
4	27	36	43
6	19	25	30
8	12	18	22
10	9	14	17
12	--	12	14
15	--	10	12
18	--	9	11
21	--	8	11
24	--	--	10
27	--	--	4
30	--	--	--
33	--	--	--
36	--	--	--

(Continued)

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Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-141 aircraft		
	Class I	Class II	Class III
4	28	36	43
6	19	26	30
8	13	19	22
10	10	15	18
12	8	13	15
15	--	11	13
18	--	11	12
21	--	9	12
24	--	--	11
27	--	--	10
30	--	--	9
33	--	--	--
36	--	--	--

(Continued)

(Sheet 6 of 8)

Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-1 aircraft Class I	Class II	Class III
4	27	36	43
6	19	26	30
8	13	19	22
10	--	14	17
12	--	12	14
15	--	10	12
18	--	9	11
21	--	--	11
24	--	--	10
27	--	--	8
30	--	--	--
33	--	--	--
36	--	--	--

(Continued)

(Sheet 7 of 8)

Table B-20. Maximum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-52 aircraft		
	Class I	Class II	Class III
4	27	36	43
6	19	25	30
8	13	19	22
10	10	14	17
12	8	13	15
15	--	11	13
18	--	10	12
21	--	9	12
24	--	--	11
27	--	--	10
30	--	--	8
33	--	--	--
36	--	--	--

(Sheet 8 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.

Pipe Diameter in.	Load: H-20 highway		
	Class I	Class II	Class III
4	1.0	1.0	1.0
6	1.0	1.0	1.0
8	1.5	1.0	1.0
10	1.5	1.5	1.0
12	1.5	1.5	1.0
15	2.0	1.5	1.5
18	2.0	1.5	1.5
21	2.0	1.5	1.5
24	2.0	1.5	1.5
27	2.0	1.5	1.5
30	2.0	1.5	1.5
33	2.0	1.5	1.5
36	2.0	1.5	1.5

(Continued)

(Sheet 1 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: 15-k single wheel		
	Class I	Class II	Class III
4	1.0	1.0	1.0
6	1.5	1.0	1.0
8	1.5	1.5	1.0
10	2.0	1.5	1.5
12	2.0	1.5	1.5
15	2.0	1.5	1.5
18	2.0	1.5	1.5
21	2.0	1.5	1.5
24	2.0	1.5	1.5
27	2.0	1.5	1.5
30	2.0	1.5	1.5
33	2.5	2.0	1.5
36	2.5	2.0	1.5

(Continued)

(Sheet 2 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: F-15 aircraft		
	Class I	Class II	Class III
4	2.0	1.5	1.5
6	2.0	2.0	1.5
8	2.5	2.0	2.0
10	2.5	2.5	2.0
12	3.0	2.5	2.5
15	3.0	2.5	2.5
18	3.0	2.5	2.5
21	3.5	2.5	2.5
24	3.5	2.5	2.5
27	3.5	3.0	2.5
30	4.0	3.0	2.5
33	4.0	3.0	2.5
36	4.0	3.0	2.5

(Continued)

(Sheet 3 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-130 aircraft		
	Class I	Class II	Class III
4	2.0	1.5	1.0
6	2.0	2.0	1.5
8	2.5	2.0	2.0
10	3.0	2.5	2.5
12	3.0	2.5	2.5
15	3.5	3.0	2.5
18	3.5	3.0	2.5
21	4.0	3.0	2.5
24	4.5	3.0	2.5
27	--	3.0	2.5
30	--	3.0	3.0
33	--	3.5	3.0
36	--	3.5	3.0

(Continued)

(Sheet 4 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-5A aircraft		
	Class I	Class II	Class III
4	1.5	1.0	1.0
6	2.0	1.5	1.5
8	3.0	2.0	1.5
10	6.0	2.5	2.0
12	--	3.0	2.5
15	--	3.5	2.5
18	--	5.5	3.0
21	--	6.5	3.0
24	--	--	3.0
27	--	--	3.5
30	--	--	--
33	--	--	--
36	--	--	--

(Continued)

(Sheet 5 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: C-141 aircraft		
	Class I	Class II	Class III
4	2.0	1.5	1.5
6	2.5	2.0	2.0
8	4.0	2.5	2.5
10	5.5	3.5	3.0
12	6.5	4.0	3.5
15	--	4.5	3.5
18	--	5.5	4.0
21	--	6.0	4.0
24	--	--	4.0
27	--	--	5.0
30	--	--	6.0
33	--	--	--
36	--	--	--

(Continued)

(Sheet 6 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-1 aircraft		
	Class I	Class II	Class III
4	2.5	2.0	2.0
6	3.0	2.5	2.0
8	4.5	3.0	2.5
10	--	4.0	3.5
12	--	4.5	4.0
15	--	6.0	4.5
18	--	6.5	5.0
21	--	--	5.0
24	--	--	5.5
27	--	--	6.5
30	--	--	--
33	--	--	--
36	--	--	--

(Continued)

(Sheet 7 of 8)

Table B-21. Minimum feet of cover for nonreinforced-concrete culverts.—Continued

Pipe Diameter in.	Load: B-52 aircraft		
	Class I	Class II	Class III
4	2.5	2.5	2.0
6	3.5	3.0	2.5
8	5.0	3.5	3.0
10	6.0	4.5	3.5
12	7.0	5.0	4.5
15	--	5.5	4.5
18	--	5.5	5.0
21	--	6.5	5.0
24	--	--	5.5
27	--	--	6.0
30	--	--	6.5
33	--	--	--
36	--	--	--

(Sheet 8 of 8)

Table B-22. Maximum feet of cover for vitrified clay culverts.

Pipe Diameter in.	Load: H-20 highway, F-15 aircraft Extra Strength	Standard Strength
4	38	23
6	27	16
8	22	14
10	20	13
12	18	12
15	16	11
18	15	10
21	15	9
24	15	8
27	14	8
30	14	8
33	14	8
36	14	8
39	14	
42	14	

(Continued)

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Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: 15-K single wheel	
	Extra Strength	Standard Strength
4	38	24
6	27	16
8	22	14
10	20	13
12	18	12
15	16	11
18	15	10
21	15	9
24	15	9
27	14	8
30	14	8
33	14	8
36	14	8
39	14	
42	14	

(Continued)

(Sheet 2 of 7)

Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-130 aircraft	
	Extra Strength	Standard Strength
4	38	23
6	27	15
8	22	13
10	19	12
12	17	11
15	15	10
18	14	8
21	14	7
24	14	7
27	14	6
30	13	6
33	13	6
36	13	6
39	13	
42	13	

(Continued)

(Sheet 3 of 7)

Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-5A aircraft Extra Strength	Standard Strength
4	37	22
6	25	13
8	21	11
10	18	9
12	16	--
15	13	--
18	12	--
21	12	--
24	12	
27	11	
30	10	--
33	10	--
36	10	--
39	10	--
42	10	

(Continued)

(Sheet 4 of 7)

Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-141 aircraft	
	Extra Strength	Standard Strength
4	38	23
6	26	14
8	22	11
10	19	--
12	16	--
15	14	--
18	13	--
21	13	--
24	13	--
27	12	--
30	11	--
33	11	--
36	11	--
39	11	
42	11	

(Continued)

(Sheet 5 of 7)

Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: B-1 aircraft Extra Strength	Standard Strength
4	38	23
6	26	14
8	21	11
10	18	--
12	16	--
15	13	--
18	12	--
21	12	--
24	12	--
27	11	--
30	10	--
33	10	--
36	10	--
39	10	
42	10	

(Continued)

(Sheet 6 of 7)

Table B-22. Maximum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: B-52 aircraft Extra Strength	Standard Strength
4	38	22
6	26	14
8	21	12
10	18	10
12	16	8
15	14	--
18	13	--
21	13	--
24	13	--
27	12	--
30	11	--
33	11	--
36	11	--
39	11	
42	11	

(Sheet 7 of 7)

Table B-23. Minimum feet of cover for vitrified clay culverts.

