DESIGN OF SMALL CANAL STRUCTURES

UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

A WATER RESOURCES TECHNICAL PUBLICATION

PB95-176848

DESIGN OF SMALL CANAL STRUCTURES

1978

Engineering Technology Pertaining Primarily to the Design of Small Canal Structures of Less Than 100-Cubic-Feet-Per-Second Capacity

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

by

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As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

Mission of the Bureau of Reclamation

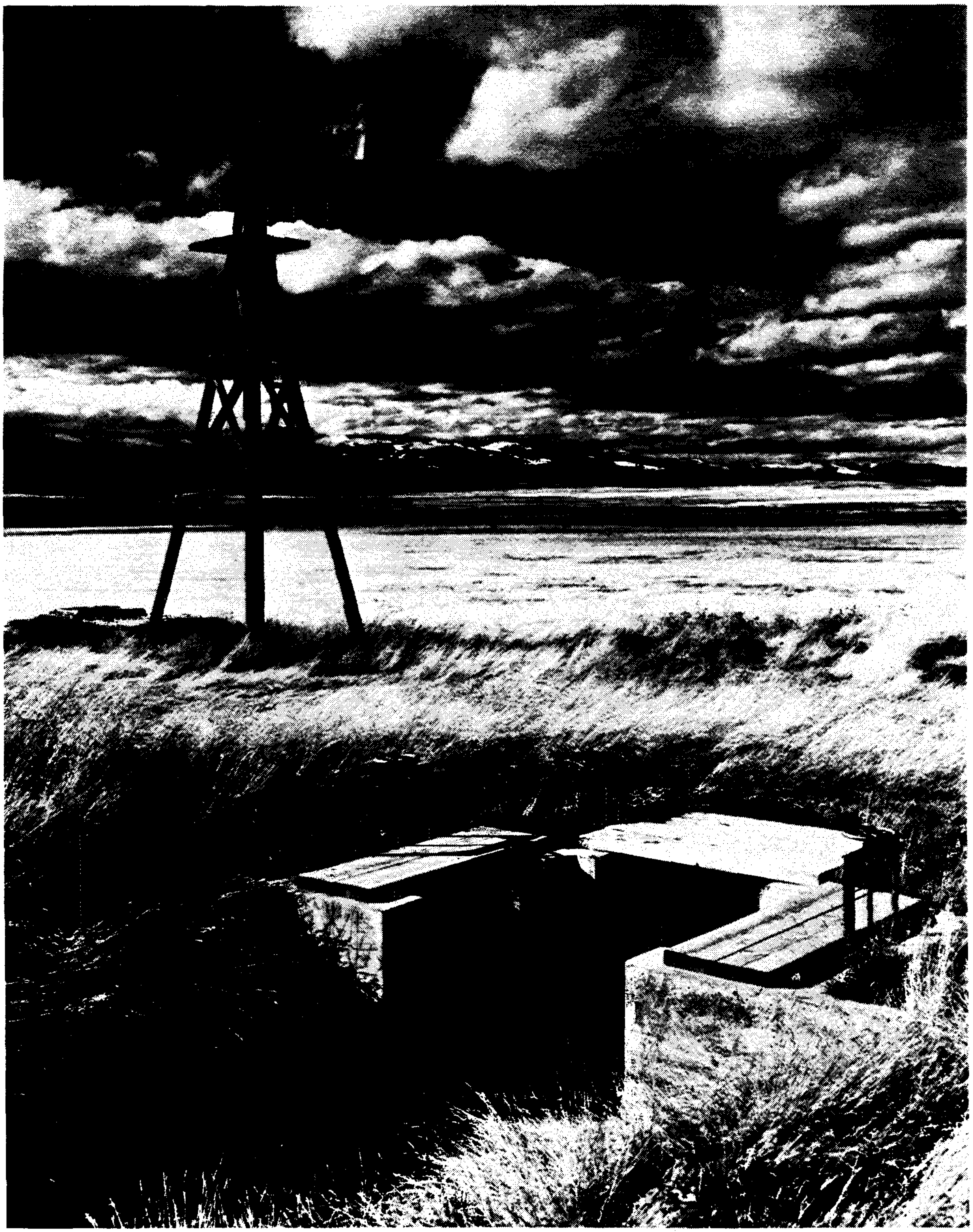
The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

FIRST EDITION 1974
REVISED REPRINT 1978
REPRINTED 1983

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, and the Bureau of Reclamation, Engineering and Research Center, Denver Federal Center, P O Box 25007, Denver, Colorado 80225-0007, Attention: D-922.



The old and the new. In the shadow of an abandoned windmill, turnouts from a division structure on an East Bench Canal lateral await water for irrigating benchlands east of Dillon, Mont. P699-600-2253NA, March 28, 1968

Preface

"Let man have dominion over the earth and let the earth bring forth grass, the herb yielding seed, and the fruit tree yielding fruit after its kind." So is it written in the Book of Genesis.

The availability of water has determined the course of empires in many civilizations. When made available, its use for irrigating crops has been practiced for thousands of years by many peoples of the world. But only since the late 1800's has man applied scientific knowledge to watering drylands to increase crop production. The wise use of soil and water resources has changed millions of acres of once barren wasteland and desert into productive farms supporting prosperous communities.

Development of water resources by the Government to provide irrigation water for the arid lands of the western United States was initiated when the Reclamation Act of 1902 was passed during President Theodore Roosevelt's administration. Because of his strong leadership ability and contempt for abusive exploitation of our natural resources, he became regarded as the steward of the public's welfare.

Reclamation engineers and scientists through the years have developed canal structures for irrigation systems which can effectively perform their intended functions. This publication has been prepared to illustrate the application of canal structures having design discharge capacities up to 100 cubic feet per second. Several types of canal structures have been standardized for this capacity range and are presented herein. Structure sizes required to discharge these flows are relatively small; however, engineering principles used in their design are also applicable to canal structures of greater capacity.

The need for this publication became apparent by the number of requests received, both foreign and domestic, during the past years. It is intended that this book provide the design engineer with a source of condensed information for use as a guide in efficiently designing small canal structures. The design engineer must realize, however, that sound engineering principles and judgment must be exercised in the selection and utilization of the structures, and that the use of design methods, procedures, and other information herein remains his responsibility.

There are occasional references to proprietary materials or products in this publication. These must not be construed in any way as an endorsement by the Bureau of Reclamation since such endorsement cannot be made for proprietary products or processes of manufacturers or the services of commercial firms for advertising, publicity, sales, or other purposes.

The text of this publication was prepared by a design team comprised of Bureau of Reclamation engineers from the Engineering and Research Center at Denver, Colorado, under the overall direction of H. G. Arthur, Director of Design and Construction.

The text was coordinated and edited by A. J. Aisenbrey, Jr., Civil Engineer, Hydraulic Structures Branch. Team members who authored the presentations for the canal structures were R. B. Hayes, D. L. Winsett, and R. B. Young, Civil Engineers, Hydraulic Structures Branch and H. J. Warren, General Engineer, Engineering Reference Branch. Editorial guidance, final review, and preparation of the manuscript for publication

was performed by W. E. Foote, General Engineer, of the Technical Services Branch. The revised reprint was edited by R. D. Mohrbacher, General Engineer, Technical Services and Publications Branch.

Many photographs used in the book were obtained through the cooperation of various Bureau of Reclamation offices and assembled for selection by R. S. Dinsmore, Civil Engineering Technician, Hydraulic Structures Branch under the guidance of D. L. Winsett.

Illustrations were prepared in the Drafting Branch, Division of Engineering Support by L. L. McGhghy and W. D. Anderson under the guidance of W. W. Groom and direction of B. F. Wilson.

Special recognition is given to Dr. J. W. Hilf, Chief, Division of Design, G. N. Thorsky, Chief, Division of Engineering Support; A. T. Lewis, Chief, Hydraulic Structures Branch; G. W. Birch, Head, Canal Structures and Bridges Section and engineers in this section for their support, guidance, and counsel.

The engineers who authored and edited this publication wish to express a special thanks to the staff of the Media Unit who worked many long and hard hours to obtain a quality publication. Other thanks are directed to the printing assistants, production unit personnel, and those who coordinated the work flow.

Food and fiber, essential to the well-being of mankind begins with the smallest viable seed, which when its fruit is harvested may provide substance to placate the hunger of man.

To believe that even the smallest seed will grow
is
To believe in the force of will.

To transform the desert into a living oasis is
To transform despair into hope,

That all mankind may be enriched
And share in the power of Charity.

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General Requirements and Design Considerations

A. J. AISENBREY, JR.[1]

A. GENERAL REQUIREMENTS

1-1. ***Purpose.***—To fulfill a project purpose of producing crops or increasing crop production, water delivery to the land must be provided by a reliable and efficient irrigation system. A sun-drenched, parched soil may need only water to change it from a sparsely vegetated, thirsty desert to a high-yield crop, farmland oasis as illustrated in figure 1-1. An increased crop yield of premium quality is very likely if the proper amount of moisture is made available to the crop when needed. (See fig. 1-2).

A canal is frequently used to convey water for farmland irrigation. In addition to transporting irrigation water, a canal may also transport water to meet requirements for municipal, industrial, and outdoor recreational uses. A variety of recreation is provided by reservoirs as shown in figures 1-3 and 1-4.

The conveyance canal and its related structures should perform their functions efficiently and competently with minimum maintenance, ease of operation, and minimum water loss.

1-2. ***Structures.***—Many different types of canal structures are required in an irrigation system to effectively and efficiently convey, regulate, and measure the canal discharge and also to protect the canal from storm runoff damage.

The design capacity of the conveyance, regulating, and water measurement structures discussed throughout this publication is limited to 100 cubic feet per second.

(a) *Conveyance Structures.*—In addition to the canal itself, it is usually necessary, because of topography or existing manmade features, to use inline canal structures to convey water along the canal route. Such structures include: (1) inverted siphons to convey canal water under natural channels, (2) road crossings to carry canal water under roadways, (3) bench flumes to conduct the water along a steep hillside, and (4) drop or chute structures to safely lower the canal water down a hillside.

(b) *Regulating Structures.*—Regulation of canal discharge begins at the source of water supply. This may be a canal headworks structure adjacent to a diversion dam on a stream or river, a turnout from a larger canal, or a pumping plant located on a reservoir or large canal. Downstream from the source of water supply, regulation of canal discharge is primarily controlled by outflow through turnout structures. Where canal flow is to be divided and directed in several directions, division structures are used to regulate the discharge in each direction. Wasteway structures also are used to control flow in a canal; however, they are more commonly thought of as protective structures as their primary function is to discharge excess canal flows and thereby protect the canal from damage.

Regulating structures are also capable of raising the canal water surface higher than would normally exist when the canal is flowing

[1]Civil Engineer, Hydraulic Structures Branch, Bureau of Reclamation.

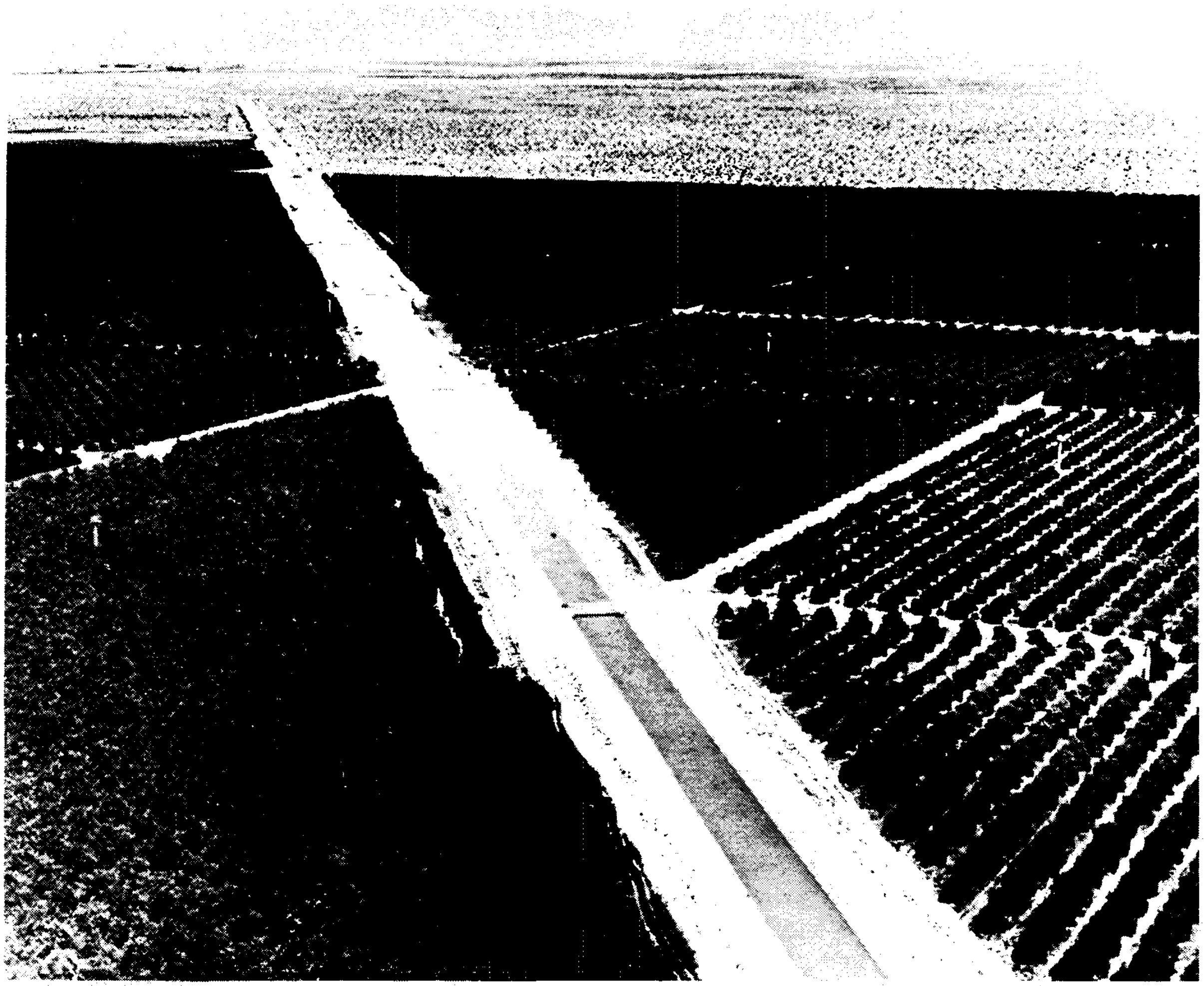

Figure 1-1. **Open canal system supplying irrigation water to citrus groves, Gila Project, Arizona. P50-300-10586**

at less than design capacity. A check structure, for example, is used to raise the canal water surface when the canal is flowing at partial capacity so that a turnout structure may still deliver its designed capacity. Check structures are spaced at appropriate intervals along a canal to provide this capability.

(c) *Water Measurement Structures.*—Efficient management of an irrigation system insists that measurement of the rate-of-flow and volume delivered be made. Equitable water distribution to the users is a primary consideration. Water measurement also tends to prevent unnecessary wasteful water management practices, thereby enhancing the conservation of this great natural resource.

Several types of water measurement structures or devices are used. Parshall flumes, weirs, weir boxes, open-flow meters, and constant head orifices are the more common types.

The constant head orifice, although sometimes used as an inline canal water measurement structure, is more commonly used in conjunction with a turnout and therefore is discussed in detail with regulating structures.

Selection of the type of structure best suited for a particular installation is discussed in chapter V.

(d) *Protective Structures.*—Provisions must be made in an open irrigation system to

Figure 1-2. Irrigating corn using siphon tubes, Red Willow Unit, Nebraska. P328-701-6140

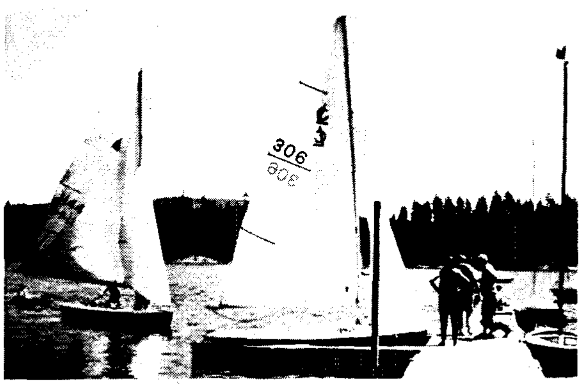

Figure 1-3. Boating on the Rogue River Project, Oregon. C448-100-148

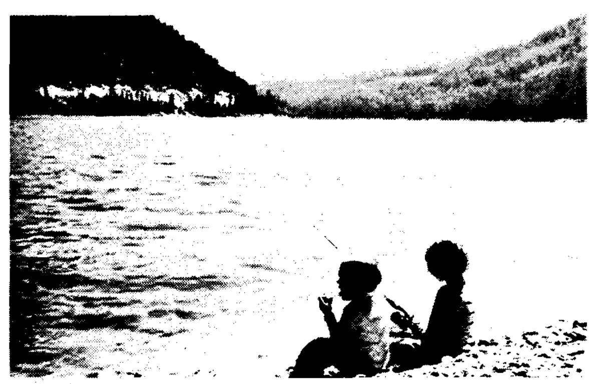

Figure 1-4. Fishing on Hungry Horse Reservoir, Montana. C447-100-50

externally protect the canal on the uphill side from damage by storm runoff water, and *internally* protect the canal from excess canal flows caused by storm waters entering the canal, or by misoperation of the canal system. Cross-drainage structures and wasteways provide this protection.

Cross-drainage structures are used to direct storm runoff flows under the canal through culverts, over the canal in overchutes, or into the canal by drain inlets.

Wasteways evacuate excess canal flows over side channel spillways, through radial gated spillways, or through siphon spillways.

(e) *Structure Components and Appurtenances.*—Nearly all canal structures are made of several different structural parts which together make up the complete structure. These parts, (components and appurtenances) include:

(1) *Pipe.*—Pipe is commonly used for that part of a structure placed underground and which may or may not be subjected to internal hydrostatic bursting pressure. Pipe is made from one of several different materials such as reinforced concrete, asbestos cement, welded steel, corrugated metal, or reinforced plastic mortar. Selection of the appropriate pipe material is dependent on several considerations discussed later in chapter VIII.

(2) *Pipe appurtenances.*—Appurtenances include such items as pipe collars, air vents, blowoffs, manholes, and are discussed later in chapter VIII.

(3) *Transitions.*—Transitions connect a canal or natural channel to a structure inlet or structure outlet. Several different transition configurations of reinforced concrete are used. Earth transitions are used as required to vary base widths and invert slopes. See chapter VII for a detailed discussion of transitions.

(4) *Energy dissipators.*—Energy dissipators are used at the outlet ends of drop or chute structures to dissipate excess energy. Energy dissipation may be achieved by a hydraulic jump in a stilling pool or by impact in a baffled outlet. Excess energy may also be dissipated by a hydraulic jump within a pipe; by impact on a baffled apron; by impact and valve losses in a high-head vertical energy dissipator; or simply by a vertical drop of a few feet into a pool of water. Energy dissipators are discussed in detail in chapter VI.

(5) *Safety features.*—Safety features are used to prevent humans, livestock, and wildlife from entering a canal and canal structures, and also to assist in their escape if inadvertently entered. Various types of fencing, guardrail, nets, racks, ladders, and signs are used as safety precautions. See chapter IX for a more detailed discussion of canal safety features.

1-3. ***Other Requirements.***—Many requirements vital to a complete and competent design of a canal water-delivery system are beyond the scope of this publication. However, the more basic considerations are briefly discussed in this section to remind or acquaint the reader of other essential design data requirements.

(a) *Canal.*—Design capacity for an irrigation canal is determined by the maximum water demand which is primarily dependent on the following considerations: (1) the area to be irrigated, (2) crops to be grown, (3) rotation or demand system for turnout deliveries, (4) water losses from evaporation and seepage, and (5) anticipated efficiency of water application to the crops. Records of actual water use for conditions similar to the system to be designed are valuable guides for estimating the quantity of water required for each acre. Soil characteristics and climatic conditions should also be evaluated. Figure 1-5 shows furrow irrigation of crops from a header ditch, and sprinkler irrigation is shown in figure 1-6.

Location of the canal with respect to the land to be irrigated is primarily influenced by topography and economic considerations. If, for example, water delivery cannot be made by gravity, the additional cost of pumping would be included in an economic study. Figure 1-7 shows water delivery by gravity from the canal, through a turnout, to the farmland.

Considerations for the selection of the canal hydraulic properties include erosion resistance for the banks and invert of an earth canal, sideslope stability, hydraulic efficiency,

Figure 1-5. **Irrigating sugar beets on the Boise Project, Idaho. C4-100-188**

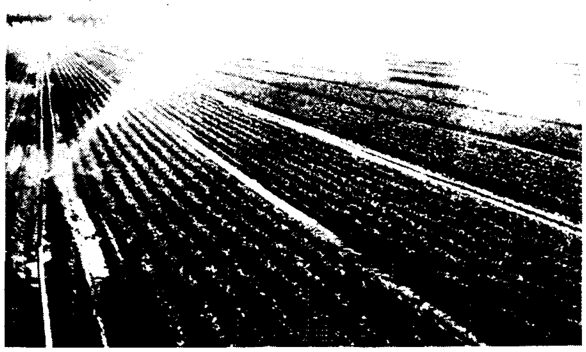

Figure 1-6. **Sprinkler irrigation in the Coachella Valley, California. LC830**

Figure 1-7. **Irrigating fruit on the Yakima Project, Washington. C-33-100-80**

operational flexibility, and economics. The need for canal lining and selection of the type of lining are also determinations which must be made.

(b) *Other Structures.*—Detailed discussions of structures such as tunnels, bridges, utility crossings, and fish screens and fish ladders are not included herein.

Tunnels are used where it is more economical to convey canal water through a ridge or hill than to: (1) pump water over the obstruction, (2) convey the water along the hillside or around the ridge, or (3) construct a canal section requiring a very deep cut.

Bridges for roadway traffic over a canal are generally used for canals of much greater capacity than 100 cubic feet per second. However, if the monetary value of hydraulic head in the canal system is particularly high, a cost comparison of a bridge and a road crossing structure may be warranted as the head loss through a pipe road crossing could be significant.

Utility crossing structures are provided for existing utilities such as gas, oil, water, sewage, electricity, and communication lines. Utility lines may cross over or under the canal, and frequently require special design considerations to comply with the utility owner's requirements and the canal design criteria.

Fish screens and ladders are special structures at or near the source of canal water supply. Screens exclude fish from a canal whereas ladders permit migration in the natural channel.

(c) *Soils Exploration.*—The following very generally and briefly discusses some of the more important considerations associated with a soils exploration program. Methods and procedures for drilling, sampling, and testing are also very important [1].[2]

Soils investigation along a canal and at structure sites is required to: (1) identify the soil materials to be excavated, (2) identify the soil for potential use as canal embankment, (3) determine the foundation adequacy for canal structures and lining, (4) determine the need for canal lining, (5) determine canal sideslope stability, (6) assess erosion resistance characteristics, (7) determine underdrain requirements for lined canals, and (8)

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[2]Numbers in brackets refer to items in the bibliography, sec. 1-25.

determine water-soluble sulfate ion concentrations.

Soil material to be excavated has one of two classifications: rock or common (soil). The materials remolded and used for canal embankment construction should ideally be nonexpansive, possess adequate remolded shearing strength, be relatively impervious, and relatively erosion resistant if hard-surface canal lining is not used. The soil in its natural state should also possess these qualities. It is sometimes necessary to locate a source of borrow material for canal embankment, for backfill adjacent to structures, and for foundation pads under the structures.

The reaches of a canal which traverse low density soils highly susceptible to hydrocompaction should be well delineated. In addition, the depth to subsurface water levels if present in any exploration hole should be recorded.

Canal structures discussed in this publication are relatively small and consequently foundation pressures are quite low. Therefore, structure settlement caused by foundation consolidation will usually be small in magnitude. Foundation treatment however, may be required to protect the structure from expansive soils or from undesirable settlement of low-density material.

Sulfate concentrations in the soil samples and water samples indicate the relative degree of potential sulfate attack on concrete [2]. Specifying the appropriate type of cement used in the concrete mix is usually all that is required to safeguard the concrete. Fly ash is also sometimes used in the concrete mix for sulfate resistance [3].

Although a well planned and executed soils exploration program may be conducted, an admonishment by the eminent soils engineer Karl Terzaghi must always be borne in mind: "*. . . results of subsurface exploration still leave a wide margin for interpretation.*"[4] Sound judgment is a requisite in all soil exploration programs.

(d) *Hydrology.*—The canal system should be reasonably well protected from damage which could occur from storm runoff. The runoff area contributing to a natural drainage channel, the ground slope and vegetative density of the area, and storm frequency to be used for design are all influential in determining the design capacity for cross drainage structures.

(e) *Sedimentation.*—Aggradation and degradation studies for a natural drainage channel provide the necessary data to determine earth cover required, for example, over the top of the pipe for an inverted siphon extending under the drainage channel. Degradation, if extensive enough could cause an empty pipe to float; whereas aggradation could create external earth loads great enough to crack or otherwise damage the pipe.

(f) *Operation and Maintenance.*—In a Bureau of Reclamation designed system, a designers' operating criteria (DOC) is usually provided to assist operation and maintenance (O&M) personnel in adequately protecting the canal system from unnecessary damage which could otherwise be caused by misoperation or inadequate maintenance. A rigorous maintenance program should include control of weeds, aquatic growths, insects, and pests.

Experience has shown that a well maintained and operated canal system usually has dedicated personnel—people who are justly proud of their work and take pride in what they do—*people who care.*

B. DESIGN CONSIDERATIONS

1-4. *General.*—Canal structures which have been standardized in this publication, show concrete dimensions and reinforcement steel, and are appropriately sized to provide for hydraulic, structural, and stability design considerations.

Hydraulic design provides: (1) adequate discharge capacity for inline canal structures, when properly selected and hydraulically set, to convey the flow at normal canal water depths, (2) adequate built-in overflow capacity for inline canal structures to limit infringement on the canal bank freeboard for emergency operation or misoperation of the canal, (3) adequate structural proportioning and appropriate hydraulic setting of structures to permit excess energy dissipation with minimum water turbulence at the downstream ends of the structures, and (4) structural proportioning of certain transitions to minimize hydraulic head loss.

Structural design provides: appropriate concrete thicknesses and reinforcement steel patterns for structural members to resist bending moment, thrust, and shear stresses imposed by reasonable loads on the structure.

Stability design provides: adequate structure dimensions so that for most soil foundation materials, the structure will: (1) resist sliding and overturning, (2) prevent percolating water from removing foundation materials, and (3) provide foundation pressures less than the maximum allowable bearing pressure.

Design examples for structures which have not been standardized illustrate a recommended hydraulic design procedure, but exclude structural design and stability analysis.

1. Loads

1-5. ***General.***—Loads which canal structures must capably resist include dead load weights, live loads on operating decks, lateral pressures, bursting and uplift pressures, and wheel loads.

1-6. ***Dead Load Weights.***—Commonly used dead load weights for small canal structures are presented in the following tabulation:

Load	Weight, (lbs./cu. ft.)
Water	62.4
Backfill	
Dry	100
Saturated	125
Compacted backfill	
Dry	120
Saturated	135
Concrete	150

To conveniently differentiate between soils which are above and below the water level in a soil mass, the terms dry and saturated are used. Obviously the soil adjacent to a structure is not ovendry and does possess a certain moisture content.

1-7. ***Operating Deck Uniform Live Loads.***—Operating decks for structures using stoplogs are designed for a uniform live load of 150 pounds per square foot; otherwise a uniform live load of 100 pounds per square foot is used. Decks for radial gate hoists require special structural design considerations which are not included herein.

1-8. ***Lateral Pressures.***—Lateral pressures from several different sources are imposed on walls of structures. Resultant forces from these pressures must be adequately resisted by the reinforced concrete.

(a) *Water.*—A fluid pressure of 62.4 pounds per square foot (psf) per foot of depth is caused by water. The pressure diagram is triangular with the resultant force acting at one-third the height above the base of the pressure diagram.

(b) *Earth.*—Active earth pressures may be determined using Rankine's solution of Coulomb's equation [5]. The pressure diagram is assumed to be triangular, the same as for water, with the resultant force acting at one-third the height above the base of the pressure diagram. Because of this similarity with the fluid pressure of water, the pressure caused by earth is sometimes referred to as an equivalent fluid pressure.

Standardized canal structures have been structurally designed to resist moist earth active lateral pressure equal to 30 psf per foot of depth and saturated earth active lateral pressure of 85 psf per foot of depth. Unless unusual soil properties exist, these values are considered adequate for design of small canal structures. For a detailed discussion of earth pressures on concrete retaining walls, see bibliography reference [5].

(c) *Construction and Operating Equipment Wheel Surcharge.*—Walls of structures should be designed to withstand construction and operating equipment wheel loads which are transmitted through the earth adjacent to the structure. An additional lateral load equivalent to 2 feet of earth surcharge is usually used. This results in an additional uniformly distributed lateral pressure (rectangular pressure diagram) of 60 psf from the backfill surface to the bottom of the wall. Standardized canal structures are designed to withstand this additional load.

(d) *Ice.*—Ice loads on structures should be considered if wintertime canal operation is required. The magnitudes for ice thrust presented in bibliography reference [5] may be used.

(e) *Wind.*—Wind loads on small irrigation structures are not included in the structural and stability analyses.

1-9. *Other Pressures.*—(a) *Uplift.*—Uplift pressures, which may be caused by water percolating under or along the sides of hydraulic structures, reduce the effective weight of a structure and are therefore particularly significant in the stability analysis.

Stilling pool floors, for example, are subjected to uplift pressure from downstream water levels which may saturate the soil behind the wall to a significantly higher elevation than the water depth in the pool just upstream from the hydraulic jump. Weep holes are used to lower the saturation level.

(b) *Hydraulic Transients.*—Pressure surges in a pipe and bore waves in a canal accompany any change of flow in the system. These transients are usually of insignificant consequence in a small, open irrigation system and are therefore generally omitted from structural design considerations. A gradual rate of change of flow minimizes the magnitude of hydraulic transients.

(c) *Seismic.*—Additional earth and water pressures imparted to small canal structures by earthquakes are not included in the design considerations. The increased loads are minor since the earth, water, and concrete masses are all small. Temporary stress increases caused by seismic loads would therefore be minor.

1-10. *Wheel Loads Transmitted to Buried Conduit.*—Pipelines crossing under highways and railroads must be designed to withstand surcharge loads from trucks and locomotives. Concentrated wheel loads are transmitted through earth cover and distributed at the top of pipe where they are considered to be uniform loads. For design convenience, the uniform loads are converted to equivalent heights of compacted earth cover and added to the actual earth cover. Pipe is selected to withstand this total equivalent earth cover.

(a) *Highways.*—Equivalent earth cover for highway wheel loads is based on criteria of the American Association of State Highway Officials [6] (AASHO). The following tabulation shows total equivalent earth cover for various heights of earth cover over the top of the pipe with H-15 and H-20 truck wheel loads.

Height of earth cover, feet	Total equivalent earth cover for wheel loads shown, feet	
	H-15 loading	H-20 loading
2	11.8	15.1
3	7.5	9.1
4	6.9	7.8
5	7.3	8.1
6	8.0	8.6
7	8.7	9.2
8	9.4	9.9

For earth covers less than 2 feet, special provisions such as concrete encasement of the pipe or slab covers are required. Wheel load effect is negligible when the earth cover is more than 8 feet.

Wheel load impact factors used for earth covers less than 3 feet are as follows: (1) 10 percent for earth covers of 2 feet 1 inch to 2 feet 11 inches, (2) 20 percent for earth covers of 1 foot 1 inch to 2 feet, and (3) 30 percent if the earth cover is 1 foot or less.

Special provisions, such as detours and safety precautions, as may be required by a highway commission should be included in the design specifications.

(b) *Railroads.*—Design and installation of pipe under a railroad often requires special provisions to comply with the railroad company requirements. Jacking casing pipe through the railroad embankment is a common requirement. Pipe used to convey the canal water (carrier pipe) is then installed in the casing pipe.

The top of pipe installed under a railroad roadbed is usually at least 3 feet below the top of the railroad tie unless otherwise specified by the railroad.

For an E-72 railroad loading [7], usual Reclamation practice is to provide a pipe class to withstand a total equivalent cover of 20 feet when the top of pipe is 3 to 15 feet below the top of the railroad tie. Additional load caused by impact is included in the total equivalent cover previously discussed.

2. Stability

1-11. ***Bearing Capacity***.—Foundation bearing pressures for small structures are of small magnitude and will ordinarily be less than allowable bearing pressures for the various soil types [5].

Foundation treatment may be required, however, for low-density or expansive foundation soils. Ordinarily, hydrocompaction by ponding will sufficiently consolidate soils of low density. Overexcavation of the foundation soil and replacement with compacted, nonexpansive soil is a usual treatment for expansive soil foundations. The nonexpansive soil surcharges the underlying expansive soil and thereby can adequately reduce the foundation movement. Overexcavation is also sometimes used for foundation treatment of low-density soils.

When installing pipe in rock, the foundation should be overexcavated and replaced with a gravel or earth cushion to permit a more uniform bearing pressure for the pipe. This is essential to effectively utilize the design strength of the pipe, and to preclude unequal settlement and resultant cracking of pipe.

1-12. ***Sliding Coefficient***.—Any structure subjected to differential lateral pressures must capably resist the tendency to slide. Resistance to sliding is developed by shearing strength along the contact surface of the structure base and the foundation, or by shearing strength within the foundation material itself. Shearing strength developed by cohesion is omitted and only that developed by mechanical friction at the base and foundation interface is used for sliding analysis of small structures. An allowable sliding coefficient equal to 0.35 is used unless unusual soil conditions exist. This may be expressed as:

$$\frac{\Sigma H}{\Sigma N} = 0.35$$

where:

ΣH = summation of lateral forces acting parallel to the assumed sliding plane, and

ΣN = summation of forces, reduced by uplift, acting normal to the assumed sliding plane.

1-13. ***Overturning***.—To prevent overturning, the sum of the stabilizing moments must exceed the sum of the overturning moments on the structure. Checks and check-drop structures are perhaps the most critically loaded small canal structures subject to overturning. Maximum upstream and minimum downstream water surfaces subject these structures to unsymmetrical loads which tend to cause overturning.

The resultant of all forces acting on the structure should fall within the middle third of the structure base to provide safety against overturning. This location of the resultant also provides a more uniform bearing pressure on the foundation.

1-14. ***Percolation***.—All standardized canal structures have sufficient cutoff and structural lengths to provide a percolation factor of 2.5 or more. This is considered adequate for most soils to prevent piping of foundation materials from beneath or adjacent to small structures. Under ordinary operation, the maximum differential hydraulic head across the structure causing percolation is of short duration. For a more detailed discussion of percolation, see subchapter VIII C.

3. Hydraulics

1-15. ***Hydraulic Control***.—In the competent hydraulic design of any hydraulic structure it is necessary to first determine the location of the water surface control.

Hydraulic control is at the downstream end of a structure if the downstream water surface influences the height to which the upstream water surface must rise. If the downstream water surface does not influence the upstream water surface, the control is at the upstream end of the structure.

Upstream control for example will usually exist for a relatively short culvert conveying storm runoff under a small canal where the outlet channel water surface is several feet below the upstream pipe invert. In this instance, the downstream water surface does not control the upstream water surface; instead, the upstream water surface is controlled at the inlet end. The upstream pipe invert elevation, the size of pipe, and the assumed entrance loss are all factors which determine the height to which the upstream water surface must rise to discharge the flow.

Downstream hydraulic control exists for a properly designed inverted siphon when flow is at design capacity. The canal water surface at the downstream end of the siphon controls the upstream canal water surface. For partial discharges, hydraulic control is at the siphon inlet if the downstream water surface is low enough to permit the water to enter the siphon at minimum energy, which is at critical depth.

For many structures, location of the hydraulic control may be determined by inspection of the structure profile and the normal channel or canal water surfaces at each end of the structure. In other cases the Bernoulli theorem [8] should be used to determine the hydraulic control location. Beginning from a known downstream water surface, a Bernoulli equation can be written between pairs of consecutive points. If a Bernoulli balance is attainable between all pairs of points from the outlet through the inlet, downstream control exists. If, however, a Bernoulli balance is not attainable between two consecutive points, a hydraulic jump between the points is indicated and hydraulic control will be at the inlet. It may then be assumed that the water depth in the pipe at the inlet is at critical depth; that is, at minimum energy.

1-16. ***Hydraulic Head Losses.***—(a) *Friction.*—The Manning formula, [8] $V = \frac{1.486}{n} r^{2/3} s^{1/2}$, is used to determine friction losses in a canal system. For all monolithic concrete canal structures, a roughness coefficient n = 0.014 is used. Roughness (friction) coefficients used for pipe are n = 0.013 for precast concrete, asbestos cement, reinforced plastic mortar, and steel, while n = 0.024 is used for corrugated-metal pipe. A 20 percent reduction in the n value is used as a safety factor in the hydraulic analysis of excess energy dissipation for those structures which perform this function.

(b) *Transitions and Bends.*—Transitions are used to geometrically change the water prism shape and cause acceleration or deceleration of the flow. Losses associated with convergence and divergence of the water prism are discussed in chapter VII.

A change in the direction of flow causes bend losses. The magnitude of the loss is dependent on the water velocity, and the angle for a miter bend or the degree of curvature of a circular curve. Pipe bend losses are discussed in chapter VIII.

1-17. ***Discharge Coefficients.***—(a) *Orifice.*—For a conduit flowing full, the discharge coefficient, C, used in the orifice equation [8], $Q = CA\sqrt{2gH}$, indicates the magnitude of the assumed entrance loss. Values of C may be related to an entrance loss coefficient, K_o in the expression $K_o h_v$, by the relationship $K_o = \frac{1}{C^2} - 1$. Values commonly used are:

C	K_o
0.6	1.78
0.7	1.04
0.75	0.78
0.8	0.56
0.82	0.50

The value for C in the orifice equation is primarily dependent on the hydraulic entrance conditions. Subsequent chapters of canal structures using the orifice equation indicate an appropriate C value for a specific design.

(b) *Weir.*—Weirs are commonly used to measure the rate of flow of water. The basic weir equation [8] is $Q = CLH^{3/2}$. The appropriate value for the discharge coefficient, C, is primarily dependent on the geometric shape of the weir and contraction of the sheet of water flowing over the weir.

Values for C and equations for the more commonly used weirs are:

Weir	Weir equation
Rectangular suppressed	$Q = 3.33\ L\ H^{3/2}$
Rectangular contracted	$Q = 3.33\ (L - 0.2H)\ H^{3/2}$
Cipolletti	$Q = 3.367\ L\ H^{3/2}$.

For a comprehensive discussion of weirs, see chapter V.

4. Structural Considerations

1-18. ***Reinforced Concrete.***—(a) *Allowable Stresses.*—Standardized canal structures included herein, which show concrete thicknesses and size and spacing of reinforcement bars, were designed by the working stress method based on a concrete strength of 4,000 pounds per square inch (psi) at 28 days (f'_c), and reinforcement steel having a specified minimum yield strength of 60,000 psi (f_y). Allowable working stresses used were 1,800 psi compression (f_c) for concrete and 24,000 psi tension (f_s) for reinforcement steel. For many of the smaller standardized structures, nominal minimum concrete thicknesses and minimal reinforcement steel patterns control the design. In these cases the concrete and steel strengths indicated could be reduced without jeopardizing structural integrity.

Reinforced precast concrete pressure pipe was designed by the ultimate strength method using $f'_c = 4{,}500$ psi, $f_y = 40{,}000$ psi, and an LF = 1.8 as a load factor.

(b) *Minimum Reinforcement Requirements.*—The minimum reinforcement used for canal structures should be No. 4 bars (1/2-inch diameter) at 12-inch spacing when reinforcement is placed in a single layer, or where exposed faces of concrete are reinforced. In unexposed faces of concrete having two-layer reinforcement, the minimum reinforcement should be No. 4 bars at 18 inches.

The following criteria should be used to determine the cross-sectional area for temperature or minimum reinforcement. The percentages of reinforcement steel areas listed are percentages of the gross cross-sectional area of the concrete to be reinforced:

(1) Single-layer reinforcement.—For slabs with joint spacing not exceeding 30 feet and:

Not exposed to freezing temperatures or direct sun . . . . 0.25 percent

Exposed to freezing temperatures or direct sun . . . . 0.30 percent

For slabs with joint spacing greater than 30 feet and:

Not exposed to freezing temperatures or direct sun . . . . 0.35 percent

Exposed to freezing temperatures or direct sun . . . . 0.40 percent

Walls and other structural members should have a total percentage of horizontal reinforcement equal to the sum of the percentages required for both faces as determined for double-layer reinforcement.

(2) Double-layer reinforcement.—For joint spacing not exceeding 30 feet and with:

Face adjacent to earth . . . . 0.10 percent

Face not adjacent to earth nor exposed to freezing temperatures or direct sun . . . . 0.15 percent

Face not adjacent to earth but exposed to freezing temperatures or direct sun . . . . 0.20 percent

If a structural member exceeds 30 feet in any direction parallel to the reinforcement, an additional 0.05 percent of reinforcement steel area is required in that direction.

If a slab is fixed along any line, the dimension from the line of fixity to the free end is doubled to determine if the reinforcement requirement should be based on a length not exceeding 30 feet or a length greater than 30 feet.

Reinforcement spacing should not exceed three times the thickness of the member for temperature reinforcement and twice the thickness of the member for stress bars.

Other minimum requirements and general notes for designing, showing, and detailing reinforcement steel are indicated on figure 1-8. When using the illustrations of standardized canal structures presented herein as construction drawings, figure 1-8 should be used for general notes and minimum requirements for detailing reinforcement.

(c) *Minimum Wall Thickness.*—To provide ease of concrete placement and insure good bond between the reinforcement and concrete, the minimum concrete thickness of cantilever walls should be 1 inch per foot of height (5 inches minimum) for walls up to 8 feet high. For walls exceeding 8 feet in height, the minimum concrete thickness should be 8 inches plus 3/4 inch for each foot of wall height greater than 8 feet.

GENERAL NOTES

UNLESS OTHERWISE SHOWN ON THE DRAWINGS, THE DETAILS AND NOTES SHOWN ARE TYPICAL FOR ALL DRAWINGS THAT REFER TO THIS DRAWING

ABBREVIATIONS:

bf = bottom face	br = bottom row	bl = bottom layer
tf = top face	tr = top row	tl = top layer
nf = near face	nr = near row	ml = middle layer
ff = far face	fr = far row	ns = near side
ef = each face	er = each row	fs = far side
if = inside face	ir = inside row	es = each side
of = outside face	or = outside row	ew = each way
	mr = middle row	ec = each corner

spc. = space or spaces
eq. spc. = equally spaced, equal spaces
D = nominal diameter of reinforcing bar
uv = uniformly varying lengths of bars between lengths shown
cl. = clear
ctr. = center or centers

SYMBOLS:

Bars shown thus #8@12 or #6 @ $7\frac{1}{2}$, indicate a group of identical bars equally spaced.
An open circle at the end of a bar indicates a bend with the bar turned away from the observer.
A closed circle at the end of a bar indicates a bend with the bar turned towards the observer
Splices shown thus indicate a lapped splice, not a bend in the bar.

DIMENSIONS:

Dimensions are to the center lines of the bars unless otherwise shown.
Clear cover dimensions are marked "cl."

COVER:

Place the reinforcement so that the clear distance between face of concrete and nearest reinforcement is $1\frac{1}{2}$" for #5 bars and smaller and 2" for #6 bars and larger; except provide a clear distance from face of concrete placed against earth or rock of 2" where member thickness is 9" or less and 3" where member thickness is greater than 9", the clear distance being to the design dimension line.
Reinforcement paralleling construction joints should have a minimum of 2" clear cover.

PLACING:

Reinforcement at small openings (max. 18") in walls and slabs may be spread apart not more than $1\frac{1}{2}$ times the bar spacing.
Reinforcement may be adjusted laterally to maintain a clear distance of at least 1" between the reinforcement and keys, waterstops, anchor bolts, form ties, conduits, and other embedded material. In heavily reinforced areas relocation of the embedded material should be considered.
Bars should not be bent to greater than 6 to 1 slope.
Reinforcement parallel to anchor bolts or other embedded material should be placed to maintain a clear distance of at least $1\frac{1}{3}$ times the maximum size aggregate.

SPACING:

The first and last bars in walls and slabs, stirrups in beams, and ties in columns should start and end at a maximum of one half of the adjacent bar spacing.

STANDARD HOOKS:

Hooks should have 180° bends and extensions of 4-bar diameters but not less than $2\frac{1}{2}$" parallel to the main leg of the bar or 90° bends and extensions of at least 12-bar diameters. Hooks for stirrup and tie anchorage only should have either a 90° or 135° bend plus an extension of at least 6-bar diameters but not less than $2\frac{1}{2}$" at the free end of the bar. Radius of bend to be as specified in the table of pin diameters.
A bar (——) with a standard 180° hook on one end is referred to as an "A" bar.
A bar (——) with a standard 180° hook on both ends is referred to as a "B" bar.

BENT BARS:

Only billet steel or axle steel shall be used for bars to be bent.
Unless other radius bends are indicated on the design drawings, all reinforcement requiring bending shall be bent around a pin having the following diameter:

TABLE OF PIN DIAMETERS IN INCHES

BAR NO. (#)	3	4	5	6	7	8	9	10	11	14	18
Standard bends	$2\frac{1}{4}$	3	$3\frac{3}{4}$	6	7	8	$10\frac{1}{8}$	$11\frac{1}{2}$	$12\frac{3}{4}$	17	$22\frac{1}{2}$
Stirrup and tie bends	$1\frac{7}{8}$	$2\frac{1}{2}$	$3\frac{1}{8}$	$5\frac{1}{4}$	7	8	$10\frac{1}{8}$	$11\frac{1}{2}$	$12\frac{3}{4}$		

REINFORCEMENT DOWELS:

Dowels indicated on the drawing, such as #8 (d), should have an embedment equal to L_e and a projection equal to that required for lap splicing a bar of the same diameter.

PLAIN DOWELS:

Plain dowels across contraction joints should be smooth bars uniformly coated with a film of oil before concrete placement. Viscosity of the oil should have a SAE rating of not less than 250.

ACCESSORIES:

Bar supports, spacers, and other accessories are not shown on the design drawings. The recommendations of ACI 315 or other approved supporting systems may be used.

REFERENCES:

Unless otherwise shown follow the recommendations established by American Concrete Institute's "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 315).

REINFORCEMENT:

Reinforcement should conform to ASTM designation A615 Grade 60 or to ASTM designation A617 Grade 60.

NOTES TO DESIGNERS:

Splice lengths shown in the tables on this drawing are for Class B splices, spaced at least 6 inches on centers, ($0.8 \times 1.3\ L_d$) in accordance with ACI 318-71. Splices requiring lap lengths other than those for Class B must be detailed on the reinforcement design drawings.
Embedment length, L_e, in the above tables is based on "other" bars with a spacing of at least 6 inches or 6 bar diameters.

SPLICES:

Unless otherwise shown the minimum length of lap for splicing parallel bars shall be as given in the table.
Splices shall be staggered, to give 12 inches clear between ends of adjacent splices, if bars are spaced closer than 6 inches or 6 bar diameters.
Noncontact lap splices shall not be spaced farther apart than one-fifth the required length of lap or 6 inches.
When reinforcing bars of different size are to be spliced, the length of lap shall be governed by the smaller diameter bar.
Splices are to be made so that the given distances to face of concrete will be maintained.

SPLICE AND EMBEDMENT LENGTHS

$f'_c = 4000$ psi $f_y = 60,000$ psi

BAR SIZE		LENGTH OF LAPPED SPLICE IN INCHES		EMBEDMENT LENGTH
NO.	DIA. (IN.)	TOP BARS *	OTHER BARS	L_e (IN.)
3	0.375	13	12	12
4	0.500	18	12	12
5	0.625	22	16	12
6	0.750	26	19	14
7	0.875	33	24	18
8	1.000	44	31	24
9	1.128	55	40	30
10	1.270	70	50	39
11	1.410	86	62	47

* Top bars are horizontal bars in beams and slabs so placed that more than 12 inches of concrete is cast in the member below the bar.

CHAMFER:

Chamfer all exposed concrete edges $\frac{3}{4}$".

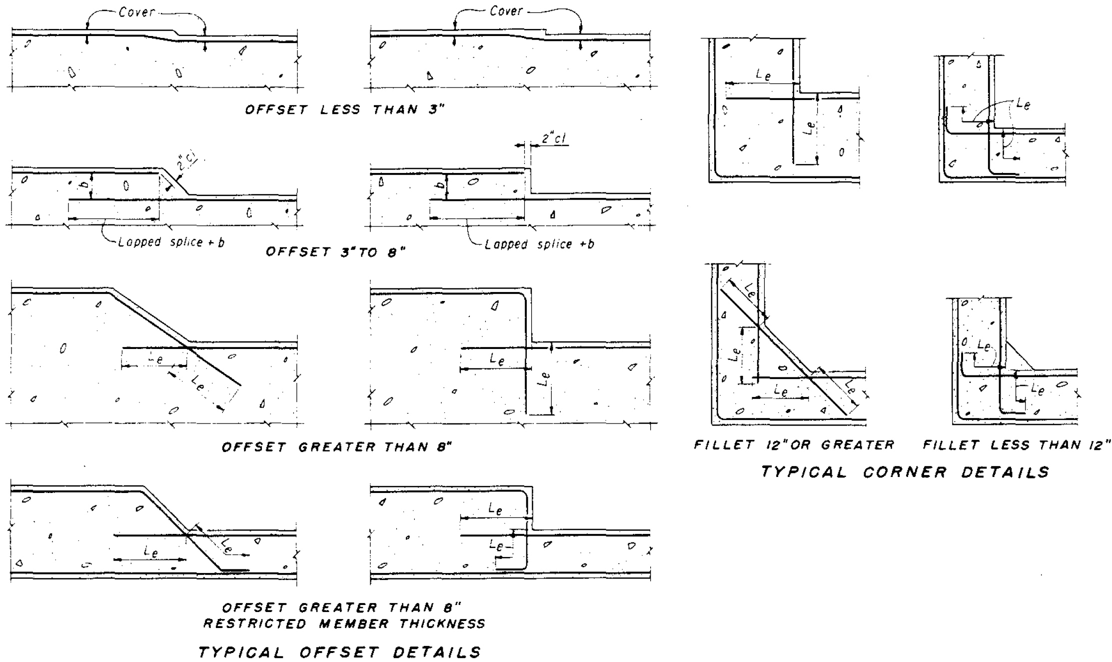
CONCRETE THICKNESS:

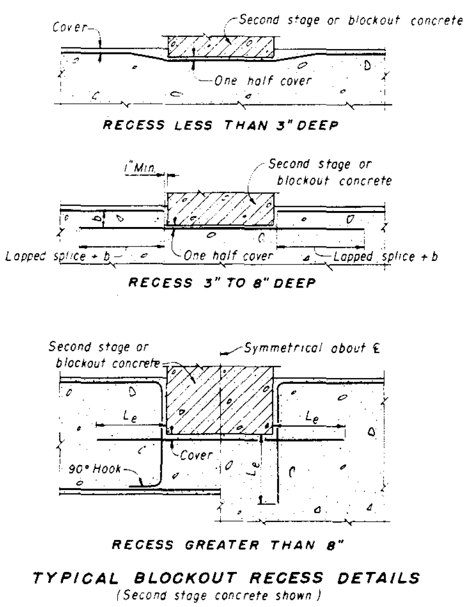
Concrete thickness to vary uniformly between dimensions shown.

DESIGN:

In this publication, concrete members were designed by the working stress method using $f'_c = 4000$ psi and $f_s = 24000$ psi.

Figure 1-8. **General notes and minimum requirements for detailing reinforcement (Sheet 1 of 2). 103-D-1194**





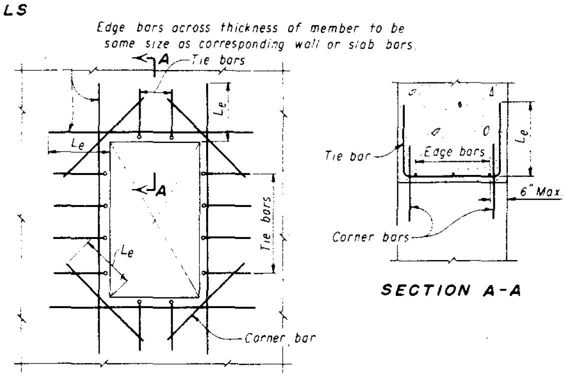


TABLE FOR REINFORCEMENT AROUND OPENINGS

MEMBER THICKNESS	TIE BARS	EDGE BARS	CORNER BARS
Less than 10"	None	1-ctr	1-#4 ctr.
10" thru 18"	None	2-(1-ef)	2-#4 (1 ef)
19" thru 36"	#4 @ 12	3-eq. spc	2-#6 (1 ef)
Over 36"	#6 @ 12	Spc @ 12	2-#8 (1 ef)

Omit edge and tie bars along sides of openings where dimension is less than 18".
Omit corner bars at sides of openings adjacent to floors, walls, or beams.
Corner bars required if either dimension of opening is greater than 18".
Use corner bars in face of recesses deeper than 4" if either dimension of recess is greater than 18".

ADDITIONAL REINFORCEMENT AROUND OPENINGS

Figure 1-8. **General notes and minimum requirements for detailing reinforcement (Sheet 2 of 2). 103-D-1194**

(d) *Fillets.*—Fillets are used to relieve stress concentrations, provide increased strength, facilitate concrete placement, and to accommodate removal of forms. For small canal structures, fillets are not ordinarily required for these purposes. However, 6-inch fillets which may be formed by excavation lines for the structure are commonly used at the junctions of cutoff walls and floor slabs. Fillets used at the base of cantilever walls are usually 3 inches for walls up to 8 feet tall.

(e) *Cutoffs.*—The primary purpose of cutoff walls is to increase the percolation path to prevent piping of foundation material and reduce percolation. Cutoffs should be used for structures in a concrete-lined canal as well as earth canals. Cutoffs also protect a structure from undermining, if excessive erosion should occur in an earth canal. Erosion protection of coarse gravel or riprap is frequently required at the ends of structures in an earth channel to control erosion. Criteria to determine if erosion protection is required, and the type of protection required, are discussed later in subchapter VII B and shown on figure 7-8. These criteria should be used where protection is indicated on structure illustrations. The passive resistance of earth behind a cutoff increases the resistance of the structure to sliding, although this resistance is usually omitted in the stability analysis. Criteria for lengths and thicknesses of cutoffs are also included in subchapter VII B.

1-19. ***Structural Steel and Welding.***—Structural steel used with small canal structures is primarily for stoplog and gate frame guides. Nominal size members are used to provide sufficient rigidity. Strength requirements for steel shapes used to fabricate safety devices such as handrail and ladders are also nominal. All dimensions and strengths of metalwork used in standard canal structures are included on the illustrations.

Many illustrations in subsequent chapters show welding requirements for metal parts. Explanations of the welding symbols and required welding procedures are included in bibliography reference [9].

5. *Canal*

1-20. ***Freeboard.***—Canal lining and canal banks are extended above the canal normal water surface as a safety measure to protect the conveyance system from overtopping. Freeboard provides for a canal water surface higher than normal which may be caused by sedimentation in the canal, temporary misoperation of the canal system, excess flows caused by storm runoff entering the canal through drain inlets, additional water depth resulting from a rougher friction coefficient than used for design, and waves produced by wind or surges which accompany sudden changes in flow.

Figure 1-9 shows the vertical distance which canal lining and canal banks should be extended above the canal normal water surface to provide adequate freeboard. Special requirements for canal bank freeboard at structures such as inverted siphons and drain inlets are discussed in the appropriate subchapters for each particular structure. Freeboard requirements for the canal structures are also discussed with each structure.

1-21. ***Winter Operation.***—Winter operation of a canal system where temperatures are subfreezing may require additional canal and structure freeboard to permit wintertime design capacity to flow under an ice cover. At subfreezing temperatures an ice cover readily forms when velocities are less than 2.2 feet per second. If velocities are fast enough to prevent formation of an ice cover, frazil ice may form, and if allowed to accumulate on racks at inlets to structures will cause backwater. Additional forces caused by expansion of the ice cover or by ice lenses in clayey foundation materials should be considered.