Pipe Diameter in.	Load: H-20 highway	
	Extra Strength	Standard Strength
4	1.0	1.0
6	1.0	1.5
8	1.0	1.5
10	1.0	1.5
12	1.0	1.5
15	1.0	2.0
18	1.0	2.0
21	1.0	2.0
24	1.0	2.0
27	1.0	2.0
30	1.0	2.0
33	1.0	2.0
36	1.0	2.0
39	1.0	
42	1.0	

(Continued)

(Sheet 1 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: 15-K single wheel Extra Strength	Standard Strength
4	1.0	1.5
6	1.0	1.5
8	1.5	1.5
10	1.5	2.0
12	1.5	2.0
15	1.5	2.0
18	1.5	2.0
21	1.5	2.0
24	1.5	2.0
27	1.5	2.0
30	1.5	2.0
33	1.5	2.0
36	1.5	2.0
39	1.5	
42	1.5	

(Continued)

(Sheet 2 of 8)

Table B–23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: F-15 aircraft	
	Extra Strength	Standard Strength
4	1.5	2.0
6	2.0	2.5
8	2.0	2.5
10	2.0	2.5
12	2.0	3.0
15	2.5	3.0
18	2.5	3.0
21	2.5	3.5
24	2.5	3.5
27	2.5	3.5
30	2.5	3.5
33	2.5	3.5
36	2.5	3.5
39	2.5	
42	2.5	

(Continued)

(Sheet 3 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-130 aircraft	
	Extra Strength	Standard Strength
4	1.5	2.0
6	2.0	2.5
8	2.0	3.0
10	2.0	3.0
12	2.5	3.0
15	2.5	3.5
18	2.5	3.5
21	2.5	4.0
24	2.5	4.0
27	2.5	4.5
30	2.5	4.5
33	2.5	4.5
36	2.5	4.5
39	2.5	
42	2.5	

(Continued)

(Sheet 4 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-5A aircraft Extra Strength	Standard Strength
4	1.0	2.0
6	1.5	3.0
8	2.0	3.5
10	2.0	6.0
12	2.0	--
15	2.5	--
18	2.5	--
21	2.5	--
24	2.5	--
27	2.5	--
30	3.0	--
33	3.0	--
36	3.0	--
39	3.0	
42	3.0	

(Continued)

(Sheet 5 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: C-141 aircraft Extra Strength	Standard Strength
4	1.5	2.5
6	2.0	3.5
8	2.5	4.5
10	2.5	5.5
12	3.0	6.5
15	3.5	--
18	3.5	--
21	3.5	--
24	3.5	--
27	3.5	--
30	4.0	--
33	4.0	--
36	4.0	--
39	4.0	
42	4.0	

(Continued)

(Sheet 6 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: B-1 aircraft	
	Extra Strength	Standard Strength
4	2.0	2.5
6	2.5	4.0
8	3.0	5.5
10	3.0	--
12	3.5	--
15	4.0	--
18	4.5	--
21	4.5	--
24	4.5	--
27	4.5	--
30	5.5	--
33	5.5	--
36	5.5	--
39	5.5	
42	5.5	

(Continued)

(Sheet 7 of 8)

Table B-23. Minimum feet of cover for vitrified clay culverts.—Continued

Pipe Diameter in.	Load: B-52 aircraft Extra Strength	Standard Strength
4	2.0	3.0
6	3.0	4.5
8	3.0	5.5
10	3.5	6.0
12	4.0	7.0
15	4.5	--
18	5.0	--
21	5.0	--
24	5.0	--
27	5.0	--
30	5.0	--
33	5.0	--
36	5.0	--
39	5.0	
42	5.0	

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APPENDIX C

OUTLET PROTECTION DESIGN PROBLEM

C-1. This appendix contains examples of recommended application to estimate the extent of scour in a cohesionless soil and alternative schemes of protection required to prevent local scour.

C-2. Circular and rectangular outlets with equivalent cross-sectional areas that will be subjected to a range of discharges for a duration of 1 hour are used with the following parameters:

Dimensions of rectangular outlet = W_o = 10 feet, D_o = 5 feet

Diameter of circular outlet, D_o = 8 feet

Range of discharge, Q = 362 to 1,086 cubic feet per second

Discharge parameter for rectangular culvert, $q/D_o^{3/2}$ = 3.2 to 9.7

Discharge parameter for circular culvert, $Q/D_o^{5/2}$ = 2 to 6

Duration of runoff event, t = 60 minutes

Maximum tailwater el = 6.4 feet above outlet invert (> 0.5 D_o)

Minimum tailwater el = 2.0 feet above outlet invert (< 0.5 D_o)

Example 1 - Determine maximum depth of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see fig 5-4)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80 \left(\frac{q}{D_o^{3/2}}\right)^{0.375} t^{0.10} \qquad \text{(eq C-1)}$$

$$D_{sm} = 0.80\ (3.2 \text{ to } 9.7)^{0.375}\ (60)^{0.1}\ (5) \qquad \text{(eq C-2)}$$

$$= \underline{\underline{9.3 \text{ ft}}} \text{ to } \underline{\underline{14.0 \text{ ft}}}$$

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left(\frac{q}{D_o^{3/2}}\right)^{0.375} t^{0.10} \qquad \text{(eq C-3)}$$

$$D_{sm} = 0.74\ (3.2 \text{ to } 9.7)^{0.375}\ (60)^{0.1}\ (5) \qquad \text{(eq C-4)}$$

$$= \underline{\underline{8.6 \text{ ft}}} \text{ to } \underline{\underline{13.0 \text{ ft}}}$$

CIRCULAR CULVERT (see fig 5-4)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80 \left(\frac{Q}{D_o^{5/2}}\right)^{0.375} t^{0.10} \qquad \text{(eq C-5)}$$

$$D_{sm} = 0.80\ (2 \text{ to } 6)^{0.375}\ (60)^{0.1}\ (8) \qquad \text{(eq C-6)}$$

$$= \underline{\underline{12.5 \text{ ft}}} \text{ to } \underline{\underline{18.9 \text{ ft}}}$$

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left(\frac{Q}{D_o^{5/2}}\right)^{0.375} t^{0.1} \qquad \text{(eq C-7)}$$

$$D_{sm} = 0.74\ (2 \text{ to } 6)^{0.375}\ (60)^{0.1}\ (8) \qquad \text{(eq C-8)}$$

$$= \underline{\underline{11.6 \text{ ft}}} \text{ to } \underline{\underline{17.5 \text{ ft}}}$$

Example 2 - Determine maximum width of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see fig 5-5)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left(\frac{q}{D_o^{3/2}}\right)^{0.915} t^{0.15} \qquad \text{(eq C-9)}$$

$$W_{sm} = 1.00\ (3.2 \text{ to } 9.7)^{0.915}\ (60)^{0.15} (5) \qquad \text{(eq C-10)}$$

$$= \underline{\underline{27 \text{ ft}}} \text{ to } \underline{\underline{74 \text{ ft}}}$$

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (27 \text{ to } 74) + \frac{10}{2} - \frac{5}{2} \qquad \text{(eq C-11)}$$

$$= \underline{\underline{29.5 \text{ ft}}} \text{ to } \underline{\underline{76.5 \text{ ft}}}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72 \left(\frac{q}{D_o^{3/2}}\right)^{0.915} t^{0.15} \qquad \text{(eq C-12)}$$

$$W_{sm} = 0.72 \ (3.2 \text{ to } 9.7)^{0.915} \ (60)^{0.015} \qquad \text{(eq C-13)}$$

$$= \underline{\underline{19 \text{ ft}}} \text{ to } \underline{\underline{53 \text{ ft}}}$$

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (19 \text{ to } 53) + \frac{10}{2} - \frac{5}{2} \qquad \text{(eq C-14)}$$

$$= \underline{\underline{21.5 \text{ ft}}} \text{ to } \underline{\underline{55.5 \text{ ft}}}$$

CIRCULAR CULVERT (see fig 5-5)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left(\frac{Q}{D_o^{5/2}}\right)^{0.915} t^{0.15} \qquad \text{(eq C-15)}$$

$$W_{sm} = 1.00 \ (2 \text{ to } 6)^{0.915} \ (60)^{0.15} \ (8) \qquad \text{(eq C-16)}$$

$$= \underline{\underline{28 \text{ ft}}} \text{ to } \underline{\underline{76 \text{ ft}}}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72 \left(\frac{Q}{D_o^{5/2}}\right)^{0.915} t^{0.15} \qquad \text{(eq C-17)}$$

$$W_{sm} = 0.72 \ (2 \text{ to } 6)^{0.915} \ (60)^{0.15} \ (8) \qquad \text{(eq C-18)}$$

$$= \underline{\underline{20 \text{ ft}}} \text{ to } \underline{\underline{55 \text{ ft}}}$$

Example 3 - Determine maximum length of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see fig 5-6)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40 \left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq C-19)}$$

$$L_{sm} = 2.4 \ (3.2 \text{ to } 9.7)^{0.71} \ (60)^{0.125} \ (5) \qquad \text{(eq C-20)}$$

$$= \underline{\underline{46 \text{ ft}}} \text{ to } \underline{\underline{101 \text{ ft}}}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10 \left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq C-21)}$$

$$L_{sm} = 4.10\ (3.2\ \text{to}\ 9.7)^{0.71}\ (60)^{0.125}\ (5) \qquad \text{(eq C-22)}$$

$$= \underline{\underline{78\ \text{ft}}}\ \text{to}\ \underline{\underline{171\ \text{ft}}}$$

CIRCULAR CULVERT (see fig 5-6)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40 \left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq C-23)}$$

$$L_{sm} = 2.4\ (2\ \text{to}\ 6)^{0.71}\ (60)^{0.125}\ (8) \qquad \text{(eq C-24)}$$

$$= \underline{\underline{52\ \text{ft}}}\ \text{to}\ \underline{\underline{114\ \text{ft}}}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10 \left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125} \qquad \text{(eq C-25)}$$

$$L_{sm} = 4.10\ (2\ \text{to}\ 6)^{0.71}\ (60)^{0.125}\ (8) \qquad \text{(eq C-26)}$$

$$= \underline{\underline{90\ \text{ft}}}\ \text{to}\ \underline{\underline{195\ \text{ft}}}$$

Example 4 - Determine profile and cross section of scour for maximum discharge and minimum tailwater conditions (see fig 5-8):

CIRCULAR CULVERT

For L_{sm} = 114 ft and D_{sm} = 18.9 ft

L_s/L_{sm}	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
L	0.0	11.4	22.8	34.2	45.6	57.0	68.4	79.8	91.2	102.6	114.0
D_s/D_{sm}	0.7	0.75	0.85	0.95	1.0	0.95	0.75	0.55	0.33	0.15	0.0
D_s	13.2	14.2	16.1	18.0	18.9	18.0	14.2	10.4	6.3	2.9	0.0

For W_{sm} = 76 ft and D_{sm} = 18.9 ft

W_s/W_{sm}	0.0	0.2	0.4	0.6	0.8	1.0
W_s	0.0	15.2	30.4	45.6	60.8	76.0
D_s/D_{sm}	1.0	0.67	0.27	0.15	0.05	0.0
D_s	18.9	12.6	5.1	2.8	0.95	0.0

(Continued)

Example 4 - (Concluded)

RECTANGULAR CULVERT

For L_{sm} = 101 ft and D_{sm} = 14.0 ft

L_s/L_{sm}	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
L	0.0	10.1	20.2	30.3	40.4	50.5	60.6	70.7	80.8	90.9	101.0
D_s/D_{sm}	0.7	0.75	0.85	0.95	1.0	0.95	0.75	0.55	0.33	0.15	0.0
D_s	9.8	10.5	11.9	13.3	14.0	13.3	10.5	7.7	4.6	2.1	0.0

For W_{sm} = 74 ft and D_{sm} = 14.0 ft

W_s/W_{sm}	0.0	0.2	0.4	0.6	0.8	1.0
W_s	0.0	14.8	29.6	44.4	59.2	74.0
D_s/D_{sm}	1.0	0.67	0.27	0.15	0.05	0.0
D_s	14.0	9.38	3.78	2.10	0.70	0.0
$W_{sr} = W_s + \frac{W_o}{2} - \frac{D_o}{2}$	0–2.5	17.3	32.1	46.9	61.7	76.5

Example 5 - Determine depth and width of cutoff wall:

RECTANGULAR CULVERT, Maximum depth and width of scour = 14 ft and 76.5 ft

From figure 5-8, depth of cutoff wall = 0.7 (D_{sm}) = 0.7 (14) = $\underline{\underline{9.8 \text{ ft}}}$

From figure 5-8, width of cutoff wall = 2 (W_{smr}) = 2 (76.5) = $\underline{\underline{153 \text{ ft}}}$

CIRCULAR CULVERT, Maximum depth and width of scour = 18.9 ft and 76.0 ft

From figure 5-8, depth of cutoff wall = 0.7 (D_{sm}) = 0.7 (18.9) = $\underline{\underline{13.2 \text{ ft}}}$

From figure 5-8, width of cutoff wall = 2 (W_{sm}) = 2(76) = $\underline{\underline{152 \text{ ft}}}$

Note: The depth of cutoff wall may be varied with width in accordance with the cross section of the scour hole at the location of the maximum depth of scour. See figures 5-8 and 5-9 of main text.

Example 6 - Determine size and extent of horizontal blanket of riprap:

RECTANGULAR CULVERT

MINIMUM TAILWATER

From figure 5-10, $$\frac{d_{50}}{D_o} = 0.020 \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq C-27)}$$

$$d_{50} = 0.020\,(5/2)(3.2 \text{ to } 9.7)^{4/3}\,(5) \qquad \text{(eq C-28)}$$

$$= \underline{\underline{1.2 \text{ ft}}} \text{ to } \underline{\underline{5.2 \text{ ft}}}$$

From figure 5-11, $$\frac{L_{sp}}{D_o} = 1.8\left(\frac{q}{D_o^{3/2}}\right) + 7 \qquad \text{(eq C-29)}$$

$$L_{sp} = [1.8(3.2 \text{ to } 9.7) + 7]\ 5 = \underline{\underline{64 \text{ ft}}} \text{ to } \underline{\underline{122 \text{ ft}}} \qquad \text{(eq C-30)}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq C-31)}$$

$$\begin{aligned} d_{50} &= 0.020\ (5/6.4)(3.2 \text{ to } 9.7)^{4/3}(5) \\ &= \underline{\underline{0.37 \text{ ft}}} \text{ to } \underline{\underline{0.76 \text{ ft}}} \end{aligned} \qquad \text{(eq C-32)}$$

$$\frac{L_{sp}}{D_o} = 3\left(\frac{q}{D_o^{3/2}}\right) \qquad \text{(eq C-33)}$$

$$L_{sp} = 3\ (3.2 \text{ to } 9.7)\ 5 = \underline{\underline{48 \text{ ft}}} \text{ to } \underline{\underline{145 \text{ ft}}} \qquad \text{(eq C-34)}$$

CIRCULAR CULVERT

MINIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq C-35)}$$

$$d_{50} = 0.020\ (8/2)(2 \text{ to } 6)^{4/3}\ (8) \qquad \text{(eq C-36)}$$

$$= \underline{\underline{1.6 \text{ ft}}} \text{ to } \underline{\underline{7.0 \text{ ft}}}$$

$$\frac{L_{sp}}{D_o} = 1.8 \left(\frac{Q}{D_o^{5/2}}\right) + 7 \qquad \text{(eq C-37)}$$

$$L_{sp} = 1.8\ (2 \text{ to } 6) + 7 \quad 8 = \underline{\underline{85 \text{ ft}}} \text{ to } \underline{\underline{142 \text{ ft}}} \qquad \text{(eq C-38)}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq C-39)}$$

$$d_{50} = 0.020\ (8/6.4)(2 \text{ to } 6)^{4/3}\ (8) \qquad \text{(eq C-40)}$$

$$= \underline{\underline{0.50 \text{ ft}}} \text{ to } \underline{\underline{2.18 \text{ ft}}}$$

$$\frac{L_{sp}}{D_o} = 3 \left(\frac{Q}{D_o^{5/2}}\right) \qquad \text{(eq C-41)}$$

$$L_{sp} = 3\ (2 \text{ to } 6)\ 8 = \underline{\underline{48 \text{ ft}}} \text{ to } \underline{\underline{144 \text{ ft}}} \qquad \text{(eq C-42)}$$

Use figure 5-12 to determine recommended configuration of horizontal blanket of riprap subject to minimum and maximum tailwaters.

Example 7 - Determine size and geometry of riprap-lined preformed scour holes 0.5- and 1.0-D_o deep for minimum tailwater conditions:

RECTANGULAR CULVERT (see fig 5-10)

0.5-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125 \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq C-43)}$$

$$d_{50} = 0.0125\ (5/2)(3.2 \text{ to } 9.7)^{4/3}\ (5) \qquad \text{(eq C-44)}$$

$$= \underline{\underline{0.73 \text{ ft}}} \text{ to } \underline{\underline{3.2 \text{ ft}}}$$

1.0-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082 \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq C-45)}$$

$$d_{50} = 0.0082\ (5/2)(3.2 \text{ to } 9.7)^{4/3}(5) \qquad \text{(eq C-46)}$$

$$= \underline{\underline{0.48 \text{ ft}}} \text{ to } \underline{\underline{2.1 \text{ ft}}}$$

CIRCULAR CULVERT

0.5-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125 \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq C-47)}$$

$$d_{50} = 0.0125\ (8/2)(2 \text{ to } 6)^{4/3}\ (8) \qquad \text{(eq C-48)}$$

$$= \underline{\underline{1.0 \text{ ft}}} \text{ to } \underline{\underline{4.4 \text{ ft}}}$$

1.0-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082\ \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq C-49)}$$

$$d_{50} = 0.0082\ (8/2)(2 \text{ to } 6)^{4/3}\ (8) \qquad \text{(eq C-50)}$$

$$= \underline{\underline{0.66 \text{ ft}}} \text{ to } \underline{\underline{2.9 \text{ ft}}}$$

See figure 5-13 for geometry.

Example 8 - Determine size and geometry of riprap-lined-channel expansion for minimum tailwaters (see fig 5-15)

RECTANGULAR CULVERT

$$\frac{d_{50}}{D_o} = 0.016\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3} \qquad \text{(eq C-51)}$$

$$d_{50} = 0.016\ (5/2)(3.2 \text{ to } 9.7)^{4/3}\ (5) \qquad \text{(eq C-52)}$$

$$= \underline{\underline{0.94 \text{ ft}}} \text{ to } \underline{\underline{4.1 \text{ ft}}}$$

CIRCULAR CULVERT

$$\frac{d_{50}}{D_o} = 0.016 \quad \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \qquad \text{(eq C-53)}$$

$$d_{50} = 0.016 \ (8/2)(2 \text{ to } 6)^{4/3} \ (8) \qquad \text{(eq C-54)}$$

$$= \underline{\underline{1.29 \text{ ft}}} \text{ to } \underline{\underline{5.6 \text{ ft}}}$$

See figure 5-14 for geometry.