1-22. ***Profile Sheet.***—Many design examples of structures in this publication refer to a canal profile sheet. The development of the canal profile requires close coordination of data obtained from several different studies including: (1) canal design capacity, (2) canal water surface requirements for turnout deliveries, (3) canal location with respect to topography within and adjacent to the canal prism, (4) canal location to minimize disturbance or relocation of existing features, and (5) the canal cross section.

Data from canal plan and profile sheets are essential for design of the canal structures.

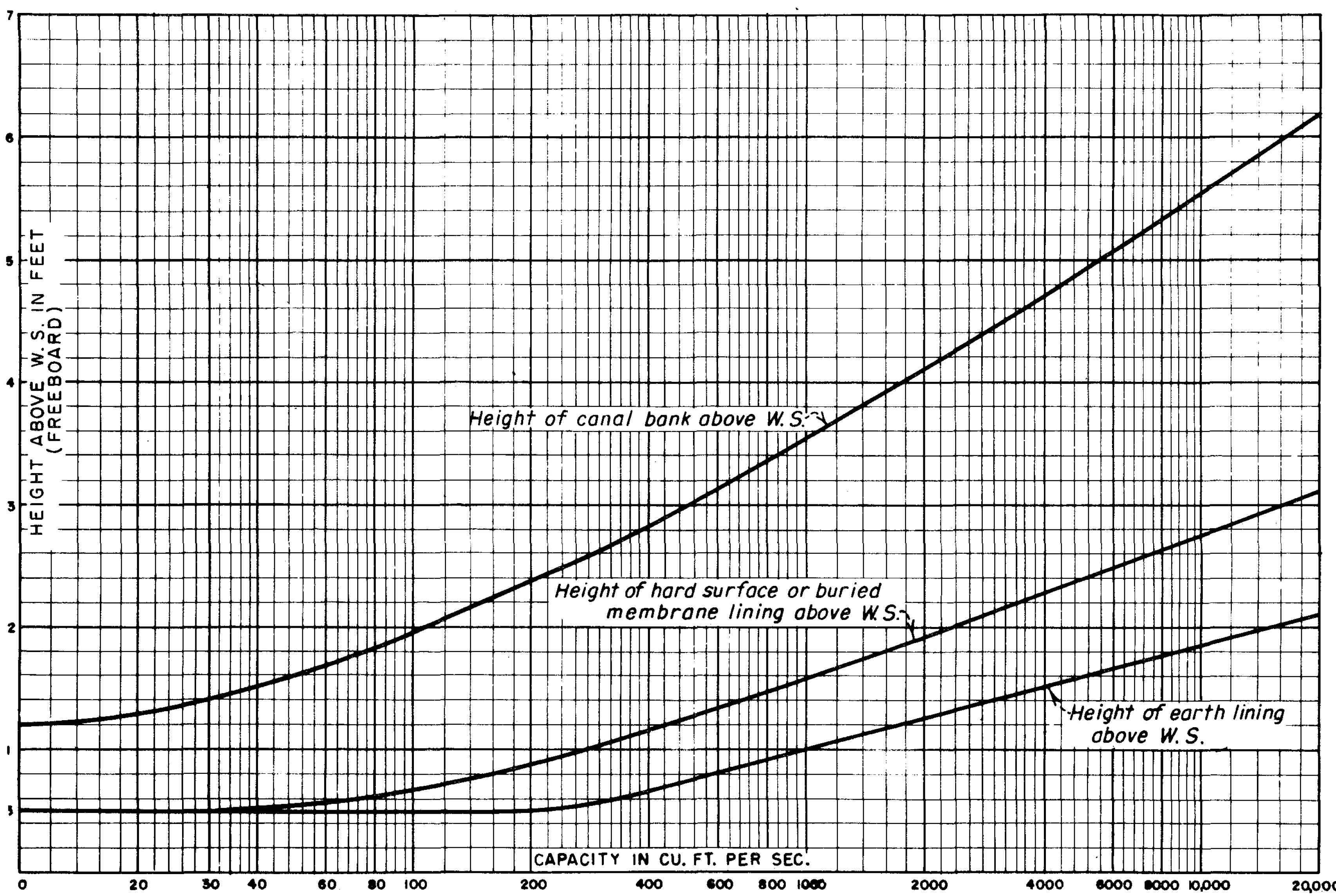


Figure 1-9. Freeboard for canal banks and freeboard for hard surface, buried membrane, and earth linings. 103-D-1195

6. *Forms for Structures*

1-23. ***Monolithic Concrete.***—It is sometimes advantageous to use "knock-down," reusable steel forms for monolithic concrete construction of small irrigation structures. A construction procedure that has been used successfully is to assemble the specially designed forms in an area remote from the construction site, place and tie the reinforcement steel in the forms, and then haul the unit to the jobsite. After carefully placing the structure-form into position, concrete is placed for the entire structure.

1-24. ***Precast Concrete.***—If a sufficient number of structures for a small canal system are identical, it may be economically advantageous to precast them in a yard and haul them to the jobsite for installation.

Figure 1-10 shows concrete being placed for a transition, and completed transitions in the precasting yard. Slings are frequently used to lower precast concrete structures into position at the jobsite, as shown in figure 1-11, for placing a precast concrete transition. Figure 1-12 shows an inplace, broken-back transition as a component part of a road crossing structure.

To provide a uniform bearing surface for precast concrete structures, a thin layer of mortar at least an inch thick should be placed on the foundation surfaces just prior to placing the structure. A mortar bedding for a precast concrete inlet to a turnout structure is shown in figure 1-13.

Standard structures discussed in this publication which require a deck for operational purposes, show either precast walk planks or a precast deck. Precast decks are used for the longer spans and are not mechanically connected to the basic structure. The bearing area at one end should have a very smooth surface to permit temperature-induced elongation or contraction of the deck without causing additional shear and bending moment stresses on the structure. Two asbestos sheets each having graphite on one side only and with the graphite surfaces in contact are sometimes used to provide such a surface.

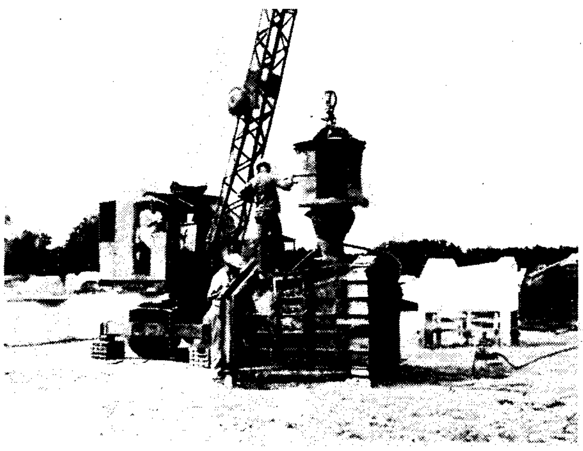


Figure 1-10. **Precast concrete transitions in various stages of completion. P-707-729-1763**

Figure 1-11. **Placing precast concrete transition. P-707-729-2134**

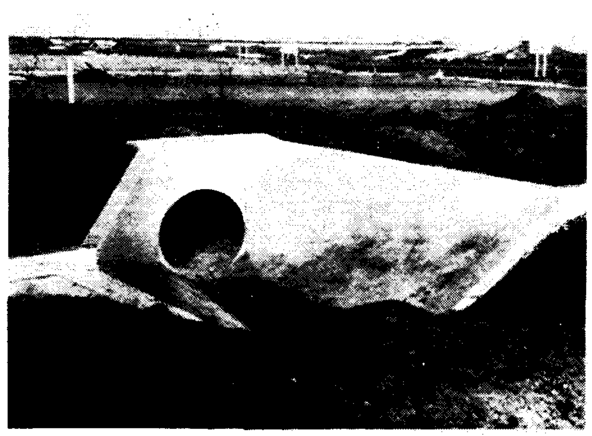


Figure 1-12. **In-place precast concrete transition, partially backfilled. PX-D-72830**

Figure 1-13. Precast concrete turnout inlet being lowered into position on mortar pad. P-707-729-1769

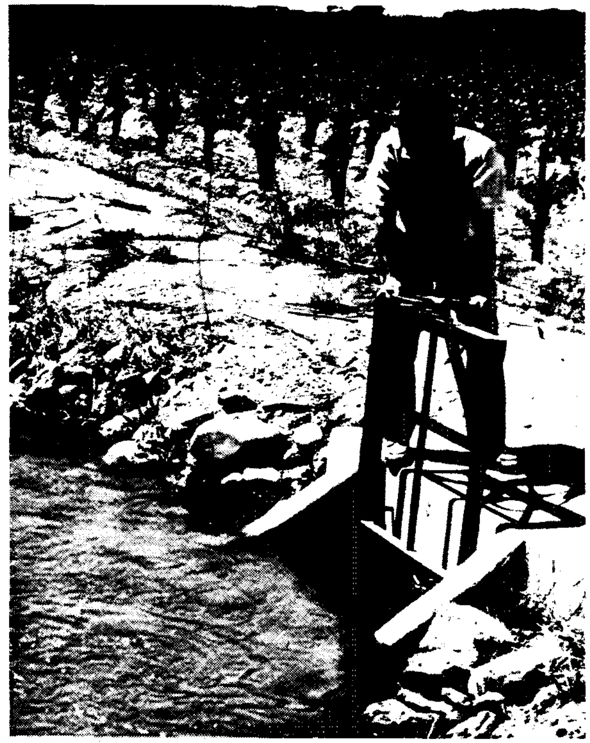


Figure 1-14. Slide gate being operated to control flow through turnout. P(F)-200-5717 NA

C. BIBLIOGRAPHY

1-25. ***Bibliography.***

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Conveyance Structures

A. GENERAL

2-1. ***Purpose.***—Conveyance structures are those structures such as road crossings, inverted siphons, drops, chutes, flumes, canals, and pipelines that are used to safely transport water from one location to another traversing various existing natural and manmade topographic features along the way. Canals and pipelines will not be discussed in detail in this publication and the technical dissertation of the other conveyance structures will be restricted to those having a capacity of 100 cfs or less.

B. ROAD CROSSINGS

R. B. YOUNG[1]

2-2. ***Purpose and Description.***—Road crossings are used to convey canal water under roads or railroads (fig. 2-1). Pipe conduit is generally used for these purposes. In accomplishing these objectives the road crossing conduit may have a straight line profile (see fig. 2-2), as discussed in these sections, or a profile like that shown in figure 2-4 with vertical bends as discussed later in subchapters II C, II E, and II F. Road crossings which have vertical bends in their profile function either as inverted siphons, drops, or chutes.

The straight line profile conduit (road crossing) is designed for flow with little or no internal hydrostatic pressure; that is, the hydraulic gradient is near or below the top of the pipe.

2-3. ***Application.***—Available hydraulic head and cost considerations usually determine the feasibility of using pipe for conveying water under a roadway or using a bridge over the waterway. Generally, for capacities up to 100

Figure 2-1. **Road crossing.**

[1]Civil Engineer, Hydraulic Structures Branch, Bureau of Reclamation.

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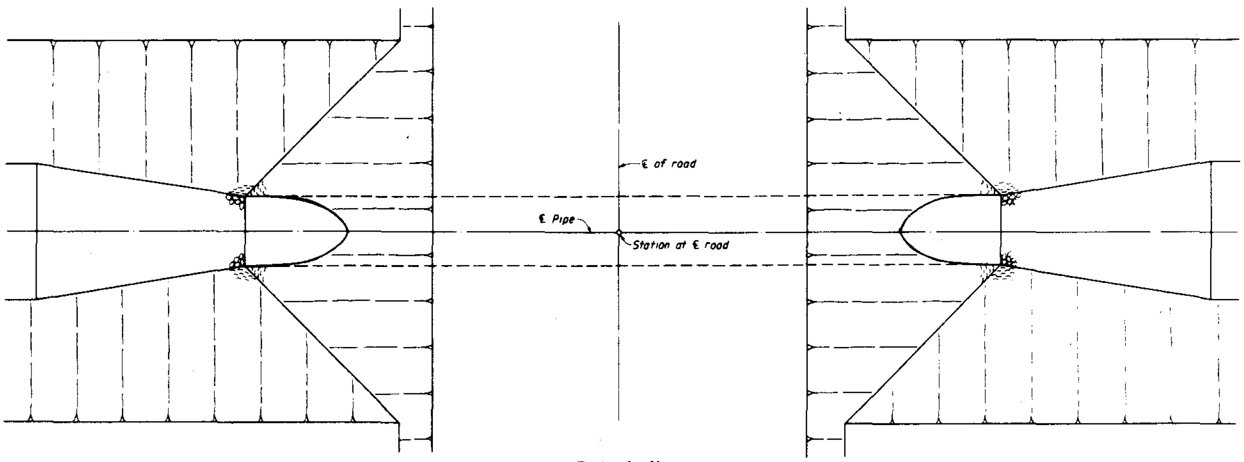
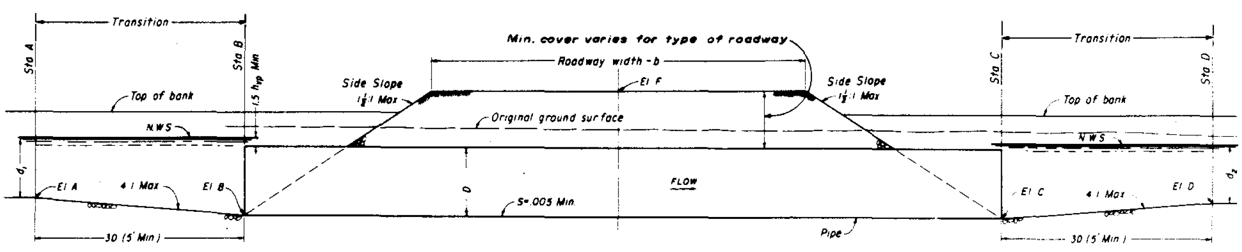
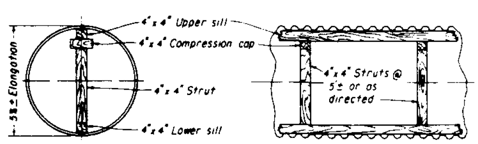


PLAN



LONGITUDINAL SECTION

PIPE DIAMETER SELECTION DATA					
MAX V=3.5 fps (EARTH TRANSITION)		MAX. V=5.0 fps (CONC. TRANSITION)		PIPE	
Q (cfs)		Q (cfs)		DIA.	AREA
FROM	INCLUDING	FROM	INCLUDING	(INCHES)	(SQ. FT.)
0	2.7	0	3.9	12	0.785
2.7	4.3	3.9	6.1	15	1.227
4.3	6.2	6.1	8.8	18	1.767
6.2	8.4	8.8	12.0	21	2.405
8.4	11.0	12.0	15.7	24	3.142
11.0	13.9	15.7	19.9	27	3.976
13.9	17.2	19.9	24.5	30	4.909
17.2	20.8	24.5	29.7	33	5.940
20.8	24.7	29.7	35.3	36	7.069
24.7	29.0	35.3	41.5	39	8.296
29.0	33.7	41.5	48.1	42	9.621
33.7	38.7	48.1	55.2	45	11.045
38.7	44.0	55.2	62.8	48	12.566
44.0	49.7	62.8	70.9	51	14.186
49.7	55.7	70.9	79.5	54	15.904
55.7	62.0	79.5	88.6	57	17.721
62.0	68.7	88.6	98.2	60	19.635
68.7	75.8			63	21.648
75.8	83.2			66	23.758
83.2	90.9			69	25.967
90.9	99.0			72	28.274



METHOD OF STRUTTING CORRUGATED METAL PIPE

No strutting required for pipes less than 54-inches in diameter. Pipe strutting to be installed before backfill is placed, and to remain in place until embankment is completed.

NOTES

Pipe may terminate in concrete structure or concrete transition at either end as directed.

Elevation F is approximate elevation of finished grade of road.

Pipe may be Corrugated Metal Pipe (CMP), Concrete Culvert Pipe (RCCP), or Precast Concrete Pressure Pipe (PCP).

Figure 2-2. Road crossing plan and section. 103-D-1253.

cfs, it is more economical to use pipe rather than a bridge. Bridge design is beyond the scope of this publication and consequently is not included.

2-4. *Advantages.*—Pipe road crossings are relatively economical, easily designed and built, and have proven a reliable means of conveying water under a roadway. Normally, canal erosion at the ends of the road crossing in earth canals is minor and can be controlled by transitions and riprap or gravel protection. Road crossings usually cause less roadway interference than a bridge both during and after construction. Pipe installation is sometimes accomplished by jacking the pipe through the roadway foundation (fig. 2-3). A road crossing permits both continuous roadbed and continuous side drain ditches which otherwise might drain into the canal.

2-5. ***Design Considerations.***—(a) *Pipe.*—The pipe may be steel corrugated-metal pipe (CMP), precast reinforced concrete culvert pipe (RCCP), asbestos-cement pressure pipe (AC), or precast reinforced concrete pressure pipe (PCP). Where watertightness of the pipe is of minor concern, the selection of either corrugated-metal pipe, asbestos-cement, or concrete pipe is often established by past experience. However, considerations involved with the selection include hydraulic efficiency, corrosion environment, and cost considerations.

The required gage (wall thickness) of CMP for a given height of earth cover and surcharge load from vehicles can be determined from manufacturers' tables. A typical set of tables is provided in bibliography reference [1].[2] If RCCP is used, this pipe design may also be determined from manufacturers' tables. A typical set of tables is provided in bibliography reference [2]. In Reclamation RCCP pipe is used for roadway crossings having little or no internal water pressure. Type B pipe joints (fig. 8-15) are generally used where RCCP pipe is permitted. However, type F joints may be substituted for type B joints. Type F joint design and gaskets for type F joints should be in accordance with the American Society for Testing and Materials [3] provided that the design of the joints in the pipe is such that the taper on the tongue and groove is not more than 3-1/2° measured from a longitudinal trace on the inside surface of the pipe.

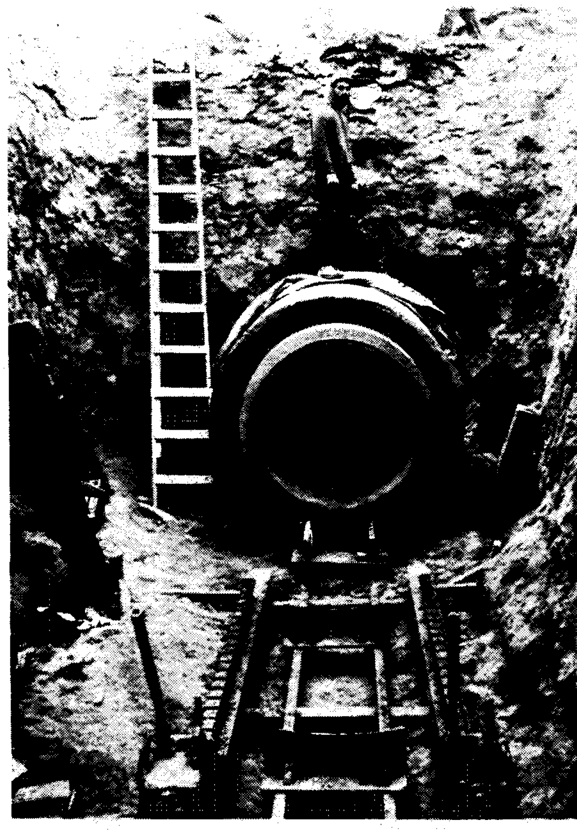


Figure 2-3. **Jacking and threading pipe under roadway. P-707-729-2007.**

All pipe subjected to internal pressure should have rubber gasket joints to insure positive watertightness. Under some roadways, watertight joints may be necessary irrespective of internal pressure. Precast concrete pressure pipe with type R joints (rubber gasket joints) or asbestos-cement pressure pipe with rubber gasket joints is used to insure watertightness. The minimum pressure pipe class permitted for each is B 25 [4] [5]. Selection of the appropriate pipe class is explained later in the discussion of Inverted Siphons, subchapter II C.

The hydraulic design of a road crossing pipe consists of selecting a pipe diameter that will

[2]Numbers in brackets refer to items in the bibliography, see section 2-38.

result in either: (1) a maximum allowable velocity of 3.5 feet per second for a pipe with earth transitions or, (2) a maximum allowable velocity of 5 feet per second for a pipe with concrete transitions or other concrete inlet and outlet structures. The maximum upstream invert elevation of the pipe is then determined by subtracting the pipe diameter and 1.5 times the velocity head of the pipe flowing full (diameter + 1.5 h_{v_p}) from the upstream normal water surface elevation in the canal. The pipe is set on a minimum slope of 0.005 from this upstream invert elevation. This provides a low point at the end of the pipe to facilitate draining should it become necessary.

The pipe hydraulic design should be examined to determine if the resulting earth cover from the top of the roadway to the top of the pipe meets the following minimum requirements:

(1) At all railroad and road crossings (except farm roads) a minimum of 3 feet of earth cover should be provided. If roadway ditches exist and are extended over the pipe, the minimum distance from the ditch invert to the top of the pipe should be 2 feet.

(2) At farm road crossings a minimum earth cover of 2 feet should be provided for both the roadway and the ditches. Farm roads are frequently ramped using 10 to 1 slopes (10 percent grade) when necessary to provide minimum earth cover requirements.

Another alternative available to the designer for achieving minimum cover requirements is to set the upstream pipe invert a distance greater than the pipe diameter plus 1.5 h_{v_p} below the upstream normal water surface elevation. However, the maximum vertical distance from the canal invert to the pipe invert should not exceed one-half the pipe diameter, except where a control structure is required. Pipe with bends in its profile to provide the required earth cover, or sagged for any other reason, is discussed with Inverted Siphons, subchapter II C.

Roadway widths and side slopes at the crossings should match existing roadway widths and side slopes, or as otherwise specified. Side slopes should not be steeper than 1-1/2 to 1.

(b) *Transitions.*—Transitions are generally used both at the inlet and outlet of structures. An accelerating water velocity usually occurs at the inlet of a structure and a decelerating velocity at the outlet. Transitions reduce head losses and prevent canal erosion by making the velocity changes less abrupt. Concrete, earth, and combination concrete-earth transitions are used for this purpose.

The following road crossings require either a concrete transition or some type of concrete control structure at the inlet and a concrete transition at the outlet:

All railroad and state highway crossings.

All crossings with 36-inch-diameter pipe or larger.

All pipe crossings with velocities in excess of 3.5 feet per second discharging into an earth canal.

Standardization of concrete transitions is a means of reducing costs. This is accomplished by a transition being designed for a range of canal and structure conditions. As might be expected, if concrete transitions are standardized, the base width and invert will seldom match that of the canal. Therefore, additional transitioning is then accomplished by an earth transition for an earth canal and by transitioning concrete lining for a concrete-lined canal. For relatively short structures, such as road crossings, it is generally more economical to omit concrete transitions if established design criteria will permit the omission. For the design of transitions see the discussion on Transitions, subchapter VII A.

If there is a need for controlling the water surface elevation upstream from the road crossing, a check inlet or a control inlet is used. (See discussion of Check and Pipe Inlet and Control and Pipe Inlet subchapters III F and III G.) If one of these structures is required, it is usually economically desirable to also use a concrete outlet structure and size the pipe based on a maximum velocity of 5 feet per second.

(c) *Pipe Collars.*—Pipe collars may be required to reduce the velocity of the water moving along the outside of the pipe or through the surrounding earth thereby preventing removal of soil particles at the point of emergence. Any road crossing where the canal water surface is significantly higher than a potential point of relief for the percolating water, should be examined to determine if collars are required. Pipe collars may also be necessary to discourage rodents from burrowing along the pipe. A detailed discussion of the design of pipe collars and cutoffs as related to percolation may be found in subchapter VIII C.

(d) *Canal Erosion Protection.*—Protection is often used adjacent to structures in earth canals where erosion may occur. For design of protection see the discussion on Protection, subchapter VII B.

2-6. ***Design Example*** *(see fig. 2-2).*—(a) *Given*:

(1) Type of waterway = earth canal.

(2) Type of roadway = farm road.

(3) Canal Q = 15 cfs.

(4) El. A = 5406.52 (from a profile sheet).

(5) d_1 = 1.58 ft. (normal depth at Sta. A).

(6) NWS at Sta. A = El. A + d_1 = 5406.52 + 1.58 = 5408.10.

(7) El. D = 5406.22 (from a profile sheet).

(8) d_2 = 1.58 ft. (normal depth at Sta. D).

(9) NWS at Sta. D = El. D + d_2 = 5406.22 + 1.58 = 5407.80.

(10) F (water surface differential)
= Upstream canal NWS El. – Downstream canal NWS El.
= 5408.10 – 5407.80
= 0.30 ft.

(11) Width of roadway = 18 ft.

(12) Side slopes of roadway = 1-1/2:1.

(13) El. top of roadway = El. F = 5411.00.

(14) Control structure at inlet not required.

(b) *Determine:*

(1) Pipe size (see table on fig. 2-2).

(2) Need for concrete transitions.

(3) Pipe type: CMP, RCCP, or PCP.

For a pipe discharge of 15 cfs, the table shows that either a 24-inch-diameter pipe with concrete transitions (max. V = 5 fps) or a 30-inch-diameter pipe with earth transitions (max. V = 3.5 fps) will be hydraulically acceptable. Since the crossing is for a farm road and both diameters are less than 36 inches, either the 24-inch-diameter pipe with concrete transitions or the 30-inch-diameter pipe with earth transitions may be used. Material costs and other considerations previously discussed will determine which pipe diameter to select, and in addition whether the pipe should be CMP, RCCP, or PCP. In this example the 30-inch-diameter concrete culvert pipe, class III with type B joints, and with earth transitions will be used.

(4) Hydraulic properties of 30-inch-diameter pipe for Q of 15 cfs:

$$A = \text{area of pipe} = 0.785 \times \text{dia.}^2 = 4.91 \text{ ft.}^2$$

$$V = \text{velocity in pipe} = \frac{Q}{A} = \frac{15}{4.91} = 3.06 \text{ f.p.s.}$$

$$h_{v_p} = \text{velocity head in pipe} = \frac{V^2}{2g} = \frac{3.06^2}{64.4} = 0.15 \text{ ft.}$$

where g = gravitational acceleration

$$wp = \text{wetted perimeter} = \pi \times \text{dia.} = 7.85 \text{ ft.}$$

$$R = \text{hydraulic radius} = \frac{A}{wp} = \frac{4.91}{7.85} = 0.63 \text{ ft.}$$

$$n = \text{assumed roughness coefficient}^3 = 0.013$$

$$s_f = \text{friction slope of pipe} = \left(\frac{1}{2.2r^{4/3}}\right) n^2 V^2 = 0.00133$$

(from Manning's equation [6])

Pipe hydraulic properties may also be taken from table 8-1.

(5) El. B = NWS El. at A – (pipe diameter + 1.5 h_{v_p}) = 5408.10 – (2.50 + 0.22) = 5405.38.

(6) Approximate length of pipe (see fig. 2-2) = 1.5 (El. F – El. B) x 2 + roadway

[3] See section 1-16.

width = 3(5411.00 – 5405.38) + 18 = 34.9 ft. The 1.5 represents a 1-1/2 to 1 side slope. If the side slope is 2 to 1 use 2 instead of 1.5 in the previous equation.

(7) Drop in pipe = length of pipe x slope of pipe = 34.9 x 0.005 = 0.17 ft

(8) El. C = El. B – drop in pipe
= 5405.38 – 0.17 = 5405.21.

(9) Length of earth transitions = 3 x dia. of pipe = 3 x 2.5 = 7.5 ft.

(10) Drop in transitions

Upstream = El. A – El. B
= 5406.52 – 5405.38
= 1.14 ft.

Downstream = El. D – El. C
= 5406.22 – 5405.21
= 1.01 ft.

(11) Assume losses in road crossing are 1.5 pipe velocity heads for inlet and outlet loss combined plus pipe friction loss, or

$$1.5\, h_{v_p} + h_f = 0.22 + 35 \times 0.00133 = 0.27 \text{ ft.}$$

where h_f = length of pipe x friction slope.

(12) *Protection.*–Use figure 7-8 to determine if protection is required. If required, select type, length, and height of protection.

For d_1 or d_2 = 1.58 ft. and for road crossings without concrete transitions:

Inlet protection = none

Outlet protection = 12-in. coarse gravel (type 2)

Length = 4d or 5 ft. min. = 4 x 1.58 ft. = 6.3 ft.

Because the transition length of 7.5 feet is not much greater than the required protection length, extend protection for full length of transition. Extend protection to 12 inches above canal water surface.

(c) *Check of Design.*–

(1) Compare computed losses with the loss provided on profile sheet. Computed losses = 0.27 foot. Loss provided, F = 0.30 foot. The excess head of 0.03 foot (0.30 – 0.27) provided on the profile sheet is inconsequential and the hydraulic design is considered adequate.

(2) *Transition slopes.*–To reduce turbulence and provide for relatively smooth transitioning of the water prism, the maximum length to drop ratio of 4 to 1 is used. The design ratio of the upstream transition is 7.5 to 1.14 or 6.6 to 1. This is flatter than 4 to 1 and therefore within the criteria limits. By inspection, the slope of the downstream transition is also satisfactory.

(3) *Cover on pipe.*–Minimum earth cover for farm road crossing is 2 feet. Approximate cover on pipe = El. F – (El. B + dia. of pipe) = 5411.00 – (5405.38 + 2.50) = 3.12 feet. As 3.12 feet is greater than the minimum required 2.0 feet the cover is satisfactory.

If concrete transitions are required for a road crossing the inlet and outlet transitions will usually be identical. Refer to subchapter VII A and design example of inverted siphons subchapter II C for procedure used in design of type 1 concrete transitions.

C. INVERTED SIPHONS

R. B. YOUNG[1]

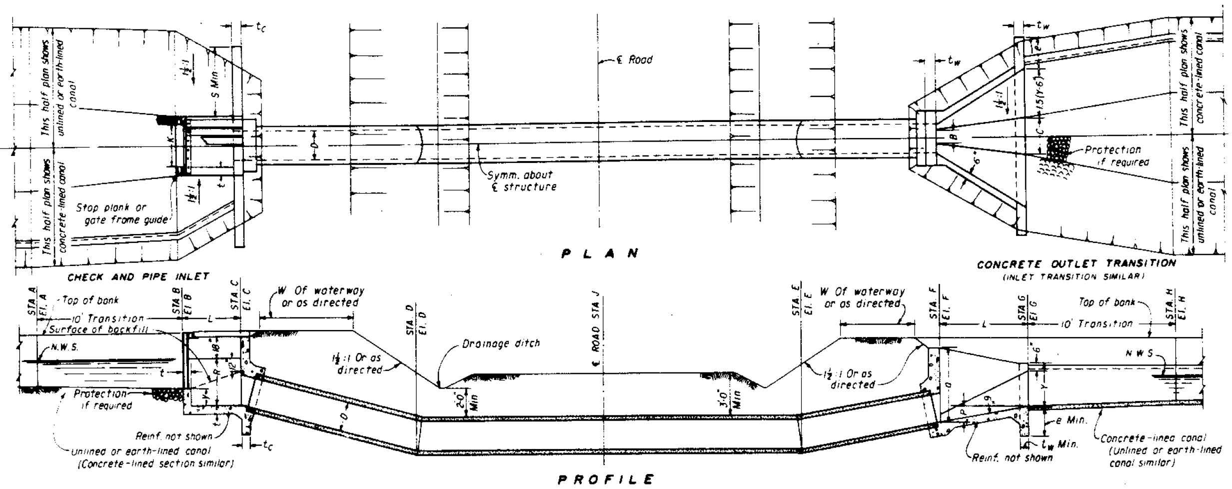
2-7. ***Purpose and Description.***–Inverted siphons figures 2-4, 2-5, and 2-6 (sometimes called sag pipes or sag lines) are used to convey canal water by gravity under roads, railroads, other structures, various types of drainage channels, and depressions. A siphon is a closed conduit designed to run full and under pressure. The structure should operate without excess head when flowing at design capacity.

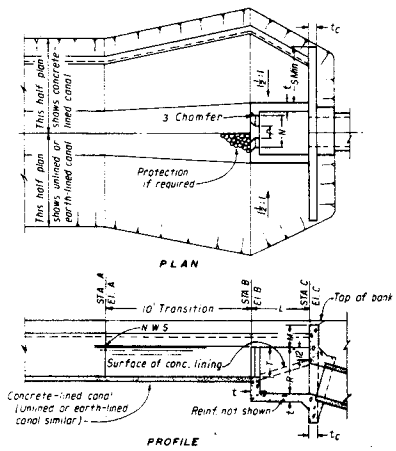
Closed conduits with excess head are discussed in subchapters II E Drops and II F Chutes.

Closed conduits with straight profiles under roadways and railroads may also function as inverted siphons with internal pressure.

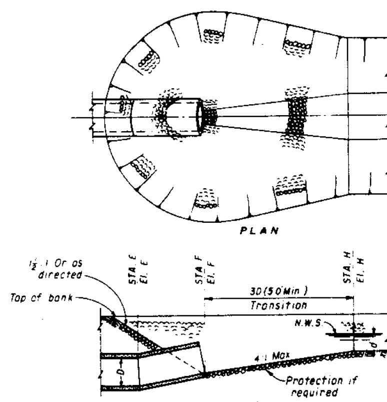
2-8. ***Application.***–Economics and other considerations determine the feasibility of using a siphon or another type of structure to

[1]Op. cit., p. 19.





CONTROL AND PIPE INLET



EARTH OUTLET TRANSITION
(INLET TRANSITION SIMILAR)

PIPE DIAMETER SELECTION DATA							
MAX. V=3.5fps (EARTH TRANSITION)		MAX. V=5.0fps (CONC. TRANSITION)		MAX. V=10.0fps (CONC. TRANSITION)		PIPE	
Q (cfs)		Q (cfs)		Q (cfs)		DIA.	AREA
FROM	INCLUDING	FROM	INCLUDING	FROM	INCLUDING	(INCHES)	(SQ. FT.)
0	2.7	0	3.9	0	7.9	12	0.785
2.7	4.3	3.9	6.1	7.9	12.3	15	1.227
4.3	6.2	6.1	8.8	12.3	17.7	18	1.767
6.2	8.4	8.8	12.0	17.7	24.1	21	2.405
8.4	11.0	12.0	15.7	24.1	31.4	24	3.142
11.0	13.9	15.7	19.9	31.4	39.8	27	3.976
13.9	17.2	19.9	24.5	39.8	49.1	30	4.909
17.2	20.8	24.5	29.7	49.1	59.4	33	5.940
20.8	24.7	29.7	35.3	59.4	70.7	36	7.069
24.7	29.0	35.3	41.5	70.7	83.0	39	8.296
29.0	33.7	41.5	48.1	83.0	96.2	42	9.621
33.7	38.7	48.1	55.2			45	11.045
38.7	44.0	55.2	62.8			48	12.566
44.0	49.7	62.8	70.9			51	14.186
49.7	55.7	70.9	79.5			54	15.904
55.7	62.0	79.5	88.6			57	17.721
62.0	68.7	88.6	98.2			60	19.635
68.7	75.8					63	21.648
75.8	83.2					66	23.758
83.2	90.9					69	25.967
90.9	99.0					72	28.274

NOTES

Locations of vertical P.I.'s at D and E are to be considered approximate only and may be adjusted to fit actual laying length of pipe.
Stations and elevations refer to invert unless otherwise shown.

Figure 2-4. **Inverted siphon plan and section. 103-D-1254.**

accomplish the previous objectives. The use of an elevated flume would be an alternative to a siphon crossing a depression, drain channel or another manmade channel. The use of a bridge over a canal would be an alternative to a siphon under a road or a railroad. Generally, for capacities up to 100 cfs, it is more economical to use a siphon rather than a bridge. Bridge design is beyond the scope of this publication and consequently is not included.

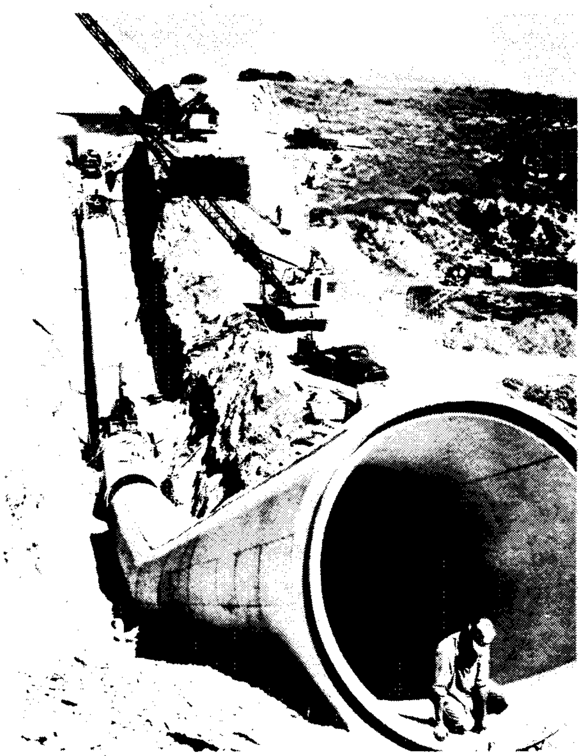


Figure 2-5. **Inverted siphon under construction. P-328-701-7259A.**

2-9. ***Advantages and Disadvantages of Inverted Siphons.***—Inverted siphons are economical, easily designed and built, and have proven a reliable means of water conveyance. Normally, canal erosion at the ends of the siphon is inconsequential if the structures in earth waterways have properly designed and constructed transitions and erosion protection.

Costs of design, construction, and maintenance are factors that may make an inverted siphon more feasible than another structure that might be used for the same purpose. There may be, however, instances where the value of the head required to operate a siphon may justify the use of another structure such as a bridge.

An inverted siphon may present a hazard to life, especially in high population density areas. See chapter IX for safety features.

2-10. ***Structure Components.***—(a) *Pipe.*—The closed conduits discussed in this publication are generally pipe. All pipe subjected to internal pressure should have watertight joints. Precast reinforced concrete pressure pipe (PCP), asbestos-cement pressure pipe (AC), or reinforced plastic mortar pressure pipe (RPM), all with rubber gasket joints, are used to insure watertightness. For heads up to 150 feet precast reinforced concrete pressure pipe is most frequently used but any of the above types may be used depending on their availability and cost considerations.

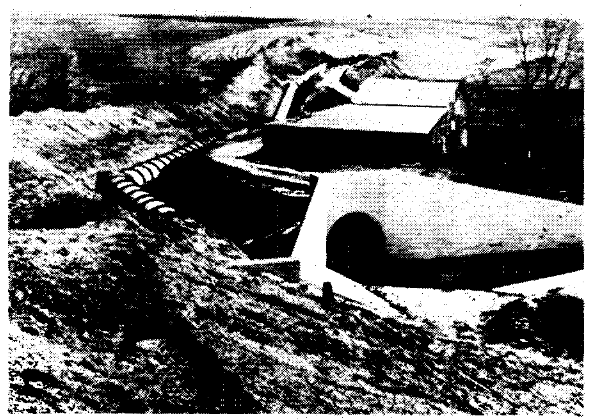


Figure 2-6. **Inverted siphon inlet transition and 60-inch-diameter precast concrete pipe. P 499-700-520.**

These pressure pipes are classed as to their capacity to withstand external loads of cover and wheel (equivalent earth cover) and internal hydrostatic head measured to the centerline of the pipe. Designations of A, B, C, and D represent 5, 10, 15, and 20 feet of cover respectively, while the associated number such as 25, 50, 75, 100, 125, and 150 represents feet of hydrostatic head. As an example, C 50 would be pressure pipe for 15-foot maximum cover and 50-foot maximum head.

Additional information regarding pipe, pipe joints, pipe bends, and other pipe appurtenances may be found in chapter VIII.

The pipe profile is determined in such a way as to satisfy certain requirements of cover, pipe slopes, bend angles, and submergence of inlet and outlet. Pipe cover requirements are:

(1) At all siphons crossing under roads other than farm roads and siphons crossing under railroads, a minimum of 3 feet of earth cover should be provided. Farm roads require only 2 feet of earth cover and are frequently ramped using 10 to 1 slopes (10 percent grade) when necessary to provide minimum cover requirements. If roadway ditches exist and are extended over the pipe, the minimum distance from the ditch to the top of the pipe should be 2 feet.

(2) At siphons crossing under cross-drainage channels, a minimum of 3 feet of earth cover should be provided unless studies indicate more cover is required because of projected future retrogressions of the channel.

(3) At siphons crossing under an earth canal, a minimum of 24 inches of earth cover should be provided.

(4) At siphons crossing under a lined canal, a minimum of 6 inches of earth cover should be provided between the canal lining and the top of pipe.

Roadway widths and side slopes at road and railroad siphon crossings should match existing roadway widths and side slopes, or as otherwise specified. Side slopes should not be steeper than 1-1/2 to 1.

Pipe slopes should not be steeper than 2 to 1 and should not be flatter than a slope of 0.005.

Changes in PCP pipe grade and alinement (bends) may be made with precast elbows, with beveled end pipe, by miter cutting pipe, or by pulling joints. Changes in AC and RPM pipe grade and alinement may be made by miter cutting pipe or by pulling joints. See chapter VIII for further information on bends.

(b) *Transitions.*—Transitions are nearly always used at the inlet and outlet of a siphon to reduce head losses and prevent canal erosion in unlined canals by causing the velocity change between the canal and pipe to be less abrupt. Concrete, earth, or a combination of concrete and earth transitions are used for this purpose.

The following siphons require either a concrete inlet transition or some type of concrete inlet control structure, and a concrete outlet transition:

All siphons crossing railroads and state highways.

All 36-inch-diameter and larger siphons crossing roads.

All siphons in unlined canals with water velocities in excess of 3.5 feet per second in the pipe.

Standardization of concrete transitions is a means of reducing costs. This is accomplished by having a single transition cover a range of canal and structure conditions. The base width and invert of standardized transitions will seldom match those of the canal. Additional transitioning is then accomplished with an earth transition where earth canals are involved and with a concrete lining transition where concrete-lined canals are involved.

For relatively short structures, such as siphons crossing roads, it is frequently more economical to omit concrete transitions even though the length of pipe will increase and size of pipe and protection may also increase. For further discussion on Transitions see chapter VII.

If there is a need for controlling the water surface elevation upstream from the siphon, a check and pipe inlet or a control and pipe inlet is used. (See discussion of Check and Pipe Inlet and Control and Pipe Inlet, subchapters III F and III G.)

(c) *Pipe Collars.*—Pipe collars are not normally required on siphons but they may be needed to reduce the velocity of the water moving along the outside of the pipe or through the surrounding earth thereby preventing removal of soil particles (piping) at the point of emergence. Pipe collars may also be necessary to discourage rodents from burrowing along the pipe. A detailed discussion for design of pipe collars and cutoffs as related to percolation may be found in chapter VIII.

(d) *Blowoff Structures.*—Blowoff structures are provided at or near the low point of relatively long inverted siphons to permit draining the pipe for inspection and maintenance or wintertime shutdown. Essentially the blowoff structure consists of a valved steel pipe tapped into the siphon barrel. Blowoffs may also be used in an emergency in conjunction with wasteways for evacuating water from canals. Short siphons are usually dewatered when necessary by pumping from either end of the siphon. If annual wintertime draining is not required, breaking into pipe smaller than 24-inch diameter for emergency draining is an economical alternative to providing a blowoff.

A manhole is often included with a blowoff on long siphons 36 inches and larger in diameter to provide an intermediate access point for inspection and maintenance.

A detailed discussion for design of blowoff structures and manholes may be found in chapter VIII.

(e) *Canal Freeboard and Erosion Protection.*—The canal bank freeboard upstream from siphons should be increased 50 percent (1.0 foot maximum) to prevent washouts at these locations due to more storm runoff being taken into the canal than anticipated or by improper operation. The increased freeboard should extend a distance from the structure such that damage caused by overtopping the canal banks would be minimal; but in any event a minimum distance of 50 feet from the structure.

Erosion protection is often used adjacent to siphons in earth canals. A discussion of Protection is presented in chapter VII.

(f) *Wasteways.*—Wasteways are often placed upstream from a siphon for the purpose of diverting the canal flow in case of emergency. For design of wasteways see the discussion on Wasteways, subchapter IV B.

(g) *Safety Features.*—Safety measures must be taken near siphons to protect persons and animals from injury and loss of life. Safety features are discussed in chapter IX.

2-11. ***Hydraulic Design Considerations.***—Available head, economy, and allowable pipe velocities determine the size of the siphon pipe. Thus, it is necessary to assume internal dimensions for the siphon and compute head losses such as entrance, pipe friction, pipe bend, and exit. The sum of all the computed losses should approximate the difference in energy grade elevation between the upstream and downstream ends of the siphon (available head).

In general, siphon velocities should range from 3.5 to 10 feet per second, depending on available head and economic considerations.

The following velocity criteria may be used in determining the diameter of the siphon:

(1) 3.5 feet per second or less for a relatively short siphon with only earth transitions provided at entrance and exit.

(2) 5 feet per second or less for a relatively short siphon with either a concrete transition or a control structure provided at the inlet and a concrete transition provided at the outlet.

(3) 10 feet per second or less for a relatively long siphon with either a concrete transition or a control structure provided at the inlet and a concrete transition provided at the outlet.

The velocity or pipe size of a long siphon is of particular importance, economically, because a slight change in pipe size can make a great change in the structure cost.

Head losses which should be considered are as follows:

(1) Convergence loss in the inlet transition.

(2) Check structure losses when a check is installed in the inlet.

(3) Control structure losses when a control is installed in the inlet.

(4) Friction and bend losses in the pipe.

(5) Divergence loss in the outlet transition.

(6) Transition friction losses are usually ignored for the size of structures in this publication.

(7) Convergence and divergence head losses in earth transitions when required between the canal and concrete transition are usually small and are usually ignored.

The total computed head loss is usually increased by 10 percent as a safety factor to insure against the possibility of the siphon causing backwater in the canal upstream from the siphon.

The hydraulic head loss in a transition is dependent on the difference of the velocity heads in the canal and the normal to centerline section of the closed conduit. Coefficients of velocity head considered adequate for determining head losses in a broken-back type of transition are 0.4 for the inlet and 0.7 for the outlet, therefore the losses would be $0.4\Delta h_v$ for inlet and $0.7\Delta h_v$ for the outlet transitions.

Coefficients of velocity head considered adequate for determining head losses in earth transitions from the canal to a pipe are 0.5 for the inlet and 1.0 for the outlet. Therefore, the losses would be $0.5\Delta h_v$ for the inlet and $1.0\Delta h_v$ for the outlet transitions.

For minimum hydraulic loss, it is desirable to provide a seal of $1.5\Delta h_v$ with 3-inch minimum at pipe inlet and no submergence at the pipe outlet. The seal is equal in height to the vertical drop from the normal canal water surface to the top of the siphon opening. If the siphon has both upstream and downstream concrete transitions it may be economically desirable to construct the downstream transition the same as the upstream transition.

If the outlet seal is greater than one-sixth the height of opening at the outlet, the head loss should be computed on the basis of a sudden enlargement and the loss for both earth and concrete outlet transitions would be $1.0\Delta h_v$.

For additional discussion on Transitions see chapter VII.

If there is a check and pipe inlet or a control and pipe inlet for the siphon see subchapters III F and III G for their hydraulic design.

Pipe friction losses are determined by using Manning's formula as is explained in section 2-13 or by using table 8-1.

Pipe bend losses are determined by using figure 8-1 as is explained in chapter VIII.

Special hydraulic considerations must be given to the inlets on long siphons where, under certain conditions, the inlet will not become sealed. On long siphons, such conditions may result when the canal is being operated at partial flows (flows less than design flow) or at full design flow when the actual coefficient of friction is less than assumed in design. Under such conditions, a hydraulic jump occurs in the pipe and may cause blowback and very unsatisfactory operating conditions. Figure 2-7, which is self-explanatory, should be used to determine proper performance of inlets to long siphons. Pipe slope or diameter should be changed to meet the requirements noted on the figure.

Another way of solving the air problem is to place properly designed air vents at locations where air might accumulate. This procedure is ordinarily used only as a remedial measure for an existing siphon with blowback problems. See discussion on Air Vents in chapter VIII.

2-12. ***Design Procedure.***—Steps required for design of a siphon include the following:

(1) Determine what inlet and outlet structures are required, and the type and approximate size of pipe.

(2) Make a preliminary layout of the siphon profile (siphon and required inlet and outlet structures) using the existing ground line, the canal properties, and the canal stations and elevations at the siphon ends (fig. 2-8). This layout should provide pipe requirements of cover, slope, bend angles, and provide pipe submergence requirements at transitions, check and pipe inlets, or control and pipe inlets.

(3) Compute the siphon head losses in this preliminary layout. If the head losses as computed are in disagreement with the available head, it may be necessary to make some adjustment such as pipe size or even the canal profile.

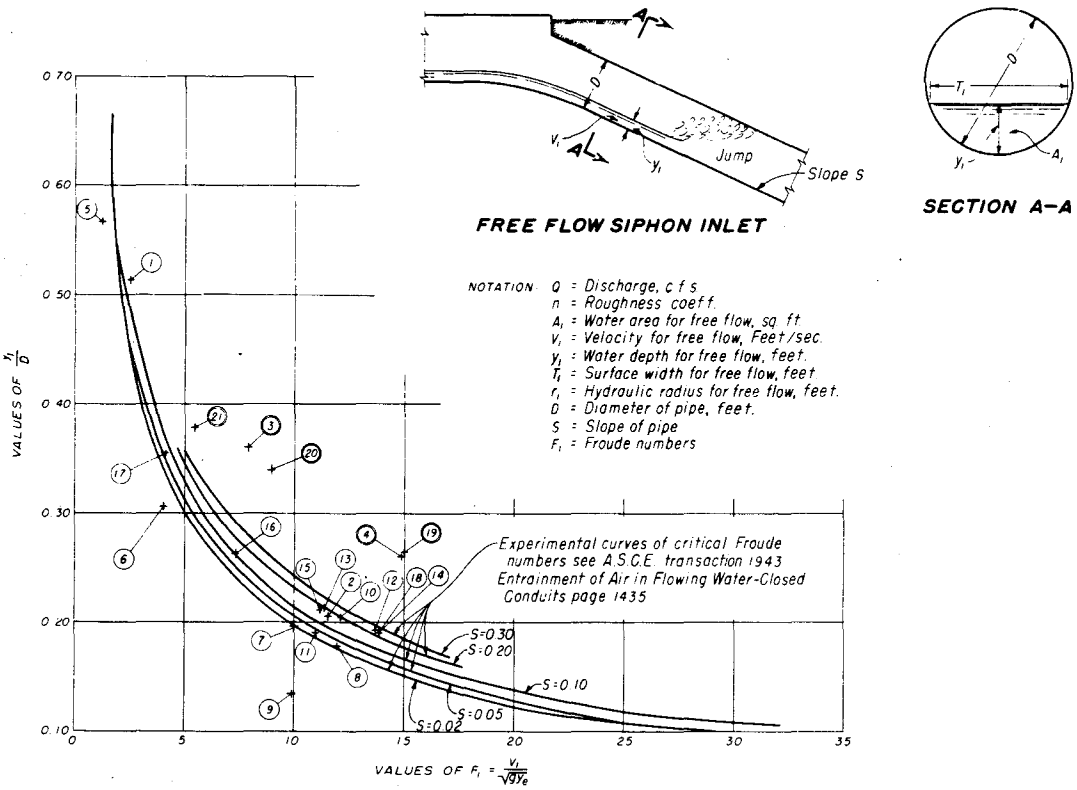
If the computed losses are greater than the difference in upstream and downstream canal water surface, the siphon will probably cause backwater in the canal upstream from the siphon. If backwater exists, the pipe size should be increased or the canal profile revised to provide adequate head.

If the computed losses are appreciably less than the difference in upstream and downstream canal water surface it may be possible to decrease the size of pipe, or the canal profile may be revised so the available head is approximately the same as the head losses.

(4) On long siphons where the inlet may not be sealed there is the possibility of blowback and unsatisfactory operating conditions. The inlet should be checked for proper performance and adjustments made if necessary.

(5) Determine the pipe class. The pipe class can be determined from the amount of external load and internal head shown on the pipe profile.

2-13. ***Design Example (see fig. 2-4.)***—Assume that an earth canal crosses a



	LOCATION	TYPE OF PIPE	Q	D	n	S	$\frac{y_1}{D}$	F_1
1	Yakima River – Yakima Project	Conc	925	111"	010	82	511	2.47
2	Malheur River – Owyhee Project	Steel	325	80"	010	213	205	11.60
3	Basin Siphon – King Hill	Wood	250	57"	012	.17	360	7.90
4	Basin Siphon – King Hill	Conc	250	54"	012	61	260	14.90
5	San Diego Sta. 620+00 N	Conc	95	54"	010	0033	.567	1.25
6	San Diego Sta. 961+00 N	Conc	95	54"	010	029	305	4.10
7	San Diego Sta. 1146+00 N	Conc	95	54"	010	.19	194	10.0
8	San Diego Sta. 1545+15 N	Conc	95	54"	010	2716	177	11.90
9	San Diego Sta. 1817+50 N	Conc	95	72"	010	1935	134	9.95
10	San Diego Sta. 1873+50 N	Conc	95	48"	010	3053	202	12.15
11	San Diego Sta. 2200+02 N	Conc	95	54"	010	2198	185	10.95
12	San Diego Sta. 1303+56 S	Conc	95	48"	010	3643	192	13.70
13	San Diego Sta. 1217+50 S	Conc	95	48"	010	2563	212	11.35
14	San Diego Sta. 1073+00 S	Conc	95	48"	010	3714	19	13.83
15	San Diego Sta. 1073+00 S	Conc	95	48"	.012	3714	211	11.20
16	San Diego Sta. 419+00 S	Conc	95	48"	010	104	262	7.33
17	San Diego Sta. 171+96 S	Conc	95	48"	010	0336	355	4.17
18	San Diego Sta. 79+92 S	Conc	95	48"	010	377	190	13.80
19	High Mesa – Uncompaghre	Steel	42	26"	010	535	264	15.00
20	Lake Valley Crossing P.G.E.	Steel	35	24"	013	367	349	8.95
21	Lake Valley Crossing P.G.E.	Steel	25	24"	016	.208	378	5.43

NOTES

Siphon inlets marked thus ◯, have given trouble in operation and air outlets were installed in some cases to relieve the blowing back of air and water.
All other siphons have not given trouble in operation.
Study made indicates that free flow siphon inlets designed so that Froude number will not fall above the critical curves established by experiments will give satisfactory performance.

Procedure to determine Froude number

For a given Q, diameter D, slope S, and coeff. n, calculate the following:

a. y_1 Using Manning's Formula
b. A_1
c. V_1
d. $T_1 = 2\sqrt{(D-Y_1)Y_1}$
e. $y_e = \frac{A_1}{T_1}$
f. $F_1 = \frac{V_1}{\sqrt{gy_e}}$

Figure 2-7. **Design of free-flow siphon inlets. 103-D-1255.**

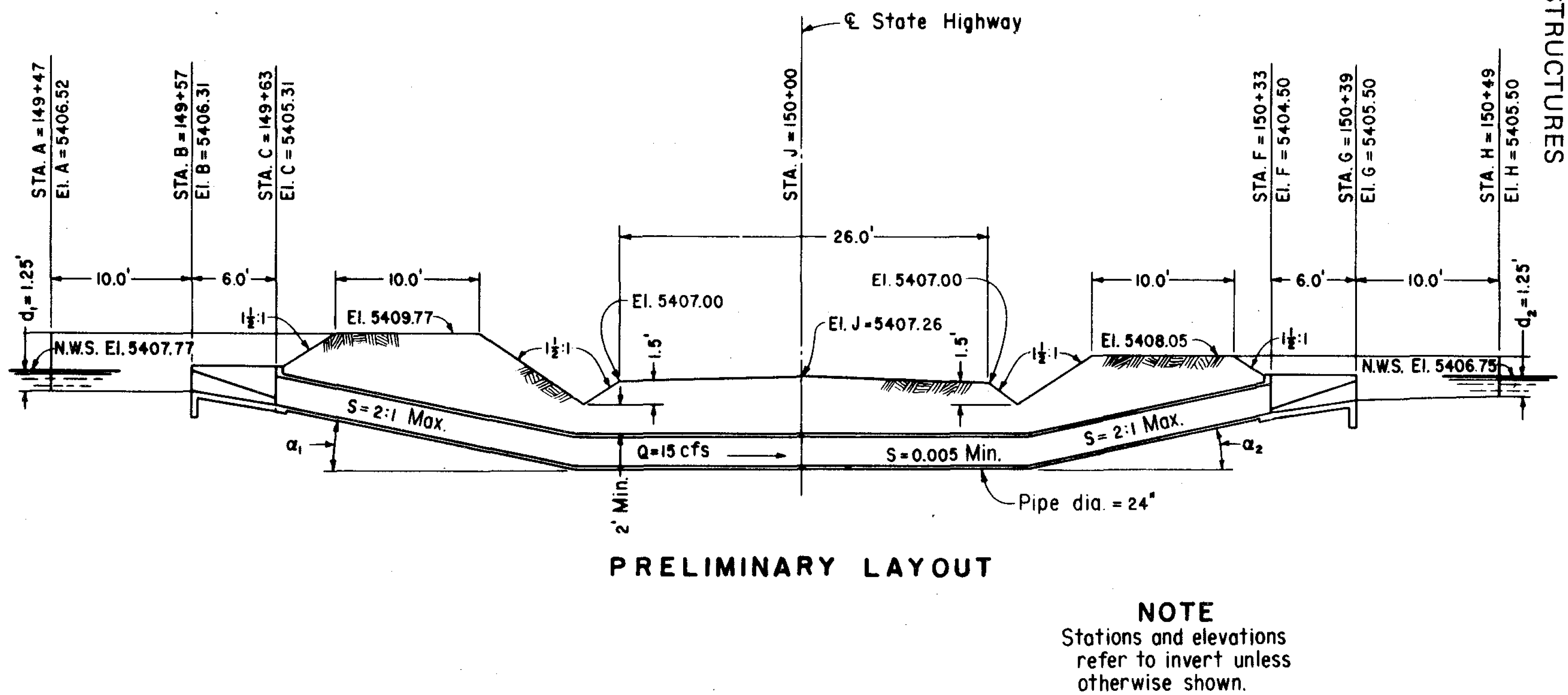


Figure 2-8. **Preliminary layout of inverted siphon. 103-D-1256**

state highway and an inverted siphon is the most feasible type of structure for conveying water past the highway.