Example 9 - Determine length and geometry of a flared outlet transition for minimum tailwaters:

RECTANGULAR CULVERT

$$\frac{L}{D_o} = 0.30 \left(\frac{D_o}{TW}\right)^2 \left(\frac{q}{D_o^{3/2}}\right)^{2.5(TW/D_o)^{1/3}} \qquad \text{(eq C-55)}$$

$$L = 0.3 \ (5/2)^2 \ (3.2 \text{ to } 9.7)^{2.5(2/5)^{1/3}} \ 5 \qquad \text{(eq C-56)}$$

$$= \underline{\underline{80 \text{ ft}}} \text{ to } \underline{\underline{616 \text{ ft}}}$$

CIRCULAR CULVERT

$$\frac{L}{D_o} = \left[0.30 \left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_o^{5/2}}\right)^{2.5(TW/D_o)^{1/3}}\right] \qquad \text{(eq C-57)}$$

$$L = \left[0.3\ (8/2)^2\ (2 \text{ to } 6)^{2.5(2/8)^{1/3}}\right] 8 \qquad \text{(eq C-58)}$$

$$= \underline{\underline{114 \text{ ft}}} \text{ to } \underline{\underline{645 \text{ ft}}}$$

See figure 5-16 for geometric details; above equations developed for H - 0 or horizontal apron at outlet invert elevation without an end sill.

Example 10 - Determine diameter of stilling well required downstream of the 8-ft-diam outlet:

From figure 5-17

$$\frac{D_W}{D_o} = 0.53 \left(\frac{Q}{D_o^{5/2}}\right)^{1.0} \qquad \text{(eq C-59)}$$

$$D_W = 0.53\ (2 \text{ to } 6)\ 8 = \underline{\underline{8.5 \text{ ft}}} \text{ to } \underline{\underline{25.4 \text{ ft}}} \qquad \text{(eq C-60)}$$

See figure 5-17 for additional dimensions.

Example 11 - Determine width of US Bureau of Reclamation type VI basin required downstream of the 8-ft-diam outlet:

From figure 5-18

$$\frac{W_{VI}}{D_o} = 1.30 \left(\frac{Q}{D_o^{5/2}}\right)^{0.55} \qquad \text{(eq C-61)}$$

$$W_{VI} = [1.3\ (2 \text{ to } 6)^{0.55}]\ 8 \qquad \text{(eq C-62)}$$

$$= \underline{\underline{15.2 \text{ ft}}} \text{ to } \underline{\underline{27.9 \text{ ft}}}$$

See figure 5-18 for additional dimensions.

Example 12 - Determine width of SAF basin required downstream of the 8-ft-diam outlet:

From figure 5-19

$$\frac{W_{SAF}}{D_o} = 0.30 \left(\frac{Q}{D_o^{5/2}}\right)^{1.0} \qquad \text{(eq C-63)}$$

$$W_{SAF} = 0.30\ (2\ \text{to}\ 6)\ 8 = \underline{\underline{4.8\ \text{ft}}}\ \text{to}\ \underline{\underline{14.4\ \text{ft}}} \qquad \text{(eq C-64)}$$

See figure 5-19 for additional dimensions.

Example 13 - Determine size of riprap required downstream of 8-ft-diam culvert and 14.4-ft-wide SAF basin with discharge of 1,086 cfs

$$q = \frac{Q}{W_{SAF}} = \frac{1086}{14.4} = 75\ \text{cfs/ft} \qquad \text{(eq C-65)}$$

$$V_1 = \frac{Q}{A} = \frac{1086}{0.785(8)^2} = 21.6\ \text{fps} \qquad \text{(eq C-66)}$$

$$d_1 = \frac{q}{V_1} = \frac{75}{21.6} = 3.5\ \text{ft} \qquad \text{(eq C-67)}$$

$d_2 = 8.4$ ft (from conjugate depth relations)

MINIMUM TAILWATER REQUIRED FOR A HYDRAULIC JUMP = 0.90 (8.4) = 7.6 ft

$$d_{50} = D \left(\frac{V}{\sqrt{gD}}\right)^3 \qquad \text{(eq C-68)}$$

$$V = \frac{q}{D} = \frac{75}{7.6} = 9.9 \text{ fps} \qquad \text{(eq C-69)}$$

$$d_{50} = 1.0 \left[\frac{9.9}{\sqrt{32.2(7.6)}}\right]^3 7.6 \qquad \text{(eq C-70)}$$

$$d_{50} = \underline{\underline{1.9 \text{ ft}}} \qquad \text{(eq C-71)}$$

APPENDIX D

CHANNEL DESIGN PROBLEM

D-1. Design procedure.

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

a. Determine gradation of material common to drainage basin from representative samples and sieve analyses.

b. Determine maximum discharges to be experienced annually and during the design storm.

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

d. Determine the sizes of material to be transported by examining the gradation of the local material (sizes and percentages of the total by weight). Particular attention should be given to the possibility of the transport of material from upper portions of the basin or drainage system and the need to prevent deposition of this material within the channel of interest.

e. Compute ratios of the diameter of the materials that should and should not be transported at the maximum depth of flow, (d_{50}/D.

f. Compute the Froude numbers of flow required to initiate transport of the selected sizes of cohesion less materials based on the equation, F 1.88 $(d_{50}/D)^{1/3}$, to determine the range of F desired in the channel.

D-2. Channel design.

a. Design the desired channel as indicated in the following steps.

(1) Assume that a channel is to be provided within and for drainage of an area composed of medium sand (grain diameter of 0.375 mm) for conveyance of a maximum rate of runoff of 400 cubic feet per second. Also assume that a channel depth of 6 feet is the maximum that can be tolerated from the standpoint of the existing groundwater level, minimum freeboard of 1 foot, and other considerations such as ease of excavation, maintenance, and aesthetics.

(2) From Figure D-1 or the equation

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

the Froude number of flow required for incipient transport and prevention of deposition of medium sand in. a channel with a 5-foot depth of flow can be estimated to be about 0.12. Further, it is indicated that a Froude number of about 0.20 would be required to prevent deposition of very coarse sand or very fine gravel. Therefore, an average Froude number of about 0.16 should not cause severe erosion or deposition of the medium sand common to the basin with a flow depth of 5 feet in the desired channel.

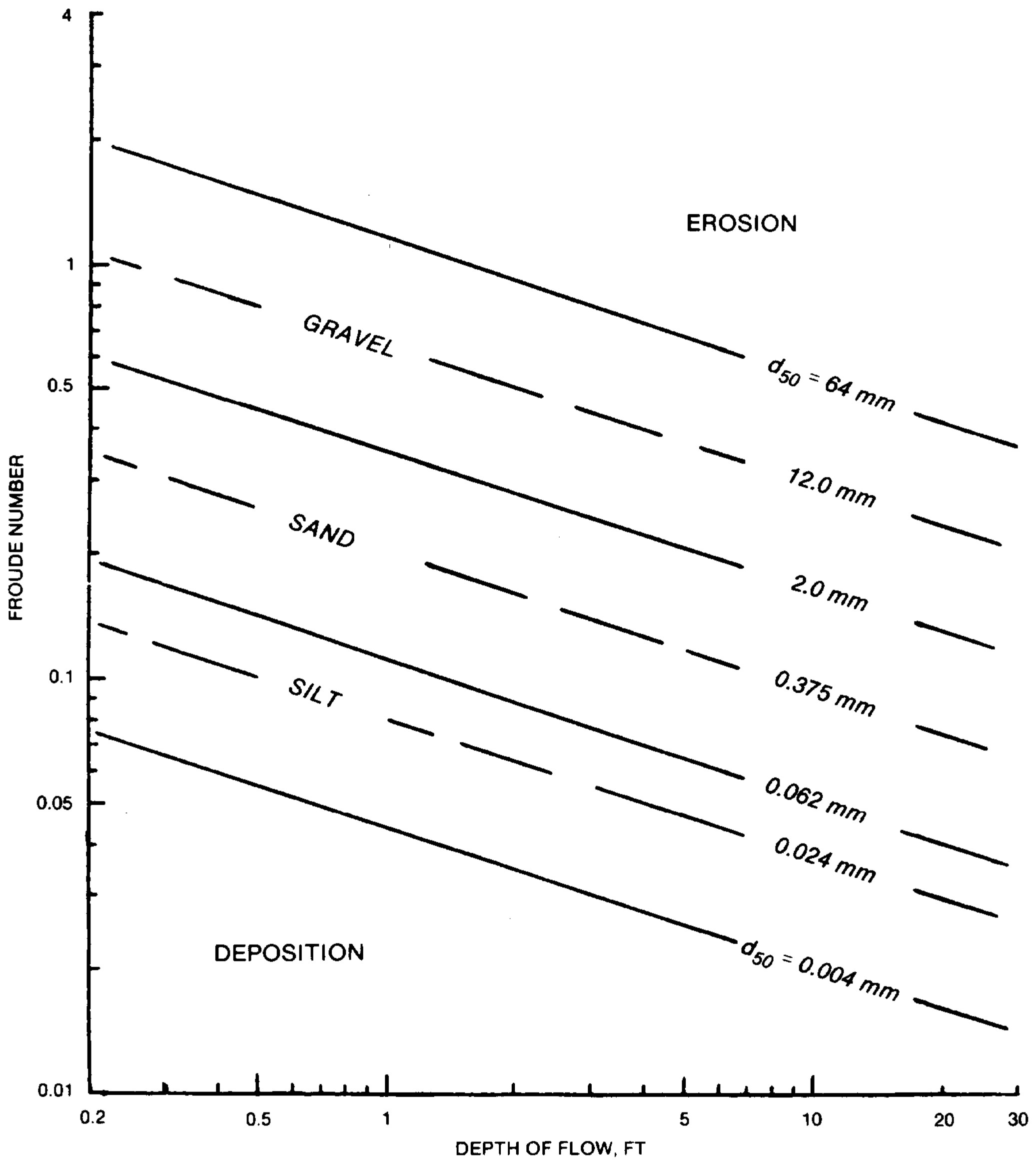

Figure D-1. Froude number and depth of flow required for incipient transport of cohesionless material.