(a) *Given:* (Refer to preliminary layout, fig. 2-8.)

(1) Type of waterway = earth canal.

(2) Feature being crossed = state highway at right angles with canal ℄.

(3) Q = 15 cfs.

(4) Sta. A = 149 + 47 and Canal Invert El. A = 5406.52 (from a canal profile sheet).

(5) d_1 = 1.25 feet (d_n, normal depth in canal).

$$V_1 = 2.1 \text{ f.p.s.}, h_{v_1} = 0.07 \text{ ft.}$$

(6) NWS El. at Sta. A = El. A + d_1 = 5406.52 + 1.25 = 5407.77.

(7) Sta. H = 150 + 49 and Canal Invert El. H = 5405.50 (from a canal profile sheet).

(8) d_2 = 1.25 ft. (d_n, normal depth in canal).

$$V_2 = 2.1 \text{ f.p.s.}, h_{v_2} = 0.07 \text{ ft.}$$

(9) NWS El. at Sta. H = El. H + d_2 = 5405.50 + 1.25 = 5406.75.

(10) Width of roadway = 26 ft.

(11) Side slopes of roadway ditch and canal embankment = 1-1/2 to 1.

(12) El. top of roadway = El. J = 5407.26.

(13) El. edge of roadway shoulders = 5407.00.

(14) Control structure at inlet not required for turnout delivery.

(15) 18-inch-deep roadway ditches.

(16) Canal Sta. J at ℄ roadway = Sta. 150+00.

(17) Canal bank width = 10.0 ft.

(18) Canal bank freeboard at outlet = normal canal bank freeboard = 1.3 ft.

(b) *Determine:*

(1) *Inlet and outlet structure requirements.*—The siphon crosses under a state highway; therefore, some kind of concrete inlet and outlet structures are required. Since a control structure is not needed at the inlet, use a concrete transition for both the inlet and the outlet. Use type 1 transitions (fig. 7-2, chapter VII) and use the same transition for both inlet and outlet.

(2) *Type of pipe.*—This pipe will have internal pressure and will be passing under a state highway, therefore, it should be either precast reinforced concrete pressure pipe (PCP), asbestos-cement pressure pipe (AC), or reinforced plastic mortar pressure pipe (RPM) each having rubber gasket joints. In this example assume that because of availability and costs it is advantageous to use PCP.

(3) *Pipe size. (See table on fig. 2-4.)*—For a relatively short siphon having concrete inlet and outlet transitions, the pipe would be sized for velocity of about 5 feet per second. Then for a discharge of 15 cfs the table suggests that a 24-inch-diameter pipe may be used.

(4) Hydraulic properties of 24-inch-diameter pipe for Q of 15 cfs.

$$A = \text{area of pipe} = 0.785 \times (\text{dia.})^2 = 3.14 \text{ ft.}^2$$

$$V = \text{velocity in pipe} = \frac{Q}{A} = \frac{15}{3.14} = 4.77 \text{ f.p.s.}$$

$$h_{v_p} = \text{velocity head in pipe} = \frac{V^2}{2g} = \frac{4.77^2}{64.4} = 0.35 \text{ ft.}$$

where g = gravitational acceleration.

$$wp = \text{wetted perimeter} = \pi \text{ dia.} = 6.28 \text{ ft.}$$

$$r = \text{hydraulic radius} = \frac{A}{wp} = \frac{3.14}{6.28} = 0.5 \text{ ft.}$$

$$n = \text{assumed roughness coefficient}^3 = 0.013$$

$$s_f = \text{friction slope of pipe} = \left(\frac{1}{2.2r^{4/3}}\right) n^2 V^2 = 0.0044$$

Hydraulic properties of pipe may also be taken from table 8-1 in chapter VIII.

(5) *Additional canal bank freeboard at upstream end of siphon.*—Additional canal bank freeboard = 0.5 of normal

[3] See section 1-16.

freeboard = 0.5 x 1.3 = 0.65 ft., use 0.7 ft.

(6) Canal bank El. at Sta. A = NWS El. + regular freeboard + additional freeboard = 5407.77 + 1.3 ft. + 0.7 ft. = 5409.77. Extend the bank at this elevation a distance of 50 feet upstream from the siphon to minimize damage which could be caused by overtopping.

(7) Canal bank El. at Sta. H = NWS El. + freeboard = 5406.75 + 1.3 ft. = 5408.05.

(8) *Inlet transition hydraulic setting.*—The transition invert elevation at the headwall (Sta. C) is based on the hydraulic seal required at the top of the headwall opening and the vertical height of the opening, Ht. Pipe slope affects this vertical dimension since $Ht = \frac{D}{\cos \alpha_1}$ where D = pipe diameter and α_1 = angle of pipe slope at headwall. The scaled value of α_1 is usually adequate since a small error in α_1 will not significantly affect Ht. The scaled value of α_1 is 12^o.

$$Ht = \frac{2.00}{\cos 12^o} = \frac{2.00}{0.978} = 2.04 \text{ ft.}$$

Hydraulic seal required = $1.5\ \Delta h_v = 1.5(h_{v_p} - h_{v_1}) = 1.5\ (0.35 - 0.07) = 0.42$ ft. which is greater than 3 inches (0.25 ft.) minimum seal required, therefore, 0.42 foot should be used. Transition invert El. C = NWS El. at Sta. A – ($1.5\ \Delta h_v$ + Ht) = 5407.77 – (0.42 + 2.04) = 5405.31

If the transition invert at the cutoff (Sta. B) is set at the canal invert, the difference in invert elevations of the transition (p) is 5406.52 – 5405.31 = 1.21 ft. = p. See figure 7-2 for p. The maximum p value for the inlet is $\frac{3}{4}$D and the maximum p value for the outlet is $\frac{1}{2}$D. Therefore, by making the inlet and outlet identical, p cannot exceed $\frac{1}{2}$D which is $\frac{1}{2}$ of 2 or 1.0 ft. (see subchapter VII A). Use a p value of 1.0 foot, then the inlet transition invert El. B will not be the same as the canal but will be El. C + p or 5405.31 + 1.00 foot = 5406.31 which is 0.21 foot lower than the canal invert at Sta. A. The invert slope for a 10-foot-long earth transition resulting from the use of p = 1.0 foot should not be steeper than 4 to 1 (see sec. 7-11). The actual slope = 10 to Δ Inv. of earth transition = 10 to 0.21 = 48 to 1 which is flatter than 4 to 1 and therefore permissible.

(9) *Outlet transition hydraulic setting.*—To minimize headwall submergence, set the downstream invert elevation (Sta. G) of the transition at canal invert. Then the transition invert El. G = canal invert El. H = 5405.50. For the inlet and outlet transitions to be identical, p = 1.0 foot. Then transition invert El. F = El. G – p = 5405.50 – 1.00 = 5404.50.

The height of headwall opening (Ht) at station F is,

$$Ht = \frac{D}{\cos \alpha_2} = \frac{2.00}{\cos 12^o} = \frac{2.00}{0.978} = 2.04 \text{ ft.}$$

Submergence of top of opening =

$$(d_2 + p) - \frac{D}{\cos \alpha_2} = (1.25 + 1.00) - 2.04 = 0.21 \text{ ft.}$$

This submergence should not exceed one-sixth Ht for minimum head loss. One-sixth Ht = $\frac{2.04}{6}$ = 0.34 foot which is greater than the submergence of 0.21 foot. Therefore, the loss for the outlet transition is minimum and may be calculated using the equation $0.7\Delta h_v$.

(10) Drop in water surface elevation (available head) = NWS El. Sta. A – NWS El. Sta. H = 5407.77 – 5406.75 = 1.02 ft.

(11) Before establishing detailed siphon elevations and dimensions, use a preliminary siphon layout (fig. 2-8) and determine approximate total head loss and compare with head provided. Scale the dimensions and angles as required. This study will indicate if the pipe diameter or canal profile should be revised.

Total siphon head loss with 10 percent safety factor = 1.1 (inlet transition convergence loss + pipe friction loss + bend losses + outlet transition divergence loss).

Pipe length scaled = about 72 feet.

Pipe bend angles scaled = about 12^o (assume single angle bends).

Approximate total head loss $H_L = 1.1\,(h_i + h_f + h_b + h_o)$ where h_i is inlet loss, h_f is pipe friction loss, h_b is pipe bend loss, and h_o is outlet loss.

$$\begin{aligned} H_L &= 1.1\,[0.4\Delta h_v + \text{pipe length x } s_f \\ &\quad + \zeta h_{v_p} \text{ x } 2 + 0.7\Delta h_v] \\ H_L &= 1.1\,[0.4\,(0.35 - 0.07) + 72 \text{ x } 0.0044 \\ &\quad + (0.04 \text{ x } 0.35)\,2 \\ &\quad + 0.7(0.35 - 0.07)] \\ &= 1.1\,[0.11 + 0.32 + 0.03 + 0.20] \\ &= 1.1\,(0.66) = 0.73 \text{ foot} \end{aligned}$$

Head provided by canal profile = 1.02 feet which is 0.29 foot more than the 0.73 foot required for this preliminary layout. This excess head will cause a slight drawdown in the canal upstream from the siphon and will result in faster than normal velocities for a short distance. For this design example assume that this velocity is noneroding so it is not necessary that the profile of the canal or the size of pipe be revised.

(12) *Transition dimension, y (fig. 7-2).*—Dimension y should be determined so that the freeboard provided at the cutoff will be 0.5 foot as indicated in subchapter VII A.

$$\begin{aligned} y &= (\text{NWS El. Sta. A} - \text{El. B}) + F_b \\ &= (5407.77 - 5406.31) + 0.5 \\ &= 1.46 + 0.5 = 1.96 \text{ ft.} \end{aligned}$$

Therefore, use 2 feet 0 inch.

(13) *Transition dimension, a (fig. 7-2).*—Freeboard at the transition headwall for pipe diameters 24 inches and smaller may be the same as the freeboard at the cutoff. Therefore the top of the headwall is set at the same elevation as the top of the wall at the cutoff and is equal to:

$$\begin{aligned} &\text{El. B} + y = 5406.31 + 2.00 = 5408.31 \\ &a = \text{El. top of wall} - \text{El. C} \\ &\quad = 5408.31 - 5405.31 = 3 \text{ ft. } 0 \text{ in.} \end{aligned}$$

(14) *Transition dimension, C.*—Refer to the table on figure 7-2, to determine C. For identical upstream and downstream transitions, the column for water surface angle of 25^o should be used.

The relationship of pipe diameter D to normal depth d in the canal is determined for use in the table and is equal to

$$D = \frac{2.00}{1.25} \text{ x } d = 1.6d.$$

By interpolating in the table between D = 1.5d and D = 2.0d, dimension C would be

$$\begin{aligned} C &= 1.8D + \left(\frac{1.6 - 1.5}{2.0 - 1.5}\right) \text{x } (2.3D - 1.8D) \\ &= 1.8D + \frac{0.1}{0.5}(0.5D) = 1.8D + 0.1D \\ &= 1.9D \end{aligned}$$

Then C = 1.9D = 1.9 x 2 = 3.8 feet

Use C = 4 feet 0 inch. This may or may not match the canal bottom width. Additional transitioning of bottom width should be included in the earth transition.

(15) *Depth and thickness of transition cutoff.*—Referring to the appropriate table on figure 7-2, for normal depth in the canal of 1.25 feet, the transition cutoff depth, e, should be 24 inches and the thickness, tw, should be 6 inches.

(16) *Concrete transition length, L (fig. 7-2).*—

$$L = 3 \text{ pipe dia.} = 3 \text{ x } 2 = 6 \text{ feet}$$

(17) *Transition dimension B (fig. 7-2).*—The width of the base at the headwall is B and is equal to 0.303 x D = 0.303 x 24 inches = 7.272 inches. Use B = 8 inches.

(18) *Pipe embedment and bends.*—Embedment details for the pipe at the headwalls and construction requirements of the pipe bends are discussed in chapter VIII and shown on figures 8-14 and 8-10, respectively. Since the deflection angles of the bends will be less than 45^o, only one miter is necessary. Also because the pipe diameter is less than 42 inches, a banded mitered pipe bend may be used with a minimum band thickness of 1 inch. Since the hydraulic thrust caused by the bend is directed into the pipe foundation, stability of the bend is probably sufficient.

Unusually poor foundation conditions in addition to high heads, large diameter pipe, and large deflection angles may, however, require that the thrust be considered when determining the foundation reaction.

(19) *Final siphon profile (fig. 2-9).*—Using the elevations, dimensions, and earth slopes previously computed or given, determine the final structure stationing, pipe elevations, and pipe slopes.

Stations C and F at the transition headwalls are controlled by the roadway earthwork dimensions and side slopes and headwall thickness. From figure 2-9 it can be determined that station C must be at least 34.36 feet upstream from the roadway centerline.

Sta. C = Sta. J – 34.36 ft.
= Sta. (150+00) – 34.36 ft.
= Sta. 149+65.64 (or less)
Use Sta. C = Sta. 149+65
Sta. B is then
= Sta. C – 6.00 feet
= Sta. (149+65) – 6.00 feet
= Sta. 149+59
and Sta. A becomes
= Sta. B – 10.00 ft.
= Sta. (149+59) – 10.00 ft.
= Sta. 149+49

The small difference in the given value for station A (149+47) and the computed station (149+49) is not significant enough to require any canal invert profile changes.

Stations F, G, and H are determined in the same manner as stations A, B, and C. From figure 2-9 it can be determined that station F must be at least 30.38 feet from the roadway centerline.

Sta. F = Sta. J + 30.38 ft.
= Sta. (150+00) + 30.38 ft.
= Sta. 150+30.38 (or greater)
Use Sta. F = Sta. 150+31
Sta. G is then
= Sta. F + 6.0 ft.
= Sta. (150+31) + 6.0 ft.
= Sta. 150+37 and
Sta. H becomes
= Sta. G + 10.0 ft.
= Sta. (150+37) + 10.0 ft.
= Sta. 150+47

Here again the difference between the given and computed values for station H is small and will not require any canal invert profile changes.

Stations D and E are selected to insure that a 2-foot minimum of earth cover on the pipe is provided at the roadway ditches. The inverts of the V-ditches are located 15.25 feet from the roadway centerline. Therefore, the pipe bend inverts should be located about 16 feet from each side of the centerline of the roadway.

Then Sta. D = Sta. J – 16.00 ft.
= Sta. (150+00) – 16.00 ft.
= Sta. 149+84

El. D is determined by subtracting the pipe diameter, the shell thickness, and the minimum cover from the elevation of the ditch invert.

El. D = (5407.00 – 1.5 ft.) – (2.00 ft. + 0.25 ft. + 2.00 ft.) = 5401.25.

Determine Sta. E = Sta. J + 16.00 ft. = Sta. (150+00) + 16.00 ft. = Sta. 150+16.

El. E is determined by subtracting the product of the distance between stations D and E and the pipe slope 0.005 (which is a minimum slope) from El. D.

El. E = El. D – 32 ft. x 0.005 = 5401.25 – 0.16 ft. = 5401.09.

Upstream pipe slope (S_1). Slope of the pipe between stations C and D is calculated as follows:

Horizontal distance = Sta. D – Sta. C
= (149+84) – (149+65) = 19 feet

Vertical distance = El. C – El. D
= 5405.31 – 5401.25 = 4.06 feet

$$S_1 = \frac{\text{vertical distance}}{\text{horizontal distance}} = \frac{4.06 \text{ ft.}}{19 \text{ ft.}}$$
$$= 0.214$$

Angle of the slope is the angle whose tangent is

$$= 0.214, \alpha_1 = 12^{\circ}05'$$

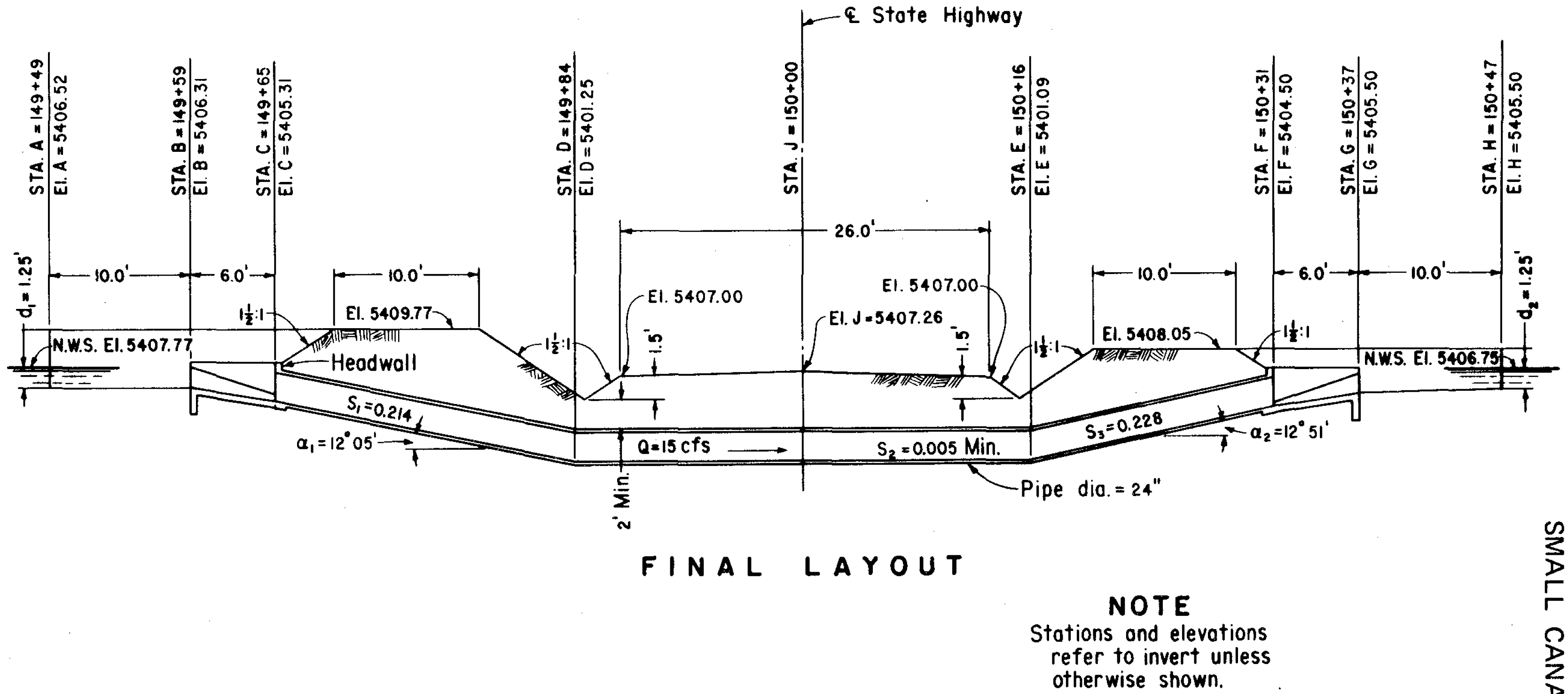


Figure 2-9. **Final layout of inverted siphon. 103-D-1257**

Downstream pipe slope (S_3). Determine slope of the pipe between stations E and F.

Horizontal distance = Sta. F – Sta. E
= Sta. (150+31) – Sta. (150+16) = 15 ft.

Vertical distance = El. F – El. E
= 5404.50 – 5401.09 = 3.41 ft.

$$S_3 = \frac{\text{vertical distance}}{\text{horizontal distance}} = \frac{3.41 \text{ ft.}}{15 \text{ ft.}}$$
$$= 0.228$$

Angle of the slope is the angle whose tangent is 0.228,

$$\alpha_2 = 12^{\circ}51'$$

(20) *Final siphon head losses.*—Total final siphon head loss with 10 percent safety factor = 1.1 (inlet transition convergence loss + pipe friction loss + pipe bend losses + outlet transition divergence loss),

$$\text{or } H_L = 1.1\,(h_i + h_f + h_b + h_o)$$

$$H_L = 1.1\,[0.4\Delta h_v + \text{pipe length} \times s_f + \zeta\, h_{v_p} \times 2 + 0.7\Delta h_v]$$

Determine pipe length:

From station C to station D

$$\text{Length} = \frac{(\text{Sta. D} - \text{Sta. C})}{\cos \alpha_1}$$
$$= \frac{19 \text{ ft.}}{\cos 12^{\circ}05'} = \frac{19 \text{ ft.}}{0.978}$$
$$= 19.4 \text{ ft.}$$

From station D to station E. Since pipe slope is relatively flat, use horizontal distance =

Sta. E – Sta. D = Sta. (150+16)
– Sta. (149+84) = 32.0 ft.

From station E to station F

$$\text{Length} = \frac{(\text{Sta. F} - \text{Sta. E})}{\cos \alpha_2} = \frac{15 \text{ ft.}}{\cos 12^{\circ}51'}$$
$$= \frac{15 \text{ ft.}}{0.975} = 15.4 \text{ ft.}$$

Total pipe length = 19.4 + 32.0 + 15.4 = 66.8 ft. Therefore, the total head loss in the siphon is:

$$H_L = 1.1\,[0.4(0.35 - 0.07) + 66.8 \times 0.0044 + (0.04 \times 0.35) \times 2 + 0.7\,(0.35 - 0.07)]$$
$$= 1.1\,(0.11 + 0.29 + 0.02 + 0.20)$$
$$= 0.68 \text{ ft.}$$

Since the head provided (1.02 feet) is greater than the head required (0.68 foot), a slight water surface drawdown will occur for a short distance upstream from the siphon. This excess head will cause faster than normal velocities. For this design example assume that these velocities are still noneroding so neither the canal profile nor the pipe size need be revised.

(21) *Erosion protection.*—Refer to figure 7-8 in chapter VII B. The water depth in the canal is less than 2 feet so protection is not required at the end of the siphon.

(22) *Pipe collars.*—Refer to subchapter VIII C. Assume that collars are not needed to discourage burrowing animals but collars may be necessary to slow the percolation of water along the pipe even without their burrows. The difference in elevation between the canal water surface and the roadway ditch is 2.3 feet (ΔH). The weighted creep ratio, percolation factor, required to prevent piping is assumed to be 3.0. Determine the weighted creep length (L_w) from the inlet transition to the first roadway ditch assuming the seepage water flows along the bottom side of the siphon from station B to station D; then along the outside of the pipe to the top of the pipe; and finally through the earth to the ditch invert. Weighted creep lengths are derived by multiplying the path length by one if the path is vertical and between structure and earth; by one-third if the path is horizontal and between structure and earth; and by two if the path is through earth.

$$L_w = (2 \times \text{vertical dimension of cutoff}) \times 1 + (\text{Sta. D} - \text{Sta. B}) \times 1/3 + (\text{outside diameter of pipe}) \times 1 + (\text{earth cover on pipe}) \times 2$$
$$= (2 \times 2) \times 1 + (25) \times 1/3 + 2.5 \times 1 + 2 \times 2$$
$$= 4.0 + 8.3 + 2.5 + 4.0 = 18.8 \text{ feet}$$

Determine the percolation factor (PF) that this weighted creep distance will provide.

$$PF = \frac{L_w}{\Delta H} = \frac{18.8 \text{ feet}}{2.3 \text{ feet}} = 8.2$$

Since the percolation factor provided (8.2) is greater than that assumed to be necessary (3.0), pipe collars are not needed.

(23) *Blowoff.*—Refer to subchapter VIII C. The structure is short and the pipe may be drained by pumping from the ends so a blowoff is not required.

(24) *Blowback.*—Refer to section 8-16. This siphon structure is short and not likely to have blowback because the air that might be entrained due to a possible freeflow inlet and hydraulic jump will probably be carried downstream and exhausted at the downstream end of the siphon.

(25) *Class of PCP.*—The equivalent earth load on the pipe will not exceed 10 feet (3 feet of earth cover + H20 loading is a total equivalent earth cover of 9.1 ft.—subchapter I B) and the hydrostatic head measured to the centerline of the pipe will not exceed 25 feet, therefore class B 25 PCP is satisfactory. The pipe designation will then be 24 B 25.

(26) *Safety features.*—Refer to chapter IX.

D. BENCH AND ELEVATED FLUMES

R. B. YOUNG[1]

2-14. ***Purpose and Description.***—(a) *General.*—Flumes are used to convey canal water along steep sidehill terrain, or to convey canal water over other waterways, or natural drainage channels. Flumes are also used at locations where there is restricted right-of-way or where lack of suitable material makes construction of canal banks undesirable or impracticable.

Flumes supported on a bench excavated into a hillside are called bench flumes (figs. 2-10 and 2-11). Flumes supported above the ground with reinforced concrete, structural steel, or timber are called elevated flumes (figs. 2-12 and 2-13).

(b) *Bench Flumes.*—A bench flume is usually rectangular in shape and made of reinforced concrete (fig. 2-17) with inlet and outlet transitions to the adjoining canal. Excavation into the hillside to form the bench should be of sufficient width to provide for an access road along the downhill side unless other provisions have been made for a road.

Some reasons for using a bench flume along steep sidehill terrain rather than a canal or pipeline, could be economy or practicality of construction and maintenance.

Where there is a possibility of falling rock damaging the flume, protective backfill should be placed to near the top of the flume wall adjacent to the hillside. Where severe rockfall problems may be encountered, especially with flumes for small discharges, it may be more economical and practicable to use precast concrete pressure pipe in a trench and provide 3 feet of earth cover as is shown in figure 2-14. If the rockfall is only over a short reach of flume it may be preferable to provide a reinforced concrete cover for this reach.

(c) *Elevated Flumes.*—Elevated steel flumes with a semicircular shape were commonly used at one time to carry canal water over natural drainage channels or depressions. Elevated flumes now are seldom used because of associated maintenance problems, environmental considerations, aesthetics, and because of the the availability of precast concrete pressure pipe with rubber gasket joints that can usually be more economically constructed under natural drainage channels or depressions.

Because the elevated flume is so limited in its usage it will not be discussed in detail in this publication. Overchutes, discussed in subchapter IV C (fig. 4-30), are a type of elevated flume used to convey cross-drainage water over a canal. These overchutes are usually steel pipe or rectangular reinforced concrete channels.

[1]Op. cit., p. 19.

Figure 2-10. Concrete bench flume.

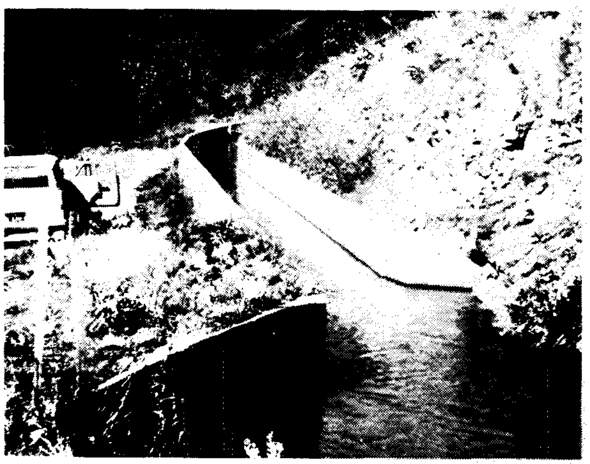


Figure 2-11. Concrete bench flume. P336-D-48829

Figure 2-12. Elevated flume crossing natural drainage. PX-D-4592

Figure 2-13. Early day elevated flume. P42-D-45751

1. Hydraulic Considerations

2-15. ***Flume Section.***–(a) *Ratio* $\frac{b}{d}$.–The most hydraulically efficient rectangular bench flume section is one with a $\frac{b}{d}$ ratio equal to 2 [6].

Although this ratio is often used, other considerations may suggest that the flume should be narrower or wider. For example, a narrower section may be more economical along a steep hillside where rock excavation is required. A narrower section may also be necessary because of limited right-of-way, yet a wider section reduces the wall heights which could effect economy.

A study of $\frac{b}{d}$ ratios in regard to hydraulic efficiency and construction costs (excluding excavation and backfill) indicates that an acceptable $\frac{b}{d}$ ratio is in the range between about 1 and 3. For any $\frac{b}{d}$ ratio in this range the respective values of area, velocity, and wetted perimeter are nearly identical for any slope between 0.0001 and 0.100 and for any flow between 0 and 100 cfs.

(b) *Slope and Velocity.*–The most economical flume section will have velocities greater than those allowed in an earth canal. Because velocities in a flume are ordinarily greater than in an earth canal, a steeper slope will be required.

Flume design velocities are ordinarily substantially slower than critical velocities, and therefore, subcritical flow exists. If, however,

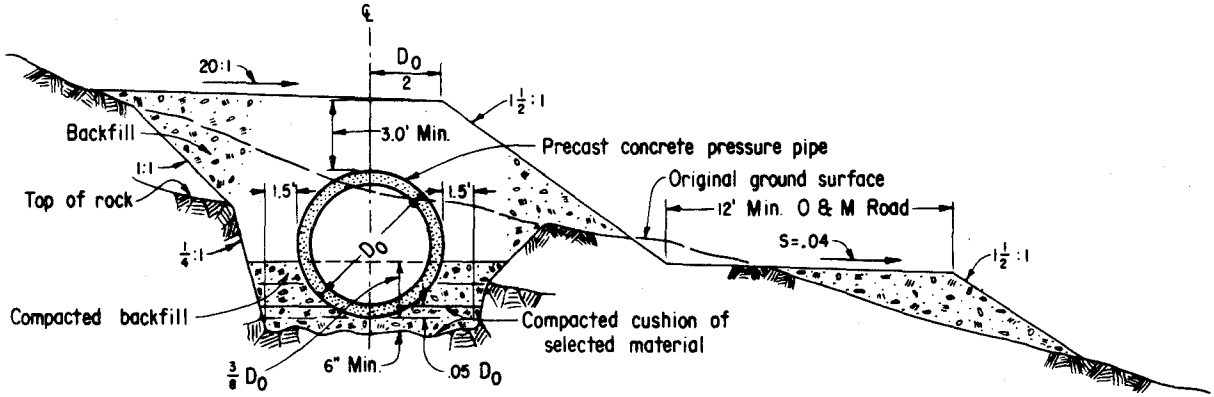


Figure 2-14. Concrete pipeline for bench construction. 103-D-1258

design velocities are faster than critical, supercritical flow exists and the bench flume performs hydraulically as a chute and an energy dissipator is required at the structure outlet. See subchapter II F for chute design.

Studies show that with the $\frac{b}{d}$ ratio equal to 1, 2, or 3 and for capacities of 100 cfs or less the slope of the flume invert should not be steeper than s = 0.002. This criteria applies not only to the flume design slope but also to the actual flume slope due to construction tolerances that could otherwise result in supercritical slopes.

Studies also show that with these ratios and capacities a rectangular flume invert slope of 0.005, for example, is nearly always steeper than critical slope resulting in supercritical flow. Flume slopes should be substantially flatter than critical slope because flows at or near critical depth tend to be rather unstable, resulting in undesirable water surface undulations.

Using an n value of 20 percent less than the nominal design n, a verification should be made to see that the depth does not approach critical depth where the bottom might be raised above design grade due to construction tolerances.

(c) *Freeboard.*—Bench flume freeboard should be correlated with the adjacent canal freeboard so that overflow from either will result in the least amount of damage to canal, flume, and facilities downhill from the overflow.

Freeboard for flumes will depend upon a number of factors, such as the size of flume, velocity of water, curvature of alinement, and anticipated method of operation. Figure 1-9 may be used as a guide for determining minimum freeboard for flumes.

2-16. ***Transitions.***—(a) *General.*—Wherever a bench flume is used with a canal, transitioning from the canal section to the flume section is usually required to provide a relatively smooth water surface and to conserve energy. This is usually also true with transitioning from flume section to canal section, especially if the canal section is unlined.

As discussed in chapter VII, where concrete transitions are required for inline structures they are usually broken-back (type 1) transitions for the capacity range in this publication. Where the flume velocity is not much more than the canal velocity, an abrupt change in water prism from canal to flume may be permitted. The canal base width should, however, be uniformly varied to match the flume base width and flume wingwalls should be provided to retain the canal banks and to serve as cutoffs to prevent flow along the outside of the flume.

(b) *Inlet.*—Usually flume velocities are greater than canal velocities, resulting in increasing velocities through the inlet transition and a drop in the water surface by an amount sufficient to produce the necessary increase in velocity and to overcome friction and transition losses. Neglecting the transition friction loss, which is usually small, and assuming 0.3 Δh_v for a broken-back inlet transition loss, the total drop in water surface is 1.3 Δh_v.

The transition invert elevation at the cutoff is set to match that of the canal, whereas the transition invert elevation at the flume is set below the flume normal water surface by an amount equal to normal depth in the flume.

The length of the inlet transition is determined by using a maximum convergence angle of 27-1/2° between the water surface and the transition centerline.

(c) *Outlet.*—The outlet transition divergence loss for a broken-back outlet transition to a rectangular channel is approximately 0.5 Δh_v. The total rise in the water surface from the flume to the canal, neglecting friction, is equal to $(1\text{-}0.5)\,\Delta h_v = 0.5\,\Delta h_v$.

The transition invert at the cutoff and at the flume is set in the same manner as the invert for the inlet transition except 0.5 Δh_v is used for difference of water surface (Δws).

The length of the outlet transition is determined by using a maximum divergence angle of 22-1/2° between the water surface and the transition centerline.

(d) *Freeboard.*—Freeboard at the transition cutoff adjacent to concrete canal lining, other hard surface, or buried-membrane canal lining is usually the same as the lining freeboard. In unlined and earth-lined canals the minimum freeboard should be 6 inches for water depths up to 1.25 feet; 9 inches for depths of 1.26 to 2.00 feet; and 12 inches for depths of 2.01 to 5.00 feet. Freeboard at the flume end of the transition should be the same as the flume freeboard.

(e) *Cutoffs.*—Transition cutoff walls should, in general, be a minimum of 24 inches deep for water depths up to 3 feet at the cutoff; 2 feet 6 inches deep for water depths of 3 to 6 feet; and 3 feet for water depths greater than 6 feet. The minimum concrete thickness should be 6 inches for 24-inch cutoffs and 8 inches for cutoffs deeper than 24 inches.

2. *Structural Considerations of a Rectangular Bench Flume*

2-17. ***Flume Section.***—(a) *Stability.*—After backfill requirements have been determined, resistance to sliding from uphill backfill pressure must be provided. If the deadweight of the empty flume does not furnish the required resistance to backfill pressure on the uphill side, the base may be extended to engage additional earth weight. The ratio of the horizontal forces to the vertical forces should not exceed the coefficient of sliding. The friction coefficient for sliding of concrete on earth is frequently assumed to be 0.35. Then $\frac{\Sigma H}{\Sigma V}$ should be equal to or less than 0.35. The resultant of all forces considered in the analysis for sliding should intersect the base of the flume within the middle-third to provide bearing pressure over the entire base width.

(b) *Wall Loads.*—Considerations for the structural design of a concrete bench flume section must include the deadweight of the flume and the backfill loads outside the flume with and without water in the flume.

If backfill is not required behind the walls, the water surface inside the flume is assumed to be at the top of the walls.

For flume walls which retain backfill, the internal water pressure is usually omitted for this size structure and only the exterior earth load due to the backfill is used. A saturated earth load and wheel loads are used where appropriate.

If wintertime operation is required, an ice load must also be considered.

(c) *Concrete Thickness.*—Concrete thickness at the base of vertical cantilever flume walls is determined by conventional reinforced concrete design methods. The floor thickness is usually made the same as the wall thickness at the junction of the wall and floor, and is usually uniform across the width of the flume.

(d) *Drainage.*—Pipe drains are intermittently placed under the flume to remove storm runoff water or any other water which might otherwise collect behind the uphill flume wall and cause stability and foundation problems.

(e) *Joints.*—The flume should be provided with rubber water stop joints every 25 feet. The joints are usually butt joints without elastic filler except that elastic filler joints should be used from the PC to the PT of curves and adjacent to transitions. Bell joints are not recommended.

(f) *Safety Requirements.*—Most flumes with capacities of 100 cfs and less do not require safety facilities because they are generally in a low exposure classification, and in addition a combination of the depth and velocity of water in the flume is not considered hazardous. Flumes in class A, B, or C exposure areas will require fencing to prevent persons from entering the flume. This is especially necessary if the flume is backfilled to the top of the hillside wall. Fencing or guardrail may also be required to keep animals and vehicles out of the flume and to protect operating personnel. Flumes with wall heights greater than 3 feet will require escape ladders at 500-foot intervals to permit a person to climb out. See chapter IX for additional safety details.

2-18. ***Design Example.***—(a) *Given.*—

(1) A bench flume is to be used along a canal alinement to convey the canal water along a steep hillside. Excavation for the bench will be in rock. Debris and loose rock is not expected to be falling so the uphill flume wall will not require backfill.

(2) Pipe drains will be required under the flume to remove hillside runoff water from behind the uphill flume wall. Assume that a 6-inch drain every 100 feet will fulfill this requirement.

(3) An access road adjacent to the flume on the downhill side will not be required since a natural bench exists a short distance downhill from the flume and will be improved to provide access.

(4) The earth canal has the following hydraulics properties at each end of the flume: Q = 100 cfs, s = 0.00056, n = 0.025, d = 3.00 ft., b = 10 ft., S:S = 1-1/2:1, A = 43.5 sq. ft., V = 2.30 f.p.s., R = 2.09, h_v = 0.08, and F_b = 1.9 ft. At the upstream end of the flume structure the canal station is 1+00 and the invert elevation of the canal is 1000.00. At the downstream end of the flume structure the canal station is 6+50 and the invert elevation of the canal is 998.80. The head loss provided for the bench flume structure is therefore 1.20 feet.

(5) Use broken-back inlet and outlet transitions from the earth canal to the flume section.

(6) Flume roughness coefficient n is assumed to be 0.014 (see sec. 1-16).

(b) *Determine.*—

(1) *Bench flume section.*—Bench excavation for construction of the flume will be in rock on a fairly steep hillside. For economy of construction including excavation costs, assume that of the acceptable flume sections previously mentioned the one with a $\frac{b}{d}$ ratio equal to about 1, for slopes 0.0001 to 0.100 and flows 0 to 100 cfs, is also the most economical.

For ideal hydraulic conditions, the drop in water surface from the canal to the flume using a broken-back transition will be equal to $1.3\Delta h_v$. At the outlet the rise in water surface ideally will be $0.5\Delta h_v$. Slight deviations from these values are not objectionable and will exist if the head loss provided by the drop in canal water surface, from the inlet to the outlet of the flume, is different from the actual flume loss.

It can be shown for these ideal hydraulic conditions and for the same canal hydraulic properties at each end of the flume, that the drop in rectangular section invert from the inlet to the outlet of the rectangular flume will be equal to:

$$\Delta \text{Inv.}_f = Y - 0.8\,\Delta h_v$$

where Y is the drop in canal invert provided for the structure. Assume that the head provided will permit a flume velocity of about 5 feet per second and h_{v_f} = 0.39 ft.

$$\begin{aligned}\text{Then } \Delta \text{Inv.}_f &= Y - 0.8\,\Delta h_v \\ &= 1.20 - 0.8\,(.39 - .08) \\ &= 1.20 - 0.25 = 0.95 \text{ ft.}\end{aligned}$$

Further, assume that preliminary computations for transition lengths show that the inlet transition will be approximately 15 feet long and the outlet transition approximately 20 feet long. The rectangular flume length, L, will then be station 6+50 minus station 1+00 minus 15 feet minus 20 feet = 515 feet.

The resulting flume invert slope is then equal to:

$$S_b = \frac{\Delta Inv._f}{L} = \frac{0.95}{515} = 0.00184,$$

rounded to 0.0018

It has been recommended that the slope should not be steeper than S = 0.002 for subcritical flow design, so this is probably satisfactory.

For S_b = 0.0018 and n = 0.014 determine the water depth and velocity for a $\frac{b}{d}$ ratio = 1 then, $A = d^2$, wp = 3d, $R = \frac{d^2}{3d} = 0.33d$ and $R^{2/3} = 0.48d^{2/3}$.

Using Manning's formula [6]

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} = 106.1 \times 0.48d^{2/3} \times (0.0018)^{1/2} = 2.16d^{2/3}$$

also

$$V = \frac{Q}{A} = \frac{100}{d^2}$$

By equating the two expressions for velocity, $2.16d^{2/3} = \frac{100}{d^2}$

$$d^{8/3} = 46.29$$
$$d = 4.21 \text{ ft.}$$

Then

$$A = d^2 = 17.72 \text{ ft.}^2$$

and

$$V = \frac{100}{17.72} = 5.64$$

and velocity head,

$$h_{v_f} = 0.49 \text{ foot}$$

This velocity is reasonably close to the assumed velocity used in calculating S_b. Design of flume section may therefore proceed.

Determine critical depth and critical slope to see if the flume invert slope is flat enough to insure that design flow will be stable. Use n = 0.011 in the computations for critical slope.

For

$$d = 4.21 \text{ ft. and } \frac{b}{d} = 1$$

$$b = 4.21 \text{ ft.}$$

Use

$$b = 4 \text{ ft. } 3 \text{ in. or } 4.25 \text{ ft.}$$

Then

$$q = \frac{Q}{b} = \frac{100}{4.25} = 23.5 \text{ cfs and } d_c = 0.314\, q^{2/3} = 0.314 \times 8.204 = 2.58 \text{ ft.}$$

Then

$$A_c = b \times d_c = 4.25 \times 2.58 = 10.98 \text{ ft.}^2$$

$$V_c = \frac{Q}{A_c} = \frac{100}{10.98} = 9.11 \text{ f.p.s.}$$

$$wp_c = 2d_c + b = 2 \times 2.58 + 4.25 = 9.41 \text{ ft.}$$

$$R_c = \frac{A_c}{wp_c} = \frac{10.98}{9.41} = 1.17$$

$$R_c^{4/3} = 1.23$$

$$S_c = \frac{1}{2.21R^{4/3}} (nV)^2 = \left(\frac{1}{2.21 \times 1.23}\right) \times (0.011 \times 9.11)^2 = 0.0037$$

Design slope is 0.0018 which is much flatter than the critical slope of 0.0037 calculated using an n of 0.011.

Assume that the tolerance for construction allows the invert to rise 1 inch (0.08 foot) at random points above the normal invert elevation. Determine what effect this will have on the flow using an n of 0.011. Studies show that with this n value and this 0.08 foot irregularity, the depth at this irregularity will always be greater than critical depth for invert slopes flatter than 0.005 when the $\frac{b}{d}$ ratio is equal to 1, 2, or 3 and capacities are not greater than 100 cfs.

Because the value b has been rounded off to 4 feet 3 inches or 4.25 feet, normal depth for the flume section must be recomputed.

Find normal depth for b = 4.25 ft., S_b = 0.0018, and n = 0.014.

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$
$$= \frac{1.486}{0.014} AR^{2/3} (0.0018)^{1/2}$$
$$= 106.14 \times 0.0424AR^{2/3} = 4.50\ AR^{2/3}$$

Assume different values for d_n until Q = 100 cfs (see tabulation below).

Normal depth, d_n, is therefore 4.15 feet, and velocity $= \frac{Q}{A} = \frac{100}{17.63} = 5.67$ f.p.s. therefore, h_{v_f} = 0.50 foot. Normal depth could also have been determined by using table 2-5.

It is desirable at this stage of the design process to compute the total head loss of the bench flume structure and compare it with the head available. The total head loss (H_T) is composed of three parts: (1) inlet transition convergence loss; (2) bench flume friction loss; and (3) outlet divergence loss.

$$H_T = 0.3\ \Delta h_v + h_f + 0.5\ \Delta h_v$$
$$= 0.8\ \Delta h_v + S_f \times L$$
$$= 0.8\ (0.50 - 0.08) + 0.0018 \times 515$$
$$= 0.34 + 0.93 = 1.27 \text{ ft.}$$

There is only 1.20 feet of drop provided in canal water surface between the flume ends so the available head is deficient by 0.07 foot. This shortage of head is small and insignificant but will cause a very slight rise in water surface upstream from the flume.

Note that in the computation for total head loss, the canal velocity head at both the upstream and downstream ends of the flume structure was based on flow assumed at normal depth in the canal. Since the computations indicate only a slight water surface rise in the canal at the upstream end, the comparison of total head loss to head available is sufficiently accurate.

If the difference in the computed head loss and the available head is considered to be significant, several alternatives are available to the designer to theoretically make them equal: (1) revise the flume width; (2) revise the flume invert slope; or (3) revise the upstream or downstream canal invert elevations as required to provide the precise, calculated head loss.

(2) *Length of transitions (fig. 2-15).*—

$$\theta_1 = 27\text{-}1/2^{\circ} \text{ max.}$$
$$\tan \theta_1 = \frac{7.38}{L_1} = 0.52057$$
$$L_1 = \frac{7.38}{0.52057} = 14.2 \text{ feet}$$

Use L_1 = 15 feet

$$\theta_2 = 22\text{-}1/2^{\circ} \text{ max.}$$
$$\tan \theta_2 = \frac{7.38}{L_2} = 0.41421$$
$$L_2 = \frac{7.38}{0.41421} = 17.8 \text{ feet}$$

Use L_2 = 20 feet

(3) *Outlet transition elevations.*—As the hydraulic control is downstream the outlet transition should be designed first, then proceed back through the structure to design the inlet transition. There is a theoretical rise in water surface in the outlet transition of 0.5 Δh_v or 0.5(0.50 – 0.08) which equals 0.21 foot. The invert elevation at the upstream end of the outlet transition is therefore set below the canal normal water surface by an amount equal to the water surface rise plus normal depth in the flume, The upstream invert elevation is then: El. 1001.80 – 0.21 – 4.15 = 997.44. The downstream invert elevation is set at the canal invert elevation which is El. 998.80 at station 6+50.

Assume d_n	A, sq. ft. (4.25d)	wp, ft. (2d+4.25)	R, ft. $\frac{A}{wp}$	$R^{2/3}$	Q, cfs ($4.50\ AR^{2/3}$)
4.20	17.85	12.65	1.41	1.26	101
4.15	17.63	12.55	1.41	1.26	100

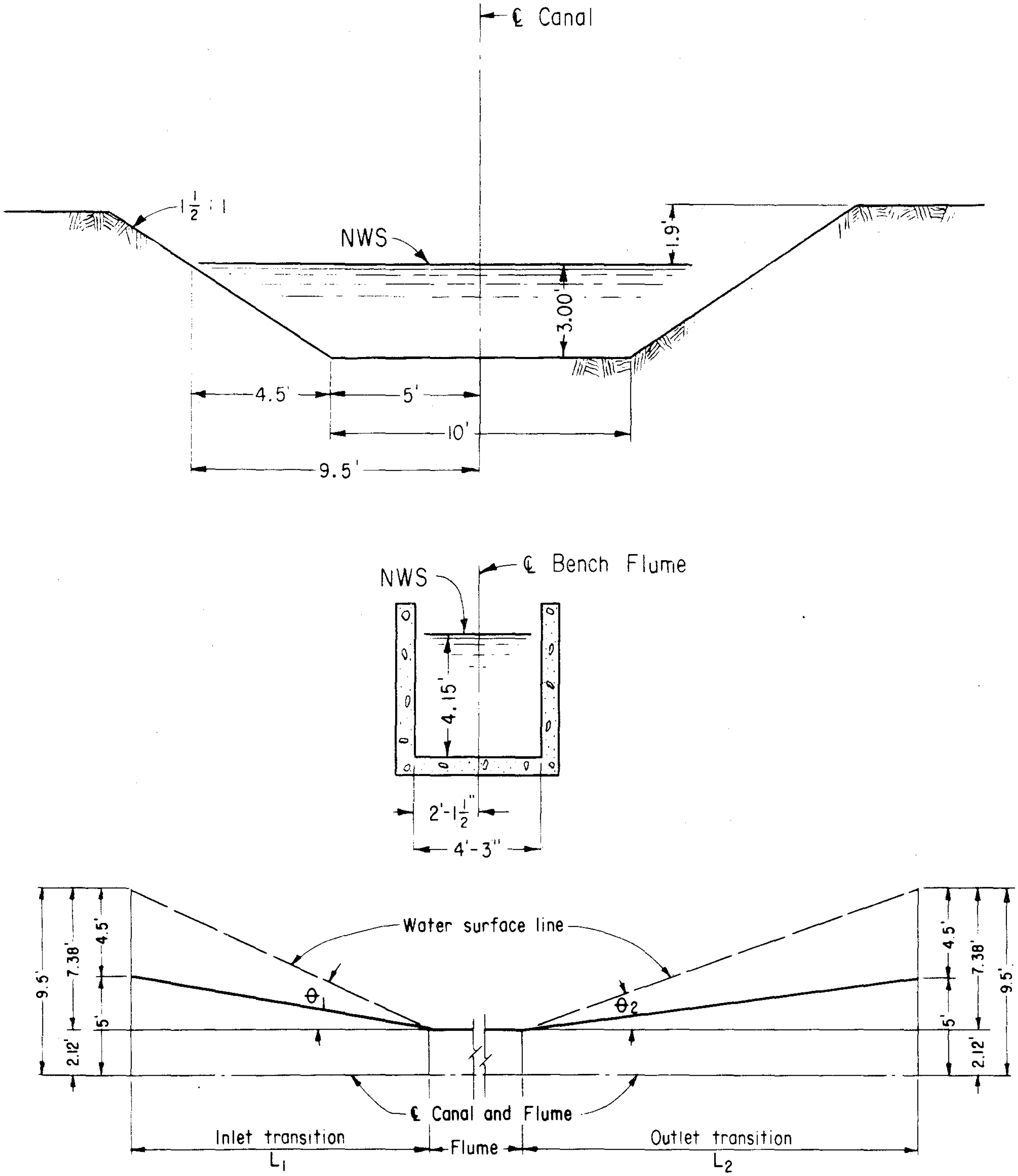


Figure 2-15. **Transition lengths. 103-D-1259**

(4) *Inlet transition elevations.*—The station of the downstream end of the inlet transition is station 1+00 plus 15 feet which equals station 1+15. The station of the upstream end of the outlet transition is station 6+50 minus 20 feet which equals station 6+30. The downstream invert of the inlet transition is set at the same elevation as the upstream invert of the flume.

$$\begin{aligned} \text{U.S. flume invert El.} &= \text{D.S. flume invert El.} + \text{length x slope} \\ &= 997.44 + (\text{Sta. } 6+30 - \text{Sta. } 1+15)\,0.0018 \\ &= 997.44 + 0.93 \\ &= 998.37 \end{aligned}$$

The upstream invert is set at the canal invert elevation which is El. 1000.00. Figure 2-16 shows a schematic profile of the flume.

The energy elevation at the downstream end of the inlet transition is determined as follows:

$$\begin{aligned} \text{Energy El.} &= \text{Flume Inv. El.} + \text{flume normal water depth} + \text{flume velocity head} \\ &= \text{El. } 998.37 + 4.15 + 0.50 \\ &= \text{El. } 1003.02 \end{aligned}$$

The energy elevation required at the upstream end of this transition to discharge design flow through the flume is determined as follows:

$$\text{Energy El. required} = \text{Energy El. at the downstream end of transition} + \text{transition loss} = \text{El. } 1003.02 + 0.3(h_{v_f} - h_v)$$

The energy elevation at this station is also = canal invert elevation + canal depth + velocity head in canal = $1000.00 + d + h_v$.

Equate the two above equations:

$$1003.02 + 0.3(h_{v_f} - h_v) = 1000.00 + d + h_v$$

or

$$d + h_v - 0.3(h_{v_f} - h_v) = 1003.02 - 1000.00 = 3.02 \text{ feet}$$

Assume different canal water depths d, until the following energy balance equation is satisfied:

$$d + h_v - 0.3(h_{v_f} - h_v) = 3.02 \text{ ft.}$$

Assume a canal water depth equal to 3.07 feet.

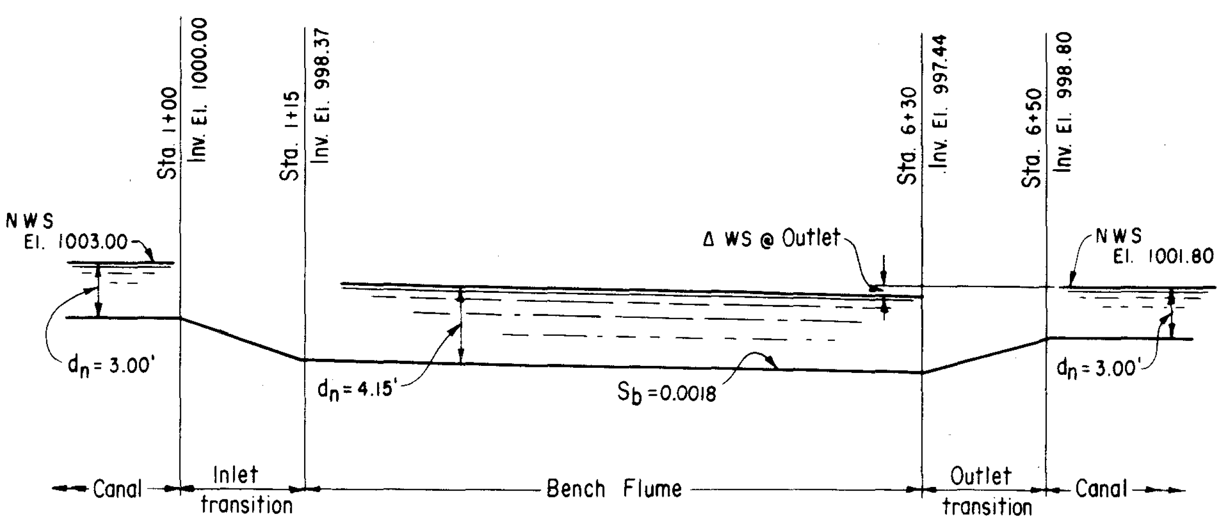


Figure 2-16. Schematic profile of bench flume. 103-D-1260

then:

$$A = 44.85 \text{ ft.}^2$$
$$V = 2.23 \text{ f.p.s.}$$
$$h_v = 0.08 \text{ foot}$$

Substitute the above appropriate values in the energy balance equation to determine if a balance exists:

$$d + h_v - 0.3(h_{v_f} - h_v) = 3.02$$

$$3.07 + 0.08 - 0.3(0.50 - 0.08) = 3.02$$
$$3.02 = 3.02$$

Since a balance does exist the assumed canal depth of 3.07 feet is also the actual canal water depth.

This means the canal depth is 3.07 feet instead of the normal depth of 3.00 feet. Assume in the design example that there is a long reach of canal between this flume and the next upstream structure and the depth in the canal will have returned to normal, therefore any backwater from the flume will not affect the next upstream structure. It can also be assumed that the loss of 0.07 foot of canal freeboard is insignificant.

(5) *Flume freeboard.*—Assume that in the design example, damage would be of less consequence if the bench flume walls were overtopped rather than the canal banks being overtopped. The canal freeboard is 1.9 feet, so determine if about 1 foot of freeboard will be sufficient for the flume.

From the curves in figure 1-9 it can be determined that about 0.75 foot will be adequate freeboard, but 0.85 foot will result in a wall height of 5 feet 0 inch. Therefore, use a 5-foot 0-inch wall height and 0.85-foot freeboard (fig. 2-17).

(6) *Erosion protection.*—Refer to figure 7-8. Protection is not required in the canal upstream from the flume. Downstream from the flume the protection requirement is a 12-inch layer of coarse gravel extending about 8 feet beyond the transition and to an elevation 12 inches above the normal water surface elevation.

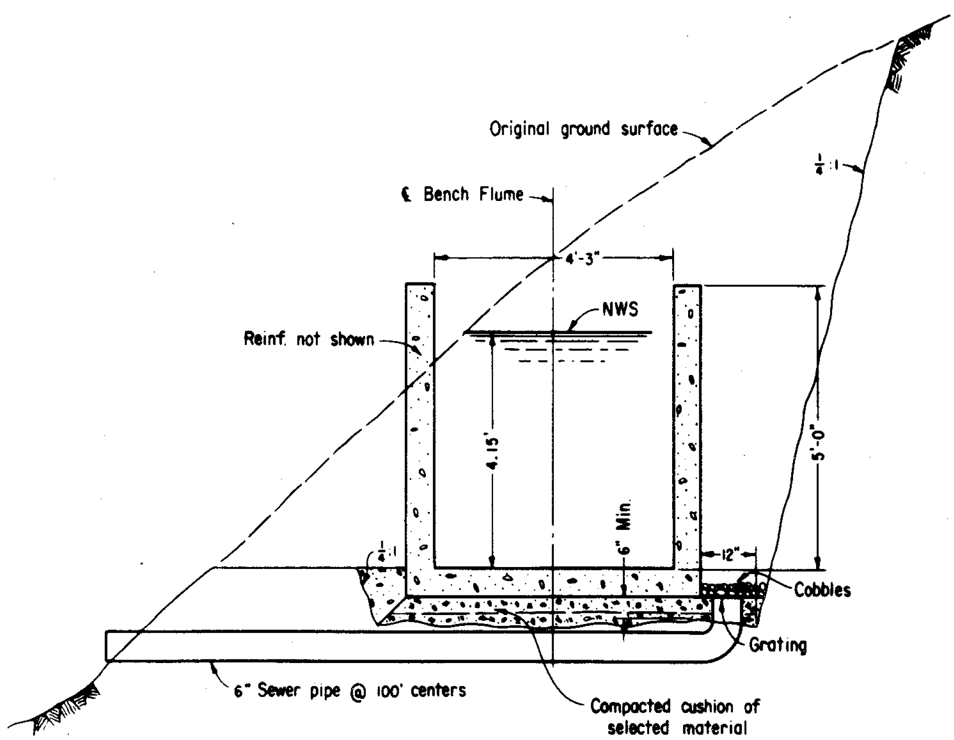


Figure 2-17. **Cross section of bench flume. 103-D-1261**

E. DROPS

R. B. YOUNG[1]

1. General

2-19. *Function.*—The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from this drop. A canal along this same terrain would ordinarily be steep enough to cause severe erosion in earth canals or disruptive flow in hard surface lined canals. The water must therefore be conveyed with a drop structure designed to safely dissipate the excess energy. Different kinds of drops that may be used are vertical, baffled apron, rectangular inclined, and pipe drops.

Subchapter III C, Check-Drops, discusses vertical drops without blocks for drops of 3 feet and less. Subchapter IV C, Overchutes, discusses vertical drops with blocks.

Baffled apron drops (fig. 6-3) may be used for nearly any decrease in water surface elevation where the horizontal distance to accomplish the drops is relatively short. They are particularly adaptable to the situation where the downstream water surface elevation may vary because of such things as degradation or an uncontrolled water surface. A further discussion on baffled aprons may be found in subchapter VI B, Baffled Aprons.

Rectangular inclined (R.I.) drops (figs. 2-18, 2-19, 2-20, 2-21, and 2-22) and pipe drops (figs. 2-23, 2-24, and 2-25) are used where the decrease in elevation is in the range of 3 to 15 feet over a relatively short distance. Economics dictate if it is more practicable to use a pipe drop or an R.I. drop. Usually a pipe drop will be selected for the smaller flows and an R.I. drop will be selected for the larger flows. If the drop crosses another waterway or a roadway it will probably be more economical to use a pipe drop.

Chutes (fig. 2-34) are usually used where the drop in elevation is greater than 15 feet and where the water is conveyed over long distances and along grades that may be flatter

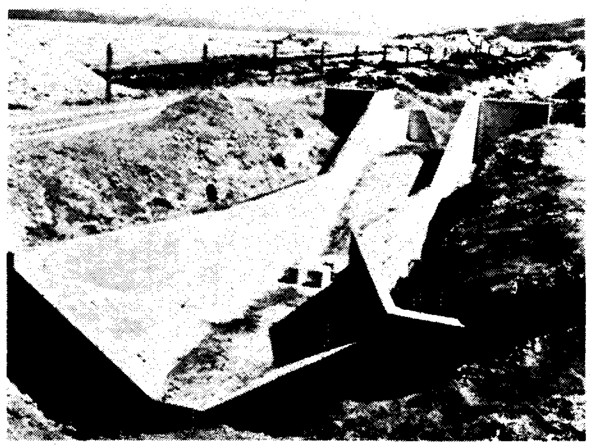


Figure 2-19. Rectangular inclined drop with control notch inlet.

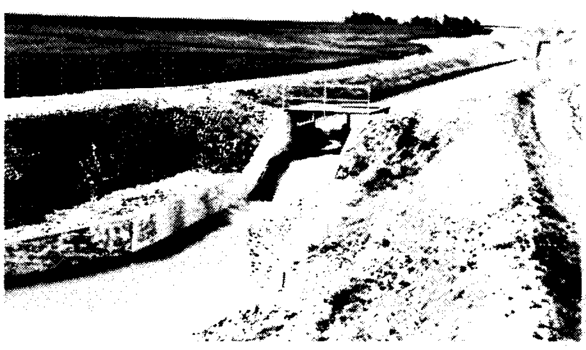


Figure 2-18. Series of rectangular inclined drops. P-328-701-7738

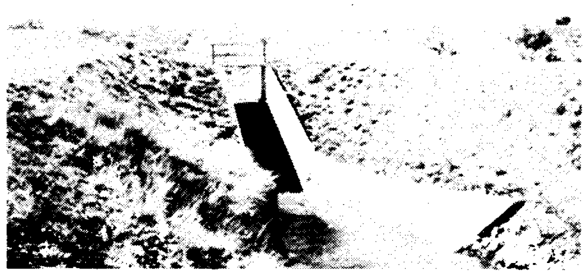


Figure 2-20. Rectangular inclined drop.

[1]Op. cit., p. 19.

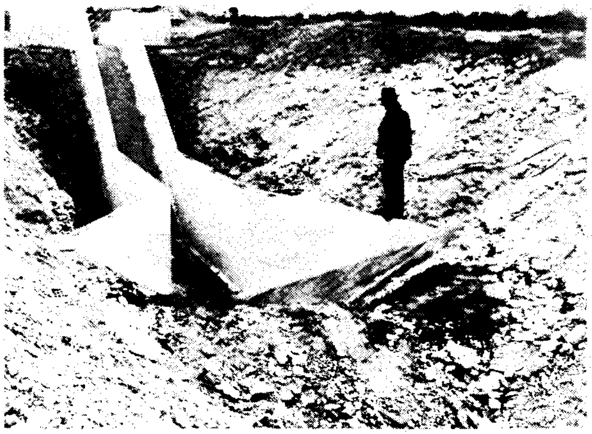


Figure 2-21. Rectangular inclined drop under construction. P 222-117-36402

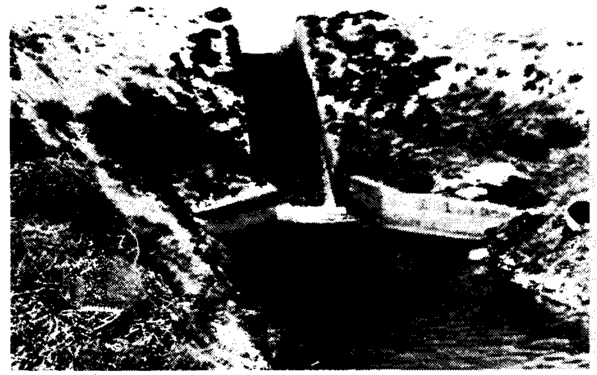


Figure 2-22. Rectangular inclined drop in operation. P 222-116-53590

Figure 2-23. Pipe drop with baffled outlet and pipe collars. P 328-701-8417

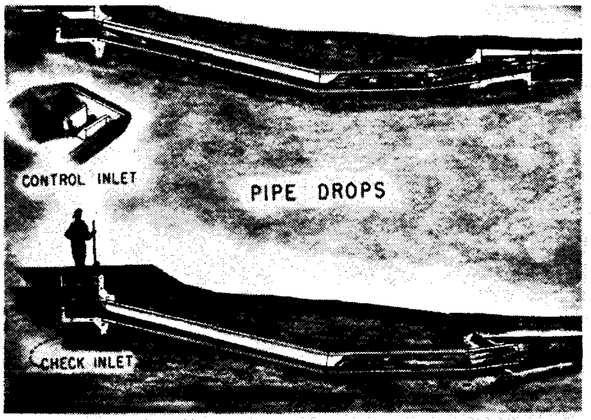


Figure 2-24. Pipe drops. PX-D-31406

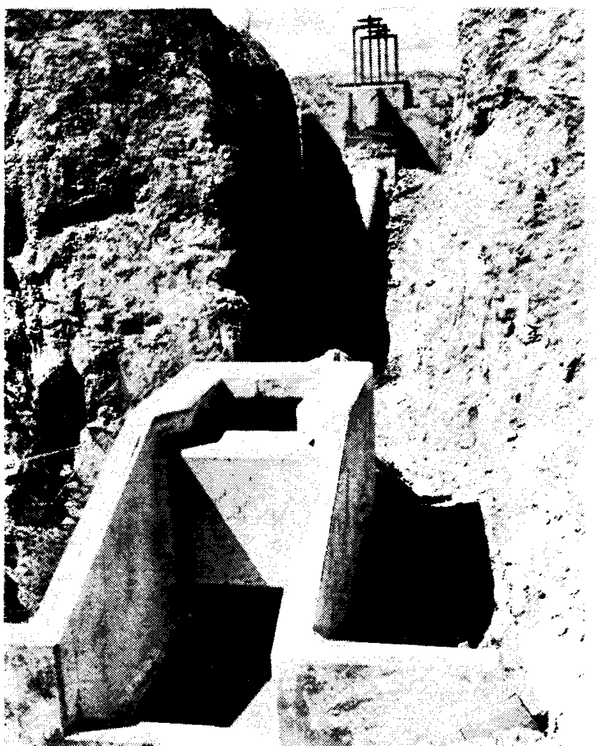


Figure 2-25. Pipe drop with baffled outlet. P-328-701-9502

than those for drops but yet steep enough to maintain supercritical velocities. The decision as to whether to use a chute or a series of drops should be based upon a hydraulic and economic study of the two alternatives. From a hydraulic viewpoint, drops should not be so closely spaced as to possibly preclude uniform flow between outlet and inlet of consecutive structures, particularly where checks or control notches are not used at the inlets. The danger is that sufficient tailwater depths may not exist to produce hydraulic jumps in the pools, and thus shooting flow may develop through the series of drops and possibly damage the canal. Also, with drops too closely spaced on a steep slope, problems of excavation and backfill may make such construction undesirable or prohibitive. Very broadly speaking, about 200 feet of canal should be the minimum between inlet and outlet cutoffs of consecutive drop structures. The economic study should compare costs of a series of drops and a chute, taking into account advantages and disadvantages pertinent to the specific conditions. Since the maintenance costs for a series of drops is usually considerably more than for a chute accomplishing the same function, it is sometimes economically justifiable to spend considerably more for a chute than for a series of drops. A more complete discussion on chutes will be found in subchapter II F.

2-20. ***Hydraulic Design.***—The hydraulic design of a drop should be completed before the structural design with only general consideration initially being given to the structural details. Minimum required field data will be the hydraulic properties and bottom grade elevations of waterway sections above and below the drop, and a profile of the ground line at the drop location with information about the foundation material. Further hydraulic design considerations are included with the discussion of structure components.

2. *Rectangular Inclined Drops*

2-21. ***Purpose and Description.***—A rectangular inclined (R.I.) drop (figs. 2-26 and 2-27) is a rectangular shaped structure with constant width that conveys water from a higher elevation to a lower elevation. This drop in elevation may be any amount between 3 and about 15 feet. The R.I. drop not only conveys water, but it also stills the water after it has reached the lower elevation resulting in excess

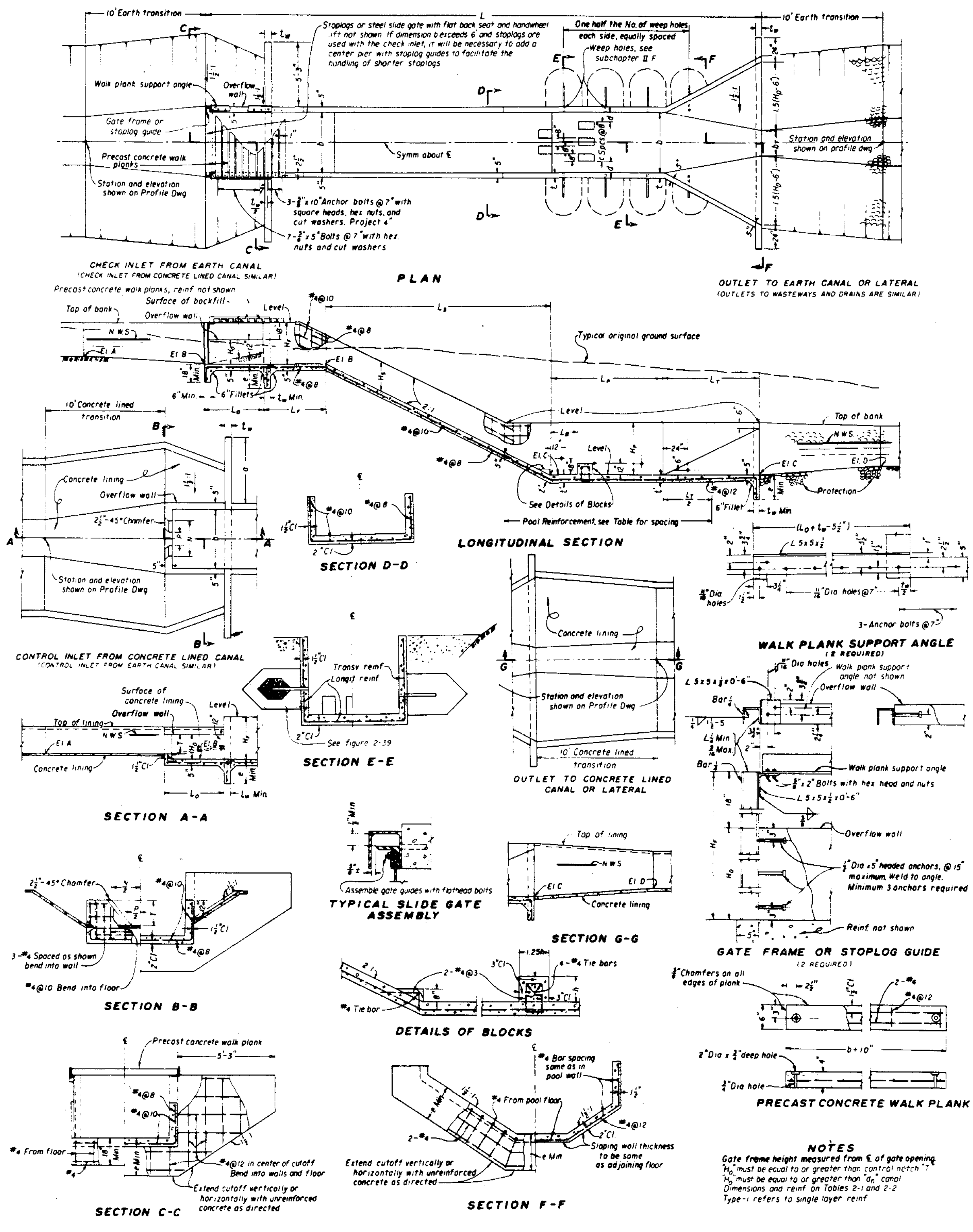


Figure 2-26. Rectangular inclined drop–type 1. 103-D-1262

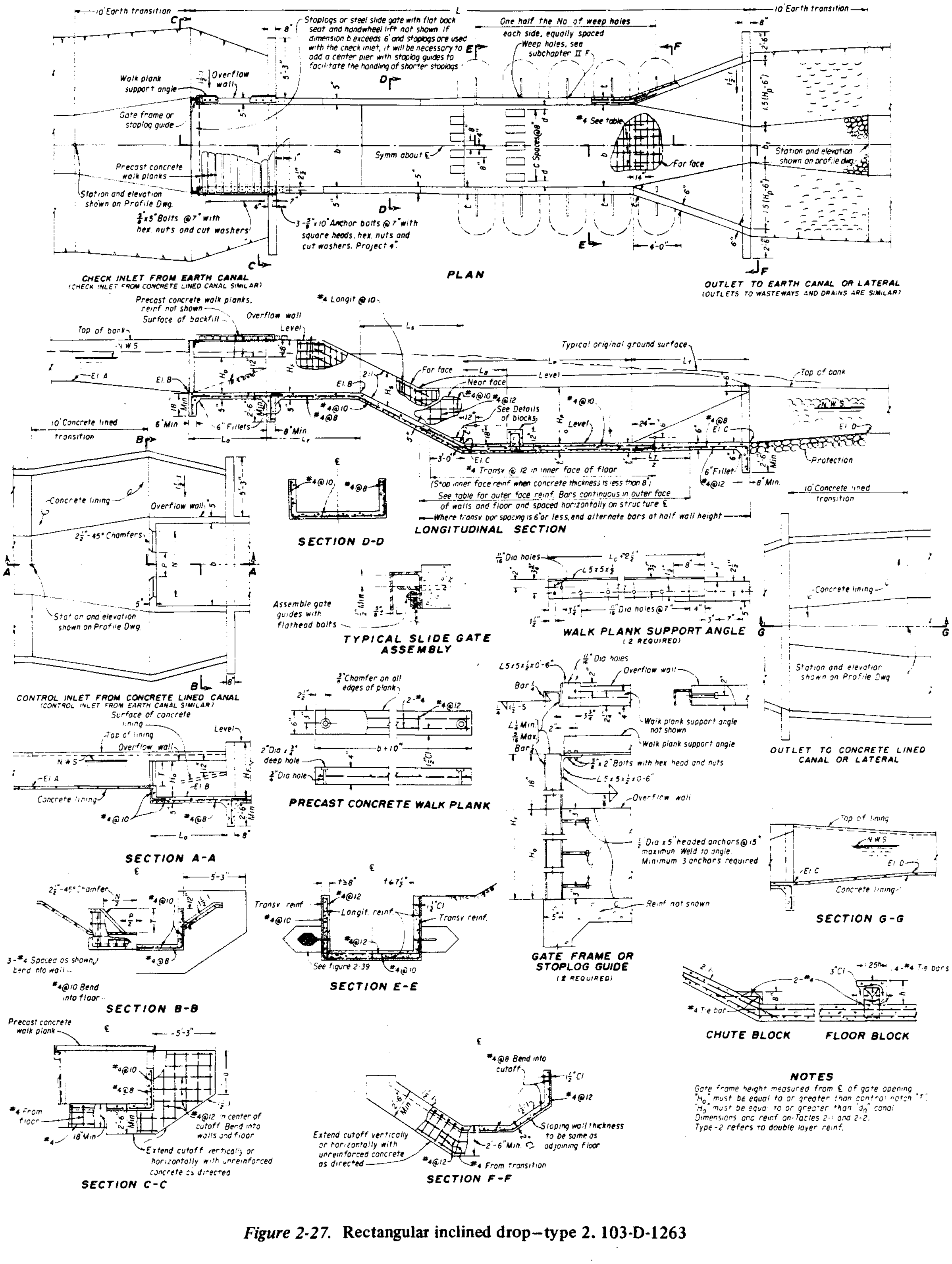


Figure 2-27. **Rectangular inclined drop–type 2. 103-D-1263**

Table 2-1.–*Canal and lateral rectangular inclined drop dimensions and reinforcement (for use with with figs. 2-26 and 2-27) (Sheet 1 of 3)*. 103-D-1264-1

TYPE I
SINGLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF			QUANTITIES		
												TRANSV. BARS IN WALLS & FLOORS	LONGIT. REINF. FLOORS	LONGIT. REINF. WALLS	CONC. (CU.YDS.)	REINF. STEEL (LBS.)	MISC. METAL (LBS.)
5-3	3.0	5	3'-8"	12"	5'-0"	2'-7"	1.28'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	4.7	470	165
5-5	5.0	5	3'-8"	14"	6'-0"	2'-9"	1.42'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	5.5	560	165
5-7	7.0	5	3'-8"	15"	6'-0"	2'-11"	1.54'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	6.1	630	165
5-9	9.0	5	4'-0"	16"	6'-6"	3'-0"	1.63'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	6.8	710	165
5-11	11.0	5	4'-4"	17"	7'-0"	3'-1"	1.71'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	7.5	790	165
5-13	13.0	5	4'-8"	17"	7'-0"	3'-2"	1.77'	8	5"	6½"	4	#4@12	#4@7½	#4@10	8.1	860	165
5-15	15.0	5	5'-0"	18"	7'-6"	3'-3"	1.84'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	8.8	940	165
10-3	3.0	10	3'-8"	16"	7'-0"	2'-10"	1.70'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	5.9	600	205
10-5	5.0	10	3'-10"	18"	7'-6"	3'-1"	1.89'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	6.9	700	205
10-7	7.0	10	4'-3"	20"	8'-0"	3'-3"	2.03'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	7.8	800	205
10-9	9.0	10	4'-8"	21"	8'-6"	3'-4"	2.15'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.6	900	205
10-11	11.0	10	5'-0"	22"	9'-0"	3'-5"	2.25'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.4	990	205
10-13	13.0	10	5'-5"	23"	9'-6"	3'-6"	2.33'	8"	5"	7"	6	#4@12	#4@7	#4@10	10.3	1090	205
10-15	15.0	10	5'-10"	24"	10'-0"	3'-7"	2.42'	8"	5"	7"	6	#4@12	#4@7	#4@10	11.1	1190	205
15-3	3.0	15	4'-0"	19"	8'-0"	3'-4"	1.96'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	7.1	710	210
15-5	5.0	15	4'-5"	21"	8'-6"	3'-7"	2.18'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.1	830	210
15-7	7.0	15	4'-10"	23"	9'-6"	3'-10"	2.34'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.3	960	210
15-9	9.0	15	5'-4"	24"	10'-0"	4'-0"	2.48'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	10.4	1080	210
15-11	11.0	15	5'-9"	2'-1"	10'-6"	4'-2"	2.58'	8"	5"	7"	6	#4@11	#4@7	#4@10	11.5	1220	210
15-13	13.0	15	6'-2"	2'-2"	11'-0"	4'-3"	2.69'	8"	5"	7"	6	#4@11	#4@7	#4@10	12.4	1330	210
15-15	15.0	15	6'-8"	2'-3"	11'-0"	4'-4"	2.77'	9"	5"	7"	6	#4@10	#4@7	#4@10	13.2	1440	210
20-3	3.0	20	5'-0"	20"	8'-6"	3'-6"	2.15'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.3	830	250
20-5	5.0	20	5'-0"	23"	9'-6"	3'-10"	2.39'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.5	980	250
20-7	7.0	20	5'-6"	24"	10'-0"	4'-1"	2.56'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	10.7	1110	250
20-9	9.0	20	6'-0"	2'-2"	11'-0"	4'-4"	2.70'	8"	5"	7"	6	#4@10	#4@7	#4@10	12.2	1300	250
20-11	11.0	20	6'-6"	2'-3"	11'-6"	4'-6"	2.83'	9"	5"	7"	6	#4@9	#4@7	#4@10	13.3	1470	250
20-13	13.0	20	7'-0"	2'-4"	12'-0"	4'-8"	2.94'	9"	5"	7"	6	#4@8	#4@7	#4@10	14.4	1630	250
20-15	15.0	20	7'-6"	2'-5"	12'-0"	4'-9"	3.03'	9"	5"	7"	6	#4@8	#4@7	#4@10	15.4	1740	250
25-3	3.0	25	5'-2"	22"	9'-0"	3'-10"	2.29'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.2	920	255
25-5	5.0	25	5'-6"	24"	10'-0"	4'-2"	2.53'	8"	5"	6½"	6	#4@11	#4@7½	#4@10	10.6	1100	255
25-7	7.0	25	6'-0"	2'-2"	11'-0"	4'-6"	2.72'	8"	5"	6½"	6	#4@9	#4@7½	#4@10	12.1	1310	255
25-9	9.0	25	6'-7"	2'-4"	11'-6'	4'-8"	2.87'	9"	5"	7"	8	#4@8	#4@7	#4@10	13.4	1500	255
25-11	11.0	25	7'-2"	2'-5"	12'-0"	4'-10"	3.00'	9"	5"	7"	8	#4@8	#4@7	#4@10	14.6	1640	255
25-13	13.0	25	7'-8"	2'-6"	12'-6"	5'-0"	3.11'	10"	6"	7"	8	#4@10	#4@7	#4@8	16.3	1780	255
25-15	15.0	25	8'-2"	2'-7"	13'-0"	5'-2"	3.22'	10"	6"	7½"	8	#4@9	#4@6½	#4@8	17.8	1980	255
30-3	3.0	30	5'-6"	23"	9'-6"	4'-0"	2.41'	8"	5"	6½"	8	#4@12	#4@7½	#4@10	10.2	1030	260
30-5	5.0	30	5'-10"	2'-1"	10'-6"	4'-4"	2.66'	8"	5"	6½"	8	#4@10	#4@7½	#4@10	11.7	1230	260
30-7	7.0	30	6'-5"	2'-3"	11'-0"	4'-7"	2.85'	9"	5"	7"	6	#4@9	#4@7	#4@10	13.2	1430	260
30-9	9.0	30	7'-0"	2'-5"	12'-0"	4'-10"	3.00'	9"	5"	7"	8	#4@8	#4@7	#4@10	14.7	1640	260

Table 2-1.–*Canal and lateral rectangular inclined drop dimensions and reinforcement (for use with figs. 2-26 and 2-27) (Sheet 2 of 3)*.–Continued. 103-D-1264-2

TYPE I
SINGLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF.			QUANTITIES		
												TRANSV. BARS IN WALLS & FLOORS	LONGIT. REINF. FLOORS	LONGIT. REINF. WALLS	CONC. (CU. YDS.)	REINF. STEEL (LBS.)	MISC. METAL (LBS.)
30-11	11.0	30	7'-7"	2'-6"	12'-6"	5'-0"	3.14'	10"	6"	7"	8	#4@10	#4@7	#4@8	16.5	1800	260
30-13	13.0	30	8'-2"	2'-7"	13'-0"	5'-2"	3.26'	10"	6"	7"	8	#4@9	#4@7	#4@8	17.8	1990	260
30-15	15.0	30	8'-9"	2'-8"	13'-6"	5'-4"	3.36'	10"	6"	7½"	8	#4@8	#4@6½	#4@6	19.4	2200	260
40-3	3.0	40	5'-9"	2'-1"	10'-6"	4'-4"	2.67'	8"	5"	6½"	8	#4@10	#4@7½	#4@10	12.4	1230	295
40-5	5.0	40	6'-5"	2'-4"	11'-6"	4'-9"	2.95'	9"	5"	6½"	8	#4@8	#4@7½	#4@10	14.3	1500	295
40-7	7.0	40	7'-1"	2'-6"	12'-6"	5'-0"	3.16'	10"	6"	7"	8	#4@10	#4@7	#4@8	16.7	1710	295
40-9	9.0	40	7'-8"	2'-8"	13'-0"	5'-3"	3.34'	10"	6"	7"	8	#4@8	#4@7	#4@8	18.2	1970	295
40-11	11.0	40	8'-4"	2'-9"	13'-6"	5'-5"	3.49'	11"	6"	7"	8	#4@8	#4@7	#4@8	19.7	2150	295
40-13	13.0	40	9'-0"	2'-11"	14'-6"	5'-7"	3.62'	11"	6"	7½"	8	#4@7½	#4@6½	#4@8	21.6	2400	295
40-15	15.0	40	9'-7"	3'-0"	15'-0"	5'-9"	3.73'	11"	6"	7½"	8	#4@7	#4@6½	#4@8	23.0	2600	295
50-3	3.0	50	6'-3"	2'-3"	11'-0"	4'-7"	2.89'	9"	5"	6½"	8	#4@9	#4@7½	#4@10	13.9	1410	310
50-5	5.0	50	7'-0"	2'-6"	12'-6"	5'-0"	3.19'	10"	6"	7"	8	#4@10	#4@7	#4@8	16.7	1700	310
50-7	7.0	50	7'-8"	2'-8"	13'-6"	5'-4"	3.41'	10"	6"	7"	8	#4@8	#4@7	#4@8	18.6	2010	310
50-9	9.0	50	8'-4"	2'-9"	14'-0"	5'-7"	3.59'	11"	6"	7"	8	#4@7½	#4@7	#4@8	20.3	2240	310
50-11	11.0	50	9'-0"	3'-0"	15'-0"	5'-10"	3.76'	12"	6"	7½"	8	#4@7	#4@6½	#4@8	22.5	2530	310
50-13	13.0	50	9'-8"	3'-1"	15'-6"	6'-0"	3.89'	12"	6"	7½"	10	#4@6½	#4@6½	#4@8	24.1	2770	310
60-3	3.0	60	6'-6"	2'-5"	12'-0"	4'-10"	3.07'	9"	5"	6½"	8	#4@8	#4@7½	#4@10	15.4	1610	345
60-5	5.0	60	7'-3"	2'-8"	13'-0"	5'-3"	3.38'	10"	6"	7"	8	#4@8	#4@7	#4@8	18.2	1950	345
60-7	7.0	60	8'-0"	2'-9"	14'-0"	5'-7"	3.61'	11"	6"	7"	8	#4@7½	#4@7	#4@8	20.3	2230	345
60-9	9.0	60	8'-8"	3'-0"	15'-0"	5'-10"	3.81'	12"	6"	7"	8	#4@7	#4@7	#4@8	22.3	2510	345
60-11	11.0	60	9'-5"	3'-2"	16'-0"	6'-1"	3.97'	12"	6"	7½"	10	#4@6	#4@6½	#4@8	24.6	2880	345
70-3	3.0	70	6'-9"	2'-6"	12'-6"	5'-0"	3.21'	10"	6"	6½"	8	#4@10	#4@7½	#4@8	17.2	1740	350
70-5	5.0	70	7'-6"	2'-9"	14'-0"	5'-5"	3.54'	11"	6"	7"	8	#4@8	#4@7	#4@8	20.0	2150	350
70-7	7.0	70	8'-3"	3'-0"	15'-0"	5'-10"	3.78'	11"	6"	7"	8	#4@7	#4@7	#4@8	22.4	2500	350
70-9	9.0	70	9'-0"	3'-2"	16'-0"	6'-1"	3.99'	12"	6"	7½"	10	#4@6	#4@6½	#4@8	24.9	2900	350
70-11	11.0	70	9'-9"	3'-4"	16'-6"	6'-4"	4.15'	13"	7"	7½"	10	#4@7½	#4@6½	#4@7	27.5	3060	350
80-3	3.0	80	7'-0"	2'-8"	13'-6"	5'-4"	3.46'	10"	6"	6½"	10	#4@8	#4@7½	#4@8	18.1	1930	355
80-5	5.0	80	7'-9"	3'-0"	15'-0"	5'-9"	3.80'	12"	6"	7"	10	#4@7	#4@7	#4@8	21.1	2350	355
80-7	7.0	80	8'-6"	3'-3"	16'-0"	6'-1"	4.06'	12"	6"	7"	10	#4@6	#4@7	#4@8	23.4	2730	355
80-9	9.0	80	9'-4"	3'-5"	17'-0"	6'-5"	4.27'	13"	7"	7½"	10	#4@7	#4@6½	#4@7	26.9	3020	355
90-3	3.0	90	7'-3"	2'-9"	13'-6"	5'-4"	3.56'	11"	6"	6½"	10	#4@8	#4@7½	#4@8	19.6	2100	395
90-5	5.0	90	8'-0"	3'-1"	15'-0"	5'-10"	3.91'	12"	6"	7"	10	#4@7	#4@7	#4@8	22.7	2530	395
90-7	7.0	90	8'-10"	3'-4"	16'-6"	6'-3"	4.18'	13"	7"	7"	10	#4@6½	#4@7	#4@7	26.3	3010	395
90-9	9.0	90	9'-8"	3'-6"	17'-6"	6'-8"	4.40'	13"	7"	7½"	10	#4@6½	#4@6½	#4@7	29.3	3360	395
100-3	3.0	100	7'-10"	2'-11"	14'-6"	5'-8"	3.78'	11"	6"	6½"	10	#4@7½	#4@7½	#4@8	20.8	2270	400
100-5	5.0	100	8'-8"	3'-3"	16'-0"	6'-2"	4.14'	12"	6"	7"	10	#4@6	#4@7	#4@8	24.0	2800	400
100-7	7.0	100	9'-6"	3'-6"	17'-6"	6'-7"	4.42'	13"	7"	7"	10	#4@5½	#4@7	#4@7	27.7	3330	400
100-9	9.0	100	10'-5"	3'-8"	18'-6"	6'-11"	4.65'	14"	7"	7½"	10	#4@5½	#4@6½	#4@7	30.8	3710	400

Table 2-1.–*Canal and lateral rectangular inclined drop dimensions and reinforcement (for use with figs. 2-26 and 2-27) (Sheet 3 of 3)*.–Continued. 103-D-1264-3

TYPE 2
DOUBLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF. OUTER LAYER: TRANSV. REINF.	POOL REINF. OUTER LAYER: LONGIT. REINF. FLOORS	POOL REINF. OUTER LAYER: LONGIT. REINF. WALLS	QUANTITIES: CONC. (CU. YDS.)	QUANTITIES: REINF. STEEL (LBS.)	QUANTITIES: MISC. METAL (LBS.)
50-15	15.0	50	10'-5"	3'-3"	16'-0"	6'-2"	4.02'	12"	6"	8"	10	#4@6	#4@16	#4@8	25.9	3160	310
60-13	13.0	60	10'-1"	3'-4"	16'-6"	6'-3"	4.12'	13"	7"	8"	10	#4@7½	#4@16	#4@7	27.4	3150	345
60-15	15.0	60	10'-10"	3'-5"	17'-0"	6'-5"	4.26'	13"	7"	8"	10	#4@7	#4@16	#4@7	29.1	3410	345
70-13	13.0	70	10'-6"	3'-5"	17'-0"	6'-6"	4.31'	13"	7"	8"	10	#4@6½	#4@16	#4@7	29.7	3530	350
70-15	15.0	70	11'-3"	3'-7"	17'-6"	6'-8"	4.44'	14"	7"	8"	10	#4@6½	#4@16	#4@7	31.5	3760	350
80-11	11.0	80	10'-1"	3'-7"	17'-6"	6'-8"	4.46'	14"	7"	8"	10	#4@6½	#4@16	#4@7	29.2	3480	355
80-13	13.0	80	10'-10"	3'-8"	18'-6"	6'-10"	4.63'	14"	7"	8"	10	#4@6	#4@16	#4@7	31.3	3810	355
80-15	15.0	80	11'-8"	3'-10"	19'-0"	7'-0"	4.77'	15"	7"	8½"	10	#4@5½	#4@15	#4@7	33.6	4160	355
90-11	11.0	90	10'-6"	3'-8"	18'-0"	7'-0"	4.59'	14"	7"	8"	10	#4@5½	#4@16	#4@7	31.9	3980	395
90-13	13.0	90	11'-4"	3'-10"	19'-0"	7'-2"	4.75'	14"	7"	8"	10	#4@5	#4@16	#4@7	34.2	4390	395
90-15	15.0	90	12'-1"	3'-11"	19'-6"	7'-4"	4.90'	15"	8"	8½"	10	#4@6	#4@15	#4@16	37.8	4880	395
100-11	11.0	100	11'-3"	3'-10"	19'-0"	7'-2"	4.85'	15"	7"	8"	10	#4@5	#4@16	#4@7	33.4	4290	400
100-13	13.0	100	12'-1"	4'-0"	20'-0"	7'-4"	5.02'	15"	8"	8½"	10	#4@6	#4@15	#4@16	37.4	4830	400
100-15	15.0	100	13'-0"	4'-2"	20'-6"	7'-6"	5.17'	16"	8"	8½"	10	#4@5½	#4@15	#4@16	39.6	5220	400

STRUCT. Q MAX.	STANDARD DIMENSIONS: b_T	a	b	L_O	H_F MIN.	H_O MIN.	H_S	c	d	L_T	CHECK INLET: GATE SIZE	FRAME HT.	NO. OF WALK PLANKS	TYPE I ONLY: e	t_w
5	24"	21"	3'-0"	2'-7"	2'-6"	12"	20"	3	6"	5'-0"	36"x12"	5'-0"	7	24"	6"
10	24"	2'-1"	3'-6"	3'-7"	2'-10"	16"	24"	3	9"	6'-0"	42"x16"	5'-6"	8	24"	6"
15	24"	2'-5"	4'-0"	3'-7"	3'-2"	20"	2'-2"	5	4"	7'-0"	48"x20"	5'-6"	8	24"	6"
20	24"	2'-7"	4'-6"	4'-7"	3'-4"	22"	2'-4"	5	7"	8'-0"	54"x22"	5'-6"	10	24"	6"
25	24"	2'-9"	5'-0"	4'-7"	3'-6"	24"	2'-5"	5	10"	8'-0"	60"x24"	5'-6"	10	24"	6"
30	24"	3'-0"	5'-6"	4'-7"	3'-9"	2'-3"	2'-6"	7	5"	9'-0"	66"x27"	6'-0"	10	24"	6"
40	24"	3'-6"	6'-0"	5'-7"	4'-0"	2'-6"	2'-8"	7	8"	9'-0"	72"x30"	6'-0"	11	2'-6"	8"
50	2'-6"	4'-0"	6'-6"	5'-7"	4'-3"	2'-9"	2'-9"	7	11"	10'-0"	78"x33"	6'-0"	11	2'-6"	8"
60	2'-6"	4'-3"	7'-0"	6'-7"	4'-6"	3'-0"	2'-10"	9	6"	10'-0"	84"x36"	6'-0"	13	2'-6"	8"
70	3'-0"	4'-6"	7'-6"	6'-7"	4'-9"	3'-3"	2'-11"	9	9"	11'-0"	90"x39"	6'-6"	13	2'-6"	8"
80	3'-0"	4'-6"	7'-6"	6'-7"	5'-0"	3'-6"	3'-0"	9	9"	11'-0"	90"x42"	6'-6"	13	2'-6"	8"
90	3'-6"	5'-0"	8'-0"	7'-7"	5'-3"	3'-9"	3'-1"	11	4"	12'-0"	96"x45"	6'-6"	15	2'-6"	8"
100	3'-6"	5'-3"	8'-0"	7'-7"	5'-6"	4'-0"	3'-3"	11	4"	12'-0"	96"x45"	6'-6"	15	2'-6"	8"

DESIGN CRITERIA

Structure No. 5-3 indicates Q = 5 c.f.s., H = 3 ft.

The dimensions of the control notch at the inlet of the structure should be determined for design Q and 20% of design Q with control notch graphs as shown in design example.

The base width of the structure is determined from the formula $b = \frac{360\sqrt{Q}}{Q+350}$

The minimum downstream energy gradient for full design capacity is determined by reducing the assumed "n" value for the channel by 20% and computing the depth.

"H" is the vertical fall from the normal upstream energy gradient to the minimum downstream energy gradient.

"d_2" is the height of the downstream end of the hydraulic jump and may be determined from the formula $d_2 = \frac{-d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}}$ or from figure 2-37.

"d_1" may be determined by using figure 2-37, then from d_1, A_1 and V_1 can be determined.

The invert El. of the stilling pool is found by subtracting $d_2 + h_{v_2}$ from the downstream energy gradient.

The minimum stilling pool length is $4d_2$.

The minimum freeboard for the stilling pool is obtained by using figure 2-33 as a guide.

The minimum distance between chute blocks and floor blocks is $.8d_2$.

Reinforcement steel design is based on working stresses of 24,000 p.s.i. with a specified minimum yield strength of 60,000 p.s.i.

Monolithic concrete design based on a compressive strength of 4,000 p.s.i. at 28 days with a working stress of 1,800 p.s.i.

When a gate with a specified height is not available, a gate with next greater available height should be used with appropriate frame height.

Table 2-2.—*Wasteway and drain rectangular inclined drop dimensions and reinforcement (for use with figs. 2-26 and 2-27) (Sheet 1 of 3).* 103-D-1265-1

TYPE I
SINGLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF. TRANSV. BARS IN WALLS & FLOORS	POOL REINF. LONGIT. BARS IN FLOOR	POOL REINF. LONGIT. BARS IN WALLS	QUANTITIES CONC. (CU.YDS.)	QUANTITIES REINF. STEEL (LBS.)	QUANTITIES MISC. METAL (LBS.)
5-3	3.0	5	3'-8"	12"	4'-0"	2'-7"	1.28'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	4.6	460	165
5-5	5.0	5	3'-8"	14"	4'-6"	2'-9"	1.42'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	5.3	540	165
5-7	7.0	5	3'-8"	15"	5'-0"	2'-11"	1.54'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	6.0	620	165
5-9	9.0	5	4'-0"	16"	5'-0"	3'-0"	1.63'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	6.7	700	165
5-11	11.0	5	4'-4"	17"	5'-6"	3'-1"	1.71'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	7.3	780	165
5-13	13.0	5	4'-8"	17"	5'-6"	3'-2"	1.77'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	8.0	860	165
5-15	15.0	5	5'-0"	18"	5'-6"	3'-3"	1.84'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	8.6	930	165
10-3	3.0	10	3'-8"	16"	5'-0"	2'-10"	1.70'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	5.8	580	205
10-5	5.0	10	3'-10"	18"	5'-6"	3'-1"	1.89'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	6.7	680	205
10-7	7.0	10	4'-3"	20"	6'-0"	3'-3"	2.03'	8"	5"	6½"	4	#4@12	#4@7½	#4@10	7.6	780	205
10-9	9.0	10	4'-8"	21"	6'-6"	3'-4"	2.15'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.4	880	205
10-11	11.0	10	5'-0"	22"	7'-0"	3'-5"	2.25'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.2	970	205
10-13	13.0	10	5'-5"	23"	7'-0"	3'-6"	2.33'	8"	5"	7"	6	#4@12	#4@7	#4@10	10.1	1070	205
10-15	15.0	10	5'-10"	24"	7'-6"	3'-7"	2.42'	8"	5"	7"	6	#4@12	#4@7	#4@10	10.8	1160	205
15-3	3.0	15	4'-0"	19"	6'-0"	3'-4"	1.96'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	6.9	690	210
15-5	5.0	15	4'-5"	21"	6'-6"	3'-7"	2.18'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	7.9	810	210
15-7	7.0	15	4'-10"	23"	7'-0"	3'-10"	2.34'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.1	940	210
15-9	9.0	15	5'-4"	24"	7'-6"	4'-0"	2.48'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	10.0	1050	210
15-11	11.0	15	5'-9"	2'-1"	8'-0"	4'-2"	2.58'	8"	5"	7"	6	#4@11	#4@7	#4@10	11.1	1180	210
15-13	13.0	15	6'-2"	2'-2"	8'-0"	4'-3"	2.69'	8"	5"	7"	6	#4@11	#4@7	#4@10	12.0	1290	210
15-15	15.0	15	6'-8"	2'-3"	8'-6"	4'-4"	2.77'	9"	5"	7"	6	#4@10	#4@7	#4@10	13.0	1420	210
20-3	3.0	20	5'-0"	20"	6'-6"	3'-6"	2.15'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.0	810	250
20-5	5.0	20	5'-0"	23"	7'-0"	3'-10"	2.39'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.3	950	250
20-7	7.0	20	5'-6"	24"	8'-0"	4'-1"	2.56'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	10.5	1090	250
20-9	9.0	20	6'-0"	2'-2"	8'-0"	4'-4"	2.70'	8"	5"	7"	6	#4@10	#4@7	#4@10	11.8	1260	250
20-11	11.0	20	6'-6"	2'-3"	8'-6"	4'-6"	2.83'	9"	5"	7"	6	#4@9	#4@7	#4@10	12.9	1420	250
20-13	13.0	20	7'-0"	2'-4"	9'-0"	4'-8"	2.94'	9"	5"	7"	6	#4@8	#4@7	#4@10	14.1	1580	250
20-15	15.0	20	7'-6"	2'-5"	9'-0"	4'-9"	3.03'	9"	5"	7"	6	#4@8	#4@7	#4@10	15.0	1700	250
25-3	3.0	25	5'-2"	22"	7'-0"	3'-10"	2.29'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	8.9	900	255
25-5	5.0	25	5'-6"	24"	7'-6"	4'-2"	2.53'	8"	5"	6½"	6	#4@11	#4@7½	#4@10	10.4	1070	255
25-7	7.0	25	6'-0"	2'-2"	8'-0"	4'-6"	2.72'	8"	5"	6½"	6	#4@9	#4@7½	#4@10	11.7	1270	255
25-9	9.0	25	6'-7"	2'-4"	8'-6"	4'-8"	2.87'	9"	5"	7"	6	#4@8	#4@7	#4@10	13.0	1460	255
25-11	11.0	25	7'-2"	2'-5"	9'-0"	4'-10"	3.00'	9"	5"	7"	6	#4@8	#4@7	#4@10	14.2	1590	255
25-13	13.0	25	7'-8"	2'-6"	9'-6"	5'-0"	3.11'	10"	6"	7"	6	#4@10	#4@7	#4@8	15.8	1740	255
25-15	15.0	25	8'-2"	2'-7"	10'-0"	5'-2"	3.22'	10"	6"	7½"	8	#4@9	#4@6½	#4@8	17.3	1930	255
30-3	3.0	30	5'-6"	23"	7'-0"	4'-0"	2.41'	8"	5"	6½"	6	#4@12	#4@7½	#4@10	9.9	1000	260
30-5	5.0	30	5'-10"	2'-1"	8'-0"	4'-4"	2.66'	8"	5"	6½"	6	#4@10	#4@7½	#4@10	11.4	1200	260
30-7	7.0	30	6'-5"	2'-3"	8'-6"	4'-7"	2.85'	9"	5"	7"	6	#4@9	#4@7	#4@10	12.9	1400	260
30-9	9.0	30	7'-0"	2'-5"	9'-0"	4'-10"	3.00'	9"	5"	7"	6	#4@8	#4@7	#4@10	14.3	1600	260

Table 2-2.—*Wasteway and drain rectangular inclined drop dimensions and reinforcement (for use with figs. 2-26 and 2-27) (Sheet 2 of 3)*.—Continued. 103-D-1265-2

TYPE I
SINGLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF.			QUANTITIES		
												TRANSV. BARS IN WALLS & FLOORS	LONGIT. BARS IN		CONC. (CU. YDS.)	REINF. STEEL (LBS.)	MISC. METAL (LBS.)
													FLOOR	WALLS			
30-11	11.0	30	7'-7"	2'-6"	9'-6"	5'-0"	3.14'	10"	6"	7"	6	#4@10	#4@7	#4@8	16.0	1750	260
30-13	13.0	30	8'-2"	2'-7"	10'-0"	5'-2"	3.26'	10"	6"	7"	8	#4@9	#4@7	#4@8	17.3	1940	260
30-15	15.0	30	8'-9"	2'-8"	10'-0"	5'-4"	3.36'	10"	6"	7½"	8	#4@8	#4@6½	#4@8	18.8	2150	260
40-3	3.0	40	5'-9"	2'-1"	8'-0"	4'-4"	2.67'	8"	5"	6½"	6	#4@10	#4@7½	#4@10	12.0	1180	295
40-5	5.0	40	6'-5"	2'-4"	9'-0"	4'-9"	2.95'	9"	5"	6½"	6	#4@8	#4@7½	#4@10	13.9	1450	295
40-7	7.0	40	7'-1"	2'-6"	9'-6"	5'-0"	3.16'	10"	6"	7"	6	#4@10	#4@7	#4@8	16.1	1660	295
40-9	9.0	40	7'-8"	2'-8"	10'-0"	5'-3"	3.34'	10"	6"	7"	8	#4@8	#4@7	#4@8	17.7	1910	295
40-11	11.0	40	8'-4"	2'-9"	10'-6"	5'-5"	3.49'	11"	6"	7"	8	#4@8	#4@7	#4@8	19.1	2090	295
40-13	13.0	40	9'-0"	2'-11"	11'-0"	5'-7"	3.62'	11"	6"	7½"	8	#4@7½	#4@6½	#4@8	20.8	2310	295
40-15	15.0	40	9'-7"	3'-0"	11'-6"	5'-9"	3.73'	11"	6"	7½"	8	#4@7	#4@6½	#4@8	22.3	2520	295
50-3	3.0	50	6'-3"	2'-3"	8'-6"	4'-7"	2.89'	9"	5"	6½"	6	#4@9	#4@7½	#4@10	13.6	1380	310
50-5	5.0	50	7'-0"	2'-6"	9'-6"	5'-0"	3.19'	10"	6"	7"	6	#4@10	#4@7	#4@8	16.2	1650	310
50-7	7.0	50	7'-8"	2'-8"	10'-0"	5'-4"	3.41'	10"	6"	7"	8	#4@8	#4@7	#4@8	18.1	1950	310
50-9	9.0	50	8'-4"	2'-10"	11'-0"	5'-7"	3.59'	11"	6"	7"	8	#4@7½	#4@7	#4@8	19.8	2180	310
50-11	11.0	50	9'-0"	3'-0"	11'-6"	5'-10"	3.76'	12"	6"	7½"	8	#4@7	#4@6½	#4@8	21.7	2440	310
50-13	13.0	50	9'-8"	3'-1"	11'-6"	6'-0"	3.89'	12"	6"	7½"	8	#4@6½	#4@6½	#4@8	23.3	2680	310
60-3	3.0	60	6'-6"	2'-5"	9'-0"	4'-10"	3.07'	9"	5"	6½"	6	#4@8	#4@7½	#4@10	14.9	1550	345
60-5	5.0	60	7'-3"	2'-8"	10'-0"	5'-3"	3.38'	10"	6"	7"	8	#4@8	#4@7	#4@8	17.7	1890	345
60-7	7.0	60	8'-0"	2'-10"	11'-0"	5'-7"	3.61'	11"	6"	7"	8	#4@7½	#4@7	#4@8	19.7	2170	345
60-9	9.0	60	8'-8"	3'-0"	11'-6"	5'-10"	3.81'	12"	6"	7"	8	#4@7	#4@7	#4@8	21.7	2440	345
60-11	11.0	60	9'-5"	3'-2"	12'-0"	6'-1"	3.97'	12"	6"	7½"	8	#4@6	#4@6½	#4@8	23.8	2780	345
70-3	3.0	70	6'-9"	2'-6"	9'-6"	5'-0"	3.21'	10"	6"	6½"	8	#4@10	#4@7½	#4@8	16.6	1680	350
70-5	5.0	70	7'-6"	2'-9"	10'-6"	5'-5"	3.54'	11"	6"	7"	8	#4@8	#4@7	#4@8	19.2	2060	350
70-7	7.0	70	8'-3"	3'-0"	11'-6"	5'-10"	3.78'	11"	6"	7"	8	#4@7	#4@7	#4@8	21.5	2400	350
70-9	9.0	70	9'-0"	3'-2"	12'-0"	6'-1"	3.99'	12"	6"	7½"	8	#4@6	#4@6½	#4@8	23.9	2790	350
70-11	11.0	70	9'-9"	3'-3"	12'-6"	6'-4"	4.15'	13"	7"	7½"	8	#4@7½	#4@6½	#4@7	26.5	2950	350
80-3	3.0	80	7'-0"	2'-8"	10'-0"	5'-4"	3.46'	10"	6"	6½"	8	#4@8	#4@7½	#4@8	17.6	1870	355
80-5	5.0	80	7'-9"	3'-0"	11'-0"	5'-9"	3.80'	12"	6"	7"	8	#4@7	#4@7	#4@8	20.3	2250	355
80-7	7.0	80	8'-6"	3'-2"	12'-0"	6'-1"	4.06'	12"	6"	7"	8	#4@6	#4@7	#4@8	22.5	2630	355
80-9	9.0	80	9'-4"	3'-4"	12'-6"	6'-5"	4.27'	13"	7"	7½"	8	#4@7	#4@6½	#4@7	25.9	2910	355
90-3	3.0	90	7'-3"	2'-9"	10'-6"	5'-4"	3.56'	11"	6"	6½"	8	#4@8	#4@7½	#4@8	19.0	2030	395
90-5	5.0	90	8'-0"	3'-1"	11' 6"	5'-10"	3.91'	12"	6"	7"	8	#4@7	#4@7	#4@8	22.1	2460	395
90-7	7.0	90	8'-10"	3'-3"	12'-6"	6'-3"	4.18'	13"	7"	7"	8	#4@6½	#4@7	#4@7	25.3	2900	395
90-9	9.0	90	9'-8"	3'-6"	13'-0"	6'-8"	4.40'	13"	7"	7½"	10	#4@6½	#4@6½	#4@7	28.3	3250	395
100-3	3.0	100	7'-10"	2'-11"	11'-0"	5'-8"	3.78'	11"	6"	6½"	8	#4@7½	#4@7½	#4@8	20.2	2200	400
100-5	5.0	100	8'-8"	3'-3"	12'-0"	6'-2"	4.14'	12"	6"	7"	8	#4@6	#4@7	#4@8	23.2	2690	400
100-7	7.0	100	9'-6"	3'-6"	13'-0"	6'-7"	4.42'	13"	7"	7"	8	#4@5½	#4@7	#4@7	26.5	3180	400
100-9	9.0	100	10'-5"	3'-8"	14'-0"	6'-11"	4.65'	14"	7"	7½"	10	#4@5½	#4@6½	#4@7	29.6	3550	400

Table 2-2.–*Wasteway and drain rectangular inclined drop dimensions and reinforcement (for use with figs. 2-26 and 2-27) (Sheet 3 of 3)*.–Continued. 103-D-1265-3

TYPE 2
DOUBLE LAYER REINFORCEMENT IN POOL FLOOR

STRUCT. NO.	H MAX.	Q MAX.	L_F	L_B	L_P	H_P	$d_2 + hv_2$	h	t	t'	NO. OF WEEP HOLES	POOL REINF. OUTER LAYER: TRANSV. REINF.	POOL REINF. OUTER LAYER: LONGIT. REINF. FLOOR	POOL REINF. OUTER LAYER: LONGIT. REINF. WALLS	QUANTITIES: CONC. (CU. YDS.)	QUANTITIES: REINF. STEEL (LBS.)	QUANTITIES: MISC. METAL (LBS.)
50-15	15.0	50	10'-5"	3'-2"	12'-0"	6'-2"	4.02'	12"	6"	8"	8	#4@ 6	#4 @16	#4@ 8	25.1	3050	310
60-13	13.0	60	10'-1"	3'-3"	12'-6"	6'-3"	4.12'	13"	7"	8"	8	#4@ 7½	#4 @16	#4@ 7	26.4	3040	345
60-15	15.0	60	10'-10"	3'-5"	13'-0"	6'-5"	4.26'	13"	7"	8"	8	#4@ 7	#4 @16	#4@ 7	28.2	3290	345
70-13	13.0	70	10'-6"	3'-5"	13'-0"	6'-6"	4.31'	13"	7"	8"	8	#4@ 6½	#4 @16	#4@ 7	28.6	3400	350
70-15	15.0	70	11'-3"	3'-6"	13'-6"	6'-8"	4.44'	14"	7"	8"	10	#4@ 6½	#4 @16	#4@ 7	30.4	3620	350
80-11	11.0	80	10'-1"	3'-6"	13'-6"	6'-8"	4.46'	14"	7"	8"	10	#4@ 6½	#4 @16	#4@ 7	28.2	3360	355
80-13	13.0	80	10'-10"	3'-8"	14'-0"	6'-10"	4.63'	14"	7"	8"	10	#4@ 6	#4 @16	#4@ 7	30.1	3650	355
80-15	15.0	80	11'-8"	3'-9"	14'-6"	7'-0"	4.77'	15"	7"	8½"	10	#4@ 5½	#4 @15	#4@ 7	32.3	3990	355
90-11	11.0	90	10'-6"	3'-7"	13'-6"	7'-0"	4.59'	14"	7"	8"	10	#4@ 5½	#4 @16	#4@ 7	30.9	3840	395
90-13	13.0	90	11'-4"	3'-10"	14'-0"	7'-2"	4.75'	14"	7"	8"	10	#4@ 5	#4 @16	#4@ 7	32.8	4200	395
90-15	15.0	90	12'-1"	3'-11"	14'-6"	7'-4"	4.90'	15"	8"	8½"	10	#4@ 6	#4 @15	#4@16	36.3	4670	395
100-11	11.0	100	11'-3"	3'-10"	14'-6"	7'-2"	4.85'	15"	7"	8"	10	#4@ 5	#4 @16	#4@ 7	32.4	4140	400
100-13	13.0	100	12'-1"	4'-0"	15'-0"	7'-4"	5.02'	15"	8"	8½"	10	#4@ 6	#4 @15	#4@16	35.9	4620	400
100-15	15.0	100	13'-0"	4'-1"	15'-6"	7'-6"	5.17'	16"	8"	8½"	10	#4@ 5½	#4 @15	#4@16	38.0	5000	400

STRUCT. Q MAX.	STANDARD DIMENSIONS: b_T	a	b	L_O	H_F MIN.	H_O MIN.	H_S	c	d	L_T	CHECK INLET: GATE SIZE	FRAME HT.	NO. OF WALK PLANKS	TYPE I ONLY: e	t_w
5	24"	21"	3'-0"	2'-7"	2'-6"	12"	20"	3	6"	5'-0"	36"x12"	5'-0"	7	24"	6"
10	24"	2'-1"	3'-6"	3'-7"	2'-10"	16"	24"	3	9"	6'-0"	42"x16"	5'-6"	8	24"	6"
15	24"	2'-5"	4'-0"	3'-7"	3'-2"	20"	2'-2"	5	4"	7'-0"	48"x20"	5'-6"	8	24"	6"
20	24"	2'-7"	4'-6"	4'-7"	3'-4"	22"	2'-4"	5	7"	8'-0"	54"x22"	5'-6"	10	24"	6"
25	24"	2'-9"	5'-0"	4'-7"	3'-6"	24"	2'-5"	5	10"	8'-0"	60"x24"	5'-6"	10	24"	6"
30	24"	3'-0"	5'-6"	4'-7"	3'-9"	2'-3"	2'-6"	7	5"	9'-0"	66"x27"	6'-0"	10	24"	6"
40	24"	3'-6"	6'-0"	5'-7"	4'-0"	2'-6"	2'-8"	7	8"	9'-0"	72"x30"	6'-0"	11	2'-6"	8"
50	2'-6"	4'-0"	6'-6"	5'-7"	4'-3"	2'-9"	2'-9"	7	11"	10'-0"	78"x33"	6'-0"	11	2'-6"	8"
60	2'-6"	4'-3"	7'-0"	6'-7"	4'-6"	3'-0"	2'-10"	9	6"	10'-0"	84"x36"	6'-0"	13	2'-6"	8"
70	3'-0"	4'-6"	7'-6"	6'-7"	4'-9"	3'-3"	2'-11"	9	9"	11'-0"	90"x39"	6'-6"	13	2'-6"	8"
80	3'-0"	4'-6"	7'-6"	6'-7"	5'-0"	3'-6"	3'-0"	9	9"	11'-0"	90"x42"	6'-6"	13	2'-6"	8"
90	3'-6"	5'-0"	8'-0"	7'-7"	5'-3"	3'-9"	3'-1"	11	4"	12'-0"	96"x45"	6'-6"	15	2'-6"	8"
100	3'-6"	5'-3"	8'-0"	7'-7"	5'-6"	4'-0"	3'-3"	11	4"	12'-0"	96"x45"	6'-6"	15	2'-6"	8"

DESIGN CRITERIA

Structure No. 5-3 indicates Q = 5 cfs., H = 3 ft.