(3) The unit discharge required for incipient transport and prevention of deposition of medium sand in a channel with a 5-foot depth of flow can be estimated to be about 7.4 cubic feet per second per foot of width from the equation

$$q = 10.66\, d_{50}^{1/3} D^{7/6} \qquad \text{(eq D-2)}$$

or figure D-2. In addition, it is indicated that a unit discharge of about 13 cubic feet per second per foot of width would be required to prevent deposition of very coarse sand or very fine gravel. Thus, an average unit discharge of about 10 cubic feet per second per foot of width should not cause severe erosion or deposition of the medium sand common to the basin and a 5-foot depth of flow in the desired channel.

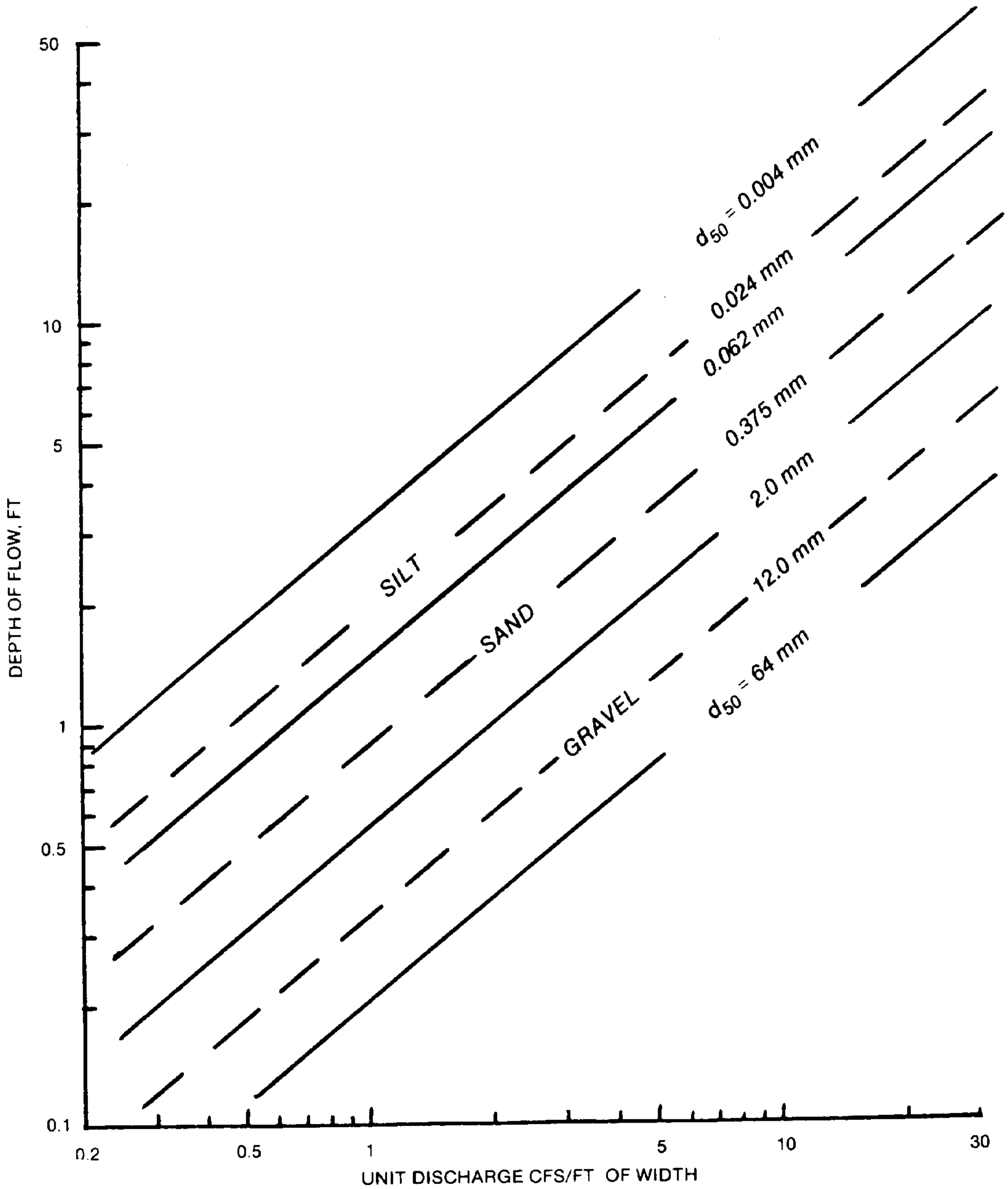

Figure D-2. Depth of flow and unit discharge for incipient transport of cohesionless material.

(4) The width of a rectangular channel and the average width of a trapezoidal channel required to convey the maximum rate of runoff of 400 cubic feet per second can be determined by dividing the design discharge by the permissible unit discharge. For the example problem an average channel width of 40 feet is required. The base width of a trapezoidal channel can be determined by subtracting the product of the horizontal component of the side slope corresponding to a vertical displacement of 1 foot and the depth of flow from the previously estimated average width. The base width of a trapezoidal channel with side slopes of IV on 3H required to convey the design discharge with a 5-foot depth of flow would be 25 feet.

(5) The values of the parameters D/B and $Q/\sqrt{g}B^5$ can now be calculated as 0.2 and 0.0225, respectively. Entering figure D-3 with these values, it is apparent that corresponding values of 0.95 and 0.185 are required for the parameters of $SB^{1/3}/n^2$ and F, respectively. Assuming a Manning's n of 0.025, a slope of 0.000203 foot per foot would be required to satisfy the $SB^{1/3}/n_2$ relation for the 5-foot deep trapezoidal channel with base width of 25 feet and IV-on-3H side slopes.

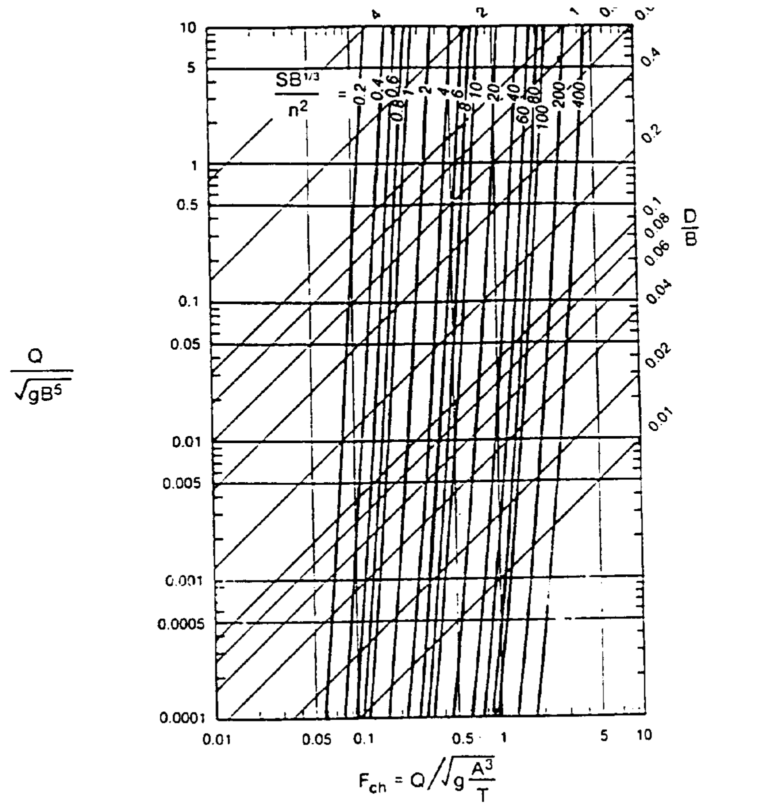

Figure D-3. Flow characteristics of trapezoidal channels with 1V-on-3H side slopes.

(6) The Froude number of flow in the channel is slightly in excess of the value of 0.16 previously estimated to be satisfactory with a depth of flow of 5 feet, but it is within the range of 0.12 and 0.20 considered to be satisfactory for preventing either severe erosion or deposition of medium to very coarse sand. However, should it be desired to convey the design discharge of 400 cubic feet per second with a Froude number of 0.16 in a trapezoidal channel of 25-foot base width and 1V-on-3H side slopes, the values of 0.0225 and 0.16 for $Q\sqrt{gB^5}$ and F, respectively, can be used in conjunction with the figure D-3 to determine corresponding values of $SB^{1/3}]/n^2$ (0.72) and D/B (0.21) required for such a channel. Thus, a depth of flow equal to 5.25 feet, and a slope of 0.000154 foot per foot would be required for the channel to convey the flow with a Froude number of 0.16.

(7) The slopes required for either the rectangular or the trapezoidal channels are extremely moderate. If a steeper slope of channel is desired for correlation with the local topography, the feasibility of a lined channel should be investigated as well as the alternative of check dams or drop structures in conjunction with the channel previously considered. For the latter case, the difference between the total drop in elevation desired due to the local topography and that permissible with the slope of an alluvial channel most adaptable to the terrain would have to be accomplished by means of one or more check dams and/or drop structures.

(8) Assume that there is a source of stone for supply of riprap with an average dimension of 3 inches. The feasibility of a riprap-lined trapezoidal channel with 1V-on-3H side slopes that will convey the design discharge of 400 cubic feet per second with depths of flow up to 5 feet can be investigated as follows. The equation, $F = 1.42(d_{50}/D)^{1/3}$, or figure D-4 can be used to estimate the Froude number of flow that will result in failure of various sizes of natural or crushed stone riprap with various depths of flow. The maximum Froude number of flow that can be permitted with average size stone of 0.25-foot-diameter and a flow depth of 5 feet is 0.52. Similarly, the maximum unit discharge permissible (33 cubic feet per second per foot of width) can be determined by the equation,

$$q = 8.05\, d_{50}^{1/3}\, D^{7/6} \qquad \text{(eq D-3)}$$

or figure D-5. For conservative design, it is recommended that the maximum unit discharge be limited to about two thirds of this value or say 22 cubic feet per second per foot of width for this example. Thus, an average channel width of about 18.2 feet is required to convey the design discharge of 400 cubic feet per second with a depth of 5 feet. The base width required of the riprap-lined trapezoidal channel with side slopes of lV on 3H would be about 3 feet.

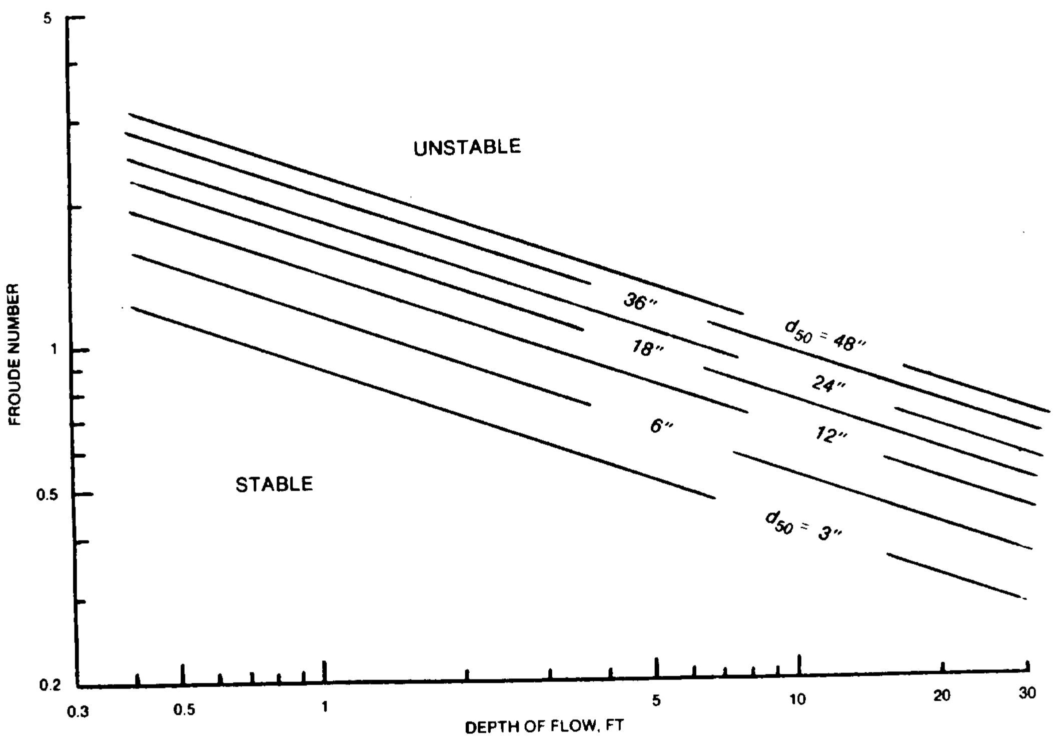

Figure D-4. Froude number and depth of flow for incipient failure of riprap-lined channel.

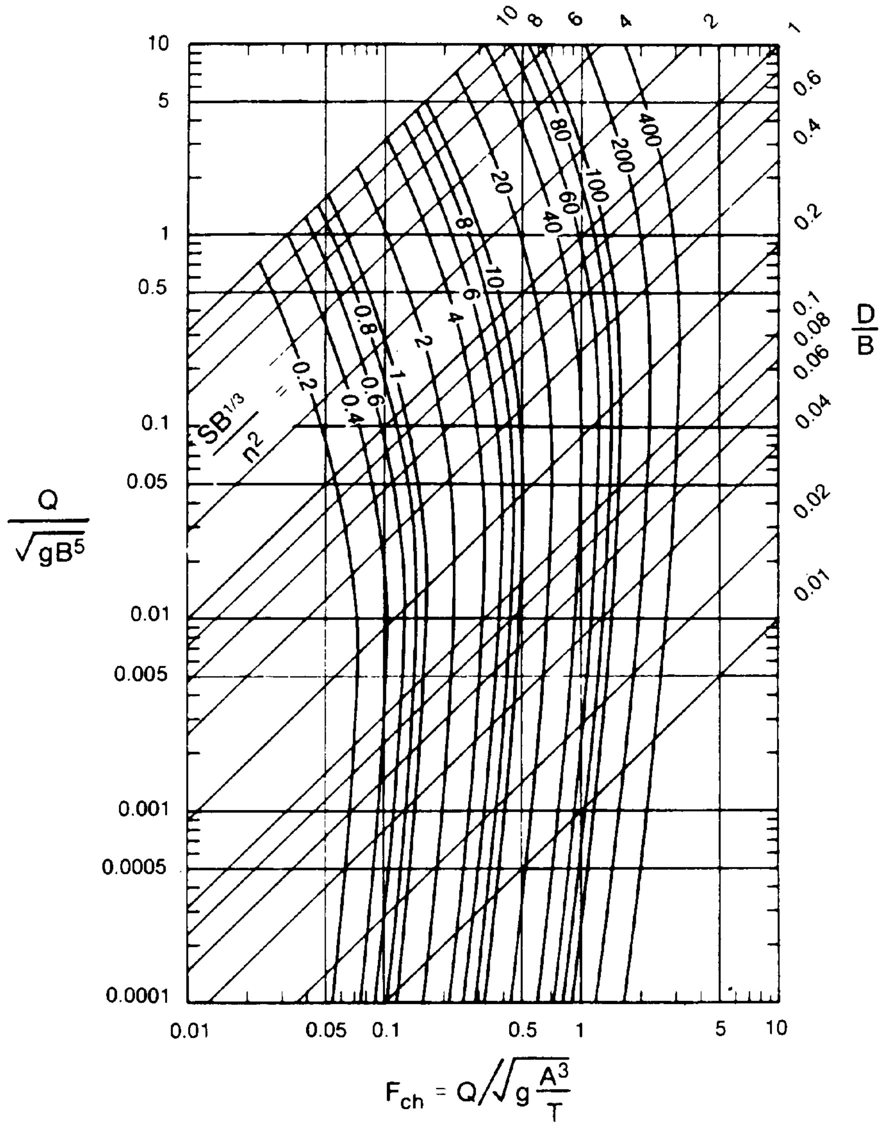

Figure D-5. Depth of flow and unit discharge for incipient failure of riprap-lined channel.

(9) The values of D/B and $Q/\sqrt{gB^5}$ can be calculated as 1.67 and 4.52, respectively. Entering figure D-3 with these values, it is apparent that corresponding values of 4.5 and 0.52 are required for the parameters of $SB^{1/3}/n^2$ and F, respectively. Assume n = 0.035 $(d_{50})^{1/6}$ and calculate Manning's roughness coefficient of 0.25-foot-stone to be 0.028. A slope of 0.00245 foot per foot would be required for the 5-foot-deep riprap-lined trapezoidal channel with base width of 3 feet and 1V-on-311 side slopes. The Froude number of flow in the channel would meet the 3-inch-diameter average size requirement for riprap as well as the maximum recommended value of 0.8 needed to prevent instabilities of flow and excessive wave heights in subcritical open channel flow.