The dimensions of the control notch at the inlet of the structure should be determined for design Q and 20% of design Q with control notch graphs as shown in design example.

The base width of the structure is determined from the formula $b = \frac{360\sqrt{Q}}{Q+350}$

The minimum downstream energy gradient for full design capacity is determined by reducing the assumed "n" value for the channel by 20% and computing the depth. When the stilling pool discharges into an uncontrolled channel, a control must be provided by the outlet structure and critical depth at the control section should be used to determine the downstream energy gradient.

"H" is the vertical fall from the normal upstream energy gradient to the minimum downstream energy gradient.

"d_2" is the height of the downstream end of the hydraulic jump and may be determined from the formula, $d_2 = \frac{-d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}}$ or from figure 2-37.

"d_1" may be determined by using figure 2-37. then from d_1, A_1 and V_1 can be determined.

The invert El. of the stilling pool is found by subtracting $d_2 + h_{v_2}$ from the downstream energy gradient.

The minimum stilling pool length is $3d_2$

The minimum freeboard for the stilling pool is obtained by using figure 2-33 as a guide.

The minimum distance between chute blocks and floor blocks is $.8d_2$

Reinforcement steel design is based on working stresses of 24,000 p.s.i. with a specified minimum yield strength of 60,000 p.s.i.

Monolithic concrete design based on a compressive strength of 4,000 p.s.i. at 28 days with a working stress of 1,800 p.s.i.

When a gate with a specified height is not available, a gate with next greater available height should be used with appropriate frame height.

energy being dissipated. The inlet of the structure may also serve as a control to regulate the water depth in the channel upstream from the drop.

2-22. ***Advantages and Disadvantages.***—Rectangular inclined drops are easily designed, built, and operated. The inlets and outlets can be easily adapted to either an earth or a lined waterway. The inlets can be made to incorporate a control notch, a check, or a weir. If there is a control or check inlet side overflow walls for emergency situations should be included. It is important to provide adequate gravel or rock protection of outlets discharging into unlined waterways. The maximum fall in water surface for any one R.I. drop is about 15 feet. The R.I. drop should have adequate percolation path and sufficient resistance to sliding. The standard R.I. drop structures in figures 2-26 and 2-27 are designed to provide this stability; however, if unusual foundation conditions are encountered, the percolation and sliding resistance should be checked, and additional stability may be obtained by increasing the length L_F.

2-23. ***Structure Components.***—The principal hydraulic elements of an R.I. drop are the upstream transition, the inlet, the short inclined channel, the stilling pool, the outlet, and the downstream transition.

(a) *Upstream Transitions.*—The upstream transition produces a gradual change of velocity from waterway to structure. When using a control inlet there is not a change in invert elevation from waterway to structure. When a check inlet is used the transition usually must provide for a gradual decrease in invert elevation from the waterway to the opening in the structure. The invert slope to a check inlet should not be steeper than 4 to 1. An earth transition may require erosion protection. For design of protection see the discussion on Erosion Protection, subchapter VII B.

(b) *Inlet to Drop.*—The inlet to an R.I. drop may be any of the following:

(1) *Critical-depth control section.*—In an earth canal requiring no check, the inlet to the drop should be designed to provide a control section which will prevent racing upstream and scouring of the canal. The inlet must be designed so that the full capacity can be discharged into the drop with normal depth in the canal. The inlet should be symmetrical about the centerline and, whenever possible, of sufficient distance from horizontal bends upstream as to limit undesirable wave action due to unsymmetrical flow. The trapezoidal critical-depth control should be proportioned in bottom width and side slopes for varying discharge from design flow to 0.2 design flow. For any flow in this range the notch causes the upstream canal water depth to be at or very near normal depth. It may also be proportioned to control only one specific discharge. Generally, the notch is designed for the varying discharge. For method of notch design for control of varying discharge see subchapter III G. The top of the notch should be at the same elevation or slightly higher than the normal upstream water surface. The invert of the notch should be the same elevation as the canal invert. The depth of the notch will be equal to or slightly more than the depth in the canal at normal flow. A control notch which requires slightly more energy than that available from normal flow conditions in the canal will cause a slight rise in water surface elevation in the canal, but this is not considered serious. For the critical condition of canal flow at or near full design flow, the water surface rise is minimized by part of the flow being able to overflow the sidewalls. The sidewalls, with tops set at the same elevation as the top of the notch, also provide for emergency overflow into the inlet structure if there should be an obstruction of the notch or if the flow in the canal is greater than design flow. The overflow sidewalls, figures 2-26 and 2-27 are of sufficient length to permit design flow to go over the sidewalls with complete blockage of the notch. If canal bank freeboard is based on figure 1-9, the resulting freeboard will still be about 0.5 foot.

The invert to the inlet structure (elevation B, figs. 2-26 and 2-27) is set low enough so that the flow at the beginning of the incline will not affect the flow through the control notch. Expressed differently, elevation B is set low enough to prevent the flow into the inclined section from controlling the canal water surface.

The inlet structure also has wingwalls and cutoffs for the purpose of retaining canal earthwork and for reducing seepage from the canal.

(2) *Check.*–Check structures are often combined with the inlet to drops. The checks in such cases are utilized as a control to prevent racing of the water upstream from the inlet, in addition to their usual function of raising the water surface to permit diversion through upstream turnouts during periods of partial flow in the canal. Checks may also be used to shut off the canal flow if there is some provision such as an upstream wasteway to allow the canal flow to go elsewhere. This inlet closure could provide for isolation of a canal reach in case of canal embankment failure or for maintenance purposes.

The area of the inlet opening should be proportioned to limit the design flow velocity to about 3.5 feet per second. This velocity is considered as the maximum desirable for ease of stoplog handling. The width is usually the same as that determined by the width requirement for the stilling pool. If the width of the inlet is greater than 6 feet and stoplogs are used, a center pier with stoplog guides should be added so shorter stoplogs can be used. Stoplogs longer than 6 feet are difficult to handle. Without a center pier, the opening has two gate frame guides for installation of a gate or for use as stoplog guides. The elevation of the inlet opening should be the same or lower than the invert of the canal, never higher. Slide gates may be operated automatically. The gate sizes in the table of dimensions (tables 2-1 and 2-2) cause the top elevation of the gates, when closed, to be approximately the same elevation as the top of the side overflow walls. If these gate sizes are not available and the next greater available height is used, the taller gate may cause the canal freeboard to be less than 0.5 foot when the gate is closed and all of the normal canal flow is spilling over the sidewalls and gate. If this freeboard is much less than 0.5 foot, the length of the sidewalls should be increased until the canal freeboard is about 0.5 foot for this overflow condition. Stoplogs may be used in vertical guides for water depths of 5 feet or less. For greater depths the guides should be placed on a slope of 1/4 to 1 to facilitate handling. The sidewalls, with their tops set at the same elevation as normal water surface in the canal, provide for emergency overflow into the structure if the gate or stoplogs are not appropriately set for a particular flow in the canal, if the inlet becomes obstructed, or if the inlet is closed. For the extreme condition of flow at canal design capacity and complete closure of the opening, the inlets in figures 2-26 and 2-27 are designed so that there will continue to be about 0.5-foot canal bank freeboard if freeboard design is based on figure 1-9. The depth of flow over the walls and closure is dependent on a coefficient of discharge, the length of the walls, the width of the structure, and the flow in the canal.

The floor of the inlet structure (elevation B) is set a distance H_0 below normal water surface in the canal. The vertical distance H_0 in the table of dimensions, tables 2-1 and 2-2 (sheet 3) is great enough so that design flow into the inclined section will not cause backwater in the canal.

Walk planks provide a platform for operating a gate or handling stoplogs. The 18-inch clearance between the top of sidewalls and bottom of walk planks (figures 2-26 and 2-27) provides clearance for the passage of floating debris that might otherwise obstruct the flow of water over sidewalls.

The check inlet structure also has wingwalls and cutoffs for the purpose of retaining the canal earthwork and for reducing seepage.

(3) *Weir.*–Sometimes a measuring weir is required at the inlet to an R.I. drop. See subchapter V E for a discussion on water measurement structures pertaining to weirs.

For a lined canal, generally the minimum freeboard at the inlet should be the same as for the lining. For unlined canals up to a capacity of 100 cfs, the minimum freeboard should be 6 inches for water depths to 1.25

feet, 9 inches for depths 1.26 to 2 feet, 12 inches for depths 2.01 to 5 feet, 15 inches for depths 5.1 to 7 feet, and 18 inches for depths 7.1 to 9 feet.

(c) *Rectangular Inclined Channel.*—The channel is rectangular in shape, and it is usually practical to make the base width the same as that required for the pool or inlet section. The vertical height of the walls may be determined by computing the depth in the section with theoretical velocity and adding freeboard of 12 inches for flows up to 100 cfs. The slope of the incline may be as steep as 1-1/2 to 1 but is usually 2 to 1. R.I. drops with flows up to 100 cfs do not require curved trajectories, therefore the incline will intersect the level inlet invert and also the level pool invert.

(d) *Stilling Pool.*—Stilling pools for hydraulic jumps are installed at the lower ends of R.I. drops to obtain the required loss of energy between the lower end of the inclined channel and the downstream pool. The outlet transition further reduces velocity and turbulence of the water thereby reducing erosion in the channel below. See subchapter II F for design of stilling pools. The necessary depth, d_2, at the lower end of the pool may be computed by the pressure-momentum equation or may be determined as demonstrated in the design example. The minimum downstream energy gradient for the hydraulic jump computations is determined by using a reduced n value. This reduced n should be 0.8 times the assumed design n for the canal. These smaller values of n are used as a factor of safety against possible lower canal water elevations than indicated using the rougher n value. The friction loss on the incline of an R.I. drop is also neglected which provides an additional factor of safety for the stilling pool. If there is not downstream water surface control in the channel, backwater for the jump may be provided by building a control into the outlet transition. See subchapter II F.

For a controlled downstream water surface, the elevation of the pool floor may be determined by subtracting $(d_2 + h_{v_2})$ from the minimum downstream energy elevation.

When use of a stilling pool is to be intermittent and for short duration of flow, such as in most wasteways or structures carrying floodwater, the minimum pool length should be 3 d_2. Where uninterrupted use or long duration of flow is expected, the minimum pool length should be 4 d_2.

For rectangular pools and discharges up to 100 cfs, the width of pool may be obtained by the empirical formula,

$$b = \frac{360\sqrt{Q}}{Q + 350}$$

where b = width of pool in feet and Q = discharge in cubic feet per second.

(e) *Outlet.*—The outlet of an R.I. drop connects the pool with an earth canal, a concrete-lined canal, or a natural channel and prevents or lessens downstream erosion.

Some of the more common types of outlets used are: Broken-back transition, straight or curved diverging vertical walls extending into the earth canal banks on either side, and a straight rectangular channel with vertical walls tapering from pool wall height to zero. A portion of the transitions may be made in earth provided the velocity at the downstream cutoff is not too great for the soil. Riprap or gravel protection is used as required on the earth transition section. See subchapter VII B for protection. Where broken-back or warped transitions are used, the tops of these transition walls should be the same elevation as the stilling pool walls.

Cutoffs at the ends of the outlets should be a minimum of 24 inches deep and 6 inches thick for water depths of 0 to 3 feet over the cutoff, and 2 feet 6 inches deep and 8 inches thick for water depths from 3 to 6 feet.

Where an earth transition is used at the inlet or outlet, it should be a minimum of 10 feet long.

2-24. *Percolation.*—Where water stands at different elevations above and below a structure, there is a tendency for the water to flow along the surfaces of the structure or through the surrounding soil from the higher elevation to the lower. The velocity that the water will attain in passing from the higher to the lower elevation will depend on the hydraulic head differential, the length of the path that the water is required to follow, and the permeability of the soil. Obviously, it is possible for percolating water to attain a velocity sufficiently great to move the soil particles with consequent hazard to the foundation of the structure.

The length of the percolation path along the structure, as computed by Lanes' weighted-creep method, should be such that the phreatic line does not intersect the tops of the inclined channel or stilling pool walls. The type of soil will govern the choice of the maximum allowable slope of the assumed percolation line. In general, the slope of the percolation line along the backside of the walls should be held to a slope not steeper than 3.5 to 1 (horizontal to vertical) from the canal water surface at the cutoff to any point at the top of the inclined channel or stilling pool walls. A slope of 5 to 1 is common. See subchapter VIII C for discussion on Lanes' weighted-creep ratio, sometimes called percolation factor.

The need for cutoff collars and the length of the portion of the inlet between the inlet cutoff and the incline may be decided by the susceptibility to percolation and seepage. It is sometimes necessary to lengthen the inlet structure or provide cutoff collars to lengthen the path of percolation on the standard R.I. drops, figures 2-26 and 2-27.

2-25. ***Design Example of R.I. Drop With Control Inlet and Stilling Pool Outlet (see figs. 2-26 and 2-27 and tables 2-1 and 2-2).***—The topography for the canal alinement is assumed to be such that the general ground drops abruptly at this point and a drop structure is the feasible way to convey the water to the lower elevation.

(a) *Given.*—

(1) Type of waterway = earth canal.

(2) Hydraulic properties of the canal at inlet and outlet of structure:

$Q = 50$ cfs	$b = 5.0$ ft.	$d_n = 2.83$ ft.
$s = 0.0005$	$s^{1/2} = 0.02235$	$S{:}S = 1\text{-}1/2{:}1$
$n = 0.025$	$V = 1.91$ f.p.s.	$h_v = 0.06$ ft.
$E = 2.89$ ft.		

(3) Type of conveyance structure = R.I. drop with stilling pool.

(4) El. A = 5406.00 (from a profile sheet).

(5) NWS El. at Sta. A = El. A + d_n = 5406.00 + 2.83 = 5408.83.

(6) Energy El. at Sta. A = NWS El. + h_v = 5408.83 + 0.06 = 5408.89.

(7) El. top of bank = 5410.4.

(8) El. D = 5392.00 (from a profile sheet).

(9) NWS El. at Sta. D = El. D + d_n = 5392.00 + 2.83 = 5394.83.

(10) Energy El. at Sta. D = El. NWS + h_v = 5394.83 + 0.06 = 5394.89.

(11) There is no requirement for a check upstream from the drop, but a control is required to minimize erosion in the canal for flows in the range of about 0.2 to full design flow.

(b) *Determine Control Inlet Dimensions.*—

(1) Standard dimensions b, L_o, H_o, H_F, and a are read from dimension table Structure Q max. of 50 (table 2-1, sheet 3) for canals and laterals.

The value for b = 6 feet 6 inches in the table was calculated from the formula:

$$b = \frac{360\sqrt{Q}}{Q + 350}$$

The value of L_o = 5 feet 7 inches in the table was determined by emergency side-overflow requirements, and the value of H_o = 2 feet 9 inches minimum was determined by the requirement that El. B is set low enough so that hydraulic control of the upstream canal water surface is at the control notch.

In the table, H_F = 4 feet 3 inches minimum, was determined by adding 18 inches to H_o. From the above table, a = 4 feet 0 inch.

(2) The procedure and calculations required to select the appropriate control notch are explained in detail in subchapter III G. The design example in subchapter III G is for the same hydraulic conditions and therefore only the design values will be repeated here:

P = 20 inches, S = 0.50, and N = 4 ft. 6 in.

(3) T should be equal to or greater than normal depth in the canal, 2.83 feet. Select T = 2 feet 10 inches.

(4) The value used for H_o must not be less than the value selected for T. Since the

value T = 2 feet 10 inches exceeds the tabulated value for H_o = 2 feet 9 inches min., use H_o = 2 feet 10 inches.

(5) The value for H_f must be H_o + 18 inches = 4 feet 4 inches.

(6) Determine minimum inside width of inlet structure with control notch.

$b_{min.}$ = N + 3 in. min. each side
= 4 ft. 6 in. + 6 in. = 5 ft. 0 in. or,
$b_{min.}$ = 6 ft. 6 in., from dimension shown on table 2-1 (sheet 3) therefor use 6 ft. 6 in.

(7) Determine El. top of notch and side overflow walls. The invert elevation of the notch is equal to El. A.

= El. A + T = 5406.00 + 2.83 = 5408.83.

(8) Determine El. B

= El. top of notch and sidewalls – H_o
= 5408.83 – 2.83 = 5406.00 which for this example happens to equal El. A.

(c) *Determine Hydraulic Setting for Stilling Pool by Using Table 2-1.*

(1) Determine the minimum downstream energy at station D for design flow using an n value of 0.8 the normal n value.
Min. energy at Sta. D = El. D + d + h_v

From Manning's equation [6]

$$AR^{2/3} = \frac{Qn}{1.486\ S^{1/2}} = \frac{50 \times 0.8 \times 0.025}{1.486 \times 0.02235} = 30.1$$

Assume different water depths, d, in the canal and solve for $AR^{2/3}$ until $AR^{2/3}$ = 30.1 (see tabulation below).

The assumed d of 2.53 feet properly balances the Manning equation, so normal depth is 2.53 feet, and A = 22.25 feet², $V = \frac{50}{22.25} = 2.25$ fps, $h_v = \frac{V^2}{2g} = \frac{5.06}{64.4} = 0.08$ foot, E = d + h_v = 2.53 + 0.08 = 2.61 feet. The normal depth could also have been determined by using table 2-5.

So minimum energy at Sta. D = El. D + E = 5392.00 + 2.61 = 5394.61.

(2) Determine H value

H = energy El. at Sta. A – minimum energy El. at Sta. D. = 5408.89 – 5394.61 = 14.28 feet. Use H = 15 feet for determining structure number. Structure number = Q-H or 50-15. This structure is for a canal so use table 2-1 (sheet 3) for standard dimensions. From this table b_t = 2 feet 6 inches, H_s = 2 feet 9 inches, c = 7 spaces, d = 11 inches, L_T = 10 feet 0 inch, L_F = 10 feet 5 inches, L_B = 3 feet 3 inches, L_P = 16 feet 0 inch, H_P = 6 feet 2 inches, $d_2 + h_{v_2}$ = 4.02 feet, h = 12 inches, t = 6 inches, and t′ = 8 inches. Other additional information regarding wingwalls, cutoffs, weep holes, concrete thicknesses, reinforcements, and estimated quantities may be taken from figure 2-27 and table 2-1.

(3) Determine stilling pool invert elevation (El. C) = minimum downstream energy elevation – ($d_2 + h_{v_2}$) = 5394.61 – 4.02 feet = 5390.59, use 5390.50.

(d) *Alternative Method for Determining Stilling Pool Invert Elevation.*—An alternate determination of stilling pool invert elevation (El. C) would be to use figure 2-37, subchapter II F and make the following computations:

(1) Determine critical depth in pool section from equation $d_c = 0.31433q^{2/3}$, where $q = \frac{Q}{b} = \frac{50}{6.5} = 7.69$ and $q^{2/3} = 3.896$.

d_c = 1.22 ft.

Assumed d, ft.	d^2, sq. ft.	$1.5d^2$, sq. ft.	b, ft.	db sq. ft.	$A = db+1.5d^2$, sq. ft.	3.606d, ft.	wp = b+3.606d, ft.	$R = \frac{A}{wp}$, ft.	$R^{2/3}$	$AR^{2/3}$
2.50	6.25	9.37	5.0	12.50	21.87	9.01	14.01	1.560	1.345	29.4
2.53	6.40	9.60	5.0	12.65	22.25	9.12	14.12	1.575	1.354	30.1

(2) Determine $\frac{H}{d_c}$

$$= \frac{14.28}{1.22} = 11.7$$

(3) Obtain values $\frac{d_1}{d_c}$ and $\frac{d_2}{d_1}$ by using above value of $\frac{H}{d_c}$ and table in figure 2-37, subchapter II F.

$$\frac{d_1}{d_c} = 0.184$$

$$\frac{d_2}{d_1} = 17.43$$

(4) Determine d_1

$$d_1 = 0.184 \times d_c = 0.184 \times 1.22 = 0.224 \text{ foot}$$

(5) Determine d_2

$$d_2 = 17.43 \times d_1 = 17.43 \times 0.224 = 3.92 \text{ feet}$$

(6) Determine A, V, and h_v for depth, d_2

$$A_2 = b\, d_2 = 6.5 \times 3.92 = 25.45 \text{ ft.}^2$$

$$V_2 = \frac{Q}{A_2} = \frac{50}{25.45} = 1.96 \text{ f.p.s.}$$

$$h_{v_2} = \frac{V^2}{2g} = \frac{1.96^2}{64.4} = 0.06 \text{ foot}$$

(7) Determine stilling pool invert elevation (El. C).

= minimum downstream energy El. $- (d_2 + h_{v_2})$

= 5394.61 – (3.92 + 0.06)
= 5390.63 use 5390.50

2-26. ***Design Example of R.I. Drop With Check Inlet and Stilling Pool Outlet (see figs. 2-26 and 2-27 and tables 2-1 and 2-2).***—The topography for the canal alinement is assumed to be such that the terrain drops abruptly at this point, and a drop structure is the feasible way to convey the water to the lower elevation.

(a) *Given.*—

(1) Type of waterway = earth canal.

(2) Hydraulic properties of the canal:

Q = 50 cfs	b = 5.0 ft.	d_n = 2.83 ft.
s = 0.0005	$s^{1/2}$ = 0.02235	S:S = 1-1/2:1
n = 0.025	V = 1.91 f.p.s.	h_v = 0.60 ft.
E = 2.89 ft.		

(3) Type of conveyance structure = R.I. drop with stilling pool.

(4) El. A = 5406.00 (from a profile sheet).

(5) NWS El. at Sta. A = El. A + d_n = 5406.00 + 2.83 = 5408.83.

(6) Energy El. at Sta. A = NWS El. + h_v = 5408.83 + 0.06 = 5408.89.

(7) El. top of bank = 5410.40.

(8) El. D = 5392.00 (from a profile sheet).

(9) NWS El. at Sta. D = El. D + d_n = 5392.00 + 2.83 = 5394.83.

(10) Energy El. at Sta. D = El. NWS + h_v = 5394.83 + 0.06 = 5394.89

(11) A check inlet structure is required at the inlet to provide diversion requirements to turnouts. The check should be furnished with a gate because of the desire to automate the check.

(b) *Determine Check Inlet Dimensions.*—

(1) Standard dimensions b, L_o and H_o are read from dimension table Structure Q max. of 50 (table 2-1) (sheet 3).

The value for b = 6 feet 6 inches in the table was calculated from the formula:

$$b = \frac{360\sqrt{Q}}{Q + 350}$$

The value of L_o = 5 feet 7 inches in the table was determined by emergency side-overflow requirements and H_o = 2 feet 9 inches minimum was determined by the requirement that El. B is set low enough so that hydraulic control of the upstream canal water surface is at the check opening.

(2) El. top of sidewalls = El. NWS in canal = 5408.83.

(3) El. B = El. NWS in canal – H_o minimum = 5408.83 – 2.75 = 5406.08, but

this is higher than canal invert. El. B should not be higher than the canal invert, therefore make El. B = El. canal invert = 5406.00, then H_o = 5408.83 – 5406.00 = 2.83 feet.

(4) Gate size, frame height, and number of walk planks are also read from the Standard Dimension tabulation on table 2-1 (sheet 3):

Gate size = 78 inches x 33 inches
Frame Ht. = 6 feet 0 inch
Number of planks = 11

(c) *Determine Hydraulic Setting for Stilling Pool.*—The following dimensions and elevations are identical to those for the R.I. drop with control inlet, section 2-25.

(1) Minimum downstream energy elevation = 5394.61.

(2) H = 15 ft.

(3) d_c = 1.22 ft.

(4) $\frac{H}{d_c} = 11.7$

(5) d_1 = 0.224 ft.

(6) d_2 = 3.92 ft.

(7) A_2 = 25.45 ft.^{2}, V_2 = 1.96 fps, h_{v_2} = 0.06 ft.

(8) El. C = 5390.50.

(9) L_P = 16 ft. 0 in.

(10) L_T = 10 ft. 0 in.

(11) H_P = 6 ft. 2 in.

(12) b_T = 2 ft. 6 in.

(13) L_B = 3 ft. 3 in.

(14) c = 7.

(15) d = 11 in.

(16) h = 12 in.

(d) *Determine Additional Dimensions.*—These dimensions are also identical to those for the R.I. Drop with control inlet discussed in section 2-25.

3. *Pipe Drops*

2-27. ***Purpose and Description.***—A pipe drop conveys water from a higher elevation to a lower elevation. This drop in elevation may be any amount between 3 and about 15 feet. A pipe drop not only conveys water but it must also dissipate the excess energy and still the water after it has reached the lower elevation. The two general types of closed conduit drops are type 1 and type 2.

(a) *Type 1.*—The type 1 pipe drop (figs. 2-28, 2-29, 2-30, and 2-31) is a practical and economical drop and is used as an inline canal structure where the possibility of clogging is minimal. Another type of drop structure should be used for cross drainage. The pipe is sumped an amount required to create a hydraulic jump in the pipe which dissipates the excess energy.

The type 1 pipe drop should have a check inlet or a control inlet, but may have only an earth outlet transition if the full pipe velocities are not greater than 3.5 feet per second. If this drop has a reinforced concrete outlet transition the full pipe velocities may be as high as 5 feet per second.

(b) *Type 2.*—The type 2 pipe drop figure 2-32 is a practical and economical drop and is used as an inline canal structure or as a cross-drainage structure. It is generally used in preference to the type 1 pipe drop if sediment and debris is carried in the water. With a type 2 pipe drop the excess energy dissipation must be accomplished at the outlet with either a baffled outlet or a stilling pool. However, if there is a possibility of having weeds in the water, a stilling pool should be used because of the difficulty with weeds clogging a baffled outlet. The type 2 pipe drop should have a check inlet or a control inlet if the drop is an inline canal structure. If the type 2 pipe drop is a cross-drainage structure it may have a reinforced concrete inlet transition.

2-28. ***Advantages and Disadvantages.***—Pipe drops are easily designed, built, and operated. The inlets can be readily adapted to either an earth or a lined waterway and the outlets can be easily adapted to an earth or a lined canal or to a waterway where there is no downstream water surface control. The inlets can be made to incorporate a control notch, a check, or a weir. If there is a control or a check inlet there should be side overflow walls for emergency flows. A pipe drop can easily be taken under another waterway or a roadway. Pipe drops are economical, especially for small discharges. Pipe drops require very little maintenance, provided they are constructed of durable pipe having good rubber gasket joints and provided the bends are properly made. It is important to provide adequate gravel or riprap protection at outlets discharging into unlined waterways. The maximum fall for any one pipe drop is about 15 feet. The type 1 pipe drop (sumped drop) should not be used if there is a strong possibility of the pipe becoming clogged with sediment. There is also the possibility of the drop becoming clogged with weeds or debris. To prevent this, the drop may have a weed screen on the inlet or the pipe may be sufficiently sized to discharge this material if it should get into the pipe drop. Usually clogging is not a problem with a type 2 pipe drop because its profile lends itself more readily to self-cleaning.

2-29. ***Type 1 Pipe Drop Structure Components.***—The principle hydraulic elements of the type 1 pipe drop are the upstream transition, the inlet structure, the pipe, and the outlet transition.

(a) *Upstream Transition.*—The transitions may be earth or concrete. With a control inlet there is no change in invert elevation from the normal waterway to the structure and the length of either an earth or concrete lining transition is 10 feet. With a check inlet the transition provides for a gradual change in invert elevation from the waterway to the opening in the check structure. This transition invert slope should not be steeper than 4 to 1, and the length of this transition is also 10 feet. An earth transition may require erosion protection which is discussed in subchapter VII B.

(b) *Inlet.*—The inlet to a type 1 pipe drop may be either of the following:

(1) *Control and pipe inlet (fig. 3-39).*—When a check structure is not required at the inlet to a type 1 pipe drop, the inlet to the drop should be designed to provide a control notch which will prevent upstream racing and scouring of earth canals. The inlet must be designed so that the full capacity can be discharged into the drop with normal depth in the canal.

A properly designed control notch will cause the upstream canal water to be at or near normal depth for discharges ranging from design flow to 0.2 design flow. It may also be proportioned to control only one specific flow. For a detailed discussion of a control and pipe inlet and a method of notch design for control of varying flow, see subchapter III G. The inlet should be an adequate distance from upstream horizontal bends as to limit undesirable wave action due to unsymmetrical flow. Figure 3-39 has standard dimensions for inlet control structures with flows up to 100 cfs.

(2) *Check and pipe inlet.*—A check and pipe inlet structure is often used at the inlet to a pipe drop. By adjusting the gate or stoplogs, the check serves as a control to prevent racing of the water upstream from the inlet, in addition to its usual function of raising the water surface to permit diversion through upstream turnouts during periods of partial discharge. The check inlet can also serve as a cutoff point in a canal provided there is a wasteway or some other structure that can take the flow that otherwise would continue down the canal. This may be desirable when water delivery downstream from the drop is not required or when maintenance downstream is required for reasons such as canal embankment failure.

For further discussion of check and pipe inlet, see subchapter III F and figure 3-38.

(c) *Pipe.*—The pipe may be precast reinforced concrete pressure pipe (PCP) or asbestos-cement pressure pipe (AC).

(d) *Outlet Transitions.*—Outlet transitions can reduce earth canal erosion and can provide smooth flow by making the velocity change less abrupt. Concrete transitions and combinations of concrete and earth transitions are used for these purposes. Concrete outlet transitions in earth canals are required on those drops that cross under railroads and state highways, on all drops with 36-inch-diameter pipe or larger, and as previously stated, on all pipe with velocities between 3.5 and 5 feet per second. A type 1 outlet transition (broken back) is ordinarily used with a type 1 pipe drop.

For the design of transitions for pipe structures and standard pipe transitions see the discussion on transitions subchapter VII A.

2-30. ***Type 1 Pipe Drop Design Considerations.***—The hydraulic design of a type

1 pipe drop includes selecting a pipe diameter for a given capacity that will result in (1) a velocity of 3.5 feet per second or less for a pipe with only an earth transition at one or both ends or (2) 5 feet per second or less for a pipe with a concrete structure at each end. The table on figure 2-31 may be used to select pipe sizes.

If the type 1 pipe drop has a control and pipe inlet or a check and pipe inlet the invert at the beginning of the pipe should be low enough so the hydraulic control is at the inlet to the inlet structure instead of at the pipe inlet. Standard dimensions for check and pipe inlet structures shown on figure 3-38 provide for this. With a properly designed control notch, the standard dimensions shown on figure 3-39 for control and pipe inlets also provide for this.

The inclined portion of the pipe should be on a slope of 0.500 or flatter.

To dissipate the excess energy from the supercritical velocities in the sloped part of the pipe profile, stilling is accomplished by providing a depressed (sumped) section of pipe near the outlet end. The depth at which the bottom of the pipe is to be depressed below the downstream water surface can be determined from the equation $(P_1 + M_1) \rightarrow = (P_2 + M_2) \leftarrow$. This is the pressure momentum equation for a hydraulic jump derived from Newton's second law of motion. See figure 2-28 and the explanation and procedure following this figure for determination of the elevation of the pipe invert at the beginning of the jump. This elevation can be determined by this procedure as is demonstrated in the design example or by using information on figures 2-29 and 2-30. The sumped section of pipe is set with the upstream end at this elevation and with a minimum slope of 0.005 towards the downstream end. The length of the sumped section must be at least $4d_2$ (6-foot min.) for a drop with an earth outlet transition. A drop with a concrete outlet transition should have at least a 5 D length for the sumped section and should also have a $4d_2$ (6-foot min.) length of pipe between the upstream sloped pipe and the transition but part of this distance can be sloped pipe as shown in figure 2-28.

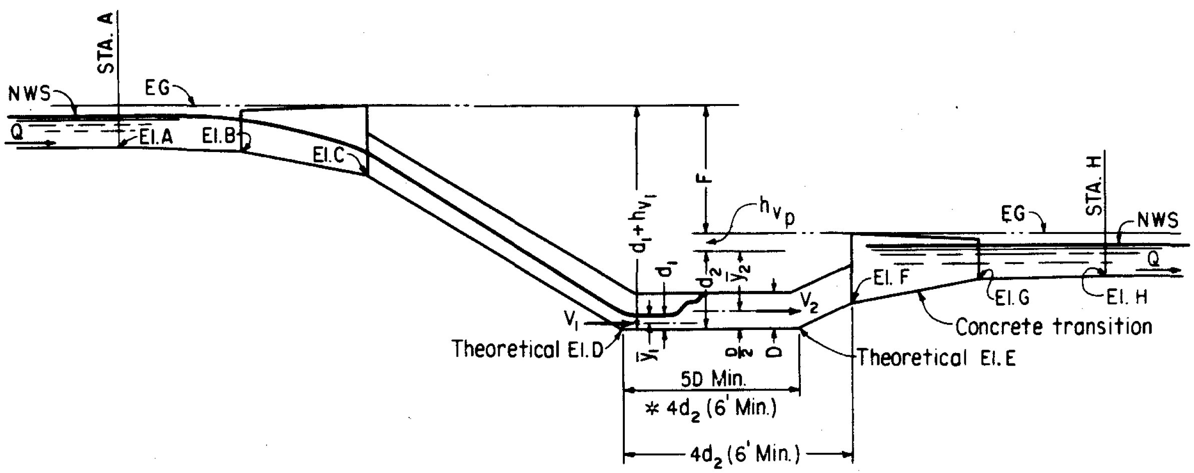
On figures 2-29 and 2-30 type 1 pipe drops have been standardized so the entire pipe profile can be designed using these figures and accompanying tables 2-3 and 2-4. There are standard designs for type 1 pipe drops with a concrete downstream transition for capacities up to 50 cfs (fig. 2-30) and standard designs for type 1 pipe drops with an earth outlet transition with capacities up to 22 cfs (fig. 2-29). Type 1 pipe drops can be designed and built for flows greater than 50 cfs but ordinarily it will be more economical to use a rectangular inclined drop for larger capacities.

(a) *Bends.*—Changes in pipe grade and alinement (bends) may be made with special fittings, by miter cutting pipe, with beveled end pipe or by pulling joints where grade and alinement can be established on a sufficiently large radius. See chapter VIII for detailed information on bends.

(b) *Pipe Cover.*—At all railroad and road crossings except farm roads a minimum of 3 feet of earth cover should be provided for pipe. Farm roads require 2 feet of earth cover and are frequently ramped using 10 to 1 slopes (10 percent grade) when necessary to provide a minimum cover requirement. If roadway ditches exist and are extended over the pipe, the minimum cover from the ditch to the top of the pipe should be 2 feet.

(c) *Percolation.*—Water has a tendency to flow through soil or along a structure to reach a lower elevation. The velocity the water will attain in passing to the lower elevation will depend upon the difference in elevation, the length of the path, and the soil itself. Obviously, it is possible for percolating water to attain a velocity sufficiently great to move the particles of the material through which it is passing, with consequent hazard to the foundation of the structure or possibility of the water developing an unrestricted channel around it. The location of the inlet structure may be decided by the susceptibility to percolation and seepage. The weighted length of the percolation path along the structure or through the surrounding material as computed by Lane's weighted-creep method, should be great enough to dissipate the difference in water surface elevations using an appropriate assumed weighted-creep ratio.

Pipe collars may be required to reduce the velocity of the water moving along the outside



PROCEDURE: Use hydraulic tables [6] [7] to determine cross section area A_1 and $\overline{y}_1$ of pipe flowing partially full.

1. Assume values of $\frac{d_1}{D}$
2. Compute d_1 and A_1 from $\frac{d_1}{D}$ and $\frac{A}{D^2}$ found in hydraulic tables [6] [7]
3. $V_1 = \frac{Q}{A_1}$
4. A_2 = Area of full pipe
5. $V_2 = \frac{Q}{A_2}$
6. Compute $\overline{y}_1$ for assumed $\frac{d_1}{D}$ by using hydraulic tables [6] [7]
7. Solve for d_2
8. Solve for F
9. By trial and error, repeat until computed F = Actual F
10. Elevation of sump = El. downstream E G $-h_{v_p} - 1.1d_2$ (10% safety factor)

Hydraulic head losses such as entrance, friction, bend, and exit are omitted because they are small.

Theoretical Equation for d_2

$$P_1 + M_1 = P_2 + M_2$$

NOTE: Wt. of water is omitted from all values.

$$A_1\overline{y}_1 + \frac{QV_1}{g} = A_2\overline{y}_2 + \frac{QV_2}{g}$$

$$\frac{Q\Delta V}{g} + A_1\overline{y}_1 = A_2\overline{y}_2$$

$$\overline{y}_2 = d_2 - \frac{D}{2}$$

$$\Delta V = V_1 - V_2$$

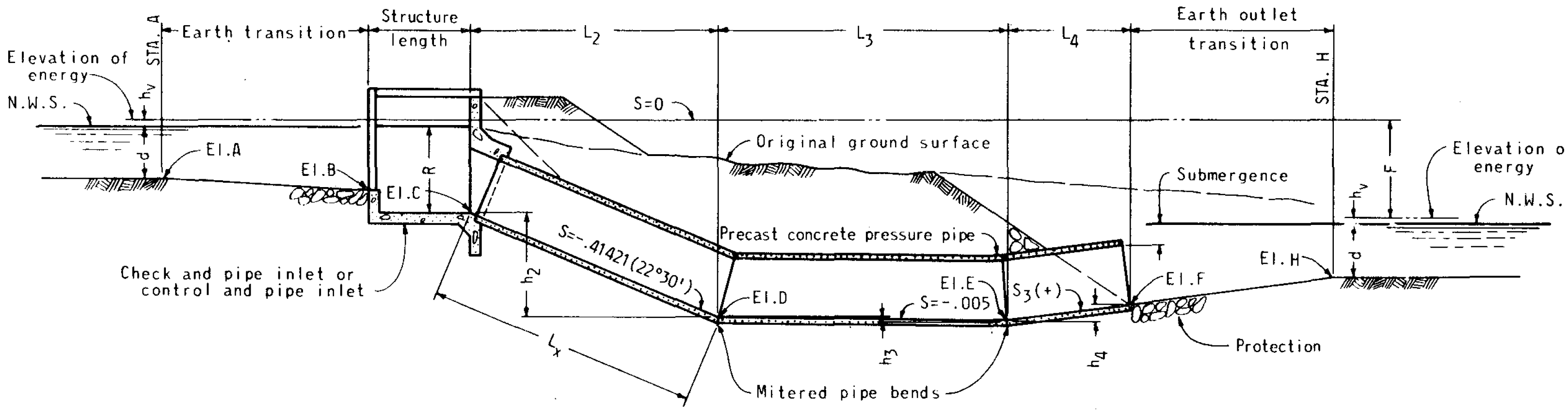
$$\frac{Q\Delta V}{g} + A_1\overline{y}_1 + \frac{A_2 D}{2} = A_2 d_2$$

$$d_2 = \frac{Q\Delta V}{A_2 g} + \frac{A_1}{A_2}\overline{y}_1 + \frac{D}{2}$$

$$F = d_1 + h_{v_1} - d_2 - h_{v_p}$$

$$h_{v_1} = \frac{V_1^2}{2g}$$

Figure 2-28. **Determination of sump elevation for type 1 pipe drop. 103-D-1266.**



LONGITUDINAL SECTION

NOTES

The pipe slopes used will allow the substitution of 7°30' precast concrete elbows for the mitered pipe bends.

The maximum velocity in the pipe at the outlet equals 3.5 feet per second for pipe flowing full.

Hydraulic head losses due to entrance, friction, bends, and exit have been neglected.

The length of the earth outlet transition equals 3 pipe diameters (5' Min.)

Precast concrete pressure pipe with mitered pipe bends shown. AC or RPM may be substituted.

These standard designs are for flows less than 23 cubic feet per second. For flows greater than 22 cubic feet per second it is probably more economical to use a concrete outlet transition and a smaller pipe Figure 2-30. For flows greater than 22 cubic per second having an earth outlet transition, design of the pipe structure must be performed.

Ordinarily the maximum water surface drop for a pipe drop structure is 15 feet. However, standard design tables extend slightly beyond this maximum.

Figure 2-29. **Type 1 pipe drop with earth outlet transition–explanation sheet for table 2-3. 103-D-1267.**

Table 2-3.–*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).* 103-D-1268-1

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

Q = 3 cfs   Diameter of pipe = 15 inches V_p = 2.44 fps hv_p = 0.09 foot   Inlet R = 2.50 feet

F	2.68	3.01	3.33	3.65	3.98	4.31	4.64	4.97	5.30	5.64	5.98	6.32	6.66	7.01	7.34	7.69	8.04	8.38	8.72	9.08	9.41
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07
L_2	5.08	6.01	6.93	7.85	8.79	9.70	10.63	11.55	12.49	13.40	14.32	15.24	16.17	17.09	18.02	18.94	19.86	20.79	21.71	22.64	23.56
L_3	7.50	7.50	7.50	8.00	8.00	8.00	8.50	8.50	8.50	9.00	9.00	9.00	9.00	9.00	9.50	9.50	10.00	10.00	10.00	10.00	10.50
L_4	2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00
S_3	7°-30'								7°-30'	15°-0'											15°-0'
Submergence	0.45	0.51	0.57	0.63	0.68	0.48	0.53	0.58	0.63	0.40	0.44	0.48	0.53	0.56	0.60	0.64	0.68	0.46	0.50	0.53	0.58
d_2	1.72	1.77	1.83	1.88	1.93	1.98	2.03	2.07	2.12	2.16	2.20	2.24	2.28	2.31	2.35	2.38	2.42	2.45	2.49	2.52	2.56
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32
F	9.76	10.12	10.46	10.80	11.15	11.49	11.84	12.19	12.54	12.89	13.24	13.60	13.94	14.29	14.64	14.98	15.33	15.69	16.04	16.38	16.73
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07
h_4	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
L_2	24.48	25.41	26.33	27.25	28.18	29.10	30.03	30.95	31.87	32.80	33.72	34.65	35.57	36.50	37.42	38.34	39.26	40.19	41.11	42.04	42.96
L_3	10.50	10.50	10.50	10.50	10.50	11.00	11.50	11.50	11.50	11.50	12.00	12.00	12.00	12.00	12.50	12.50	12.50	12.50	13.00	13.00	13.00
L_4	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
S_3	15°-0'																				15°-0'
Submergence	0.61	0.64	0.67	0.45	0.48	0.54	0.57	0.60	0.64	0.67	0.70	0.41	0.45	0.48	0.52	0.55	0.58	0.61	0.65	0.69	0.72
d_2	2.59	2.62	2.65	2.69	2.72	2.76	2.79	2.82	2.85	2.88	2.91	2.94	2.97	3.00	3.03	3.06	3.09	3.12	3.15	3.18	3.21
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	12.70	13.08	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35

Q = 4 cfs   Diameter of pipe = 15 inches V_p = 3.26 fps hv_p = 0.17 foot   Inlet R = 2.50 feet

F	2.28	2.59	2.90	3.20	3.52	3.82	4.15	4.46	4.79	5.10	5.43	5.75	6.08	6.40	6.73	7.04	7.38	7.70	8.04	8.37	8.70
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	0.53	0.53	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	8.50	9.00	9.00	9.50	9.50	10.00	10.00	10.00	10.50	10.50	11.00	11.00	11.50	11.50	11.50	12.00	12.00	12.00	12.50	12.50	13.00
L_4	2.00	2.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	15°-0'														15°-0'	22°-30'					22°-30'
Submergence	0.58	0.66	0.46	0.54	0.61	0.43	0.48	0.55	0.61	0.41	0.46	0.52	0.59	0.65	0.70	0.45	0.51	0.56	0.60	0.66	0.71
d_2	2.01	2.08	2.15	2.22	2.28	2.35	2.40	2.46	2.52	2.58	2.63	2.68	2.74	2.79	2.84	2.90	2.95	3.00	3.04	3.09	3.14
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32
F	9.03	9.37	9.70	10.04	10.38	10.71	11.06	11.39	11.74	12.08	12.42	12.76	13.09	13.43	13.79	14.12	14.47	14.81	15.15	15.50	15.85
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.50
L_4	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
S_3	22°-30'																				22°-30'
Submergence	0.76	0.39	0.46	0.50	0.55	0.59	0.63	0.68	0.71	0.75	0.38	0.42	0.47	0.51	0.55	0.60	0.63	0.67	0.72	0.75	0.78
d_2	3.18	3.22	3.27	3.31	3.35	3.39	3.43	3.47	3.50	3.54	3.58	3.62	3.66	3.70	3.73	3.77	3.80	3.84	3.88	3.91	3.94
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	12.70	13.08	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ $\text{Submergence} = 1.1d_2 + hv_p + h_3 - h_4 - \text{Diameter of pipe} - hv$

Table 2-3.–*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*–Continued. 103-D-1268-2

PIPE DROP WITH MITER BENDS

Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 5 cfs Diameter of pipe = 18 inches V_p = 2.83 fps hv_p = 0.12 foot Inlet R = 2.50 feet

F	2.35	2.65	2.96	3.27	3.59	3.92	4.24	4.57	4.90	5.23	5.55	5.88	6.22	6.55	6.89	7.22	7.57	7.90	8.24	8.59	8.92
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06
h_4	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	8.00	8.50	8.50	9.00	9.00	9.50	9.50	10.00	10.00	10.00	10.50	10.50	11.00	11.00	11.00	11.50	11.50	11.50	12.00	12.00	12.00
L_4	2.00	2.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00
S_3	7°-30'						7°-30'	15°-0'													15°-0'
Submergence	0.53	0.62	0.43	0.50	0.58	0.63	0.69	0.45	0.51	0.56	0.61	0.67	0.46	0.51	0.55	0.61	0.65	0.70	0.47	0.51	0.56
d_2	1.99	2.07	2.14	2.20	2.26	2.31	2.36	2.41	2.46	2.51	2.56	2.61	2.65	2.70	2.74	2.78	2.82	2.86	2.90	2.94	2.98
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32
F	9.27	9.60	9.96	10.30	10.64	10.98	11.33	11.67	12.02	12.36	12.71	13.05	13.40	13.74	14.09	14.44	14.78	15.13	15.48	15.82	16.17
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	12.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50	15.00	15.00
L_4	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'			15°-0'	22°-30'																22°-30'
Submergence	0.59	0.63	0.67	0.71	0.43	0.47	0.51	0.56	0.59	0.63	0.67	0.71	0.74	0.78	0.82	0.44	0.48	0.51	0.55	0.59	0.62
d_2	3.01	3.05	3.08	3.12	3.15	3.19	3.23	3.26	3.29	3.33	3.36	3.40	3.43	3.46	3.50	3.53	3.56	3.59	3.63	3.66	3.69
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	12.70	13.08	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35

Q = 6 cfs Diameter of pipe = 18 inches V_p = 3.40 fps hv_p = 0.18 foot Inlet R = 2.50 feet

F	2.07	2.37	2.66	2.97	3.29	3.59	3.89	4.21	4.54	4.85	5.16	5.49	5.81	6.14	6.47	6.80	7.13	7.47	7.81	8.13	8.45
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07
h_4	0.52	0.52	0.52	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	9.00	9.00	9.50	10.00	10.00	10.50	10.50	11.00	11.00	11.50	11.50	12.00	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.50	13.50
L_4	4.00	4.00	4.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	7°-30'		7°-30'	15°-0'											15°-0'	22°-30'					22°-30'
Submergence	0.56	0.65	0.73	0.52	0.59	0.66	0.49	0.55	0.61	0.68	0.49	0.54	0.61	0.66	0.71	0.44	0.50	0.54	0.58	0.65	0.70
d_2	2.19	2.27	2.35	2.42	2.48	2.55	2.63	2.68	2.74	2.80	2.86	2.91	2.97	3.02	3.06	3.11	3.16	3.20	3.24	3.30	3.35
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32
F	8.79	9.13	9.47	9.80	10.12	10.46	10.80	11.14	11.47	11.80	12.14	12.47	12.81	13.14	13.49	13.83	14.15	14.49	14.83	15.17	15.49
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.90	2.90
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	13.50	14.00	14.00	14.00	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00
L_4	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00
S_3	22°-30'																				22°-30'
Submergence	0.76	0.80	0.44	0.49	0.55	0.59	0.63	0.68	0.73	0.79	0.42	0.47	0.51	0.55	0.60	0.64	0.70	0.74	0.79	0.42	0.47
d_2	3.39	3.43	3.47	3.52	3.57	3.61	3.65	3.69	3.74	3.78	3.83	3.87	3.91	3.95	3.99	4.03	4.08	4.12	4.16	4.20	4.25
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	12.70	13.08	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4$ – Diameter of pipe – hv

Table 2-3.--*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*--Continued. 103-D-1268-3

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 7 cfs Diameter of pipe = 21 inches V_p = 2.91 fps hv_p = 0.13 foot Inlet R = 3.00 feet

F	2.57	2.89	3.20	3.52	3.84	4.16	4.48	4.81	5.14	5.48	5.81	6.14	6.47	6.81	7.13	7.47	7.81	8.14	8.48	8.82	9.16
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	9.00	9.50	9.50	10.00	10.00	10.50	10.50	10.50	11.00	11.00	11.50	11.50	11.50	12.00	12.00	12.00	12.50	12.50	12.50	13.00	13.00
L_4	2.00	2.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00
S_3	7°-30'						7°-30'	15°-0'													15°-0'
Submergence	0.57	0.64	0.45	0.51	0.58	0.64	0.70	0.46	0.50	0.56	0.63	0.67	0.47	0.51	0.57	0.61	0.67	0.44	0.48	0.53	0.57
d_2	2.24	2.30	2.36	2.42	2.48	2.54	2.59	2.64	2.69	2.73	2.78	2.82	2.87	2.91	2.96	3.00	3.05	3.09	3.13	3.17	3.21
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.16	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.37	9.76	10.14	10.52	10.90	11.29	11.67	12.05	12.44	12.82
F	9.51	9.84	10.19	10.52	10.86	11.20	11.55	11.89	12.24	12.57	12.93	13.27	13.61	13.95	14.30	14.64	14.99	15.32	15.68	16.02	16.36
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08
h_4	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00
L_4	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'		15°-0'	22°-30'																	22°-30'
Submergence	0.60	0.64	0.70	0.43	0.47	0.51	0.55	0.59	0.62	0.67	0.70	0.74	0.78	0.82	0.44	0.50	0.53	0.58	0.61	0.65	0.69
d_2	3.24	3.28	3.32	3.36	3.40	3.44	3.47	3.51	3.54	3.58	3.61	3.65	3.68	3.72	3.75	3.79	3.82	3.86	3.89	3.93	3.96
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.17	18.56	18.94	19.32	19.70	20.09	20.47	20.85

Q = 8 cfs Diameter of pipe = 21 inches V_p = 3.33 fps hv_p = 0.17 foot Inlet R = 3.00 feet

F	2.35	2.67	2.97	3.28	3.59	3.91	4.22	4.55	4.87	5.19	5.51	5.83	6.17	6.50	6.83	7.16	7.50	7.83	8.15	8.49	8.81
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07
h_4	0.52	0.52	0.52	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	10.00	10.00	10.50	10.50	11.00	11.00	11.50	11.50	12.00	12.00	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00
L_4	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00
S_3	7°-30'		7°-30'	15°-0'													15°-0'	22°-30'			22°-30'
Submergence	0.53	0.60	0.68	0.46	0.54	0.60	0.68	0.47	0.54	0.60	0.66	0.45	0.50	0.55	0.60	0.65	0.71	0.44	0.50	0.55	0.61
d_2	2.40	2.46	2.54	2.60	2.67	2.73	2.79	2.84	2.90	2.95	3.01	3.06	3.11	3.15	3.20	3.25	3.29	3.34	3.39	3.44	3.49
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.16	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.37	9.76	10.14	10.52	10.90	11.29	11.67	12.05	12.44	12.82
F	9.14	9.47	9.82	10.15	10.49	10.82	11.15	11.49	11.81	12.15	12.49	12.83	13.16	13.49	13.83	14.16	14.49	14.82	15.17	15.49	15.82
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09
h_4	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.90
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	16.00	16.00	16.00	16.00	16.50	16.50	17.00	17.00	17.00	17.50	17.50	17.50	18.00
L_4	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00
S_3	22°-30'																				22°-30'
Submergence	0.66	0.71	0.75	0.80	0.43	0.48	0.55	0.59	0.65	0.69	0.74	0.78	0.82	0.46	0.52	0.56	0.61	0.67	0.71	0.77	0.41
d_2	3.54	3.58	3.62	3.66	3.70	3.75	3.80	3.84	3.89	3.93	3.97	4.01	4.05	4.10	4.15	4.19	4.24	4.28	4.32	4.37	4.42
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.17	18.56	18.94	19.32	19.70	20.09	20.47	20.85