(10) Similar analyses could be made for design of stable channels with different sizes of riprap protection should other sizes be available and steeper slopes be desired. This could reduce the number of drop structures required to provide the necessary grade change equal to the difference in elevation between that of the local terrain and the drop provided by the slope and length of the selected channel design.

(11) The feasibility of a paved rectangular channel on a slope commensurate with that of the local terrain for conveyance of the design discharge at either subcritical or supercritical velocities should also be investigated. Such a channel should be designed to convey the flow with a Froude number less than 0.8 if subcritical, or greater than

1.2 and less than 2.0 if supercritical to prevent flow instabilities and excessive wave heights. It should also be designed to have a depth-to-width ratio as near 0.5 (the most efficient hydraulic rectangular cross section) as practical depending upon the local conditions of design discharge, maximum depth of flow permissible, and commensuration of a slope with that of the local terrain.

(12) For example, assume that a paved rectangular channel is to be provided with a Manning's n = 0.015 and a slope of 0.01 foot per foot (average slope of local terrain) for conveyance of a design discharge of 400 cubic feet per second at supercritical conditions. A depth-to-width ratio of 0.5 is desired for hydraulic efficiency and a Froude number of flow between 1.2 and 2.0 is desired for stable supercritical flow. The range of values of the parameter $SB^{1/3}]/n^2$(70-180) required to satisfy the desired D/B and range of Froude number of supercritical flow can be determined from figure D-6. Corresponding values of the parameter $\sqrt{g}B^5$ (0.44-0.68) can also be determined from figure D-6 for calculation of the discharge capacities of channels that will satisfy the desired conditions. The calculated values of discharge and channel widths can be plotted on log-log paper as shown in figure D-7 to determine the respective relations for supercritical rectangular channels with a depth-to-width ratio of 0.5, a slope of 0.01 foot per foot, and a Manning's n of 0.015. Figure D-7 may then be used to select a channel width of 7.5 feet for conveyance of the design discharge of 400 cubic feet per second. The exact value of the constraining parameter $SB^{1/3}/n^2$ can be calculated to be 87 and used in conjunction with a D/B ratio of 0.5 and figure D-6 to obtain corresponding values of the remaining constraining parameters, $Q\sqrt{g}B^5$ = 0.48 and F = 1.4, required to satisfy all of the dimensionless relations shown in figure D-6. The actual discharge capacity of the selected 7.5-foot-wide channel with a depth of flow equal to 3.75 feet can be calculated based on these relations to ensure the adequacy of the selected design. For example, based on the magnitude of a discharge parameter equal to 0.48, the channel should convey 419 cubic feet per second:

$$Q = 0.48\sqrt{g}(7.5)^{5/2} = 419 \text{ cubic feet per second} \qquad \text{ed D-4)}$$

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

$$Q = 1.4 \frac{\sqrt{g}(7.5\text{x}3.75)^3}{7.5} = 432 \text{ cubic feet per second} \qquad \text{(ed D-5)}$$

Obviously, the capacity of the 7.5-foot-wide channel is adequate for the design discharge of 400 cubic feet per second.

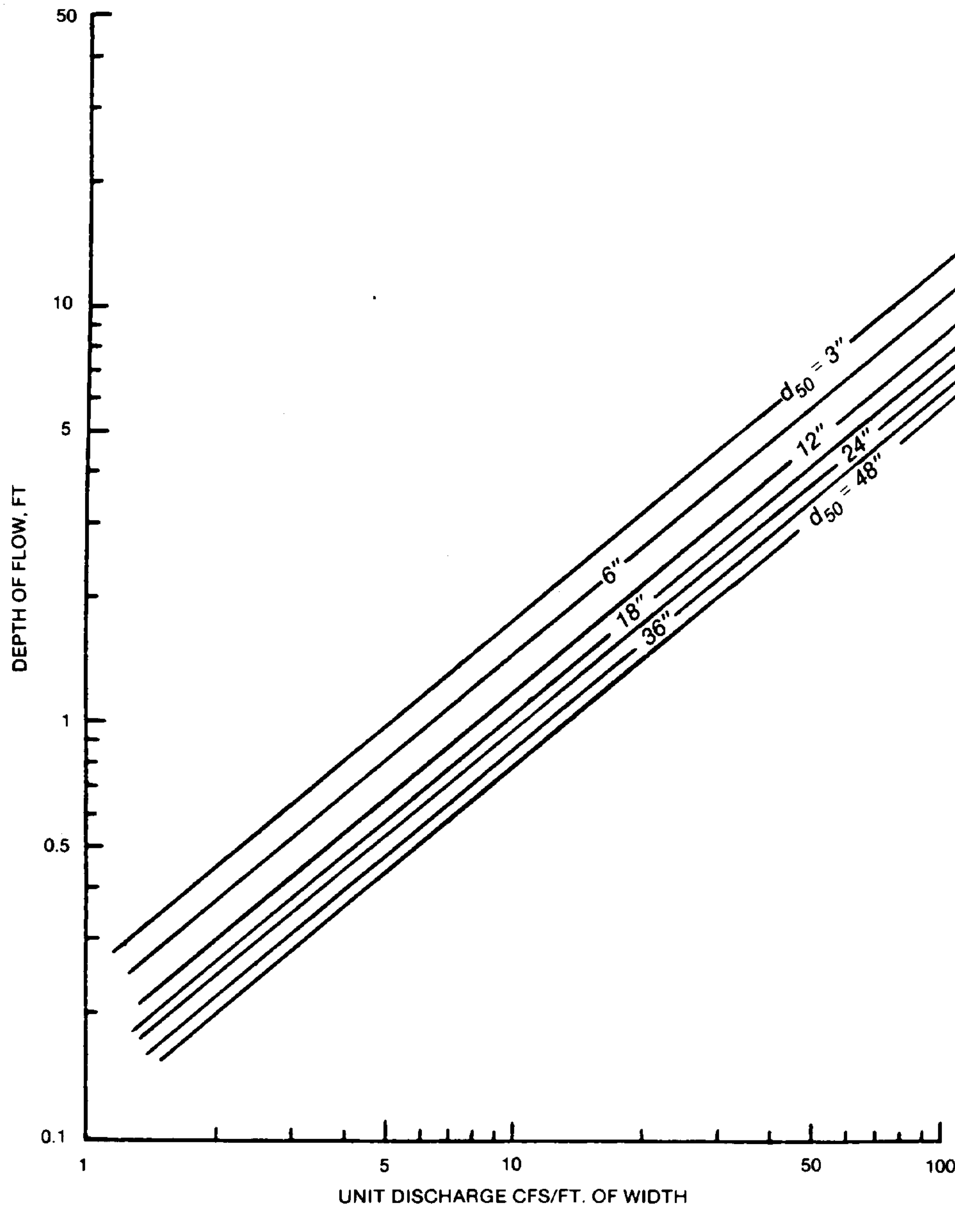

Figure D-6. Flow characteristics of rectangular channels.