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − Diameter of pipe − hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-4

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet   Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 9 cfs   Diameter of pipe = 24 inches V_p = 2.86 fps hv_p = 0.13 foot   Inlet R = 3.25 feet

F	2.72	3.03	3.33	3.64	3.95	4.27	4.60	4.92	5.25	5.58	5.91	6.24	6.58	6.92	7.24	7.58	7.92	8.26	8.59	8.94	9.28
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07
h_4	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.34
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	9.50	10.00	10.00	10.50	10.50	11.00	11.00	11.00	11.50	11.50	12.00	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50
L_4	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00
S_3	7°-30'							7°-30'	15°-0'												15°-0'
Submergence	0.42	0.50	0.58	0.65	0.47	0.53	0.58	0.64	0.42	0.47	0.52	0.57	0.62	0.66	0.46	0.50	0.55	0.59	0.63	0.69	0.46
d_2	2.33	2.40	2.47	2.54	2.61	2.66	2.71	2.76	2.82	2.86	2.91	2.95	3.00	3.04	3.09	3.13	3.17	3.21	3.25	3.29	3.33
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07

F	9.61	9.95	10.29	10.63	10.97	11.31	11.66	12.00	12.33	12.67	13.01	13.35	13.69	14.03	14.39	14.74	15.08	15.43	15.79	16.13	16.48
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	13.50	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.50	16.50
L_4	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'					15°-0'	22°-30'														22°-30'
Submergence	0.50	0.55	0.60	0.64	0.68	0.72	0.44	0.48	0.53	0.57	0.62	0.67	0.71	0.75	0.78	0.81	0.44	0.47	0.50	0.53	0.56
d_2	3.37	3.41	3.45	3.49	3.53	3.56	3.60	3.64	3.68	3.72	3.76	3.80	3.84	3.87	3.90	3.93	3.96	3.99	4.02	4.05	4.08
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.45	13.83	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10

Q = 10 cfs   Diameter of pipe = 24 inches V_p = 3.18 fps hv_p = 0.16 foot   Inlet R - 3.25 feet

F	2.51	2.81	3.09	3.42	3.74	4.06	4.37	4.70	5.03	5.37	5.69	6.02	6.35	6.68	7.00	7.33	7.67	8.01	8.33	8.68	9.00
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	10.50	10.50	10.50	11.00	11.00	11.50	11.50	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.50
L_4	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00
S_3	7°-30'				7°-30'	15°-0'														15°-0'	22°-30'
Submergence	0.37	0.46	0.56	0.61	0.68	0.46	0.53	0.58	0.64	0.68	0.48	0.53	0.59	0.64	0.44	0.49	0.54	0.58	0.64	0.68	0.41
d_2	2.49	2.57	2.66	2.71	2.77	2.83	2.89	2.94	2.99	3.03	3.08	3.13	3.18	3.23	3.28	3.33	3.37	3.41	3.46	3.50	3.55
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07

F	9.34	9.68	10.02	10.35	10.68	11.01	11.36	11.69	12.03	12.36	12.71	13.05	13.38	13.72	14.06	14.40	14.73	15.07	15.42	15.75	16.09
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09
h_4	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00	17.50	17.50	17.50	18.00
L_4	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00
S_3	22°-30'																				22°-30'
Submergence	0.46	0.50	0.55	0.60	0.65	0.71	0.75	0.80	0.43	0.48	0.52	0.56	0.61	0.65	0.71	0.74	0.79	0.42	0.46	0.51	0.55
d_2	3.59	3.63	3.67	3.72	3.76	3.81	3.85	3.89	3.93	3.97	4.01	4.05	4.09	4.13	4.17	4.21	4.25	4.29	4.33	4.37	4.41
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.45	13.83	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – Diameter of pipe – hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-5

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 11 cfs Diameter of pipe = 24 inches V_p = 3.50 fps hv_p = 0.19 foot Inlet R = 3.25 feet

F	2.38	2.68	2.96	3.26	3.56	3.88	4.20	4.51	4.84	5.16	5.47	5.80	6.12	6.45	6.76	7.09	7.41	7.74	8.06	8.41	8.73
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08
h_4	0.52	0.52	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	10.50	11.00	11.00	11.50	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.50	13.50	14.00	14.00	14.00	14.50	14.50	15.00	15.00	15.50
L_4	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	7°-30'	7°-30'	15°-0'												15°-0'	22°-30'					22°-30'
Submergence	0.54	0.62	0.41	0.49	0.58	0.63	0.44	0.51	0.57	0.63	0.43	0.49	0.56	0.61	0.68	0.41	0.48	0.53	0.59	0.63	0.70
d_2	2.62	2.69	2.75	2.83	2.91	2.96	3.02	3.08	3.14	3.19	3.25	3.30	3.36	3.41	3.47	3.52	3.58	3.63	3.68	3.72	3.77
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07
F	9.07	9.39	9.74	10.07	10.40	10.73	11.07	11.40	11.73	12.06	12.40	12.73	13.06	13.39	13.74	14.06	14.40	14.72	15.07	15.39	15.73
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.49	2.91	2.91	2.91
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	15.50	15.50	16.00	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00	17.50	17.50	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00
L_4	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00
S_3	22°-30'																				22°-30'
Submergence	0.74	0.39	0.43	0.48	0.53	0.58	0.63	0.68	0.73	0.78	0.41	0.47	0.51	0.57	0.61	0.67	0.71	0.77	0.39	0.45	0.49
d_2	3.81	3.86	3.90	3.95	3.99	4.04	4.08	4.13	4.17	4.22	4.26	4.31	4.35	4.40	4.44	4.49	4.53	4.58	4.62	4.67	4.71
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.45	13.83	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.12	18.81	19.19	19.57	19.95	20.34	20.72	21.10

Q = 12 cfs Diameter of pipe = 30 inches V_p = 2.44 fps hv_p = 0.09 foot Inlet R = 3.75 feet

F	3.05	3.38	3.71	4.03	4.37	4.70	5.04	5.38	5.72	6.07	6.40	6.74	7.09	7.42	7.76	8.01	8.46	8.80	9.15	9.50	9.84
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07
h_4							0	0.26	0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.81
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	10.50	10.50	10.50	11.00	11.00	11.00	11.50	11.50	11.50	12.00	12.00	12.00	12.50	12.50	12.50	12.50	13.00	13.00	13.00	13.00	13.50
L_4								2.00	2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00
S_3								7°-30'												7°-30'	15°-0'
Submergence	0.35	0.41	0.46	0.52	0.57	0.62	0.67	0.45	0.50	0.63	0.57	0.62	0.66	0.45	0.49	0.52	0.56	0.60	0.63	0.67	0.43
d_2	2.52	2.57	2.62	2.67	2.72	2.76	2.80	2.84	2.88	2.91	2.95	2.99	3.03	3.07	3.11	3.14	3.17	3.21	3.24	3.27	3.31
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57
F	10.19	10.53	10.88	11.23	11.57	11.92	12.28	12.62	12.97	13.32	13.69	14.04	14.39	14.74	15.10	15.45	15.81	16.16	16.52	16.87	17.22
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	30.10	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	13.50	13.50	14.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.50	15.50	15.50	15.50	15.50	16.00
L_4	3.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'																				15°-0'
Submergence	0.46	0.50	0.54	0.57	0.61	0.64	0.67	0.45	0.48	0.51	0.53	0.56	0.59	0.62	0.65	0.69	0.44	0.47	0.50	0.53	0.56
d_2	3.34	3.37	3.41	3.44	3.47	3.50	3.53	3.56	3.59	3.62	3.64	3.66	3.69	3.72	3.75	3.77	3.79	3.82	3.85	3.87	3.90
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − Diameter of pipe − hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-6

PIPE DROP WITH MITER BENDS

Earth Outlet Transition

Check and Pipe Inlet   Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 13 cfs   Diameter of pipe = 30 inches V_p = 2.65 fps hv_p = 0.11 foot   Inlet R = 3.75 feet

F	2.93	3.25	3.58	3.90	4.25	4.59	4.91	5.23	5.58	5.91	6.24	6.57	6.91	7.25	7.59	7.93	8.28	8.61	8.95	9.30	9.63
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07
h_4					0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	10.50	11.00	11.00	11.00	11.50	11.50	12.00	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00
L_4					2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00
S_3					7°-30'										7°-30'	15°-0'					15°-0'
Submergence	0.47	0.54	0.59	0.65	0.44	0.49	0.54	0.60	0.64	0.43	0.48	0.53	0.58	0.62	0.66	0.42	0.46	0.51	0.55	0.59	0.64
d_2	2.61	2.67	2.72	2.77	2.81	2.85	2.90	2.95	2.99	3.04	3.08	3.13	3.17	3.21	3.25	3.28	3.32	3.36	3.40	3.44	3.48
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57
F	9.98	10.32	10.66	11.04	11.36	11.70	12.06	12.41	12.75	13.09	13.47	13.82	14.16	14.51	14.87	15.22	15.57	15.93	16.30	16.64	16.99
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.50	15.50	15.50	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.50	16.50	16.50
L_4	3.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00
S_3	15°-0'																		15°-0'	22°-30'	22°-30'
Submergence	0.67	0.46	0.50	0.53	0.56	0.60	0.63	0.66	0.71	0.46	0.49	0.52	0.56	0.59	0.62	0.65	0.68	0.70	0.72	0.44	0.47
d_2	3.51	3.55	3.59	3.62	3.65	3.68	3.71	3.74	3.77	3.79	3.82	3.85	3.88	3.91	3.94	3.96	3.99	4.01	4.03	4.06	4.09
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

Q = 14 cfs   Diameter of pipe = 30 inches V_p = 2.85 fps hv_p = 0.13 foot   Inlet R = 3.75 feet

F	2.80	3.12	3.43	3.75	4.09	4.41	4.75	5.08	5.42	5.75	6.09	6.42	6.76	7.10	7.43	7.77	8.12	8.45	8.79	9.13	9.47
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4		0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	11.00	11.00	11.50	11.50	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50
L_4		2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00
S_3		7°-30'										7°-30'	15°-0'								15°-0'
Submergence	0.60	0.41	0.49	0.55	0.60	0.66	0.44	0.49	0.54	0.59	0.63	0.69	0.45	0.49	0.54	0.58	0.62	0.67	0.45	0.50	0.54
d_2	2.71	2.77	2.84	2.89	2.94	2.99	3.03	3.07	3.12	3.16	3.20	3.25	3.29	3.33	3.37	3.41	3.45	3.49	3.53	3.57	3.61
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57
F	9.81	10.14	10.49	10.83	11.18	11.53	11.87	12.22	12.56	12.88	13.06	13.61	13.96	14.30	14.66	15.00	15.34	15.69	16.05	16.39	16.74
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09
h_4	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	15.00	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.00	16.50	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.50	17.50
L_4	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	15°-0'											15°-0'	22°-30'								22°-30'
Submergence	0.58	0.63	0.67	0.71	0.48	0.51	0.56	0.59	0.63	0.67	0.70	0.73	0.45	0.48	0.51	0.55	0.59	0.62	0.65	0.70	0.73
d_2	3.65	3.69	3.73	3.76	3.79	3.82	3.86	3.89	3.93	3.96	3.99	4.02	4.05	4.08	4.11	4.15	4.18	4.21	4.24	4.27	4.30
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – Diameter of pipe – hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-7

PIPE DROP WITH MITER BENDS

Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 15 cfs Diameter of pipe = 30 inches V_p = 3.05 fps hv_p = 0.15 foot Inlet R = 3.75 feet

F	2.67	3.00	3.33	3.65	3.98	4.29	4.62	4.94	5.27	5.59	5.92	6.25	6.60	6.93	7.11	7.59	7.94	8.27	8.59	8.94	9.29
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.34
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	11.50	11.50	12.00	12.00	12.00	12.50	12.50	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00
L_4	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00
S_3	7°-30'								7°-30'	15°-0'											15°-0'
Submergence	0.48	0.54	0.59	0.65	0.45	0.52	0.57	0.63	0.69	0.47	0.52	0.57	0.61	0.66	0.45	0.50	0.54	0.59	0.65	0.68	0.44
d_2	2.81	2.86	2.91	2.96	3.02	3.08	3.13	3.18	3.24	3.29	3.34	3.38	3.42	3.46	3.51	3.55	3.59	3.64	3.69	3.72	3.75
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57
F	9.63	9.97	10.31	10.65	10.98	11.33	11.68	12.01	12.35	12.68	13.04	13.38	13.72	14.05	14.40	14.73	15.07	15.40	15.75	16.09	16.42
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.10	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	15.50	15.50	15.50	16.00	16.00	16.00	16.50	16.50	16.50	16.50	17.00	17.00	17.00	17.50	17.50	17.50	18.00	18.00	18.00	18.50	18.50
L_4	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'						22°-30'														22°-30'
Submergence	0.50	0.54	0.59	0.63	0.67	0.71	0.43	0.48	0.52	0.55	0.60	0.64	0.68	0.74	0.78	0.42	0.46	0.50	0.55	0.59	0.64
d_2	3.79	3.83	3.87	3.91	3.95	3.98	4.02	4.06	4.10	4.13	4.17	4.21	4.25	4.29	4.33	4.37	4.41	4.45	4.49	4.53	4.57
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

Q = 16 cfs Diameter of pipe = 30 inches V_p = 3.26 fps hv_p = 0.17 foot Inlet R = 3.75 feet

F	2.57	2.90	3.20	3.52	3.86	4.18	4.51	4.82	5.15	5.47	5.79	6.11	6.46	6.78	7.12	7.45	7.79	8.11	8.44	8.79	9.12
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08
h_4	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	11.50	12.00	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50
L_4	2.00	2.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00
S_3	7°-30'						7°-30'	15°-0'													15°-0'
Submergence	0.58	0.64	0.46	0.52	0.57	0.63	0.68	0.46	0.53	0.59	0.65	0.45	0.49	0.55	0.59	0.64	0.42	0.48	0.54	0.58	0.63
d_2	2.88	2.94	3.01	3.06	3.11	3.16	3.21	3.27	3.33	3.38	3.44	3.49	3.53	3.58	3.62	3.66	3.71	3.76	3.81	3.85	3.89
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57
F	9.45	9.78	10.12	10.45	10.79	11.13	11.47	11.81	12.14	12.46	12.83	13.16	13.50	13.82	14.17	14.50	14.84	15.17	15.51	15.85	16.18
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10
h_4	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00	17.50	17.50	17.50	17.50	18.00	18.00	18.00	18.50	18.50	19.00	19.00	19.00	19.50
L_4	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00
S_3	15°-0'	15°-0'	22°-30'																		22°-30'
Submergence	0.68	0.73	0.46	0.51	0.55	0.59	0.64	0.68	0.74	0.78	0.41	0.46	0.50	0.56	0.60	0.65	0.69	0.74	0.79	0.41	0.47
d_2	3.94	3.98	4.03	4.07	4.11	4.15	4.19	4.23	4.27	4.31	4.35	4.39	4.43	4.48	4.52	4.56	4.60	4.65	4.69	4.73	4.77
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − Diameter of pipe − hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-8

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet — Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 17 cfs — Diameter of pipe = 30 inches — V_p = 3.46 fps — hv_p = 0.19 foot — Inlet R = 3.75 feet

F	2.47.	2.80	3.10	3.42	3.74	4.05	4.43	4.69	5.02	5.34	5.66	5.98	6.31	8.63	6.96	7.28	7.62	7.94	8.26	8.60	8.94
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.00	12.00	12.50	12.50	13.00	13.00	13.50	13.50	14.00	14.00	14.50	14.50	15.00	15.00	15.00	15.50	15.50	16.00	16.00	16.00	16.50
L_4	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00
S_3	7°-30'				7°-30'	15°-0'												15°-0'	22°-30'		22°-30'
Submergence	0.41	0.48	0.56	0.62	0.69	0.47	0.54	0.60	0.66	0.46	0.52	0.58	0.64	0.43	0.48	0.55	0.60	0.66	0.39	0.45	0.49
d_2	2.95	3.01	3.08	3.14	3.20	3.26	3.32	3.37	3.43	3.48	3.54	3.59	3.65	3.70	3.75	3.80	3.85	3.90	3.95	4.00	4.04
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57

F	9.26	9.59	9.93	10.25	10.59	10.92	11.26	11.60	11.94	12.24	12.61	12.93	13.27	13.59	13.94	14.26	14.60	14.93	15.27	15.60	15.93
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10
h_4	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49	2.49
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.30	39.30	40.20	41.10	42.00	43.00
L_3	16.50	16.50	17.00	17.00	17.50	17.50	17.50	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00	19.00	19.50	19.50	20.00	20.00	20.00
L_4	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00
S_3	22°-30'																				22°-30'
Submergence	0.55	0.60	0.65	0.70	0.76	0.40	0.45	0.49	0.53	0.59	0.63	0.69	0.73	0.79	0.41	0.45	0.52	0.57	0.62	0.67	0.72
d_2	4.09	4.14	4.18	4.23	4.27	4.32	4.36	4.40	4.44	4.49	4.53	4.58	4.62	4.67	4.71	4.76	4.80	4.85	4.89	4.94	4.98
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	13.95	14.33	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60

Q = 18 cfs — Diameter of pipe = 36 inches — V_p = 2.54 fps — hv_p = 0.10 foot — Inlet R = 4.25 feet

F	3.14	3.48	3.80	4.13	4.46	4.80	5.13	5.47	5.81	6.14	6.48	6.81	7.16	7.49	7.83	8.18	8.54	8.88	9.24	9.60	9.94
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4										0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.52
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.00	12.00	12.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50	14.50	14.50	15.00
L_4										2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3										7°-30'											7°-30'
Submergence	0.27	0.32	0.37	0.42	0.47	0.53	0.58	0.62	0.67	0.48	0.52	0.56	0.60	0.64	0.43	0.46	0.49	0.53	0.55	0.58	0.62
d_2	2.88	2.93	2.98	3.03	3.08	3.12	3.16	3.20	3.25	3.29	3.33	3.37	3.41	3.45	3.49	3.52	3.55	3.58	3.60	3.63	3.66
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07

F	10.29	10.64	11.00	11.34	11.69	12.04	12.39	12.74	13.09	13.43	13.79	14.14	14.49	14.84	15.20	15.55	15.91	16.25	16.61	16.97	17.31
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	15.00	15.00	15.00	15.50	15.50	15.50	15.50	16.00	16.00	16.00	16.00	16.50	16.50	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.00
L_4	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00
S_3	7°-30'	7°-30'	15°-0'																		15°-0'
Submergence	0.65	0.68	0.42	0.47	0.50	0.53	0.57	0.60	0.63	0.67	0.44	0.47	0.50	0.53	0.56	0.59	0.61	0.65	0.68	0.43	0.48
d_2	3.69	3.72	3.75	3.78	3.81	3.84	3.87	3.90	3.93	3.96	3.99	4.02	4.05	4.07	4.10	4.13	4.15	4.18	4.21	4.23	4.26
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	14.45	14.83	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10

$D = (d_1 + hv_1) - (1.1d_2 + hv_p)$ — $d_1 + hv_1 = R + h_2 + 0.06$ — Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − Diameter of pipe − hv

Table 2-3.–*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*–Continued. 103-D-1268-9

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 19 cfs Diameter of pipe = 36 inches V_p = 2.68 fps hv_p = 0.11 foot Inlet R = 4.25 feet

F	3.05	3.39	3.73	4.05	4.39	4.71	5.05	5.38	5.72	6.05	6.39	6.73	7.07	7.41	7.74	8.08	8.44	8.78	9.13	9.49	9.82
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08
h_4								0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.00	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50
L_4								2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00
S_3								7°-30'										7°-30'	15°-0'		15°-0'
Submergence	0.36	0.41	0.45	0.51	0.56	0.62	0.66	0.46	0.51	0.56	0.60	0.64	0.43	0.47	0.52	0.56	0.59	0.63	0.37	0.40	0.45
d_2	2.95	3.00	3.04	3.09	3.14	3.19	3.23	3.27	3.32	3.36	3.40	3.44	3.48	3.52	3.56	3.60	3.63	3.66	3.69	3.72	3.76
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07

F	10.17	10.52	10.88	11.22	11.57	11.92	12.27	12.62	12.96	13.30	13.66	14.00	14.34	14.69	15.05	15.39	15.74	16.09	16.45	16.79	17.15
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09
h_4	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.00	16.50	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.50	17.50	17.50	17.50	18.00
L_4	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00
S_3	15°-0'																				15°-0'
Submergence	0.49	0.52	0.55	0.59	0.62	0.65	0.43	0.46	0.50	0.54	0.57	0.61	0.65	0.68	0.44	0.48	0.52	0.55	0.58	0.62	0.64
d_2	3.79	3.82	3.85	3.88	3.91	3.94	3.97	4.00	4.04	4.07	4.10	4.14	4.17	4.20	4.23	4.26	4.29	4.32	4.35	4.38	4.40
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	14.45	14.83	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10

Q = 20 cfs Diameter of pipe = 36 inches V_p = 2.82 fps hv_p = 0.12 foot Inlet R = 4.25 feet

F	2.98	3.31	3.64	3.96	4.30	4.62	4.95	5.27	5.60	5.92	6.27	6.61	6.95	7.29	7.62	7.96	8.31	8.64	8.98	9.32	9.66
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08
h_4						0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00
L_4						2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00
S_3						7°-30'										7°-30'	15°-0'				15°-0'
Submergence	0.43	0.49	0.54	0.60	0.65	0.45	0.51	0.57	0.63	0.69	0.46	0.50	0.55	0.59	0.64	0.68	0.43	0.49	0.53	0.58	0.62
d_2	3.01	3.06	3.11	3.16	3.21	3.26	3.31	3.36	3.42	3.47	3.50	3.54	3.58	3.62	3.66	3.70	3.74	3.78	3.82	3.86	3.90
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07

F	10.01	10.35	10.71	11.06	11.39	11.73	12.08	12.42	12.77	13.12	13.47	13.82	14.17	14.51	14.87	15.22	15.57	15.91	16.27	16.62	16.96
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.81	0.81	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	16.00	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.00	17.50	17.50	17.50	17.50	18.00	18.00	18.00	18.00	18.50	18.50
L_4	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00
S_3	15°-0'																15°-0'	22°-30'			22°-30'
Submergence	0.65	0.69	0.46	0.49	0.54	0.58	0.62	0.66	0.42	0.45	0.49	0.53	0.56	0.60	0.63	0.66	0.69	0.41	0.44	0.47	0.51
d_2	3.93	3.96	3.99	4.02	4.06	4.10	4.14	4.17	4.20	4.23	4.26	4.29	4.32	4.35	4.38	4.41	4.44	4.47	4.50	4.53	4.56
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	14.45	14.83	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – Diameter of pipe – hv

Table 2-3.—*Type 1 pipe drop with earth outlet transition (to be used with fig. 2-29).*—Continued. 103-D-1268-10

PIPE DROP WITH MITER BENDS
Earth Outlet Transition

Check and Pipe Inlet — Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 21 cfs — Diameter of pipe = 36 inches — V_p = 2.97 fps — hv_p = 0.14 foot — Inlet R = 4.25 feet

F	2.89	3.23	3.55	3.87	4.20	4.52	4.84	5.17	5.51	5.84	6.17	6.50	6.84	7.18	7.52	7.85	8.20	8.52	8.86	9.20	9.54
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08
h_4					0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	1.07
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.50	12.50	13.00	13.00	13.50	13.50	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00
L_4					2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00
S_3					7°-30'									7°-30'	15°-0'						15°-0'
Submergence	0.52	0.57	0.63	0.69	0.50	0.56	0.62	0.67	0.46	0.51	0.56	0.61	0.66	0.70	0.45	0.51	0.55	0.61	0.65	0.70	0.48
d_2	3.07	3.12	3.17	3.23	3.28	3.34	3.39	3.44	3.48	3.53	3.57	3.62	3.66	3.70	3.74	3.78	3.82	3.87	3.91	3.95	3.99
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07

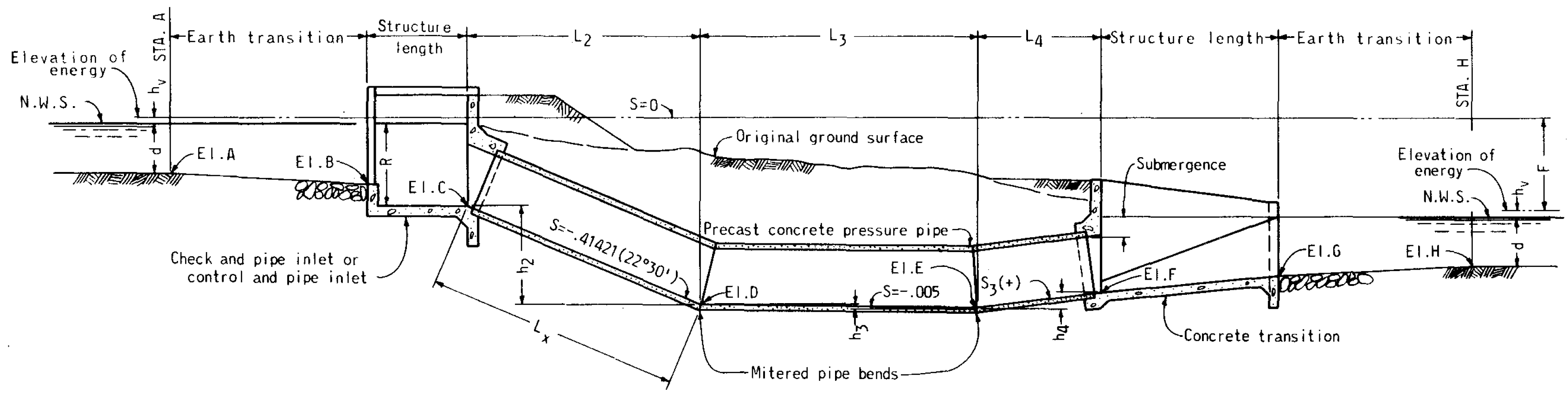
F	9.88	10.21	10.57	10.92	11.26	11.61	11.96	12.30	12.64	12.99	13.33	13.68	14.02	14.35	14.70	15.04	15.39	15.74	16.05	16.43	16.78
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	1.07	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	16.50	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.50	17.50	17.50	18.00	18.00	18.00	18.50	18.50	18.50	18.50	19.00	19.00	19.00
L_4	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	15°-0'												15°-0'	22°-30'							22°-30'
Submergence	0.52	0.57	0.60	0.63	0.67	0.70	0.47	0.52	0.56	0.59	0.64	0.67	0.71	0.44	0.48	0.52	0.55	0.58	0.63	0.66	0.69
d_2	4.03	4.07	4.10	4.13	4.16	4.19	4.23	4.26	4.30	4.33	4.37	4.40	4.44	4.48	4.52	4.55	4.58	4.61	4.65	4.68	4.71
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	14.45	14.83	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10

Q = 22 cfs — Diameter of pipe = 36 inches — V_p = 3.11 fps — hv_p = 0.15 foot — Inlet R = 4.25 feet

F	2.82	3.14	3.47	3.79	4.13	4.45	4.76	5.09	5.42	5.75	6.07	6.41	6.76	7.08	7.42	7.85	8.09	8.42	8.76	9.12	9.45
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76
h_3	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4			0.26	0.26	0.26	0.26	0.26	0.52	0.52	0.52	0.52	0.52	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1.07	1.07
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60
L_3	12.50	13.00	13.00	13.50	13.50	14.00	14.00	14.00	14.50	14.50	15.00	15.00	15.00	15.50	15.50	15.50	16.00	16.00	16.00	16.50	16.50
L_4			2.00	2.00	2.00	2.00	2.00	4.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00
S_3			7°-30'									7°-30'	15°-0'								15°-0'
Submergence	0.59	0.66	0.45	0.52	0.57	0.63	0.70	0.49	0.55	0.60	0.66	0.70	0.45	0.52	0.56	0.61	0.66	0.71	0.75	0.52	0.57
d_2	3.13	3.19	3.24	3.29	3.34	3.39	3.45	3.50	3.55	3.60	3.65	3.69	3.73	3.78	3.82	3.86	3.91	3.95	3.99	4.02	4.06
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07

F	9.79	10.14	10.48	10.83	11.17	11.51	11.87	12.21	12.64	12.87	13.22	13.56	13.89	14.24	14.60	14.94	15.28	15.62	15.96	16.31	16.66
h_2	10.14	10.52	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79
h_3	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10
h_4	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66
L_2	24.50	25.40	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00
L_3	16.50	16.50	17.00	17.00	17.00	17.50	17.50	17.50	17.50	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.50	19.50
L_4	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	15°-0'										15°-0'	22°-30'									22°-30'
Submergence	0.61	0.64	0.69	0.72	0.76	0.54	0.57	0.61	0.66	0.70	0.75	0.47	0.50	0.55	0.58	0.62	0.66	0.70	0.75	0.79	0.82
d_2	4.10	4.13	4.17	4.20	4.24	4.27	4.30	4.34	4.38	4.42	4.46	4.50	4.53	4.57	4.60	4.64	4.67	4.71	4.75	4.78	4.81
L_x	26.50	27.50	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50
$d_1 + hv_1$	14.45	14.83	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $\qquad d_1 + hv_1 = R + h_2 + 0.06$ $\qquad$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4 -$ Diameter of pipe $-$ hv



LONGITUDINAL SECTION

NOTES

The pipe slopes used will allow the substitution of 7°30' precast concrete elbows for the mitered pipe bends.
The maximum velocity in the pipe at the outlet equals 5 feet per second for pipe flowing full.
Hydraulic head losses due to entrance, friction, bends, and exit have been neglected.
Precast concrete pressure pipe with mitered pipe bends shown. AC or RPM may be substituted.
These standard designs are for flows less than 50 cubic feet per second. For flows greater than 49 cubic feet per second it is probably more economical to use a rectangular inclined drop Figures 2-26 and 2-27. If it should be desirable to have a pipe drop with a flow greater than 49 cubic feet per second, it will be necessary to actually design the pipe drop.
Ordinarily the maximum water surface drop for a pipe drop structure is 15 feet. However, standard design tables extend slightly beyond this maximum.

Figure 2-30. **Type 1 pipe drop with concrete outlet transition–explanation sheet for table 2-4. 103-D-1269.**

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).* 103-D-1270-1

PIPE DROP WITH MITER BENDS

Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 3 cfs Diameter of pipe = 15 inches V_p = 2.44 fps hv_p = 0.09 foot Inlet R = 2.50 feet

F	2.68	3.01	3.33	3.65	3.98	4.31	4.64	4.97	5.30	5.64	5.98	6.32	6.66	7.01	7.34	7.69	8.04	8.38	8.72	9.08	9.41	9.76	10.12
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
h_4	0.26	0.26	0.26	0.26	0.26	0.26	0.39	0.39	0.39	0.39	0.39	0.39	0.53	0.80	0.80	0.80	0.80	0.80	0.80	1.07	1.07	1.07	1.07
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00
L_4	2.00	2.00	2.00	2.00	2.00	2.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00
S_3	7°-30'												7°-30'	15°									15°
Submergence	0.43	0.49	0.55	0.61	0.66	0.72	0.64	0.69	0.74	0.79	0.83	0.87	0.78	0.52	0.57	0.60	0.64	0.68	0.72	0.48	0.53	0.56	0.59
d_2	1.72	1.77	1.83	1.88	1.93	1.98	2.03	2.07	2.12	2.16	2.20	2.24	2.28	2.31	2.35	2.38	2.42	2.45	2.49	2.52	2.56	2.59	2.62
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	10.46	10.80	11.15	11.49	11.84	12.19	12.54	12.89	13.24	13.60	13.94	14.29	14.64	14.98	15.33	15.69	16.04	16.38	16.73	17.09	17.43	17.78	18.13
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05
h_4	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.61	1.61	1.61	1.61	1.61	1.61	1.61	1.61	1.86	1.86	1.86
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	9.50	9.50	9.50
L_4	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	4.50	4.50	4.50
S_3	15°																			15°	22°-30'		22°-30'
Submergence	0.63	0.67	0.70	0.48	0.51	0.54	0.58	0.61	0.64	0.67	0.71	0.74	0.50	0.54	0.57	0.60	0.64	0.67	0.70	0.73	0.47	0.50	0.53
d_2	2.65	2.69	2.72	2.76	2.79	2.82	2.85	2.88	2.91	2.94	2.97	3.00	3.03	3.06	3.09	3.12	3.15	3.18	3.21	3.24	3.27	3.30	3.33
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

Q = 4 cfs Diameter of pipe = 15 inches V_p = 3.26 fps hv_p = 0.17 foot Inlet R = 2.50 feet

F	2.28	2.59	2.90	3.20	3.52	3.82	4.15	4.46	4.79	5.10	5.43	5.75	6.08	6.40	6.73	7.04	7.38	7.70	8.04	8.37	8.70	9.03	9.37
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.03
h_4	0.53	0.53	0.53	0.53	0.66	0.66	0.66	0.66	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.74	1.74	1.74	1.74	1.74
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	6.50	6.50	6.50	6.50	6.50
L_4	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	6.50	6.50	6.50	6.50	6.50
S_3	7°-30'							7°-30'	15°														15°
Submergence	0.56	0.64	0.72	0.79	0.73	0.81	0.86	0.93	0.56	0.63	0.68	0.74	0.80	0.59	0.64	0.71	0.77	0.82	0.45	0.51	0.56	0.61	0.65
d_2	2.01	2.08	2.15	2.22	2.28	2.35	2.40	2.46	2.52	2.58	2.63	2.68	2.74	2.79	2.84	2.90	2.95	3.00	3.04	3.09	3.14	3.18	3.22
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	9.70	10.04	10.38	10.71	11.06	11.39	11.74	12.08	12.42	12.76	13.09	13.43	13.79	14.12	14.47	14.81	15.15	15.50	15.85	16.19	16.54	16.89	17.22
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	1.88	1.88	1.88	1.88	1.88	1.88	1.88	2.07	2.07	2.07	2.07	2.07	2.07	2.48	2.48	2.48	2.48	2.48	2.48	2.48	2.69	2.69	2.69
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	7.00	7.00	7.00	7.00	7.00	7.00	7.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.50	10.50	10.50
L_4	7.00	7.00	7.00	7.00	7.00	7.00	7.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.50	6.50	6.50
S_3	15°						15°	22°-30'															22°-30'
Submergence	0.58	0.62	0.67	0.71	0.75	0.80	0.83	0.63	0.68	0.72	0.77	0.81	0.84	0.48	0.51	0.55	0.60	0.63	0.66	0.71	0.53	0.56	0.61
d_2	3.27	3.31	3.35	3.39	3.43	3.47	3.50	3.54	3.58	3.62	3.66	3.70	3.73	3.77	3.80	3.84	3.88	3.91	3.94	3.98	4.01	4.04	4.08
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – diameter of pipe – hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-2

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 5 cfs Diameter of pipe = 15 inches V_p = 4.07 fps hv_p = 0.26 foot Inlet R = 2.50 feet

F	1.91	2.19	2.48	2.77	3.07	3.36	3.65	3.96	4.26	4.56	4.86	5.18	5.50	5.81	6.12	6.44	6.76	7.08	7.40	7.72	8.03	8.36	8.67
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	0.66	1.07	1.07	1.21	1.21	1.21	1.47	1.47	1.47	1.47	1.74	1.74	1.74	1.74	2.07	2.07	2.07	2.07	2.28	2.28	2.28	2.28	2.49
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	5.00	6.00	6.00	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	6.50	9.00	9.00	9.00	9.00	9.50	9.50	9.50	9.50	10.00
L_4	5.00	4.00	4.00	4.50	4.50	4.50	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	5.00	5.00	5.00	5.00	5.50	5.50	5.50	5.50	6.00
S_3	7°-30'	15°												15°	22°-30'								22°-30'
Submergence	0.80	0.47	0.57	0.51	0.60	0.69	0.52	0.59	0.68	0.76	0.57	0.63	0.70	0.78	0.47	0.53	0.60	0.66	0.51	0.58	0.65	0.70	0.56
d_2	2.26	2.36	2.45	2.53	2.61	2.69	2.77	2.84	2.92	2.99	3.06	3.12	3.18	3.25	3.31	3.36	3.43	3.48	3.54	3.60	3.66	3.71	3.77
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	9.01	9.33	9.66	9.97	10.31	10.63	10.96	11.28	11.62	11.94	12.28	12.60	12.94	13.26	13.60	13.92	14.27	14.61	14.94	15.28	15.61	15.94	16.27
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73	3.73
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.50	10.50	10.50	10.50	10.50	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00
L_4	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00
S_3	22°-30'																						22°-30'
Submergence	0.61	0.67	0.72	0.79	0.43	0.49	0.54	0.60	0.65	0.50	0.54	0.60	0.65	0.71	0.56	0.62	0.66	0.70	0.75	0.80	0.43	0.48	0.53
d_2	3.82	3.87	3.92	3.98	4.03	4.08	4.13	4.18	4.23	4.28	4.32	4.37	4.42	4.47	4.51	4.56	4.60	4.64	4.68	4.73	4.77	4.82	4.86
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.70	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

Q = 6 cfs Diameter of pipe = 15 inches V_p = 4.89 fps hv_p = 0.37 foot Inlet R = 2.50 feet

F	1.61	1.88	2.13	2.40	2.68	2.97	3.25	3.53	3.82	4.11	4.40	4.69	5.00	5.29	5.58	5.88	6.20	6.50	6.80	7.10	7.42	7.72	8.02
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	1.21	1.34	1.34	1.61	1.61	1.61	1.74	1.74	1.74	2.07	2.07	2.07	2.49	2.49	2.49	2.69	2.69	2.69	2.69	2.90	2.90	2.90	3.11
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	5.50	6.00	6.00	6.00	6.00	6.00	6.50	6.50	6.50	9.00	9.00	9.00	9.00	9.00	9.00	9.50	9.50	9.50	9.50	10.00	10.00	10.00	10.50
L_4	4.50	5.00	5.00	6.00	6.00	6.00	6.50	6.50	6.50	5.00	5.00	5.00	6.00	6.00	6.00	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.50
S_3	15°								15°	22°-30'													22°-30'
Submergence	0.52	0.52	0.64	0.48	0.59	0.69	0.65	0.75	0.85	0.57	0.66	0.75	0.41	0.50	0.59	0.47	0.54	0.62	0.70	0.58	0.64	0.72	0.59
d_2	2.44	2.55	2.66	2.76	2.86	2.95	3.04	3.13	3.22	3.30	3.38	3.46	3.54	3.62	3.70	3.77	3.84	3.91	3.98	4.06	4.12	4.19	4.26
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	8.34	8.65	8.95	9.27	9.59	9.91	10.22	10.54	10.87	11.18	11.51	11.82	12.15	12.47	12.80	13.11	13.44	13.77	14.10	14.42	14.73	15.06	15.38
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	3.11	3.11	3.11	3.52	3.52	3.52	3.52	3.52	3.73	3.73	3.73	3.73	4.14	4.14	4.14	4.14	4.14	4.35	4.35	4.35	4.35	4.56	4.56
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	10.50	10.50	10.50	10.50	10.50	10.50	10.50	10.50	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.50	11.50	11.50	11.50	12.00	12.00
L_4	7.50	7.50	7.50	8.50	8.50	8.50	8.50	8.50	9.00	9.00	9.00	9.00	10.00	10.00	10.00	10.00	10.00	10.50	10.50	10.50	10.50	11.00	11.00
S_3	22°-30'																						22°-30'
Submergence	0.66	0.73	0.81	0.46	0.53	0.59	0.66	0.72	0.58	0.65	0.70	0.77	0.42	0.48	0.53	0.60	0.66	0.50	0.56	0.62	0.69	0.53	0.59
d_2	4.33	4.39	4.46	4.52	4.58	4.64	4.70	4.75	4.81	4.87	4.92	4.98	5.04	5.09	5.14	5.20	5.25	5.30	5.35	5.41	5.47	5.52	5.57
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30)*.—Continued. 103-D-1270-3

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

Q = 7 cfs Diameter of pipe = 18 inches V_p = 3.96 fps hv_p = 0.24 foot Inlet R = 2.50 feet

F	1.82	2.12	2.41	2.70	3.00	3.30	3.60	3.90	4.21	4.53	4.83	5.14	5.46	5.77	6.08	6.40	6.72	7.04	7.35	7.67	7.98	8.30	8.63
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05
h_4	0.66	0.66	1.07	1.07	1.07	1.07	1.07	1.34	1.34	1.47	1.47	1.47	1.47	1.74	1.74	1.74	1.74	2.01	2.01	2.01	2.01	2.28	2.28
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	5.00	5.00	7.00	7.00	7.00	8.00	8.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	10.50	10.50
L_4	5.00	5.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	7.50	7.50	7.50	7.50	5.50	5.50
S_3	7°-30'	7°-30'	15°																		15°	22°-30'	22°-30'
Submergence	0.64	0.74	0.38	0.47	0.56	0.64	0.72	0.53	0.61	0.54	0.62	0.69	0.76	0.56	0.63	0.69	0.76	0.55	0.62	0.69	0.76	0.49	0.54
d_2	2.36	2.45	2.53	2.61	2.69	2.76	2.84	2.91	2.98	3.04	3.11	3.17	3.24	3.30	3.36	3.42	3.48	3.54	3.60	3.66	3.73	3.78	3.83
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	8.96	9.28	9.60	9.93	10.26	10.59	10.91	11.25	11.58	11.92	12.25	12.58	12.91	13.25	13.58	13.92	14.26	14.59	14.93	15.26	15.60	15.94	16.27
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	2.28	2.28	2.28	2.49	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.31	3.31	3.31	3.31
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	10.50	10.50	10.50	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.50	11.50	11.50	11.50	11.50	12.00	12.00	12.00	12.00
L_4	5.50	5.50	5.50	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	8.00	8.00	8.00	8.00
S_3	22°-30'																						22°-30'
Submergence	0.60	0.66	0.72	0.57	0.63	0.68	0.74	0.78	0.43	0.47	0.52	0.57	0.63	0.67	0.52	0.55	0.60	0.65	0.69	0.55	0.59	0.63	0.68
d_2	3.88	3.94	3.99	4.04	4.09	4.14	4.19	4.23	4.28	4.32	4.36	4.41	4.46	4.50	4.55	4.58	4.63	4.67	4.71	4.76	4.80	4.84	4.88
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

Q = 8 cfs Diameter of pipe = 18 inches V_p = 4.53 fps hv_p = 0.32 foot Inlet R = 2.50 feet

F	1.58	1.86	2.14	2.42	2.71	2.99	3.28	3.58	3.87	4.16	4.46	4.77	5.08	5.38	5.68	5.99	6.29	6.60	6.92	7.23	7.53	7.85	8.16
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	1.07	1.07	1.07	1.34	1.34	1.34	1.61	1.61	1.61	1.88	1.88	1.88	2.01	2.01	2.01	2.01	2.28	2.28	2.28	2.28	2.69	2.69	2.69
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	10.50	10.50	10.50	10.50	10.50	10.50	10.50
L_4	4.00	4.00	4.00	5.00	5.00	5.00	6.00	6.00	6.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	5.50	5.50	5.50	5.50	6.50	6.50	6.50
S_3	15°															15°	22°-30'						22°-30'
Submergence	0.44	0.55	0.65	0.48	0.58	0.68	0.50	0.58	0.68	0.50	0.58	0.65	0.60	0.68	0.76	0.83	0.59	0.66	0.72	0.80	0.47	0.53	0.60
d_2	2.51	2.61	2.70	2.79	2.88	2.97	3.05	3.13	3.22	3.30	3.37	3.44	3.51	3.58	3.65	3.72	3.80	3.86	3.92	3.99	4.06	4.12	4.18
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08
F	8.49	8.80	9.11	9.43	9.76	10.09	10.40	10.72	11.04	11.36	11.69	12.02	12.35	12.68	13.00	13.32	13.66	13.98	14.31	14.64	14.97	15.31	15.64
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	2.69	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.52	3.52	3.52	3.52	3.52	3.73	3.73	3.73	3.73	3.73	3.93	3.93	3.93
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	10.50	11.00	11.00	11.00	11.00	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	12.00	12.00	12.00	12.00	12.00	12.50	12.50	12.50
L_4	6.50	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	8.50	8.50	8.50	8.50	8.50	9.00	9.00	9.00	9.00	9.00	9.50	9.50	9.50
S_3	22°-30'																						22°-30'
Submergence	0.66	0.53	0.60	0.66	0.72	0.56	0.63	0.69	0.76	0.82	0.46	0.52	0.57	0.63	0.68	0.53	0.58	0.64	0.69	0.75	0.60	0.64	0.69
d_2	4.24	4.30	4.36	4.42	4.47	4.52	4.58	4.64	4.70	4.75	4.80	4.85	4.90	4.95	5.00	5.05	5.10	5.15	5.20	5.25	5.30	5.34	5.38
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-4

PIPE DROP WITH MITER BENDS

Concrete Outlet Transition

Check and Pipe Inlet — Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 9 cfs — Diameter of pipe = 18 inches — V_p = 5.09 fps — hv_p = 0.40 foot — Inlet R = 2.50 feet

F	1.39	1.66	1.91	2.18	2.46	2.73	3.00	3.26	3.56	3.84	4.13	4.42	4.72	5.00	5.30	5.59	5.90	6.19	6.50	6.81	7.11	7.42	7.72
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06
h_4	1.07	1.07	1.34	1.34	1.61	1.61	1.61	1.88	1.88	2.01	2.01	2.01	2.49	2.49	2.49	2.69	2.69	2.69	2.69	2.90	2.90	2.90	3.31
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	10.00	10.00	10.00	10.50	10.50	10.50	10.50	11.00	11.00	11.00	11.00
L_4	4.00	4.00	5.00	5.00	6.00	6.00	6.00	7.00	7.00	7.50	7.50	7.50	6.00	6.00	6.00	6.50	6.50	6.50	6.50	7.00	7.00	7.00	8.00
S_3	15°											15°	22°-30'										22°-30'
Submergence	0.63	0.75	0.61	0.72	0.56	0.67	0.78	0.64	0.72	0.69	0.78	0.87	0.42	0.52	0.60	0.49	0.57	0.66	0.73	0.61	0.69	0.76	0.43
d_2	2.61	2.72	2.84	2.94	3.04	3.14	3.24	3.35	3.43	3.52	3.60	3.68	3.76	3.85	3.93	4.01	4.08	4.16	4.23	4.30	4.37	4.44	4.51
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	4.66	5.05	5.43	5.81	6.20	6.58	6.96	7.34	7.73	8.11	8.49	8.87	9.26	9.64	10.02	10.40	10.79	11.17	11.55	11.94	12.32	12.70	13.08

F	8.03	8.35	8.65	8.96	9.29	9.59	9.90	10.22	10.54	10.86	11.17	11.48	11.82	12.13	12.44	12.77	13.10	13.41	13.72	14.06	14.37	14.70	15.02
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07
h_4	3.31	3.31	3.31	3.52	3.52	3.52	3.52	3.73	3.73	3.73	3.73	4.14	4.14	4.14	4.14	4.14	4.35	4.35	4.35	4.35	4.56	4.56	4.56
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	11.00	11.00	11.00	11.50	11.50	11.50	11.50	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.50	12.50	12.50	12.50	13.00	13.00	13.00
L_4	8.00	8.00	8.00	8.50	8.50	8.50	8.50	9.00	9.00	9.00	9.00	10.00	10.00	10.00	10.00	10.00	10.50	10.50	10.50	10.50	11.00	11.00	11.00
S_3	22°-30'																						22°-30'
Submergence	0.51	0.57	0.65	0.51	0.57	0.65	0.72	0.57	0.64	0.70	0.77	0.43	0.48	0.55	0.62	0.67	0.52	0.59	0.66	0.71	0.58	0.63	0.69
d_2	4.58	4.64	4.71	4.77	4.83	4.90	4.96	5.02	5.08	5.14	5.20	5.26	5.31	5.37	5.44	5.48	5.54	5.60	5.66	5.71	5.77	5.82	5.87
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.47	13.85	14.23	14.61	15.00	15.38	15.76	16.14	16.53	16.91	17.29	17.67	18.06	18.44	18.82	19.20	19.59	19.97	20.35	20.74	21.12	21.50	21.88

Q = 10 cfs — Diameter of pipe = 21 inches — V_p = 4.16 fps — hv_p = 0.27 foot — Inlet R = 3.00 feet

F	1.99	2.29	2.57	2.85	3.15	3.44	3.74	4.03	4.35	4.64	4.96	5.27	5.58	5.89	6.20	6.52	6.84	7.15	7.47	7.79	8.12	8.44	8.76
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	0.80	0.80	1.07	1.07	1.07	1.07	1.34	1.34	1.24	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.49	2.49
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	9.00	9.50	9.50	10.00	10.00	10.00	10.00	10.50	11.00	11.00	11.00	11.00	11.50	12.00	12.50	13.00	13.00	13.00	13.50	14.00	14.00	14.00	14.00
L_4	3.00	3.00	4.00	4.00	4.00	4.00	5.00	5.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00
S_3	15°							15°	22°-30'														22°-30'
Submergence	0.55	0.64	0.47	0.57	0.66	0.75	0.56	0.65	0.74	0.41	0.47	0.54	0.62	0.70	0.77	0.43	0.49	0.56	0.63	0.70	0.75	0.39	0.46
d_2	2.64	2.72	2.81	2.90	2.98	3.06	3.14	3.22	3.28	3.36	3.42	3.48	3.55	3.62	3.68	3.74	3.80	3.86	3.92	3.98	4.03	4.08	4.14
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.16	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.37	9.76	10.14	10.52	10.90	11.29	11.67	12.05	12.44	12.82	13.20	13.58

F	9.09	9.41	9.73	10.06	10.39	10.72	11.04	11.36	11.70	12.02	12.35	12.69	13.02	13.35	13.68	14.01	14.34	14.67	15.00	15.34	15.67	16.01	16.35
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09
h_4	2.49	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.00	14.00	14.50	14.50	15.00	15.00	15.00	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.50	16.50	17.00	17.00	17.50	17.50	17.50
L_4	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00
S_3	22°-30'																						22°-30'
Submergence	0.51	0.58	0.64	0.69	0.75	0.39	0.44	0.50	0.56	0.62	0.67	0.71	0.77	0.41	0.45	0.51	0.57	0.62	0.67	0.72	0.77	0.40	0.44
d_2	4.19	4.25	4.30	4.35	4.40	4.45	4.50	4.55	4.60	4.65	4.70	4.74	4.79	4.84	4.88	4.93	4.98	5.03	5.07	5.12	5.16	5.20	5.24
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.17	18.56	18.94	19.32	19.70	20.09	20.47	20.85	21.24	21.62	22.00	22.38

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ — $d_1 + hv_1 = R + h_2 + 0.06$ — Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-5

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

Q = 11 cfs Diameter of pipe = 21 inches V_p = 4.57 fps hv_p = 0.33 foot Inlet R = 3.00 feet

F	1.81	2.07	2.36	2.64	2.92	3.21	3.50	3.79	4.09	4.39	4.68	4.99	5.29	5.60	5.90	6.21	6.53	6.84	7.14	7.47	7.78	8.10	8.41
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07
h_4	0.80	1.07	1.07	1.07	1.24	1.24	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.49
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	9.00	9.00	9.50	9.50	10.00	10.00	10.50	10.50	11.00	11.00	11.00	11.50	11.50	12.00	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50	14.00
L_4	3.00	4.00	4.00	4.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00
S_3	15°			15°	22°-30'																		22°-30'
Submergence	0.73	0.58	0.68	0.78	0.64	0.73	0.40	0.49	0.58	0.67	0.76	0.83	0.50	0.58	0.66	0.73	0.80	0.46	0.54	0.60	0.67	0.74	0.81
d_2	2.75	2.86	2.95	3.04	3.14	3.22	3.30	3.38	3.46	3.54	3.62	3.68	3.76	3.83	3.90	3.96	4.03	4.09	4.16	4.22	4.28	4.34	4.40
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.16	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.37	9.76	10.14	10.52	10.90	11.29	11.67	12.05	12.44	12.82	13.20	13.58

F	8.73	9.05	9.36	9.68	10.01	10.33	10.65	10.96	11.29	11.61	11.94	12.26	12.60	12.92	13.24	13.57	13.91	14.23	14.56	14.89	15.22	15.54	15.88
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09
h_4	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73	3.73	3.73	3.73	3.73	3.73	3.73	4.14	4.14
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.00	14.00	14.00	14.50	14.50	15.00	15.00	15.00	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.00	16.50	16.50	16.50	17.00	17.00	17.00	17.00
L_4	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	10.00	10.00
S_3	22°-30'																						22°-30'
Submergence	0.46	0.52	0.59	0.66	0.71	0.78	0.42	0.49	0.56	0.61	0.67	0.72	0.78	0.42	0.48	0.53	0.59	0.64	0.70	0.75	0.81	0.45	0.49
d_2	4.46	4.52	4.58	4.64	4.69	4.75	4.80	4.86	4.92	4.97	5.02	5.07	5.12	5.17	5.23	5.27	5.32	5.37	5.42	5.47	5.52	5.57	5.61
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.17	18.56	18.94	19.32	19.70	20.09	20.47	20.85	21.24	21.62	22.00	22.38

Q = 12 cfs Diameter of pipe = 21 inches V_p = 4.99 fps hv_p = 0.39 foot Inlet R = 3.00 feet

F	1.64	1.90	2.17	2.44	2.72	2.99	3.28	3.57	3.86	4.15	4.43	4.73	5.04	5.32	5.62	5.91	6.24	6.53	6.83	7.14	7.46	7.76	8.08
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07
h_4	1.07	1.07	1.24	1.24	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90	2.90
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	9.00	9.00	10.00	10.00	10.00	10.00	10.00	11.00	11.00	11.00	11.50	11.50	12.00	12.00	12.00	12.00	12.50	12.50	13.00	13.00	13.00	13.50	13.50
L_4	4.00	4.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00
S_3	15°	15°	22°-30'																				22°-30'
Submergence	0.63	0.75	0.62	0.73	0.42	0.53	0.63	0.72	0.83	0.50	0.60	0.68	0.78	0.44	0.52	0.61	0.67	0.76	0.44	0.52	0.58	0.66	0.73
d_2	2.85	2.96	3.06	3.16	3.26	3.36	3.45	3.53	3.62	3.70	3.79	3.86	3.94	4.03	4.10	4.18	4.24	4.32	4.39	4.46	4.52	4.59	4.65
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.16	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.37	9.76	10.14	10.52	10.90	11.29	11.67	12.05	12.44	12.82	13.20	13.58

F	8.39	8.70	9.02	9.33	9.65	9.97	10.28	10.60	10.91	11.22	11.55	11.86	12.19	12.51	12.82	13.15	13.47	13.80	14.12	14.45	14.77	15.10	15.42
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09
h_4	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73	3.73	3.73	3.73	3.73	4.14	4.14	4.14	4.14	4.14	4.14	4.14	4.56	4.56	4.56
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.00	14.00	14.00	14.00	14.50	14.50	14.50	15.00	15.00	15.00	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.50	16.50	17.00	17.00	17.00	17.00
L_4	7.00	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00	9.00	9.00	9.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	11.00	11.00	11.00
S_3	22°-30'																						22°-30'
Submergence	0.80	0.45	0.52	0.59	0.65	0.72	0.78	0.45	0.52	0.59	0.64	0.71	0.78	0.42	0.49	0.54	0.61	0.66	0.72	0.79	0.43	0.48	0.54
d_2	4.72	4.78	4.84	4.90	4.96	5.02	5.08	5.14	5.21	5.27	5.32	5.38	5.44	5.49	5.55	5.60	5.66	5.71	5.76	5.82	5.87	5.92	5.97
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.17	18.56	18.94	19.32	19.70	20.09	20.47	20.85	21.24	21.62	22.00	22.38

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-6

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 10 cfs Diameter of pipe = 24 inches V_p = 3.18 fps hv_p = 0.16 foot Inlet R = 3.25 feet

F	2.50	2.81	3.12	3.43	3.75	4.07	4.38	4.71	5.04	5.36	5.69	6.02	6.35	6.68	7.02	7.34	7.68	8.02	8.34	8.69	9.03	9.35	9.69
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	0.40	0.46	0.46	0.46	0.66	0.66	0.66	0.66	0.66	0.94	0.94	0.94	0.94	0.94	1.21	1.21	1.21	1.21	1.21	1.21	1.47	1.47	1.47
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	7.00	7.50	7.50	7.50	7.00	7.00	7.00	7.00	7.00	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50
L_4	3.00	3.50	3.50	3.50	5.00	5.00	5.00	5.00	5.00	3.50	3.50	3.50	3.50	3.50	4.50	4.50	4.50	4.50	4.50	4.50	5.50	5.50	5.50
S_3	7°-30'								7°-30'	15°													15°
Submergence	0.47	0.49	0.56	0.63	0.50	0.56	0.63	0.68	0.74	0.48	0.53	0.58	0.64	0.69	0.47	0.52	0.57	0.61	0.67	0.71	0.49	0.55	0.59
d_2	2.50	2.57	2.64	2.70	2.76	2.82	2.88	2.93	2.98	3.04	3.08	3.13	3.18	3.23	3.27	3.32	3.36	3.40	3.45	3.49	3.53	3.58	3.62
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07	13.45	13.83
F	10.03	10.37	10.71	11.04	11.39	11.72	12.06	12.40	12.75	13.09	13.44	13.77	14.12	14.46	14.81	15.15	15.49	15.84	16.19	16.54	16.89	17.23	17.57
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	1.47	1.47	1.47	1.74	1.74	1.74	1.74	1.74	1.74	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.28	2.28	2.28	2.28	2.28	2.28	2.28
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50
L_4	5.50	5.50	5.50	6.50	6.50	6.50	6.50	6.50	6.50	7.50	7.50	7.50	7.50	7.50	7.50	7.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50
S_3	15°																						15°
Submergence	0.64	0.68	0.72	0.50	0.54	0.59	0.63	0.67	0.71	0.48	0.51	0.56	0.60	0.64	0.67	0.71	0.49	0.52	0.55	0.59	0.62	0.66	0.70
d_2	3.66	3.70	3.74	3.78	3.82	3.86	3.90	3.94	3.97	4.01	4.04	4.08	4.12	4.15	4.18	4.22	4.26	4.29	4.32	4.35	4.38	4.42	4.45
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10	21.49	21.87	22.25	22.63

Q = 12 cfs Diameter of pipe = 24 inches V_p = 3.82 fps hv_p = 0.23 foot Inlet R = 3.25 feet

F	2.20	2.50	2.80	3.10	3.40	3.71	4.01	4.33	4.64	4.96	5.27	5.58	5.91	6.22	6.54	6.85	7.17	7.49	7.81	8.15	8.47	8.80	9.12
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.03	0.03	0.03	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	0.66	0.72	0.72	0.72	0.94	0.94	0.94	0.94	1.21	1.21	1.21	1.21	1.47	1.47	1.47	1.47	1.74	1.74	1.74	1.74	1.74	2.01	2.01
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	6.00	6.50	6.50	6.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50	9.50
L_4	5.00	5.50	5.50	5.50	3.50	3.50	3.50	3.50	4.50	4.50	4.50	4.50	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	6.50	7.50	7.50
S_3	7°-30'			7°-30'	15°																		15°
Submergence	0.50	0.53	0.61	0.69	0.53	0.60	0.68	0.74	0.55	0.61	0.68	0.75	0.55	0.62	0.68	0.75	0.55	0.61	0.67	0.72	0.78	0.56	0.62
d_2	2.71	2.79	2.86	2.94	3.02	3.08	3.15	3.21	3.28	3.34	3.40	3.46	3.52	3.58	3.64	3.70	3.76	3.82	3.87	3.92	3.97	4.02	4.07
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07	13.45	13.83
F	9.46	9.78	10.11	10.43	10.77	11.10	11.44	11.78	12.12	12.45	12.77	13.11	13.45	13.79	14.13	14.46	14.80	15.13	15.47	15.81	16.19	16.49	16.82
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	2.01	2.01	2.01	2.28	2.28	2.28	2.28	2.28	2.28	2.49	2.49	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90	2.90	2.90	3.11	3.11
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	9.50	9.50	9.50	12.50	12.50	12.50	12.50	12.50	12.50	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.50	13.50
L_4	7.50	7.50	7.50	5.50	5.50	5.50	5.50	5.50	5.50	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50
S_3	15°		15°	22°-30'																			22°-30'
Submergence	0.67	0.73	0.78	0.49	0.54	0.59	0.63	0.67	0.72	0.57	0.63	0.67	0.72	0.76	0.80	0.44	0.49	0.54	0.58	0.63	0.67	0.50	0.55
d_2	4.12	4.17	4.22	4.27	4.32	4.36	4.40	4.44	4.48	4.53	4.58	4.62	4.66	4.70	4.74	4.78	4.83	4.87	4.91	4.95	4.99	5.03	5.07
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10	21.49	21.87	22.25	22.63

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – diameter of pipe – hv

Table 2-4.–*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*–Continued. 103-D-1270-7

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

Q = 13 cfs Diameter of pipe = 24 inches V_p = 4.14 fps hv_p = 0.27 foot Inlet R = 3.25 feet

F	2.04	2.33	2.62	2.91	3.21	3.52	3.82	4.12	4.44	4.75	5.05	5.37	5.69	5.99	6.32	6.63	6.95	7.26	7.59	7.91	8.22	8.55	8.86
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06
h_4	0.66	0.66	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.61	1.61	1.61	1.61	1.88	1.88	1.88	1.88	2.14	2.14	2.14	2.14	2.28
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	7.00	7.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	12.50
L_4	5.00	5.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	5.50
S_3	7°-30'	7°-30'	15°																			15°	22°-30'
Submergence	0.67	0.77	0.41	0.50	0.59	0.66	0.47	0.55	0.62	0.69	0.50	0.56	0.63	0.71	0.49	0.56	0.63	0.70	0.49	0.56	0.63	0.68	0.53
d_2	2.82	2.91	2.99	3.07	3.15	3.22	3.29	3.36	3.43	3.49	3.56	3.62	3.68	3.75	3.80	3.86	3.93	3.99	4.04	4.10	4.16	4.21	4.27
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07	13.45	13.83

F	9.20	9.52	9.84	10.16	10.50	10.82	11.15	11.47	11.82	12.13	12.47	12.78	13.13	13.46	13.80	14.11	14.46	14.80	15.12	15.47	15.80	16.14	16.46
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	2.28	2.28	2.28	2.28	2.69	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.31	3.31	3.31
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	12.50	12.50	12.50	12.50	12.50	12.50	12.50	12.50	12.50	13.00	13.00	13.00	13.00	13.00	13.00	13.50	13.50	13.50	13.50	13.50	14.00	14.00	14.00
L_4	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	8.00	8.00	8.00
S_3	22°-30'																						22°-30'
Submergence	0.58	0.64	0.70	0.76	0.40	0.46	0.51	0.57	0.61	0.48	0.52	0.59	0.63	0.68	0.72	0.58	0.62	0.66	0.72	0.76	0.61	0.65	0.71
d_2	4.32	4.37	4.43	4.48	4.53	4.58	4.63	4.68	4.72	4.78	4.82	4.88	4.92	4.96	5.00	5.06	5.10	5.14	5.19	5.23	5.27	5.31	5.36
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10	21.49	21.87	22.25	22.63

Q = 14 cfs Diameter of pipe = 24 inches V_p = 4.46 fps hv_p = 0.31 foot Inlet R = 3.25 feet

F	1.92	2.20	2.48	2.77	3.06	3.36	3.64	3.94	4.25	4.55	4.84	5.15	5.46	5.77	6.08	6.38	6.70	7.01	7.33	7.65	7.96	8.28	8.59
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
h_4	0.94	0.94	1.07	1.07	1.07	1.34	1.34	1.34	1.61	1.61	1.61	1.88	1.88	1.88	1.88	2.14	2.14	2.14	2.14	2.41	2.41	2.41	2.41
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	8.50	8.50	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
L_4	3.50	3.50	4.00	4.00	4.00	5.00	5.00	5.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00	9.00
S_3	15°																						15°
Submergence	0.46	0.57	0.55	0.64	0.74	0.55	0.65	0.73	0.54	0.62	0.71	0.51	0.59	0.66	0.73	0.55	0.62	0.69	0.75	0.55	0.62	0.68	0.75
d_2	2.89	2.99	3.08	3.16	3.25	3.33	3.42	3.49	3.56	3.64	3.72	3.78	3.85	3.92	3.98	4.05	4.12	4.18	4.24	4.30	4.36	4.42	4.48
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07	13.45	13.83

F	8.92	9.23	9.55	9.87	10.20	10.52	10.83	11.17	11.52	11.82	12.14	12.46	12.79	13.12	13.45	13.78	14.11	14.44	14.76	15.11	15.43	15.76	16.09
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.52	3.52	3.52	3.52	3.52	3.73	3.73	3.73	3.73
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	12.50	12.50	12.50	12.50	13.00	13.00	13.00	13.00	13.00	13.50	13.50	13.50	13.50	13.50	13.50	13.50	13.50	13.50	13.50	14.00	14.00	14.00	14.00
L_4	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	8.50	8.50	8.50	8.50	8.50	9.00	9.00	9.00	9.00
S_3	22°-30'																						22°-30'
Submergence	0.45	0.52	0.58	0.64	0.50	0.56	0.63	0.67	0.74	0.58	0.64	0.70	0.76	0.81	0.45	0.50	0.56	0.61	0.67	0.50	0.56	0.61	0.66
d_2	4.54	4.60	4.65	4.71	4.76	4.82	4.88	4.92	4.98	5.03	5.08	5.14	5.19	5.24	5.28	5.33	5.38	5.43	5.48	5.52	5.57	5.62	5.66
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10	21.49	21.87	22.25	22.63

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ – diameter of pipe – hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30)*.—Continued. 103-D-1270-8

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 16 cfs Diameter of pipe = 24 inches V_p = 5.09 fps hv_p = 0.40 foot Inlet R = 3.25 feet

F	1.64	1.91	2.18	2.45	2.72	3.01	3.28	3.58	3.86	4.15	4.43	4.73	5.03	5.33	5.62	5.92	6.22	6.54	6.84	7.14	7.46	7.77	8.07
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	1.21	1.21	1.47	1.47	1.47	1.74	1.74	1.74	2.01	2.01	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.69	2.69	2.69	2.69	3.11	3.11
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.50	12.50	12.50	12.50	12.50	12.50
L_4	4.50	4.50	5.50	5.50	5.50	6.50	6.50	6.50	7.50	7.50	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.50	6.50	6.50	6.50	7.50	7.50
S_3	15°									15°	22°-30'												22°-30'
Submergence	0.47	0.59	0.44	0.55	0.67	0.49	0.60	0.68	0.52	0.61	0.58	0.66	0.75	0.83	0.50	0.58	0.67	0.53	0.61	0.70	0.76	0.41	0.49
d_2	3.06	3.17	3.27	3.37	3.48	3.56	3.66	3.74	3.84	3.92	4.01	4.08	4.16	4.24	4.32	4.39	4.47	4.53	4.60	4.68	4.74	4.80	4.87
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.41	5.80	6.18	6.56	6.95	7.33	7.71	8.09	8.48	8.86	9.24	9.62	10.01	10.39	10.77	11.15	11.54	11.92	12.30	12.69	13.07	13.45	13.83
F	8.40	8.70	9.01	9.33	9.65	9.97	10.28	10.58	10.90	11.21	11.53	11.85	12.17	12.50	12.81	13.13	13.45	13.72	14.10	14.42	14.75	15.07	15.39
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	3.11	3.11	3.31	3.31	3.31	3.31	3.52	3.52	3.52	3.52	3.93	3.93	3.93	3.93	4.14	4.14	4.14	4.14	4.14	4.35	4.35	4.35	4.35
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	12.50	12.50	13.00	13.00	13.00	13.00	13.50	13.50	13.50	13.50	13.50	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50
L_4	7.50	7.50	8.00	8.00	8.00	8.00	8.50	8.50	8.50	8.50	9.50	9.50	9.50	9.50	10.00	10.00	10.00	10.00	10.00	10.50	10.50	10.50	10.50
S_3	22°-30'																						22°-30'
Submergence	0.55	0.63	0.51	0.57	0.64	0.70	0.56	0.64	0.71	0.78	0.43	0.49	0.56	0.61	0.47	0.53	0.60	0.67	0.71	0.57	0.62	0.68	0.74
d_2	4.93	5.00	5.06	5.12	5.18	5.24	5.30	5.37	5.44	5.50	5.55	5.61	5.67	5.72	5.78	5.84	5.90	5.96	6.00	6.06	6.11	6.16	6.22
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.22	14.60	14.98	15.36	15.75	16.13	16.51	16.89	17.28	17.66	18.04	18.42	18.81	19.19	19.57	19.95	20.34	20.72	21.10	21.49	21.87	22.25	22.63

Q = 17 cfs Diameter of pipe = 30 inches V_p = 3.46 fps hv_p = 0.19 foot Inlet R = 3.75 feet

F	2.51	2.83	3.15	3.46	3.77	4.08	4.39	4.71	5.03	5.35	5.68	6.00	6.33	6.65	6.98	7.30	7.63	7.95	8.28	8.61	8.94	9.27	9.61
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07
h_4	0.39	0.39	0.46	0.46	0.46	0.46	0.80	0.80	0.80	0.80	0.94	0.94	0.94	0.94	0.94	1.21	1.21	1.21	1.21	1.21	1.45	1.45	1.45
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	9.00	9.00	9.50	9.50	9.50	9.50	11.00	11.00	11.00	11.00	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	13.50	13.50	13.50
L_4	3.00	3.00	3.50	3.50	3.50	3.50	3.00	3.00	3.00	3.00	3.50	3.50	3.50	3.50	3.50	4.50	4.50	4.50	4.50	4.50	3.50	3.50	3.50
S_3	7°-30'					7°-30'	15°													15°	22°-30'		22°-30'
Submergence	0.48	0.55	0.54	0.62	0.69	0.75	0.43	0.50	0.56	0.62	0.53	0.59	0.65	0.71	0.76	0.56	0.61	0.67	0.72	0.78	0.48	0.53	0.57
d_2	2.92	2.98	3.04	3.11	3.17	3.23	3.30	3.36	3.42	3.47	3.52	3.57	3.63	3.68	3.73	3.79	3.84	3.89	3.94	3.99	4.04	4.08	4.12
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57	13.95	14.33
F	9.95	10.28	10.62	10.95	11.29	11.62	11.96	12.29	12.63	12.97	13.31	13.65	13.99	14.33	14.68	15.02	15.37	15.72	16.07	16.42	16.76	17.10	17.46
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	1.45	1.45	1.45	1.66	1.66	1.66	1.66	1.66	1.86	1.86	1.86	1.86	1.86	1.86	1.86	2.28	2.28	2.28	2.28	2.28	2.28	2.28	2.28
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50
L_4	3.50	3.50	3.50	4.00	4.00	4.00	4.00	4.00	4.50	4.50	4.50	4.50	4.50	4.50	4.50	5.50	5.50	5.50	5.50	5.50	5.50	5.50	5.50
S_3	22°-30'																						22°-30'
Submergence	0.62	0.67	0.72	0.55	0.60	0.65	0.69	0.75	0.59	0.64	0.68	0.71	0.76	0.80	0.83	0.45	0.48	0.52	0.56	0.59	0.63	0.67	0.69
d_2	4.16	4.21	4.25	4.29	4.34	4.38	4.42	4.47	4.51	4.55	4.59	4.62	4.66	4.70	4.73	4.76	4.79	4.83	4.86	4.89	4.93	4.96	4.98
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60	21.99	22.37	22.75	23.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4 -$ diameter of pipe $- hv$

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-9

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet — Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 19 cfs — Diameter of pipe = 30 inches — V_p = 3.87 fps — hv_p = 0.23 foot — Inlet R = 3.75 feet

F	2.30	2.62	2.92	3.22	3.53	3.84	4.15	4.46	4.77	5.08	5.40	5.71	6.03	6.35	6.67	6.98	7.30	7.63	7.96	8.29	8.61	8.93	9.26
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07
h_4	0.59	0.59	0.59	0.72	0.72	0.72	0.72	1.07	1.07	1.07	1.07	1.21	1.21	1.21	1.21	1.47	1.47	1.47	1.47	1.47	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	8.50	8.50	8.50	8.50	8.50	8.50	8.50	11.00	11.00	11.00	11.00	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	11.50	14.00	14.00	14.00
L_4	4.50	4.50	4.50	5.50	5.50	5.50	5.50	4.00	4.00	4.00	4.00	4.50	4.50	4.50	4.50	5.50	5.50	5.50	5.50	5.50	4.00	4.00	4.00
S_3	7°-30'						7°-30'	15°												15°	22°-30'		22°-30'
Submergence	0.48	0.55	0.63	0.58	0.66	0.73	0.80	0.48	0.55	0.62	0.68	0.61	0.68	0.74	0.80	0.61	0.68	0.74	0.79	0.85	0.60	0.67	0.71
d_2	3.07	3.14	3.21	3.28	3.35	3.42	3.48	3.55	3.62	3.68	3.74	3.80	3.86	3.92	3.97	4.04	4.10	4.15	4.20	4.25	4.30	4.36	4.40
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57	13.95	14.33
F	9.59	9.92	10.25	10.58	10.92	11.25	11.59	11.92	12.26	12.60	12.92	13.25	13.59	13.92	14.26	14.60	14.93	15.26	15.60	15.93	16.27	16.60	16.95
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.28	2.28	2.28	2.28	2.28	2.28	2.49	2.49	2.49	2.49	2.49	2.49	2.90	2.90	2.90	2.90
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00
L_4	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.50	5.50	5.50	5.50	5.50	5.50	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00
S_3	22°-30'																						22°-30'
Submergence	0.77	0.82	0.47	0.52	0.56	0.61	0.65	0.50	0.54	0.59	0.64	0.70	0.74	0.58	0.63	0.67	0.73	0.78	0.83	0.47	0.51	0.56	0.60
d_2	4.45	4.50	4.55	4.60	4.64	4.68	4.72	4.77	4.81	4.85	4.90	4.95	4.99	5.03	5.07	5.11	5.16	5.21	5.25	5.30	5.34	5.38	5.42
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60	21.99	22.37	22.75	23.13

Q = 21 cfs — Diameter of pipe = 30 inches — V_p = 4.28 fps — hv_p = 0.28 foot — Inlet R = 3.75 feet

F	2.13	2.42	2.71	3.00	3.30	3.59	3.89	4.19	4.49	4.79	5.10	5.42	5.74	6.06	6.38	6.70	7.02	7.33	7.64	7.96	8.28	8.60	8.92
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.04	0.04	0.04	0.04	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
h_4	0.72	0.72	0.72	0.72	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.61	1.61	1.61	1.61	1.61	1.88	1.88	1.88	1.88	2.14	2.14	2.14
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	7.50	8.50	8.50	8.50	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00
L_4	5.50	5.50	5.50	5.50	4.00	4.00	4.00	4.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00
S_3	7°-30'			7°-30'	15°																		15°
Submergence	0.52	0.62	0.72	0.80	0.49	0.58	0.66	0.75	0.55	0.64	0.71	0.50	0.57	0.63	0.69	0.76	0.55	0.62	0.69	0.76	0.57	0.62	0.69
d_2	3.18	3.27	3.36	3.44	3.52	3.60	3.67	3.75	3.82	3.90	3.96	4.02	4.08	4.14	4.19	4.25	4.31	4.37	4.44	4.50	4.56	4.61	4.67
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57	13.95	14.33
F	9.24	9.56	9.89	10.21	10.54	10.86	11.19	11.52	11.85	12.17	12.50	12.83	13.16	13.48	13.81	14.14	14.48	14.81	15.15	15.49	15.82	16.15	16.48
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	2.14	2.28	2.28	2.28	2.28	2.28	2.69	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	2.90	3.11	3.11	3.11	3.11	3.11	3.11	3.52
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	11.00	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.50	15.50	15.50	15.50	15.50	15.50	15.50
L_4	8.00	5.50	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	7.00	7.50	7.50	7.50	7.50	7.50	7.50	8.50
S_3	15°	22°-30'																					22°-30'
Submergence	0.75	0.56	0.61	0.67	0.73	0.79	0.43	0.48	0.54	0.60	0.65	0.51	0.56	0.62	0.67	0.72	0.55	0.61	0.65	0.70	0.75	0.81	0.45
d_2	4.73	4.78	4.83	4.88	4.94	4.99	5.04	5.08	5.14	5.19	5.24	5.29	5.34	5.39	5.44	5.48	5.52	5.57	5.61	5.65	5.70	5.75	5.80
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60	21.99	22.37	22.75	23.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ — $d_1 + hv_1 = R + h_2 + 0.06$ — Submergence $= 1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.–*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*–Continued. 103-D-1270-10

PIPE DROP WITH MITER BENDS

Concrete Outlet Transition

Check and Pipe Inlet   Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 23 cfs   Diameter of pipe = 30 inches V_p = 4.68 fps hv_p = 0.34 foot   Inlet R = 3.75 feet

F	1.94	2.22	2.49	2.77	3.06	3.35	3.65	3.95	4.25	4.54	4.84	5.14	5.44	5.75	6.06	6.37	6.69	7.01	7.32	7.63	7.94	8.25	8.57
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	0.94	0.94	0.94	1.21	1.21	1.34	1.34	1.34	1.34	1.61	1.61	1.61	1.88	1.88	1.88	1.88	2.07	2.07	2.07	2.07	2.49	2.49	2.49
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	10.50	10.50	10.50	10.50	10.50	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00
L_4	3.50	3.50	3.50	4.50	4.50	5.00	5.00	5.00	5.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00
S_3	15°															15°	22°-30'						22°-30'
Submergence	0.43	0.54	0.65	0.49	0.58	0.55	0.62	0.71	0.80	0.62	0.71	0.79	0.60	0.67	0.73	0.81	0.58	0.64	0.72	0.78	0.43	0.51	0.57
d_2	3.30	3.40	3.50	3.60	3.68	3.76	3.83	3.91	3.99	4.07	4.15	4.23	4.30	4.36	4.42	4.49	4.55	4.61	4.68	4.74	4.80	4.87	4.93
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57	13.95	14.33
F	8.89	9.21	9.52	9.84	10.16	10.47	10.79	11.10	11.42	11.75	12.08	12.42	12.76	13.08	13.40	13.72	14.05	14.38	14.71	15.03	15.36	15.70	16.03
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	2.49	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.52	3.52	3.52	3.52	3.52	3,52	3.73	3.73	3.73
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.00	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.50	15.50	15.50	15.50	15.50	15.50	16.00	16.00	16.00
L_4	6.00	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.50	8.50	8.50	8.50	8.50	8.50	9.00	9.00	9.00
S_3	22°-30'																						22°-30'
Submergence	0.64	0.51	0.57	0.64	0.70	0.57	0.64	0.71	0.77	0.40	0.46	0.50	0.56	0.61	0.47	0.52	0.58	0.62	0.68	0.73	0.58	0.63	0.69
d_2	4.99	5.05	5.11	5.17	5.23	5.29	5.35	5.42	5.47	5.51	5.56	5.60	5.65	5.70	5.76	5.81	5.86	5.90	5.95	6.00	6.05	6.10	6.15
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60	21.99	22.37	22.75	23.13

Q = 25 cfs   Diameter of pipe = 30 inches V_p = 5.09 fps hv_p = 0.40 foot   Inlet R = 3.75 feet

F	1.75	2.03	2.30	2.57	2.86	3.14	3.42	3.70	4.00	4.29	4.58	4.88	5.18	5.49	5.79	6.08	6.38	6.68	6.98	7.30	7.61	7.92	8.23
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07
h_4	1.07	1.07	1.07	1.07	1.45	1.45	1.45	1.66	1.66	1.66	1.86	1.86	1.86	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.69	2.69	2.69
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	10.00	11.00	11.00	11.00	12.50	12.50	12.50	13.00	13.00	13.00	13.50	13.50	13.50	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.50	14.50	14.50
L_4	4.00	4.00	4.00	4.00	3.50	3.50	3.50	4.00	4.00	4.00	4.50	4.50	4.50	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.50	6.50	6.50
S_3	15°			15°	22°-30'																		22°-30'
Submergence	0.49	0.61	0.72	0.83	0.43	0.53	0.64	0.53	0.62	0.72	0.60	0.68	0.77	0.62	0.71	0.81	0.47	0.55	0.63	0.70	0.57	0.64	0.71
d_2	3.42	3.52	3.62	3.72	3.81	3.90	4.00	4.08	4.16	4.25	4.33	4.40	4.48	4.54	4.62	4.71	4.78	4.85	4.93	4.99	5.05	5.12	5.18
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	5.91	6.30	6.68	7.06	7.45	7.83	8.21	8.59	8.98	9.36	9.74	10.12	10.51	10.89	11.27	11.65	12.04	12.42	12.80	13.19	13.57	13.95	14.33
F	8.55	8.87	9.18	9.50	9.82	10.14	10.44	10.76	11.09	11.40	11.71	12.02	12.34	12.66	12.97	13.28	13.62	13.95	14.27	14.60	14.92	15.24	15.58
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	2.69	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.52	3.52	3.52	3.52	3.73	3.73	3.73	3.73	3.73	4.14	4.14	4.14	4.14	4.14
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.50	15.50	15.50	15.50	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00
L_4	6.50	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.50	8.50	8.50	8.50	9.00	9.00	9.00	9.00	9.00	10.00	10.00	10.00	10.00	10.00
S_3	22°-30'																						22°-30'
Submergence	0.77	0.64	0.71	0.77	0.84	0.49	0.56	0.63	0.69	0.55	0.61	0.69	0.76	0.61	0.68	0.75	0.80	0.86	0.50	0.56	0.62	0.68	0.72
d_2	5.24	5.30	5.36	5.42	5.48	5.54	5.60	5.66	5.72	5.78	5.84	5.91	5.97	6.03	6.09	6.15	6.20	6.25	6.30	6.35	6.41	6.46	6.50
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	14.72	15.10	15.48	15.86	16.25	16.63	17.01	17.39	17.78	18.16	18.54	18.92	19.31	19.69	20.07	20.45	20.84	21.22	21.60	21.99	22.37	22.75	23.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ $\text{Submergence} = 1.1d_2 + hv_p + h_3 - h_4 - \text{diameter of pipe} - hv$

Table 2-4.–*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*–Continued. 103-D-1270-11

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 26 cfs Diameter of pipe = 36 inches V_p = 3.67 fps hv_p = 0.21 foot Inlet R = 4.25 feet

F	2.53	2.83	3.14	3.44	3.78	4.08	4.41	4.71	5.03	5.35	5.66	5.99	6.31	6.63	6.95	7.28	7.61	7.94	8.26	8.60	8.93	9.26	9.59
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09
h_4	0.40	0.40	0.40	0.40	0.40	0.40	0.53	0.80	0.80	0.80	0.80	0.80	0.80	1.07	1.07	1.07	1.07	1.24	1.24	1.24	1.24	1.24	1.24
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	11.00	11.00	11.00	12.00	12.00	12.00	12.00	13.00	13.50	13.50	14.00	14.00	14.50	15.00	15.00	15.50	16.00	16.00	16.00	16.00	16.50	16.50	17.00
L_4	3.00	3.00	3.00	3.00	3.00	3.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	3.00
S_3	7°-30'						7°-30'	15°									15°	22°-30'					22°-30'
Submergence	0.45	0.54	0.61	0.68	0.74	0.82	0.74	0.48	0.54	0.61	0.68	0.73	0.80	0.60	0.66	0.71	0.77	0.51	0.57	0.62	0.67	0.72	0.78
d_2	3.34	3.42	3.48	3.55	3.60	3.67	3.72	3.79	3.85	3.91	3.97	4.02	4.08	4.14	4.19	4.24	4.29	4.34	4.39	4.44	4.48	4.53	4.57
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07	14.45	14.83
F	9.92	10.25	10.58	10.91	11.25	11.58	11.91	12.25	12.59	12.93	13.25	13.59	13.93	14.28	14.62	14.95	15.30	15.65	15.98	16.34	16.68	17.01	17.35
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10
h_4	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.48	2.48	2.48	2.48	2.48
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.00	17.00	17.00	17.00	17.50	17.50	17.50	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.00	19.00
L_4	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00
S_3	22°-30'																						22°-30'
Submergence	0.42	0.47	0.52	0.57	0.62	0.66	0.72	0.76	0.81	0.44	0.50	0.54	0.58	0.62	0.66	0.71	0.75	0.79	0.43	0.46	0.50	0.55	0.59
d_2	4.63	4.67	4.72	4.76	4.81	4.85	4.90	4.94	4.98	5.02	5.07	5.11	5.15	5.18	5.22	5.26	5.30	5.33	5.37	5.40	5.44	5.48	5.52
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38:50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10	22.49	22.87	23.25	23.63

Q = 29 cfs Diameter of pipe = 36 inches V_p = 4.10 fps hv_p = 0.26 foot Inlet R = 4.25 feet

F	2.31	2.60	2.90	3.20	3.51	3.80	4.10	4.42	4.73	5.05	5.36	5.66	6.00	6.30	6.63	6.95	7.26	7.59	7.91	8.24	8.57	8.89	9.21
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.40	0.53	0.53	0.53	0.80	0.80	0.80	0.80	1.07	1.07	1.07	1.07	1.24	1.24	1.24	1.24	1.24	1.66	1.66	1.66	1.66	1.66	1.66
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	11.00	12.00	12.50	13.00	13.00	13.00	13.50	14.00	14.00	14.50	15.00	15.00	15.00	15.00	16.50	16.50	17.00	17.00	17.00	17.00	17.00	17.50	17.50
L_4	3.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00
S_3	7°-30'			7°-30'	15°							15°	22°-30'										22°-30'
Submergence	0.67	0.64	0.72	0.81	0.54	0.63	0.70	0.77	0.58	0.64	0.72	0.80	0.54	0.62	0.67	0.73	0.82	0.45	0.51	0.57	0.62	0.68	0.74
d_2	3.49	3.58	3.65	3.73	3.80	3.88	3.95	4.01	4.08	4.14	4.20	4.27	4.32	4.39	4.44	4.49	4.56	4.61	4.66	4.72	4.76	4.82	4.87
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07	14.45	14.83
F	9.55	9.87	10.19	10.51	10.85	11.17	11.49	11.82	12.16	12.54	12.82	13.14	13.49	13.81	14.15	14.49	14.83	15.17	15.50	15.84	16.17	16.52	16.85
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
h_4	1.66	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.48	2.48	2.48	2.48	2.48	2.48	2.48	2.48	2.90	2.90	2.90	2.90	2.90	2.90	2.90
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.50	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.50	19.50	19.50	20.00	20.00	20.00	20.00	20.00	20.00	20.50
L_4	4.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7:00	7.00
S_3	22°-30'																						22°-30'
Submergence	0.79	0.44	0.50	0.56	0.61	0.67	0.73	0.79	0.42	0.48	0.53	0.59	0.63	0.69	0.73	0.77	0.40	0.44	0.49	0.54	0.58	0.62	0.67
d_2	4.92	4.97	5.03	5.08	5.13	5.18	5.24	5.28	5.33	5.37	5.42	5.47	5.51	5.56	5.60	5.64	5.68	5.72	5.76	5.81	5.85	5.88	5.93
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10	22.49	22.87	23.25	23.63

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4 -$ diameter of pipe $- hv$

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-12

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

Q = 32 cfs   Diameter of pipe = 36 inches V_p = 4.52 fps hv_p = 0.32 foot   Inlet R = 4.25 feet

F	2.10	2.39	2.67	2.95	3.25	3.54	3.85	4.15	4.45	4.75	5.06	5.37	5.67	5.98	6.30	6.60	6.93	7.24	7.56	7.87	8.19	8.51	8.83
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.53	0.80	0.80	0.80	0.80	1.07	1.07	1.07	1.24	1.24	1.24	1.24	1.66	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	11.00	12.00	13.00	13.00	13.50	14.00	14.50	15.00	15.00	15.00	15.50	16.00	16.00	16.00	16.00	16.00	16.50	17.00	17.00	17.00	17.00	17.00	17.50
L_4	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00
S_3	7°-30'	15°						15°	22°-30'														22°-30'
Submergence	0.75	0.50	0.61	0.71	0.80	0.62	0.70	0.78	0.56	0.63	0.71	0.78	0.45	0.52	0.58	0.66	0.72	0.80	0.45	0.53	0.58	0.65	0.71
d_2	3.63	3.72	3.81	3.90	3.98	4.06	4.13	4.20	4.28	4.35	4.42	4.48	4.56	4.63	4.68	4.75	4.81	4.87	4.93	5.00	5.05	5.11	5.16
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07	14.45	14.83
F	9.15	9.47	9.80	10.12	10.45	10.76	11.08	11.47	11.74	12.07	12.39	12.72	13.05	13.38	13.72	14.04	14.38	14.70	15.03	15.36	15.70	16.02	16.36
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
h_4	2.07	2.48	2.48	2.48	2.48	2.48	2.48	2.48	2.90	2.90	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.31
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.50	18.00	18.00	18.00	18.00	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.00	19.50	19.50	19.50	20.00	20.00	20.00	20.00	20.50	20.50	20.50
L_4	5.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00
S_3	22°-30'																						22°-30'
Submergence	0.78	0.43	0.48	0.54	0.60	0.67	0.72	0.78	0.42	0.48	0.54	0.59	0.64	0.70	0.74	0.80	0.44	0.50	0.55	0.61	0.65	0.71	0.75
d_2	5.23	5.28	5.33	5.38	5.44	5.50	5.55	5.60	5.65	5.70	5.75	5.80	5.85	5.90	5.94	5.99	6.04	6.09	6.14	6.19	6.23	6.28	6.32
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10	22.49	22.87	23.25	23.63

Q = 34 cfs   Diameter of pipe = 36 inches V_p = 4.81 fps hv_p = 0.36 foot   Inlet R = 4.25 feet

F	1.96	2.23	2.51	2.80	3.13	3.37	3.66	3.96	4.27	4.57	4.86	5.16	5.47	5.77	6.09	6.38	6.70	7.00	7.32	7.63	7.95	8.26	8.57
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.80	0.80	0.80	1.07	1.07	1.07	1.24	1.24	1.24	1.24	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.07	2.48	2.48	2.48
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	12.00	13.00	13.50	13.50	14.00	14.50	15.00	15.00	15.50	16.00	16.00	16.00	16.50	16.50	17.00	17.00	17.00	17.00	17.50	17.50	18.00	18.00	18.00
L_4	3.00	3.00	3.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00
S_3	15°					15°	22°-30'																22°-30'
Submergence	0.54	0.67	0.77	0.59	0.69	0.79	0.58	0.66	0.74	0.82	0.49	0.57	0.65	0.73	0.80	0.48	0.55	0.62	0.69	0.77	0.42	0.49	0.56
d_2	3.72	3.83	3.92	4.00	4.09	4.18	4.26	4.34	4.41	4.48	4.56	4.64	4.71	4.78	4.84	4.92	4.98	5.05	5.11	5.18	5.24	5.30	5.36
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07	14.45	14.83
F	8.89	9.21	9.53	9.84	10.17	10.49	10.81	11.13	11.46	11.78	12.09	12.42	12.75	13.08	13.41	13.73	14.05	14.37	14.70	15.03	15.37	15.70	16.01
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.11	0.11	0.11	0.11	0.11	0.11	0.11
h_4	2.48	2.48	2.48	2.48	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73	3.73	3.73	3.73	3.73
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	18.00	18.50	18.50	19.00	19.00	19.00	19.00	19.50	19.50	19.50	20.00	20.00	20.00	20.00	20.50	20.50	21.00	21.00	21.00	21.00	21.00	21.00	21.50
L_4	6.00	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00	9.00	9.00	9.00
S_3	22°-30'																						22°-30'
Submergence	0.63	0.69	0.75	0.83	0.46	0.53	0.59	0.65	0.71	0.77	0.43	0.48	0.54	0.59	0.64	0.70	0.78	0.42	0.47	0.52	0.57	0.62	0.69
d_2	5.43	5.48	5.54	5.60	5.65	5.71	5.76	5.82	5.87	5.93	5.99	6.04	6.09	6.14	6.18	6.24	6.30	6.35	6.40	6.45	6.49	6.54	6.60
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10	22.49	22.87	23.25	23.63

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-13

PIPE DROP WITH MITER BENDS

Concrete Outlet Transition

Check and Pipe Inlet — Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 36 cfs — Diameter of pipe = 36 inches — V_p = 5.09 fps — hv_p = 0.40 foot — Inlet R = 4.25 feet

F	1.82	2.09	2.36	2.64	2.92	3.20	3.50	3.78	4.09	4.38	4.67	4.97	5.27	5.58	5.87	6.17	6.49	6.79	7.09	7.42	7.73	8.03	8.34
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.80	1.07	1.07	1.07	1.24	1.24	1.24	1.24	1.66	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.07	2.48	2.48	2.48	2.48	2.48	2.48
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	13.00	14.00	14.00	14.50	15.00	15.00	15.50	16.00	16.00	16.00	16.00	16.50	16.50	17.00	17.00	17.50	17.50	18.00	18.00	18.00	18.00	18.50	18.50
L_4	3.00	4.00	4.00	4.00	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.00
S_3	15°			15°	22°-30'																		22°-30'
Submergence	0.70	0.54	0.65	0.75	0.56	0.66	0.74	0.84	0.50	0.59	0.68	0.76	0.84	0.52	0.61	0.69	0.76	0.43	0.51	0.57	0.64	0.72	0.79
d_2	3.81	3.92	4.02	4.11	4.21	4.30	4.37	4.46	4.54	4.62	4.70	4.77	4.85	4.92	5.00	5.07	5.14	5.21	5.28	5.34	5.40	5.47	5.54
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.41	6.80	7.18	7.56	7.95	8.33	8.71	9.09	9.48	9.86	10.24	10.62	11.01	11.39	11.77	12.15	12.54	12.92	13.30	13.69	14.07	14.45	14.83
F	8.66	8.97	9.29	9.60	9.93	10.23	10.55	10.87	11.19	11.51	11.82	12.13	12.47	12.78	13.11	13.42	13.76	14.08	14.41	14.73	15.06	15.38	15.71
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11
h_4	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.31	3.73	3.73	3.73	3.73	3.73	3.73	3.73	4.14	4.14	4.14	4.14
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	19.00	19.00	19.00	19.50	19.50	20.00	20.00	20.00	20.00	20.00	20.50	20.50	21.00	21.00	21.00	21.00	21.50	21.50	21.50	22.00	22.00	22.00	22.00
L_4	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	10.00	10.00	10.00	10.00
S_3	22°-30'																						22°-30'
Submergence	0.45	0.52	0.58	0.65	0.71	0.79	0.44	0.50	0.57	0.63	0.70	0.77	0.41	0.48	0.53	0.60	0.65	0.71	0.76	0.42	0.47	0.53	0.58
d_2	5.60	5.66	5.72	5.78	5.84	5.91	5.96	6.02	6.08	6.14	6.20	6.26	6.31	6.37	6.42	6.48	6.53	6.58	6.63	6.69	6.74	6.79	6.84
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.22	15.60	15.98	16.36	16.75	17.13	17.51	17.89	18.28	18.66	19.04	19.42	19.81	20.19	20.57	20.95	21.34	21.72	22.10	22.49	22.87	23.25	23.63

Q = 38 cfs — Diameter of pipe = 42 inches — V_p = 3.95 fps — hv_p = 0.24 foot — Inlet R = 4.75 feet

F	2.53	2.84	3.14	3.44	3.76	4.06	4.36	4.67	4.97	5.31	5.63	5.94	6.26	6.58	6.90	7.22	7.54	7.87	8.19	8.53	8.85	9.18	9.50
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.39	0.39	0.39	0.39	0.53	0.53	0.53	0.53	0.94	0.94	0.94	0.94	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.34	1.61	1.61
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00
L_4	3.00	3.00	3.00	3.00	4.00	4.00	4.00	4.00	3.50	3.50	3.50	3.50	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	5.00	6.00	6.00
S_3	7°-30'							7°-30'	15°														15°
Submergence	0.47	0.55	0.63	0.71	0.65	0.72	0.80	0.87	0.43	0.50	0.56	0.63	0.58	0.64	0.70	0.76	0.56	0.61	0.67	0.72	0.78	0.56	0.62
d_2	3.76	3.84	3.91	3.98	4.05	4.12	4.19	4.25	4.31	4.37	4.43	4.49	4.55	4.61	4.66	4.72	4.78	4.83	4.88	4.93	4.98	5.03	5.08
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.91	7.30	7.68	8.06	8.45	8.83	9.21	9.59	9.98	10.36	10.74	11.12	11.51	11.89	12.27	12.65	13.04	13.42	13.80	14.19	14.57	14.95	15.33
F	9.84	10.16	10.50	10.82	11.16	11.48	11.82	12.16	12.49	12.83	13.16	13.49	13.83	14.16	14.49	14.83	15.18	15.51	15.85	16.19	16.53	16.87	17.20
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	1.61	1.61	1.61	1.88	1.88	1.88	1.88	1.88	1.88	2.07	2.07	2.07	2.07	2.07	2.28	2.28	2.28	2.28	2.28	2.28	2.28	2.69	2.69
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50
L_4	6.00	6.00	6.00	7.00	7.00	7.00	7.00	7.00	7.00	5.00	5.00	5.00	5.00	5.00	5.50	5.50	5.50	5.50	5.50	5.50	5.50	6.50	6.50
S_3	15°								15°	22°-30'													22°-30'
Submergence	0.67	0.73	0.77	0.56	0.61	0.67	0.71	0.76	0.81	0.50	0.55	0.60	0.65	0.70	0.54	0.58	0.62	0.67	0.71	0.76	0.80	0.43	0.48
d_2	5.13	5.18	5.22	5.27	5.32	5.37	5.41	5.45	5.50	5.54	5.58	5.63	5.67	5.72	5.76	5.80	5.84	5.88	5.92	5.96	6.00	6.04	6.08
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.72	16.10	16.48	16.86	17.25	17.63	18.01	18.39	18.78	19.16	19.54	19.92	20.31	20.69	21.07	21.45	21.84	22.22	22.60	22.99	23.37	23.75	24.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4 -$ diameter of pipe $- hv$

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-14

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet

All values are in feet unless otherwise noted

Q = 40 cfs Diameter of pipe = 42 inches V_p = 4.16 fps hv_p = 0.27 foot Inlet R = 4.75 feet

F	2.41	2.71	3.00	3.30	3.62	3.92	4.22	4.52	4.84	5.15	5.45	5.77	6.09	6.41	6.73	7.04	7.37	7.69	8.01	8.33	8.66	8.98	9.31
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
h_4	0.53	0.53	0.66	0.66	0.66	0.66	0.94	0.94	0.94	0.94	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.61	1.61	1.61	1.61	1.61
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	12.00	12.00	12.00	12.00	12.00	12.00	14.50	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00
L_4	4.00	4.00	5.00	5.00	5.00	5.00	3.50	3.50	3.50	3.50	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00
S_3	7°-30'					7°-30'	15°																15°
Submergence	0.45	0.53	0.49	0.57	0.65	0.72	0.44	0.52	0.59	0.66	0.62	0.68	0.75	0.81	0.60	0.67	0.73	0.79	0.58	0.65	0.70	0.76	0.81
d_2	3.85	3.93	4.01	4.08	4.15	4.22	4.29	4.36	4.43	4.49	4.56	4.62	4.68	4.74	4.79	4.85	4.91	4.96	5.02	5.08	5.13	5.18	5.23
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.91	7.30	7.68	8.06	8.45	8.83	9.21	9.59	9.98	10.36	10.74	11.12	11.51	11.89	12.27	12.65	13.04	13.42	13.80	14.19	14.57	14.95	15.33

F	9.64	9.97	10.29	10.62	10.95	11.29	11.61	11.94	12.27	12.61	12.93	13.27	13.62	13.94	14.28	14.60	14.95	15.27	15.61	15.96	16.29	16.62	16.95
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10
h_4	1.86	1.86	1.86	1.86	1.86	2.07	2.07	2.07	2.07	2.07	2.28	2.28	2.28	2.28	2.28	2.28	2.69	2.69	2.69	2.69	2.69	2.69	2.90
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.50	17.50	17.50	17.50	17.50	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50	19.00
L_4	4.50	4.50	4.50	4.50	4.50	5.00	5.00	5.00	5.00	5.00	5.50	5.50	5.50	5.50	5.50	5.50	6.50	6.50	6.50	6.50	6.50	6.50	7.00
S_3	22°-30'																						22°-30'
Submergence	0.46	0.51	0.57	0.62	0.68	0.51	0.57	0.62	0.68	0.72	0.57	0.61	0.65	0.71	0.75	0.81	0.44	0.50	0.54	0.59	0.63	0.68	0.53
d_2	5.28	5.33	5.38	5.43	5.48	5.52	5.57	5.62	5.67	5.71	5.76	5.80	5.84	5.89	5.93	5.98	6.02	6.07	6.11	6.15	6.19	6.24	6.28
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.72	16.10	16.48	16.86	17.25	17.63	18.01	18.39	18.78	19.16	19.54	19.92	20.31	20.69	21.07	21.45	21.84	22.22	22.60	22.99	23.37	23.75	24.13

Q = 43 cfs Diameter of pipe = 42 inches V_p = 4.47 fps hv_p = 0.31 foot Inlet R = 4.75 feet

F	2.24	2.55	2.84	3.13	3.43	3.72	4.03	4.32	4.64	4.95	5.25	5.55	5.88	6.18	6.49	6.81	7.13	7.43	7.75	8.07	8.40	8.71	9.04
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.66	0.66	0.66	0.66	0.94	0.94	0.94	1.07	1.07	1.07	1.07	1.34	1.34	1.34	1.34	1.66	1.66	1.66	1.66	1.86	1.86	1.86	1.86
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	11.00	12.00	12.00	12.00	14.50	14.50	14.50	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	17.00	17.00	17.00	17.00	17.50	17.50	17.50	17.50
L_4	5.00	5.00	5.00	5.00	3.50	3.50	3.50	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.00	4.00	4.00	4.00	4.00	4.50	4.50	4.50	4.50
S_3	7°-30'			7°-30'	15°										15°	22°-30'							22°-30'
Submergence	0.48	0.56	0.65	0.74	0.47	0.56	0.63	0.60	0.67	0.74	0.82	0.63	0.69	0.77	0.84	0.42	0.49	0.57	0.63	0.50	0.55	0.62	0.67
d_2	3.96	4.04	4.12	4.20	4.28	4.36	4.43	4.51	4.57	4.64	4.71	4.78	4.84	4.91	4.97	5.03	5.09	5.16	5.22	5.28	5.33	5.39	5.44
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.91	7.30	7.68	8.06	8.45	8.83	9.21	9.59	9.98	10.36	10.74	11.12	11.51	11.89	12.27	12.65	13.04	13.42	13.80	14.19	14.57	14.95	15.33

F	9.36	9.68	10.01	10.32	10.65	10.97	11.30	11.62	11.96	12.28	12.61	12.93	13.27	13.60	13.93	14.25	14.60	14.92	15.26	15.60	15.93	16.26	16.58
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
h_4	1.86	2.07	2.07	2.07	2.07	2.49	2.49	2.49	2.49	2.49	2.69	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	2.90	2.90	3.31	3.31
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.50	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00
L_4	4.50	5.00	5.00	5.00	5.00	6.00	6.00	6.00	6.00	6.00	6.50	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	7.00	7.00	8.00	8.00
S_3	22°-30'																						22°-30'
Submergence	0.74	0.59	0.64	0.71	0.77	0.41	0.46	0.52	0.57	0.63	0.48	0.54	0.59	0.64	0.69	0.55	0.59	0.65	0.69	0.74	0.79	0.43	0.49
d_2	5.50	5.55	5.60	5.66	5.72	5.77	5.82	5.87	5.92	5.97	6.02	6.07	6.12	6.16	6.21	6.26	6.30	6.35	6.39	6.44	6.48	6.53	6.58
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.72	16.10	16.48	16.86	17.25	17.63	18.01	18.39	18.78	19.16	19.54	19.92	20.31	20.69	21.07	21.45	21.84	22.22	22.60	22.99	23.37	23.75	24.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence $= 1.1d_2 + hv_p + h_3 - h_4 -$ diameter of pipe $- hv$

Table 2-4.—*Type 1 pipe drop with concrete outlet transition (to be used with fig. 2-30).*—Continued. 103-D-1270-15

PIPE DROP WITH MITER BENDS
Concrete Outlet Transition

Check and Pipe Inlet Control and Pipe Inlet
All values are in feet unless otherwise noted

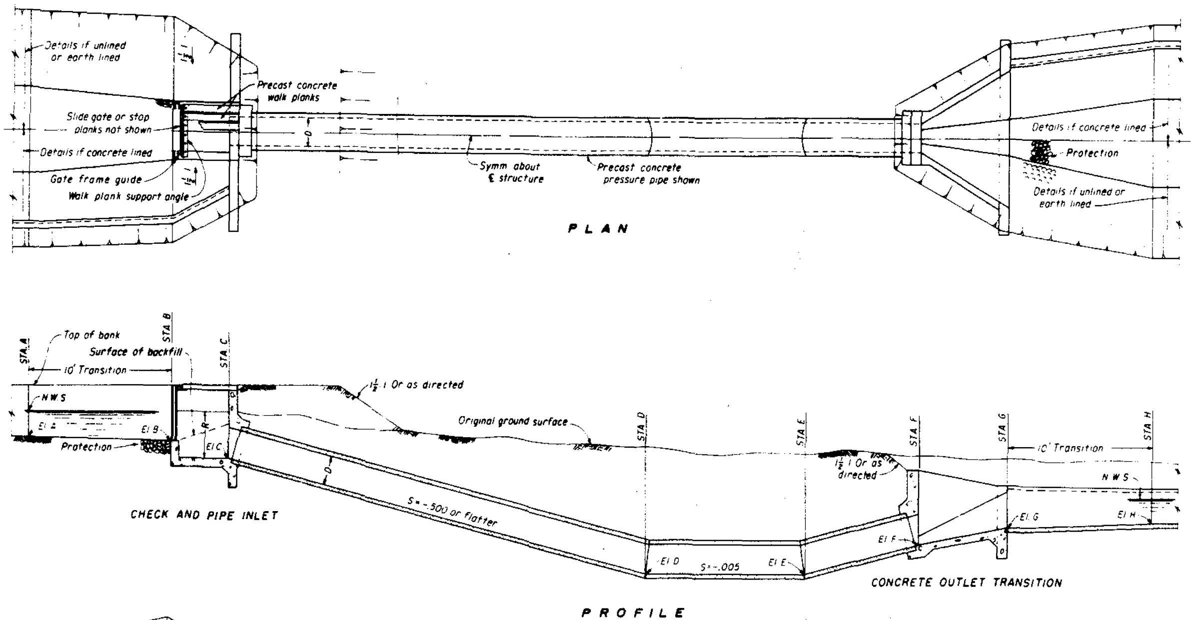
Q = 46 cfs Diameter of pipe = 42 inches V_p = 4.78 fps hv_p = 0.36 foot Inlet R = 4.75 feet

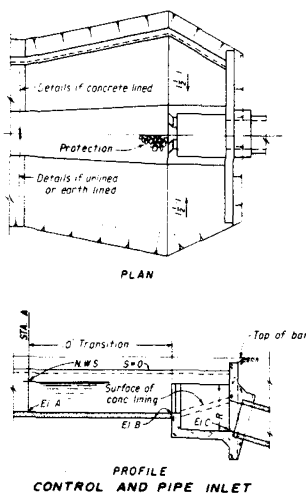
F	2.08	2.38	2.66	2.95	3.24	3.52	3.81	4.12	4.43	4.73	5.02	5.33	5.64	5.94	6.26	6.56	6.87	7.19	7.50	7.81	8.13	8.44	8.76
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.06	0.06	0.06	0.06	0.06	0.06	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.09	0.09	0.09
h_4	0.79	0.79	0.79	0.92	0.92	0.92	1.34	1.34	1.34	1.47	1.47	1.47	1.74	1.74	1.74	1.74	2.01	2.01	2.01	2.01	2.28	2.28	2.28
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	11.00	11.00	11.00	11.00	11.00	11.00	14.00	14.00	14.00	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	14.50	17.50	17.50	17.50
L_4	6.00	6.00	6.00	7.00	7.00	7.00	5.00	5.00	5.00	5.50	5.50	5.50	6.50	6.50	6.50	6.50	7.50	7.50	7.50	7.50	5.50	5.50	5.50
S_3	7°-30'					7°-30'	15°													15°	22°-30'		22°-30'
Submergence	0.51	0.61	0.70	0.66	0.76	0.86	0.45	0.53	0.60	0.55	0.64	0.71	0.52	0.60	0.66	0.74	0.55	0.61	0.68	0.76	0.40	0.47	0.54
d_2	4.06	4.15	4.24	4.32	4.41	4.50	4.58	4.65	4.72	4.79	4.87	4.94	5.01	5.08	5.14	5.21	5.28	5.34	5.40	5.47	5.53	5.59	5.65
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.91	7.30	7.68	8.06	8.45	8.83	9.21	9.59	9.98	10.36	10.74	11.12	11.51	11.89	12.27	12.65	13.04	13.42	13.80	14.19	14.57	14.95	15.33
F	9.09	9.40	9.72	10.03	10.36	10.67	11.00	11.32	11.64	11.97	12.29	12.61	12.94	13.27	13.59	13.92	14.25	14.58	14.90	15.24	15.57	15.90	16.22
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
h_4	2.28	2.49	2.49	2.49	2.49	2.49	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.52	3.52	3.52
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	17.50	18.00	18.00	18.00	18.00	18.00	18.50	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.50	19.50	19.50
L_4	5.50	6.00	6.00	6.00	6.00	6.00	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.50	8.50	8.50
S_3	22°-30'																						22°-30'
Submergence	0.60	0.45	0.51	0.58	0.64	0.71	0.57	0.62	0.69	0.74	0.60	0.66	0.72	0.77	0.83	0.47	0.53	0.58	0.64	0.69	0.53	0.58	0.64
d_2	5.71	5.76	5.82	5.88	5.94	6.00	6.05	6.10	6.16	6.21	6.26	6.32	6.37	6.42	6.47	6.52	6.57	6.62	6.67	6.72	6.76	6.81	6.86
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.72	16.10	16.48	16.86	17.25	17.63	18.01	18.39	18.78	19.16	19.54	19.92	20.31	20.69	21.07	21.45	21.84	22.22	22.60	22.99	23.37	23.75	24.13

Q = 49 cfs Diameter of pipe = 42 inches V_p = 5.09 fps hv_p = 0.40 foot Inlet R = 4.75 feet

F	1.93	2.23	2.50	2.78	3.07	3.35	3.64	3.93	4.22	4.53	4.82	5.12	5.42	5.73	6.03	6.32	6.63	6.95	7.25	7.56	7.88	8.18	8.50
h_2	2.10	2.49	2.87	3.25	3.64	4.02	4.40	4.78	5.17	5.55	5.93	6.31	6.70	7.08	7.46	7.84	8.23	8.61	8.99	9.38	9.76	10.14	10.52
h_3	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
h_4	0.94	0.94	1.07	1.07	1.21	1.21	1.21	1.45	1.45	1.45	1.66	1.66	1.66	1.66	2.07	2.07	2.07	2.28	2.28	2.28	2.28	2.49	2.49
L_2	5.10	6.00	7.00	7.90	8.80	9.70	10.60	11.60	12.50	13.40	14.30	15.20	16.20	17.10	18.00	19.00	19.90	20.80	21.70	22.60	23.60	24.50	25.40
L_3	13.50	13.50	14.00	14.00	14.50	14.50	14.50	16.50	16.50	16.50	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.50	17.50	17.50	17.50	18.00	18.00
L_4	3.50	3.50	4.00	4.00	4.50	4.50	4.50	3.50	3.50	3.50	4.00	4.00	4.00	4.00	5.00	5.00	5.00	5.50	5.50	5.50	5.50	6.00	6.00
S_3	15°						15°	22°-30'															22°-30'
Submergence	0.43	0.53	0.51	0.60	0.56	0.66	0.75	0.44	0.54	0.61	0.50	0.58	0.67	0.74	0.41	0.50	0.58	0.43	0.51	0.59	0.65	0.52	0.59
d_2	4.16	4.25	4.35	4.44	4.53	4.62	4.70	4.78	4.87	4.94	5.02	5.09	5.17	5.24	5.31	5.39	5.46	5.52	5.59	5.66	5.72	5.79	5.85
L_x	5.50	6.50	7.50	8.50	9.50	10.50	11.50	12.50	13.50	14.50	15.50	16.50	17.50	18.50	19.50	20.50	21.50	22.50	23.50	24.50	25.50	26.50	27.50
$d_1 + hv_1$	6.91	7.30	7.68	8.06	8.45	8.83	9.21	9.59	9.98	10.36	10.74	11.12	11.51	11.89	12.27	12.65	13.04	13.42	13.80	14.19	14.57	14.95	15.33
F	8.81	9.12	9.44	9.75	10.07	10.39	10.70	11.02	11.34	11.65	11.97	12.29	12.62	12.94	13.27	13.59	13.92	14.24	14.57	14.89	15.23	15.55	15.88
h_2	10.91	11.29	11.67	12.05	12.44	12.82	13.20	13.58	13.97	14.35	14.73	15.11	15.50	15.88	16.26	16.64	17.03	17.41	17.79	18.18	18.56	18.94	19.32
h_3	0.09	0.09	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
h_4	2.49	2.49	2.69	2.69	2.69	2.69	2.90	2.90	2.90	2.90	3.31	3.31	3.31	3.31	3.31	3.52	3.52	3.52	3.52	3.52	3.93	3.93	3.93
L_2	26.30	27.30	28.20	29.10	30.00	31.00	31.90	32.80	33.70	34.70	35.60	36.50	37.40	38.40	39.30	40.20	41.10	42.00	43.00	43.90	44.80	45.70	46.70
L_3	18.00	18.00	18.50	18.50	18.50	18.50	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.50	19.50	19.50	19.50	19.50	19.50	19.50	19.50
L_4	6.00	6.00	6.50	6.50	6.50	6.50	7.00	7.00	7.00	7.00	8.00	8.00	8.00	8.00	8.00	8.50	8.50	8.50	8.50	8.50	9.50	9.50	9.50
S_3	22°-30'																						22°-30'
Submergence	0.66	0.73	0.59	0.66	0.73	0.79	0.66	0.72	0.79	0.86	0.51	0.57	0.63	0.69	0.74	0.59	0.65	0.71	0.76	0.83	0.46	0.52	0.57
d_2	5.92	5.98	6.04	6.10	6.16	6.22	6.28	6.34	6.40	6.46	6.52	6.57	6.63	6.68	6.73	6.78	6.84	6.89	6.94	7.00	7.04	7.09	7.14
L_x	28.50	29.50	30.50	31.50	32.50	33.50	34.50	35.50	36.50	37.50	38.50	39.50	40.50	41.50	42.50	43.50	44.50	45.50	46.50	47.50	48.50	49.50	50.50
$d_1 + hv_1$	15.72	16.10	16.48	16.86	17.25	17.63	18.01	18.39	18.78	19.16	19.54	19.92	20.31	20.69	21.07	21.45	21.84	22.22	22.60	22.99	23.37	23.75	24.13

$F = (d_1 + hv_1) - (1.1d_2 + hv_p)$ $d_1 + hv_1 = R + h_2 + 0.06$ Submergence = $1.1d_2 + hv_p + h_3 - h_4$ − diameter of pipe − hv





PIPE DIAMETER SELECTION DATA					
MAX. V=3.5 fps (EARTH TRANSITION)		MAX. V=5.0 fps (CONC TRANSITION)		PIPE	
Q (cfs)		Q (cfs)		DIA.	AREA
FROM	INCLUDING	FROM	INCLUDING	(INCHES)	(SQ. FT.)
0	2.7	0	3.9	12	0.785
2.7	4.3	3.9	6.1	15	1.227
4.3	6.2	6.1	8.8	18	1.767
6.2	8.4	8.8	12.0	21	2.405
8.4	11.0	12.0	15.7	24	3.142
11.0	13.9	15.7	19.9	27	3.976
13.9	17.2	19.9	24.5	30	4.909
17.2	20.8	24.5	29.7	33	5.940
20.8	24.7	29.7	35.3	36	7.069
24.7	29.0	35.3	41.5	39	8.296
29.0	33.7	41.5	48.1	42	9.621
33.7	38.7	48.1	55.2	45	11.045
38.7	44.0	55.2	62.8	48	12.566
44.0	49.7	62.8	70.9	51	14.186
49.7	55.7	70.9	79.5	54	15.904
55.7	62.0	79.5	88.6	57	17.721
62.0	68.7	88.6	98.2	60	19.635
68.7	75.8			63	21.648
75.8	83.2			66	23.758
83.2	90.9			69	25.967
90.9	99.0			72	28.274

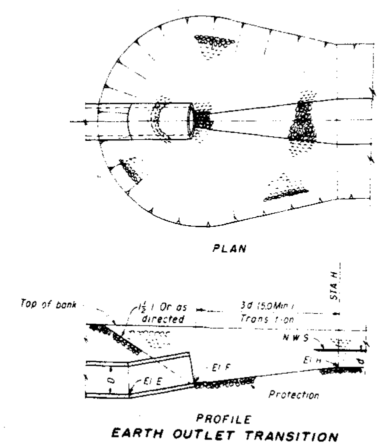
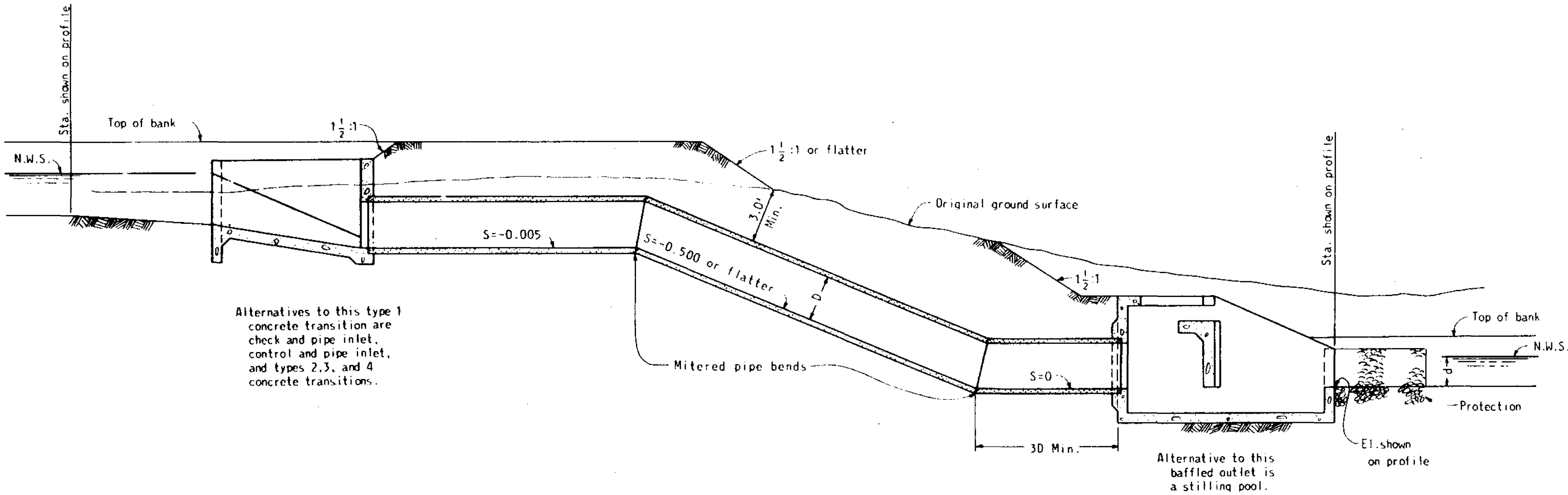


Figure 2-31. **Type 1 pipe drop–plan and sections. 103-D-1271.**

of the pipe or through the surrounding earth, thereby preventing removal of soil particles (piping) at the point of emergence. Pipe collars may also be necessary to discourage rodents from burrowing along pipe. Pipe drops should be examined to see if collars are required. A detailed discussion on pipe collars and percolation may be found in subchapter VIII C.