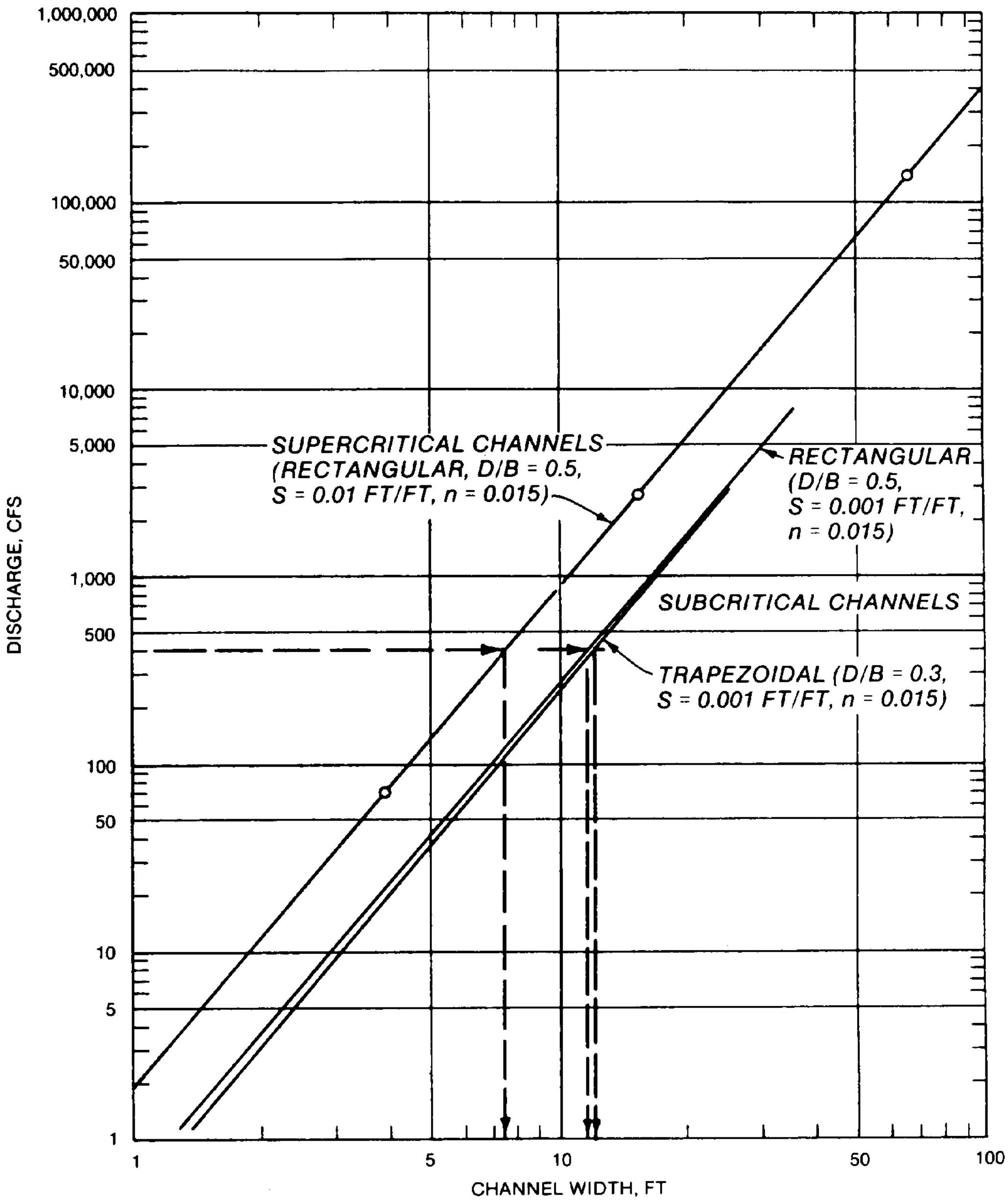

Figure D-7. Discharge characteristics of various channels.

(13) The feasibility of a paved channel with a slope compatible with that of the local terrain for conveyance of the design discharge at subcritical conditions should be investigated. However, it may not be feasible with slopes of 1 percent or greater. Paved channels for subcritical conveyance of flows should be designed to provide Froude numbers of flow ranging from about 0.25 to 0.8 to prevent excessive deposition and flow instabilities, respectively, If rectangular, paved channels should be designed to have a depth of width radio as near 0.5 as practical for hydraulic efficiency; if trapezoidal, they should be designed to have side slopes of lV on 3H and a depth-to-width ratio of 0.3.

(14) For example, assume a subcritical paved channel with a Manning's n of 0.015 and slope of 0.01 foot per foot is to be provided for a design discharge of 400 cubic feet per second. The maximum slope and discharge permissible for conveying flow with a Froude number less than 0.8 in a hydraulically efficient rectangular channel with a minimum practical width of 1.0 foot can be determined from figure D-6. For a D/B = 0.5 and

Froude number of flow of 0.8, the corresponding values of $SB^{1/3}/n^2$ and $Q\sqrt{gB^5}$ are determined as 30 and 0.275, respectively. Solving these regulations for S and Q based on n = 0.015 and B = 1 foot yields

$$S = 30\ n^2/B^{1/3} = 0.00675 \text{ foot per foot} \tag{eq D-6}$$

$$Q = 0.275\sqrt{g}B^{5/2} = 1.56 \text{ cubic feet per second} \tag{eq D-7}$$

Greater widths of hydraulically efficient rectangular channels would convey greater discharges, but slopes flatter than 0.00675 foot per foot would be required to prevent the Froude number of flow from exceeding 0.8. Therefore, a rectangular channel of the most efficient cross section and a slope as steep as 0.01 foot per foot are not practical for subcritical conveyance of the design discharge and the example problem. A similar analysis for any shape of channel would result in the same conclusion; stable subcritical conveyance of the design discharge on a slope of 0.01 foot per foot is not feasible.

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

(16) Based on the desired range of Froude numbers of flow (0.25 to 0.8) in a rectangular channel of efficient cross section (D/B = 0.5), figure D-6 indicates the corresponding range of values of the restraining parameters $SB^{1/3}/n^2$ and $Q\sqrt{gB^5}$ to be from 3 to 30 and 0.085 to 0.275, respectively. The relations between discharge and channel width for subcritical rectangular channels with a depth-to- width ratio of 0.5, a slope of 0.001 foot per foot, and a Manning's n of 0.015 can be plotted as shown in figure D-7 to select the 11.5-foot-width of channel required to convey the design discharge of 400 cubic feet per second.

(17) As a check, the exact value of $SB^{1/3}/n^2$ can be calculated to be 10.1 and used in conjunction with a D/B ratio of 0.5 and figure D-6 to obtain corresponding values of the remaining constraining parameters, $Q\sqrt{gB^5}$ = 0.16 and F = 0.47, required to satisfy all of the dimensionless relations for rectangular channels. The actual discharge capacity of the selected 11.5-foot-wide channel with a depth of 5.75 feet can be calculated based on these relations to ensure the adequacy of the selected design. For example, based on the magnitude of the discharge parameter (0.16), the channel should convey 407 cubit feet per second:

$$Q = 0.16\sqrt{g}(11.5)^{5/2} = 407 \text{ cubic feet per second} \tag{eq D-8}$$

Similarly, based on the Froude number of flow to 0.47, the channel should convey a discharge of 422 cubic feet per second:

$$Q = 0.47\sqrt{\frac{g(11.5\times 5.75)^3}{11.5}} = 422 \text{ cubic feet per second} \tag{eq D-9}$$

Therefore, the 11 .5-foot-wide channel is sufficient for subcritical conveyance of the design discharge of 400 cubic feet per second and, based on figure D-1, is sufficient for transporting materials as large as average size gravel.

(18) A similar procedure would be followed to design a trapezoidal channel with a depth-to-width ratio of 0.3, a slope of 0.001 foot per foot, and a Manning's n of 0.015 utilizing figure D-3. For example, in order to maintain a Froude number

of flow between 0.25 and 0.75 in a trapezoidal channel with side slopes lV on 3H and a depth-to-width ratio of 0.3, the constraining parameter of $SB^{1/3}/n^2$ would have to have a value between 2 and 15 (fig. D-3). The relations between discharge and base width for these subcritical trapezoidal channels were plotted as shown in figure D-7 to select the 12-foot-base width required to convey the design discharge of 400 cubic feet per second.

(19) As a check, the exact value of $SB^{1/3}/n^2$ was calculated to be 10.2 and used in conjunction with D/B of 0.3 and figure D-3 to obtain corresponding values of the remaining constraining parameters, $Q\sqrt{gB^5}$=0 15 and F=0.63, required to satisfy the dimensionless relations of trapezoidal channels. The actual discharge capacity of the selected trapezoidal channel with a base width of 12 feet and a flow depth of 3.6 feet based on these relations would be 425 and 458 cubic feet per second, respectively.

$$Q = 0.15\sqrt{g}\,(12)^{5/2} = 425 \text{ cubic feet per second} \qquad \text{(eq D-10)}$$

$$Q = 0.63\sqrt{\frac{g\,\frac{45.6\times 3.6^3}{2}}{33.6}} = 458 \text{ cubic feet per second} \qquad \text{(eq D-11)}$$

Therefore, the selected trapezoidal channel is sufficient for subcritical conveyance of the design discharge of 400 cubic feet per second and based on figure D-1 is sufficient for transporting materials as large as coarse gravel.

b. Having determined a channel that will satisfy the conditions desired for the design discharge, determine the relations that will occur with the anticipated maximum annual discharge and ensure that deposition and/or erosion will not occur under these conditions. It may be necessary to compromise and permit some erosion during design discharge conditions in order to prevent deposition under annual discharge conditions. Lime stabilization can be effectively used to confine clay soils, and soil-cement stabilization may be effective in areas subject to sparse vegetative cover. Sand-cement and rubble protection of channels may be extremely valuable in areas where rock protection is unavailable or costly. Appropriate filters should be provided to prevent leaching of the natural soil through the protective material. Facilities for subsurface drainage or relief of hydrostatic pressures beneath channel linings should be provided to prevent structural failure.

APPENDIX E

NOTATION

A — Cross-sectional area, ft^2.

a — Offset for weir notch ventilation, ft.

B — Base width of channel, ft.

b_n — Length of notch, ft.

B_s — Bottom width of approach channel, ft.

C — Coefficient.

D — Depth of flow in channel, ft.

D_o — Diameter of circular culverts, ft.

D_s — Depth of scour, ft.

D_{sm} — Maximum depth of scour, ft.

D_w — Diameter of stilling well, ft.

d — Depth of uniform flow in culvert, ft.

d_c — Critical depth, ft.

d_s — Depth of approach flow, ft.

d_1 — Depth of flow upstream of hydraulic jump, ft.

d_2 — Theoretical depth of flow required for hydraulic jump, ft.

d_{50} — Diameter of average size stone, ft.

F — Froude number.

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 sill, 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, $\bar{v}$ 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.

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