(d) *Freeboard.*—Type 1 pipe drop structures have either a check and pipe inlet or a control and pipe inlet. Figures 3-38 and 3-39 are standard designs for these structures and have adequate freeboard incorporated into their design. Freeboard for concrete outlet transitions should be in accordance with chapter VII.

2-31. ***Type 2 Pipe Drop Structure Components and Design Considerations (fig. 2-32).***—The principle hydraulic elements of the type 2 pipe drop are the upstream transition, the inlet structure, the pipe, and the outlet structure. The type 2 pipe drop is similar to the type 1 pipe drop except that a reinforced concrete transition of the type 1, 2, 3, or 4 may be used at the inlet if the pipe drop is a



PIPE DIA. SELECTION DATA			
MAX. V=12.0 fps		PIPE	
Q (cfs)		DIA.	AREA
FROM	INCLUDING	(INCHES)	(SQ FT.)
0	9.4	12	0.785
9.4	14.7	15	1.227
14.7	21.2	18	1.767
21.2	28.9	21	2.405
28.9	37.7	24	3.142
37.7	47.7	27	3.976
47.7	58.9	30	4.909
58.9	71.3	33	5.940
71.3	84.8	36	7.069
84.8	99.6	39	8.296

NOTE

The maximum full pipe velocity for sizing pipe is 12 feet per second.

Figure 2-32. **Type 2 pipe drop–plan and sections. 103-D-1272.**

cross-drainage structure and a baffled outlet or a stilling pool is always used at the outlet end.

(a) *Baffled Outlets.*—Baffled outlets are generally used at the outlet end of a type 2 pipe drop to dissipate excess energy. See chapter VI for a discussion on baffled outlets.

(b) *Stilling Pools.*—Stilling pools are sometimes used at the outlet end of a type 2 pipe drop to dissipate excess energy. The pool reduces velocity and turbulence of the water at the outlet end and thereby reduces erosion in the channel below. See subchapter II F for design of stilling pools.

When the stilling pool discharges into an uncontrolled channel, a control such as a weir must be provided in the outlet structure. Critical depth over this weir control section should be used to determine the downstream energy gradient. These controls are especially suitable for wasteways discharging into natural channels. Figure 2-36 of subchapter II F shows such a control. The aperture in the weir permits the pool to drain. In general, for any pool depending on downstream tailwater for sufficient d_2 to cause the hydraulic jump, the elevation of the pool floor may be determined by subtracting $(d_2 + h_{v_2})$ from minimum energy elevation downstream of the pool.

Where use of the stilling pool is to be intermittent and for short duration of flow, such as is most wasteways or structures carrying flood water, the minimum pool length should be $3d_2$. Where uninterrupted use or long duration of the flow is expected, the minimum pool length should be $4d_2$.

For rectangular pools with discharges up to 100 cfs, the width of pool may be determined by the empirical formula

$$b = \frac{360\sqrt{Q}}{Q + 350}$$

where b = width of pool in feet and
Q = discharge in cubic feet per second.

(c) *Outlet Transition.*—The outlet from a stilling pool connects the pool with the earth or concrete-lined canal downstream and prevents or lessens canal erosion. Some of the more common types of stilling pool outlets used are: broken-back transitions and straight or curved diverging vertical walls extending into the earth canal banks on each side. A portion of a transition may be made in earth provided the velocity at the downstream cutoff of the concrete transition is not too great for erosion. Riprap or gravel protection is generally used on the earth transition section. See subchapter VII B for protection. Where broken-back transitions are used, the top of the walls should be the same elevation as the pool walls.

Cutoffs at the ends of the concrete transitions should be a minimum of 24 inches deep and 6 inches thick for water depths 0 to 3 feet over the cutoff, and 2 feet 6 inches deep and 8 inches thick for water depths from 3 to 6 feet.

Where an earth transition is used at the inlet or outlet, it should be a minimum of 10 feet long.

(d) *Design Layout.*—While exact methods of design of pipe drop cannot be laid down, it will usually be helpful to first plot the profile of the natural ground surface on the centerline of the drop to natural scale on cross-section paper, and lay out a tentative bottom grade line. From this same profile a study can be made to determine the tentative location of the inlet of the structure and the proper location of the stilling pool or the baffled outlet.

(e) *Freeboard.*—Type 2 pipe drop structures have either a check and pipe inlet, a control and pipe inlet, or a type 1, 2, 3, or 4 concrete transition. Figures 3-38 and 3-39 are standard designs for check and pipe inlet and control and pipe inlet and have adequate freeboard incorporated into their design. Freeboard for concrete transitions should be in accordance with chapter VII.

2-32. *Design Example of a Type 1 Pipe Drop with a Control and Pipe Inlet (see fig. 2-31).*—The topography for the canal at this station is assumed to have a road crossing the canal and an abrupt drop in the general terrain. A type 1 pipe drop sumped to dissipate energy and to cross under the roadway is assumed to be the feasible structure to use.

Assume that a control and pipe inlet will be the feasible type of inlet structure to minimize erosion in the earth canal upstream from the drop.

Assume that for this drop structure it will be more economical to have a concrete downstream transition which will permit the use of a smaller pipe because the pipe velocity can be as much as 5 feet per second.

(a) *Given.*—

(1) Type of waterway = earth canal.

(2) Hydraulic properties of the canal at upstream and downstream ends of structure:

Q = 49 cfs	b = 5.0 ft.	d_n = 2.81 ft.
S:S = 1-1/2:1	A = 25.90 ft.^{2}	wp = 15.15 ft.
R = 1.71	V = 1.89 f.p.s.	h_v = 0.06 ft.
E = 2.87 ft.	s = 0.0005	$s^{1/2}$ = 0.02235
n = 0.025		

(3) El. A = 5406.00 (from a canal profile sheet).

(4) NWS El. at Sta. A = El. A + d_n = 5406.00 + 2.81 = 5408.81.

(5) Energy El. at Sta. A = El. NWS + h_v = 5408.81 + 0.06 = 5408.87.

(6) El. H = 5392.0 (from a canal profile sheet).

(7) NWS at Sta. H = 5392.0 + 2.81 = 5394.81.

(8) Energy El. at Sta. H = 5394.81 + 0.06 = 5394.87.

(b) *Determine.*—

(1) *Pipe size (see table on fig. 2-31).*—For a discharge of 49 cfs and a maximum allowable velocity of 5 feet per second the table would permit the use of a 45-inch-diameter pipe.

However, a 42-inch diameter is considered satisfactory since V = 5.09 feet per second and does not exceed the maximum velocity appreciably.

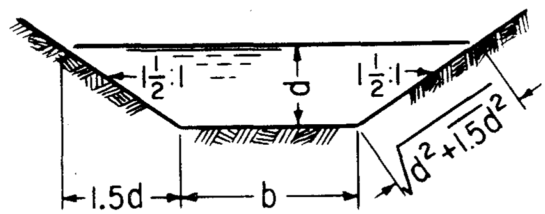
(2) *Inlet Structure Number.*—Figure 3-39 provides standard dimensions for control and pipe inlets. A structure inlet with a 42-inch-diameter pipe and a maximum allowable V_p of 5 feet per second is numbered 42-1.

(3) Determine control and pipe inlet standard dimensions R and L by entering the structure table figure 3-39 for structure no. 42-1. Read R = 4 ft. 9 in. and L = 4 ft. 6 in.

(4) Determine the normal depth in the canal and its velocity head for a Q of 0.2 the design Q.

$$AR^{2/3} = \frac{0.2\,Qn}{1.486\,s^{1/2}} = \frac{0.2 \times 49 \times 0.025}{1.486 \times 0.02235} = 7.38$$

Assume different water depths, d, in the canal and solve for $AR^{2/3}$ until $AR^{2/3}$ = 7.38 (see tabulation below).



CANAL SECTION

$$A = db + 1.5d^2$$
$$wp = b + 2\sqrt{d^2 + 1.5d^2} = b + 3.606d$$

when d = 1.19 ft., $AR^{2/3}$ = 7.35 which is close to the desired value of 7.38. Therefore for a normal depth of d = 1.19 ft., A = 8.07 ft.^{2}, $V = \frac{0.2 \times 49}{8.07}$ = 1.21 feet per second, $h_v = \frac{V^2}{2g} = \frac{1.21^2}{64.4}$ = 0.02 ft.

$$E = d + h_v = 1.21 \text{ ft. and } Q = 9.8 \text{ cfs}$$

The normal depth, d, could also have been determined by using table 2-5.

(5) Determine the control notch dimensions using the procedure in subchapter III G, Control and Pipe Inlets.

Assumed d, ft.	d^2, sq. ft.	$1.5d^2$, sq. ft.	b, ft.	db, sq. ft.	$A = db+1.5d^2$, sq. ft.	3.606d, ft.	wp = b+3.606d, ft.	$R = \frac{A}{wp}$, ft.	$R^{2/3}$	$AR^{2/3}$
1.19	1.415	2.12	5.0	5.95	8.07	4.29	9.29	0.868	0.910	7.35
1.2	1.44	2.16	5.0	6.0	8.16	4.33	9.33	0.875	0.920	7.51

P = 20 inches T = d_n = 2.81 feet
S = 0.50 make T = 2 ft. 10 in.
N = P+2ST
= 4 ft. 6 in.

(6) Determine inlet box inside width.

Minimum width = pipe dia. + 6 inches
= (3 ft. 6 in.) + 6 inches
= 4 ft. 0 in.

or

Minimum width = N+6 inches
= (4 ft. 6 in.) + 6 inches
= 5 ft. 0 in.

Use width = 5 ft. 0 in.

(7) Determine El. top of notch and sidewalls = El. A + T = 5406.00 + 2.83 ft. = 5408.83.

(8) Determine El. D figure 3-39 (invert of box) = El. C of figure 2-31 = El. top of sidewalls − R = 5408.83 − 4.75 = 5404.08.

(9) Determine vertical distance F shown on figure 2-28 = Energy El. at Sta. A − Energy El. at Sta. H = 5408.87 − 5394.87 = 14 ft.

(10) *Determine El. D – Refer to figure 2-28.*—Use the procedure associated with the above figure and hydraulic tables that may be found in different handbooks to compute the cross-sectional area (A_1) and $\overline{Y}_1$ for a pipe flowing partially full.

Assume $\frac{d_1}{D}$ = 0.20. Then from hydraulic tables [6] or [7] for area of pipe flowing partially full:

$$C = 0.1118$$
$$A_1 = C \times D^2 = 0.1118 \times 3.5^2 = 1.37 \text{ ft.}^2$$
$$d_1 = 0.20 \times D = 0.20 \times 3.50 = 0.70 \text{ ft.}$$
$$V_1 = \frac{Q}{A_1} = \frac{49}{1.37} = 35.8 \text{ f.p.s.}$$
$$h_{v_1} = \frac{35.8^2}{64.4} = 19.9 \text{ ft.}$$

From hydraulic tables [6] for $\frac{d_1}{D}$ = 0.20, compute $\overline{Y}_1$ for pipe flowing partially full:

$$C = 0.082$$
$$\overline{Y}_1 = C \times D = 0.082 \times 3.5 = 0.287 \text{ ft.}$$

For full pipe:

$$A_2 = 9.62 \text{ ft.}^2$$
$$V_2 = \frac{49}{9.62} = 5.09 \text{ f.p.s.},$$
$$h_{v_2} = 0.40 \text{ ft., also} = h_{v_p}$$

Now, following the procedure outlined with figure 2-28,

$$d_2 = \frac{Q \Delta V}{A_2 g} + \frac{A_1}{A_2} \overline{Y}_1 + \frac{D}{2}$$
$$d_2 = \frac{49\,(35.8 - 5.09)}{9.62 \times 32.2} + \frac{1.37}{9.62}\ 0.287 + \frac{3.5}{2}$$
$$= 4.86 + 0.04 + 1.75 = 6.65 \text{ ft.}$$

$$F = d_1 + h_{v_1} - d_2 - h_{v_p} = 0.7 + 19.9 - 6.65 - 0.40 = 13.55 \text{ ft.}$$

This is not compatible with the actual drop of 14.00 feet.

Try $\frac{d_1}{D}$ = 0.195. Then from hydraulic tables [6] or [7] for area of pipe flowing partially full.

$$C = 0.1079$$
$$A_1 = C \times D^2 = 0.1079 \times 3.5^2 = 1.32 \text{ ft.}^2$$
$$d_1 = 0.195 \times 3.5 = 0.68 \text{ ft.}$$
$$V_1 = \frac{49}{1.32} = 37.2 \text{ f.p.s.}$$
$$h_{v_1} = \frac{37.2^2}{64.4} = 21.52 \text{ ft.}$$

From hydraulic tables [6] for $\frac{d_1}{D}$ = 0.195, compute $\overline{Y}_1$ for pipe partially full:

$$C = 0.08$$
$$\overline{Y}_1 = C \times D$$
$$\overline{Y}_1 = 0.08 \times 3.5 = 0.28 \text{ ft.}$$

For full pipe:

$$A_2 = 9.62 \text{ ft.}^2$$
$$V_2 = 5.09 \text{ f.p.s.,}$$
$$h_{v_2} = 0.40 \text{ ft., also} = h_{v_p}$$

$$d_2 = \frac{Q\Delta V}{A_2 g} + \frac{A_1}{A_2}\overline{Y}_1 + \frac{D}{2}$$
$$= \frac{49\,(37.2 - 5.09)}{9.62 \times 32.2} + \left(\frac{1.32}{9.62}\right) 0.28 + \frac{3.5}{2}$$
$$= 5.07 + 0.04 + 1.75 = 6.86 \text{ ft.}$$

$$F = d_1 + h_{v_1} - d_2 - h_{v_p} = 0.68 + 21.52 - 6.86 - 0.40 = 14.94 \text{ ft.}$$

The $\frac{d_1}{D}$ ratio of 0.20 produces a computed F value closer to actual (14 ft.) so use this ratio and d_2 = 6.65 ft. to get El. D.

El. D = Energy El. at Sta. H – (d_2 + h_{v_p} + 0.1 d_2) where 0.1d_2 is a factor of safety. El. D = 5394.87 – 6.65 – 0.40 – 0.67 = 5387.15 Use El. D = 5387.00.

The other portions of the pipe drop may be designed as explained in the narrative. The designer may use figures 3-38 and 3-39 for design of the inlet structures.

The entire pipe profile may be extracted from table 2-4 which is for use with figure 2-30. Difference in elevations and distances that correspond to a discharge of 49 cfs, a pipe diameter of 42 inches, and an F value of 14.24 feet are taken from this table. There is no tabulation for F = 14 feet, therefore use the next greater value which is 14.24 feet. By using these tables El. D = NWS El. in the canal at the upstream end of the drop minus R and minus h_2 = 5408.81 – 4.75 – 17.41 = 5386.65 The first solution was El. D = 5387.15 which is 0.5 foot higher. Either solution is considered satisfactory.

F. CHUTES

D. L. WINSETT [4]

2-33. ***Purpose and Description.***–Chutes are used to convey water from a higher elevation to a lower elevation. A chute structure may consist of an inlet, a chute section, an energy dissipator, and an outlet transition. Figure 2-33 shows the relationship of the different parts of the structure. The chute section may be pipe as in a pipe chute or an open section as in an open channel chute. Chutes are similar to drops except that they carry the water over longer distances, over flatter slopes, and through greater changes in grade. Figure 2-34 shows an open channel chute on a steep slope. Figures 2-35 and 2-36 show stilling pools and outlets at the lower ends of chute structures.

The inlet portion of the structure transitions the flow from the channel upstream of the structure to the chute section. The inlet should provide a control to prevent racing and scouring in the channel. Control is achieved by combining a check, a weir, or a control notch with the inlet. The inlet used should be symmetrical about the centerline of the chute, it should allow the passage of the full capacity of the upstream channel into the chute with normal water surface upstream and where desirable, it should allow the channel upstream to be drained when operations are suspended. It should have cutoffs, as required, to provide a sufficient length of percolation path as computed by Lane's weighted-creep method. Head losses through the inlet may be neglected provided they are small enough that they do not significantly affect the end result. Otherwise the losses through the inlet should be computed and used in the determination of the energy level at the beginning of the chute section. If the bottom grade of the inlet is flat, it may be assumed that critical flow occurs where the flat grade of the inlet portion meets the steeper grade of the chute section. If the

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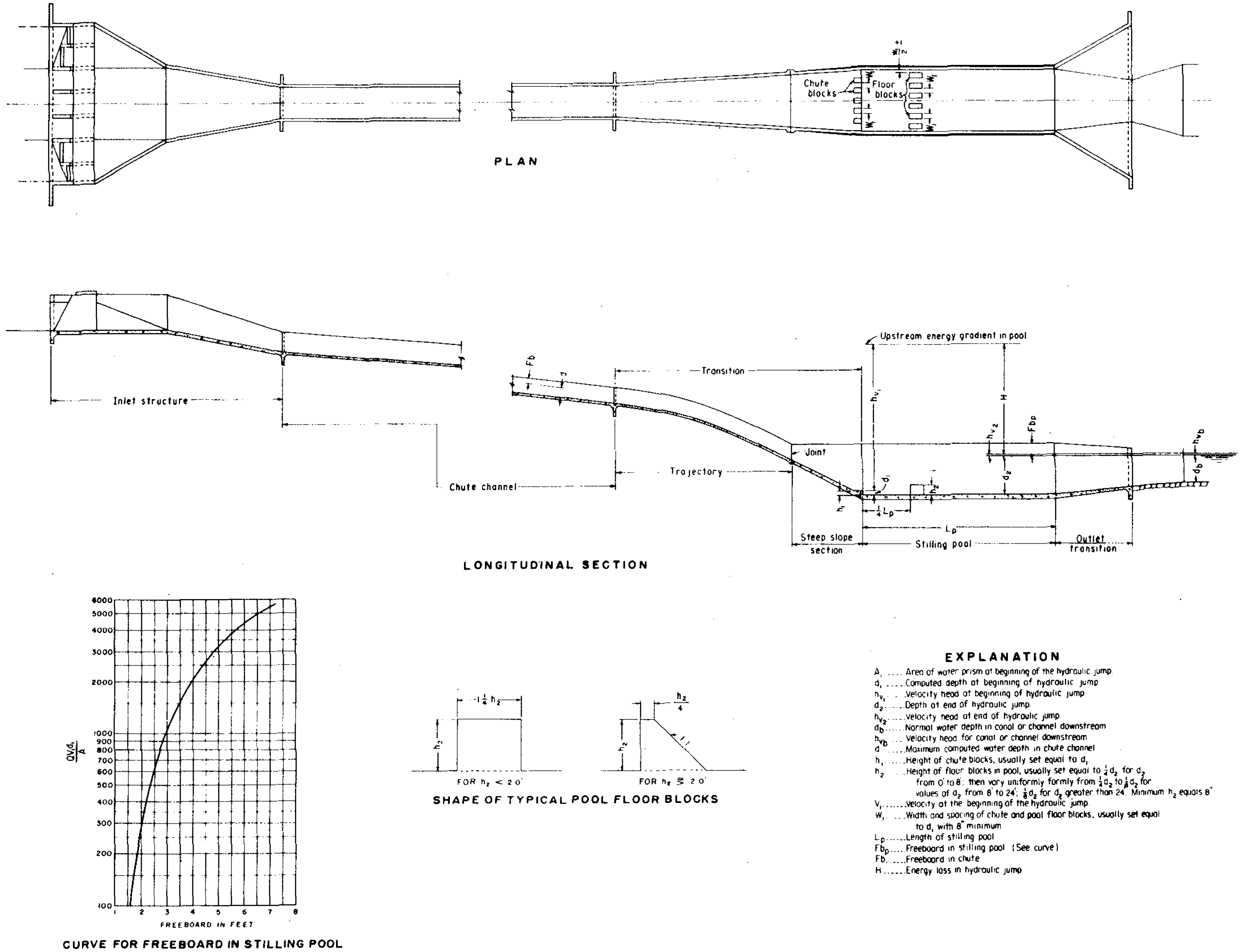


Figure 2-33. **Typical rectangular chutes. 103-D-1273**

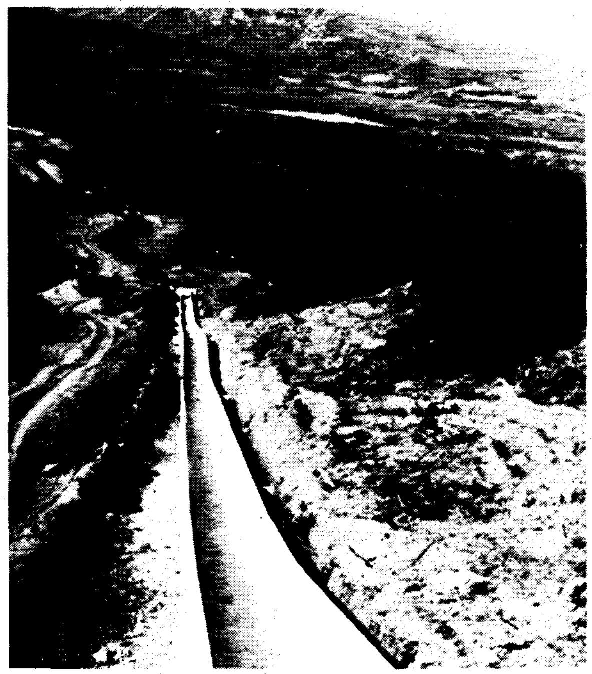


Figure 2-34. **Rectangular concrete chute on a steep slope. P 482-417-908**

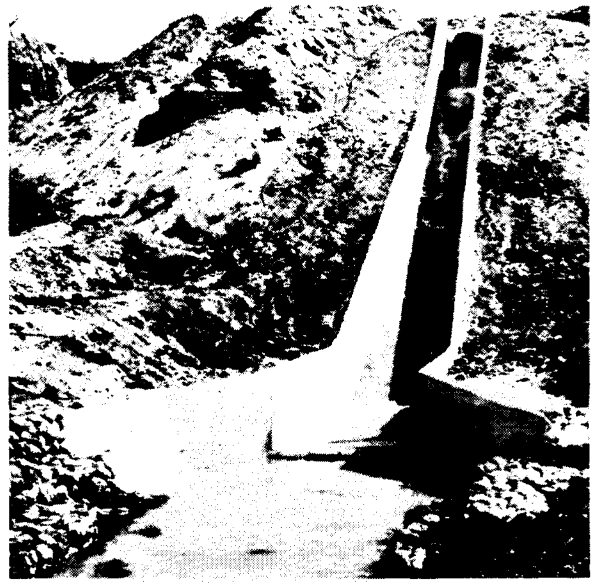


Figure 2-35. **Terminal wasteway with a rectangular inclined drop into a stilling pool. P 343-529-112 NA**

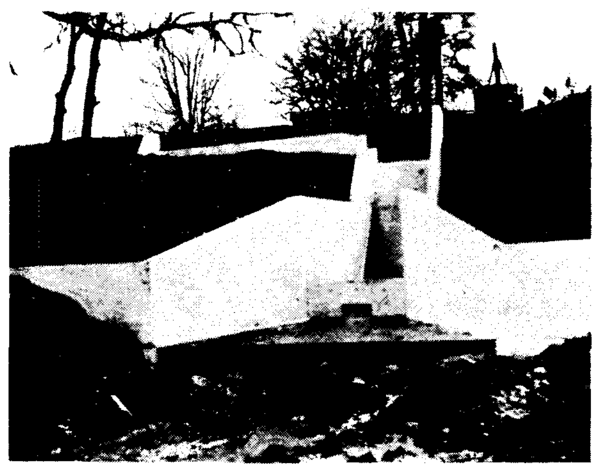


Figure 2-36. **Stilling pool and outlet portions of a spillway structure. Control built into the stilling pool maintains tailwater for the hydraulic jump. P-833-128-70**

slope of the inlet portion is made steep enough to support a velocity greater than the critical velocity, this velocity and its water depth should be computed and used to determine the energy gradient at the beginning of the chute section.

The chute section, either pipe or open channel, generally follows the original ground surface and connects to an energy dissipator at the lower end. Most fluid mechanics textbooks discuss the behavior of water on steep slopes and in hydraulic jumps and derive the equations used to determine the characteristics of flow under these conditions. Some of the solutions are obtained through trial and error. The use of a computer and programs similar to the one presented in appendix C makes these solutions less tedious.

Stilling pools or baffled outlets are used as energy dissipators on chute structures. Baffled outlets are discussed in chapter VI [8].

An outlet transition is used when it is necessary to transition the flow between the energy dissipator and the downstream channel. If it is necessary to provide tailwater for the energy dissipator, the water surface at the outlet must be controlled. If a concrete outlet transition is provided and there is no downstream control of flow in the channel, the transition may be used to provide backwater by raising the floor of the transition at the cutoff as shown on figure 2-33. Tailwater may also be provided by building a control into the outlet transition (fig. 2-36). Head loss in the outlet transition is neglected.

1. Open Channel Chute

2-34. ***Design Considerations.***—(a) *Manning's Coefficient of Roughness (n).*—In computing characteristics of flow in a chute structure, conservative values of n are used. When computing wall heights in a concrete chute, an n of 0.014 is assumed. In computing energy values in a concrete chute, n is assumed to be 0.010.

(b) *Transitions.*—Transitions in an open channel chute should be designed to prevent the formation of waves. An abrupt change in section, either a convergence or a divergence may set up waves that can be troublesome as they travel through the chute section and the energy dissipator [9]. To avoid the formation of waves, the cotangent of the deflection angle of the water surface in developed plan of each side of a chute transition should not be less than 3.375 times the Froude number (F). This restriction on deflection angles should apply to any change in section made in the inlet, the chute section, or in the stilling pool. If this restriction does not control the angle of deflection, the maximum deflection angle in the water surface in the inlet transition may be about 30°. The angle of the water surface with the centerline in the outlet transition may be about 25° maximum.

The maximum angle of deflection is computed as follows:

$$\text{Cotangent } \alpha = 3.375 \times F \qquad (1)$$

where:

$$F = \frac{V}{\sqrt{(1-K) g\, d \cos\theta}}, \qquad (2)$$

The mean of the values of F at the beginning and end of the transition may be used.

d = water depth normal to the floor of the chute. Using

$$d = \frac{\text{Area of the section}}{\text{Top width of the section}}$$

allows for various shapes of section,

g = acceleration of gravity, $g = 32.2$ ft./sec.^{2}

K = an acceleration factor as determined below:

With the floor of the transition in a plane:

$$K = 0$$

With the floor of the transition on a circular curve:

$$K = \frac{V^2}{gR\cos\theta} \qquad (3)$$

With the floor of the transition on a parabolic curve:.

$$K = \frac{(\tan\theta_L - \tan\theta_o)\, 2\, h_v \cos^2\theta_o}{L_T} \qquad (4)$$

The Bureau of Reclamation limits the value of K to 0.5 maximum to insure positive pressure on the floor.

In (3) and (4):

h_v = velocity head at the origin of the trajectory,

L_T = length of trajectory,
R = radius of curvature of the floor,
V = velocity at the point being considered,
θ = slope angle of the floor at the point being considered,
θ_o = slope angle of the floor at the beginning of the trajectory, and
θ_L = slope angle of the floor at the end of the trajectory.

The flare angle and the widths for several points along the transition may be computed and plotted. A chord which approximates the theoretical curve can be drawn to determine the flare to be used. Limiting the angle of flare in an inlet transition minimizes the possibility of separation and pulsating flow being initiated in that part of the structure. Unsymmetrical inlet transitions and changes in alinement immediately upstream of the structure are to be avoided because they may set up cross waves or transverse flow that will continue in the chute section.

(c) *Chute Section.*—The usual section for an open channel chute is rectangular but the flow characteristics of other shapes should be considered where wave suppression is an important part of the design. Economics and ease of construction are always considered in choosing a section. When it is necessary to increase the resistance of the chute section to sliding, cutoffs are used to key the structure into the foundation.

For chutes less than about 30 feet long, friction in the chute may be neglected. Bernoulli's equation is used to compute the flow variables at the bottom of the chute. The equation:

$$d_1 + h_{v_1} + Z = d_2 + h_{v_2} \qquad (5)$$

is solved by trial. The distance Z is the change in elevation in the chute floor. For chutes longer than 30 feet, friction losses are included and the equation becomes:

$$d_1 + h_{v_1} + Z = d_2 + h_{v_2} + h_f \qquad (6)$$

The quantity h_f is the friction loss in the reach and is equal to the average friction slope s_a in the reach times the length of the reach L. Manning's n is assumed to be 0.010. The friction slope, s_f, at a point in the chute section is computed as:

$$s_f = \frac{n^2 V^2}{2.2R^{4/3}}$$

R = hydraulic radius of the section.

In equations (5) and (6):

d_1 = Depth at the upstream end of the reach,
h_{v_1} = Velocity head at the upstream end of the reach,
d_2 = Depth at the downstream end of the reach, and
h_{v_2} = Velocity head at the downstream end of the reach.

Using either equation (5) or (6), d_2 is assumed and the energy levels are computed and compared. Additional trials are made until the two energy levels balance.

Another form of the equation in which friction is considered is:

$$L = \frac{(d_1 + h_{v_1}) - (d_2 + h_{v_2})}{s_a - s} \qquad (7)$$

where:

s_a = average friction slope
s = slope in channel floor.

In using equation (7) a stepwise procedure is used in which small changes in energy are assumed and the corresponding change in length is computed. This procedure is repeated until the total of the increments of length is equal to the length of the reach of chute being considered. The smaller the increment of length is, the greater the accuracy will be.

The height of the walls in the open channel chute section should be equal to the maximum depth computed in the section plus a freeboard allowance or to 0.4 times the critical depth in the chute section plus freeboard, whichever is greater. The recommended minimum freeboard for open channel chute sections (capacity 100 cfs or less) is 12 inches. Depth and freeboard are measured normal to the floor of the chute section.

At velocities greater than about 30 feet per second, water may take on additional bulk due to air being entrained by the water. The recommended freeboard allowance for chute walls will result in a wall of sufficient height to contain this additional bulk.

(d) *Trajectory.*—When the energy dissipator is a stilling pool, a short steep section should connect the trajectory with the stilling pool. The slope of this steep section should be between 1.5 to 1 and 3 to 1 with a slope of 2 to 1 preferred. Flatter slopes may be used in special cases but slopes flatter than 6 to 1 should not be used. A vertical curve is required between the chute section and the steep slope section. A parabolic curve will result in a constant value of K over the length of the curve and is generally used. A parabolic trajectory may be determined from the following equation:

$$Y = X \tan \theta_o + \frac{(\tan \theta_L - \tan \theta_o) X^2}{2L_T} \qquad (8)$$

where:

X = Horizontal distance from the origin to a point on the trajectory,
Y = Vertical distance from the origin to point X on the trajectory,
L_T = Horizontal length from the origin to the end of the trajectory,
θ_o = The angle of inclination of the chute channel at the origin of the trajectory, and
θ_L = The angle of inclination of the chute channel at the end of the trajectory.

A length of trajectory (L_T) can be selected which, when substituted into equation (4), will result in a value of K of 0.5 or less. The length L_T is then used in the computation of Y using equation (8).

The trajectory should end at or upstream from the intersection of the chute section walls with the stilling pool walls. A long-radius curve slightly flatter than the computed trajectory may be used. If practicable the trajectory should coincide with any transition required.

The variables of flow on the trajectory and the steep section are computed in the same way as those in the chute section. An elevation for the floor of the stilling pool is assumed and the energy gradient at the junction of the chute section and the pool floor is computed. The flow variables at this point are used as the variables ahead of the hydraulic jump in the design of the stilling pool.

(e) *Stilling Pool.*—In a stilling pool the water flows down the steep slope section at a velocity greater than the critical velocity. The abrupt change in slope where the flat grade of the stilling pool floor meets the steep slope section forces the water into a hydraulic jump and energy is dissipated in the resulting turbulence. The stilling pool is proportioned to contain the jump. For a stilling pool to operate properly the Froude number $\left(F = \frac{V_1}{\sqrt{gd_1}}\right)$ where the water enters the stilling pool should be between 4.5 and 15. Special studies or model testing are required for structures with Froude numbers outside this range. If the Froude number is less than about 4.5 a stable hydraulic jump may not occur. If the Froude number is greater than about 10 a stilling pool may not be the best choice of energy dissipator. Stilling pools require tailwater to force the jump to occur where the turbulence can be contained.

Stilling pools usually have a rectangular cross section, parallel walls, and a level floor. The following equations to determine pool width and water depth after the jump apply to this type of pool. Flared pools are sometimes used and require special consideration. For discharges up to 100 cfs, the equation:

$$b = \frac{360\sqrt{Q}}{Q + 350}$$

where:

b = width of pool in feet, and
Q = discharge in cfs.

may be used to determine the width of a pool for the initial calculations. The water depth after the hydraulic jump may be computed from the formula:

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2V_1^{\,2} d_1}{g} + \frac{d_1^{\,2}}{4}} \qquad (9)$$

where

d_1 = Depth before the jump,
V_1 = Velocity before the jump,
d_2 = Depth after the jump, and
g = Acceleration of gravity

For structures where the vertical drop is less than 15 feet the water depth after the jump may be obtained from figure 2-37. The elevation of the energy gradient after the

H/d_c	0		0.1		0.2		0.3		0.4		0.5		0.6		0.7		0.8		0.9	
	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c
0	1.0	1.0	2.07	.680	2.48	.614	2.81	.572	3.09	.541	3.35	.516	3.60	.494	3.82	.477	4.04	.461	4.24	.448
1	4.44	.436	4.64	.425	4.82	.415	5.00	.405	5.18	.397	5.36	.389	5.53	.381	5.69	.375	5.86	.368	6.02	.362
2	6.18	.356	6.33	.351	6.49	.345	6.64	.340	6.79	.336	6.94	.331	7.09	.327	7.23	.323	7.38	.319	7.52	.315
3	7.66	.311	7.80	.308	7.94	.304	8.07	.301	8.21	.298	8.34	.295	8.48	.292	8.61	.289	8.74	.286	8.87	.284
4	9.00	.281	9.13	.278	9.26	.276	9.39	.274	9.51	.271	9.64	.269	9.76	.267	9.89	.265	10.01	.263	10.13	.261
5	10.25	.259	10.38	.257	10.50	.255	10.62	.253	10.73	.251	10.85	.250	10.97	.248	11.09	.246	11.21	.244	11.32	.243
6	11.44	.241	11.55	.240	11.67	.238	11.78	.237	11.90	.235	12.01	.234	12.12	.233	12.24	.231	12.35	.230	12.46	.228
7	12.57	.227	12.68	.226	12.79	.225	12.90	.223	13.01	.222	13.12	.221	13.23	.220	13.34	.219	13.45	.218	13.56	.216
8	13.66	.215	13.77	.214	13.88	.213	13.98	.212	14.09	.211	14.19	.210	14.30	.209	14.41	.208	14.51	.207	14.61	.206
9	14.72	.205	14.82	.204	14.93	.203	15.03	.202	15.13	.202	15.23	.201	15.34	.200	15.44	.199	15.54	.198	15.64	.197
10	15.74	.197	15.84	.196	15.95	.195	16.05	.194	16.15	.193	16.25	.193	16.35	.192	16.45	.191	16.54	.191	16.64	.190
11	16.74	.189	16.84	.188	16.94	.187	17.04	.187	17.13	.186	17.23	.185	17.33	.185	17.43	.184	17.52	.183	17.62	.183
12	17.72	.182	17.81	.181	17.91	.181	18.01	.180	18.10	.180	18.20	.179	18.29	.178	18.39	.178	18.48	.177	18.58	.176
13	18.67	.176	18.77	.175	18.80	.175	18.95	.174	19.05	.174	19.14	.173	19.24	.173	19.33	.172	19.42	.171	19.52	.171
14	19.61	.170	19.70	.170	19.79	.169	19.89	.169	19.98	.168	20.07	.168	20.16	.167	20.25	.167	20.34	.166	20.44	.166
15	20.53	.165	20.62	.165	20.71	.164	20.80	.164	20.89	.164	20.98	.163	21.07	.163	21.16	.162	21.25	.162	21.34	.161
16	21.43	.161	21.52	.160	21.61	.160	21.70	.160	21.79	.159	21.88	.159	21.97	.158	22.05	.158	22.14	.157	22.23	.157
17	22.32	.157	22.41	.156	22.50	.156	22.58	.155	22.67	.155	22.76	.155	22.85	.154	22.93	.154	23.02	.154	23.11	.153
18	23.19	.153	23.28	.152	23.37	.152	23.45	.152	23.54	.151	23.63	.151	23.71	.151	23.80	.150	23.89	.150	23.97	.150
19	24.06	.149	24.14	.149	24.23	.148	24.31	.148	24.40	.148	24.49	.147	24.57	.147	24.66	.147	24.74	.146	24.83	.146
20	24.91	.146	24.99	.145	25.08	.145	25.16	.145	25.25	.145	25.33	.144	25.42	.144	25.50	.144	25.58	.143	25.67	.143
21	25.75	.143	25.83	.142	25.92	.142	26.00	.142	26.08	.141	26.17	.141	26.25	.141	26.33	.141	26.42	.140	26.50	.140
22	26.58	.140	26.66	.139	26.75	.139	26.83	.139	26.91	.139	26.99	.138	27.08	.138	27.16	.138	27.24	.138	27.32	.137
23	27.40	.137	27.48	.137	27.57	.136	27.65	.136	27.73	.136	27.81	.136	27.89	.135	27.97	.135	28.05	.135	28.13	.135
24	28.22	.134	28.30	.134	28.38	.134	28.46	.134	28.54	.133	28.62	.133	28.70	.133	28.78	.133	28.86	.132	28.94	.132
25	29.02	.132	29.10	.132	29.18	.131	29.26	.131	29.34	.131	29.42	.131	29.50	.131	29.58	.130	29.66	.130	29.74	.130
26	29.82	.130	29.89	.129	29.97	.129	30.05	.129	30.13	.129	30.21	.128	30.29	.128	30.37	.128	30.45	.128	30.52	.128
27	30.60	.127	30.68	.127	30.76	.127	30.84	.127	30.92	.127	31.00	.126	31.07	.126	31.15	.126	31.23	.126	31.31	.126
28	31.38	.125	31.46	.125	31.54	.125	31.62	.125	31.69	.125	31.77	.124	31.85	.124	31.93	.124	32.00	.124	32.08	.124
29	32.16	.123	32.23	.123	32.31	.123	32.39	.123	32.46	.123	32.54	.122	32.62	.122	32.69	.122	32.77	.122	32.85	.122
30	32.92	.121	33.00	.121	33.08	.121	33.15	.121	33.23	.121	33.31	.121	33.38	.120	33.46	.120	33.53	.120	33.61	.120
31	33.68	.120	33.76	.119	33.84	.119	33.91	.119	33.99	.119	34.06	.119	34.14	.119	34.21	.118	34.29	.118	34.36	.118
32	34.44	.118	34.51	.118	34.59	.118	34.66	.117	34.74	.117	34.81	.117	34.89	.117	34.96	.117	35.04	.117	35.11	.116
33	35.19	.116	35.26	.116	35.34	.116	35.41	.116	35.49	.116	35.56	.115	35.63	.115	35.71	.115	35.78	.115	35.86	.115
34	35.93	.115	36.00	.115	36.08	.114	36.15	.114	36.23	.114	36.30	.114	36.37	.114	36.45	.114	36.52	.113	36.59	.113
35	36.67	.113	36.74	.113	36.81	.113	36.89	.113	36.96	.112	37.03	.112	37.11	.112	37.18	.112	37.25	.112	37.33	.112
36	37.40	.112	37.47	.112	37.55	.111	37.62	.111	37.69	.111	37.76	.111	37.84	.111	37.91	.111	37.98	.111	38.05	.110
37	38.13	.110	38.20	.110	38.27	.110	38.34	.110	38.42	.110	38.49	.110	38.56	.109	38.63	.109	38.70	.109	38.78	.109
38	38.85	.109	38.92	.109	38.99	.109	39.06	.109	39.14	.108	39.21	.108	39.28	.108	39.35	.108	39.42	.108	39.49	.108
39	39.56	.108	39.64	.107	39.71	.107	39.78	.107	39.85	.107	39.92	.107	39.99	.107	40.06	.107	40.14	.107	40.21	.106
40	40.28	.106	40.35	.106	40.42	.106	40.49	.106	40.56	.106	40.63	.106	40.70	.106	40.77	.105	40.84	.105	40.91	.105

Relation of energy loss, critical depth, and depth before and after jump, for hydraulic jumps in rectangular channels with level floor.

H = Difference in energy levels at upstream and downstream ends of jump.

d_c = Critical depth for flow considered, based on pool width. $d_c = \sqrt[3]{\frac{q^2}{g}}$

d_1 = Depth at upstream end of jump.

d_2 = Depth at downstream end of jump.

$K = d_2 - d_1$

$El.\ C_2 = El.\ C_1 - (d_1 + h_{v_1}) = El.\ C_1 - (d_2 + h_{v_2} + H)$

$$\frac{d_1}{d_c} = \sqrt[3]{\frac{2}{K\,(K+1)}}$$

$$\frac{H}{d_c} = \frac{(K-1)^3}{4K}\sqrt[3]{\frac{2}{K(K+1)}}$$

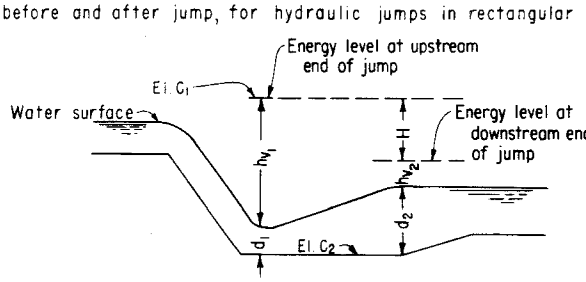


Figure 2-37. **Energy loss in hydraulic jump. 103-D-1274**

hydraulic jump should be balanced by the energy gradient in the channel downstream of the structure. If the gradients do not balance, a new elevation for the pool floor or a new pool width should be assumed and the energy levels recomputed. Trials are repeated until a balance is obtained.

The elevations selected should be reviewed to insure that the stilling pool will operate effectively at partial flow. Designs are normally checked at one-third the design flow or curves may be drawn as on figure 2-38. If the review indicates it to be necessary, the pool floor should be lowered or a different pool width assumed and the design procedure repeated.

The minimum pool length (L_p on fig. 2-33) for structures used on canals is normally 4 times d_2. For structures on wasteways or drains where the flow will be intermittent and of short duration the minimum length may be about 3 times d_2. The recommended freeboard for stilling pools may be determined from figure 2-33. Freeboard is measured above the maximum downstream energy gradient.

When the stilling pool discharges intermittently or discharges into a natural or other uncontrolled channel, a control should be built into the outlet of the stilling pool to provide the necessary tailwater. The critical depth at the control section should be used to determine the downstream energy gradient. When the stilling pool discharges into a controlled channel, the depth in the channel should be computed with the n value of the channel reduced by 20 percent and this depth used to determine the downstream energy gradient. If a flared pool is used, the deflection angle of the sidewalls should not exceed the angle permitted in the chute walls. Weep holes with gravel pockets (fig. 2-39) may be used to relieve hydrostatic pressure on the floor and walls of the stilling pool and outlet transition.

Chute and floor blocks are provided to break up jet flow and to stabilize the hydraulic jump. The location, spacing, and details of the blocks are shown on figure 2-33.

If a concrete outlet transition is not provided, a solid end sill is required (fig. 2-40). The upstream side of the sill should have a 2 to 1 slope and the downstream side should be vertical. The elevation of the top of the sill should be set to provide tailwater for the hydraulic jump.

A stilling pool and outlet transition built to the dimensions recommended may not completely contain the spray caused by the turbulent water but the structure should contain enough of the turbulence to prevent erosion damage downstream of the structure.

(f) *Wave Formation.*—Waves in a chute structure are objectionable because they may overtop the chute walls and they cause surging in the energy dissipator. A stilling pool would not be an effective energy dissipator with this type of flow because a stable hydraulic jump could not form (fig. 2-41). An unstable pulsating type of flow known as slug flow or roll waves may develop in long, steep chute structures (figs. 2-42 and 2-43). These waves generally form in chute channels that are longer than about 200 feet and have bottom grade slopes flatter than about 20° [9]. The maximum wave height that can be expected is twice the normal depth for the slope and the maximum momentary slug flow capacity is twice the normal capacity. Transverse flow or cross waves may also develop in a chute. These waves are caused by:

(1) Abrupt transitions from one channel section to another,
(2) Unsymmetrical structures, and
(3) Curves or angles in the alinement of the chute.

The probability of these waves being generated in the structure may be reduced by following the recommendations concerning deflection angles and symmetry made in the discussions pertaining to transitions and by avoiding changes in direction in the structure.

Some chute sections are more likely to sustain waves than others. Shallow dished sections appear to be particularly susceptible to transverse flow while deep, narrow sections resist both transverse and slug flow. Chute sections that theoretically should prevent the formation of waves have been developed [9][11]. A theoretically waveless chute section is shown on figure 2-44.

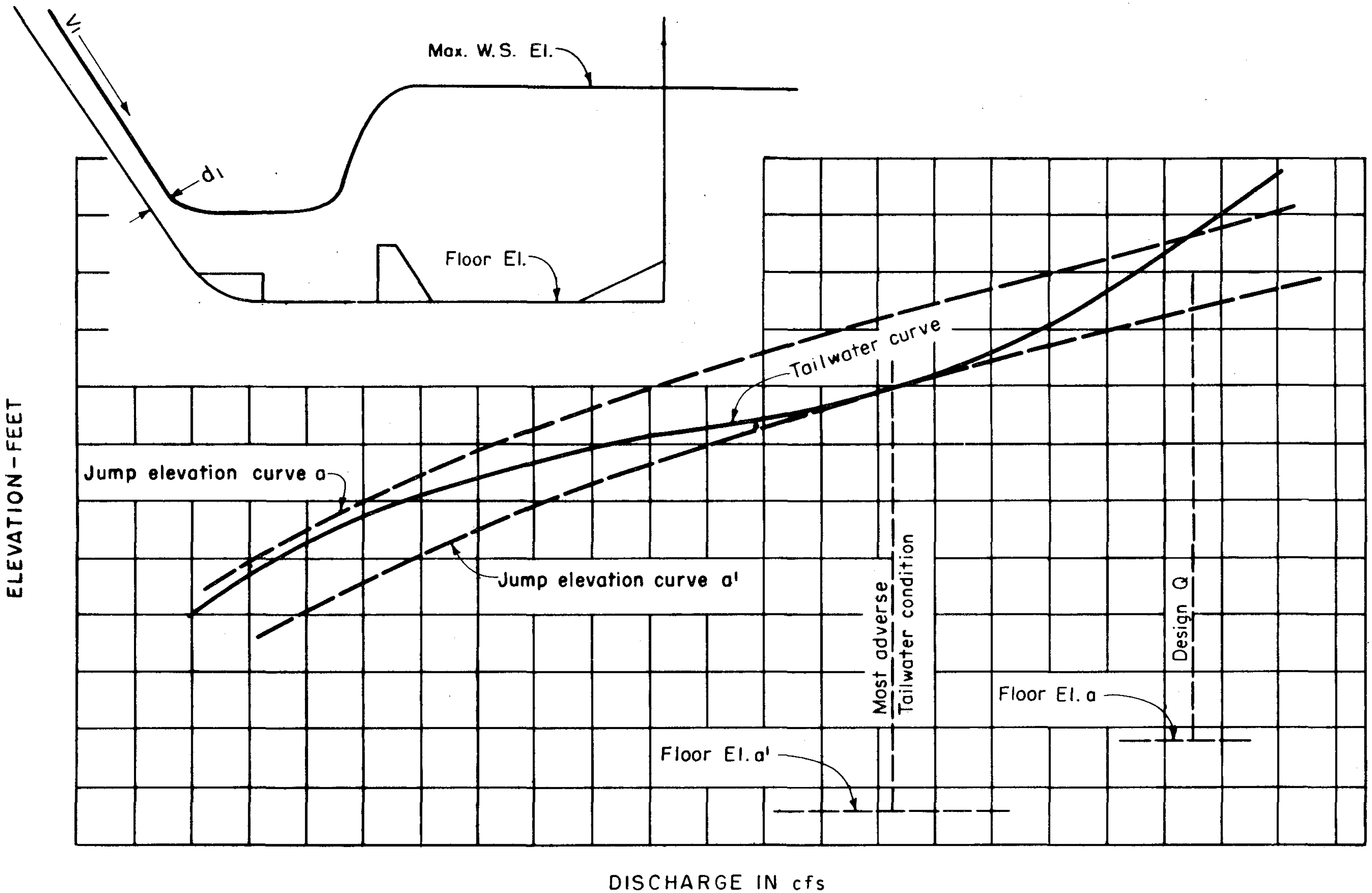
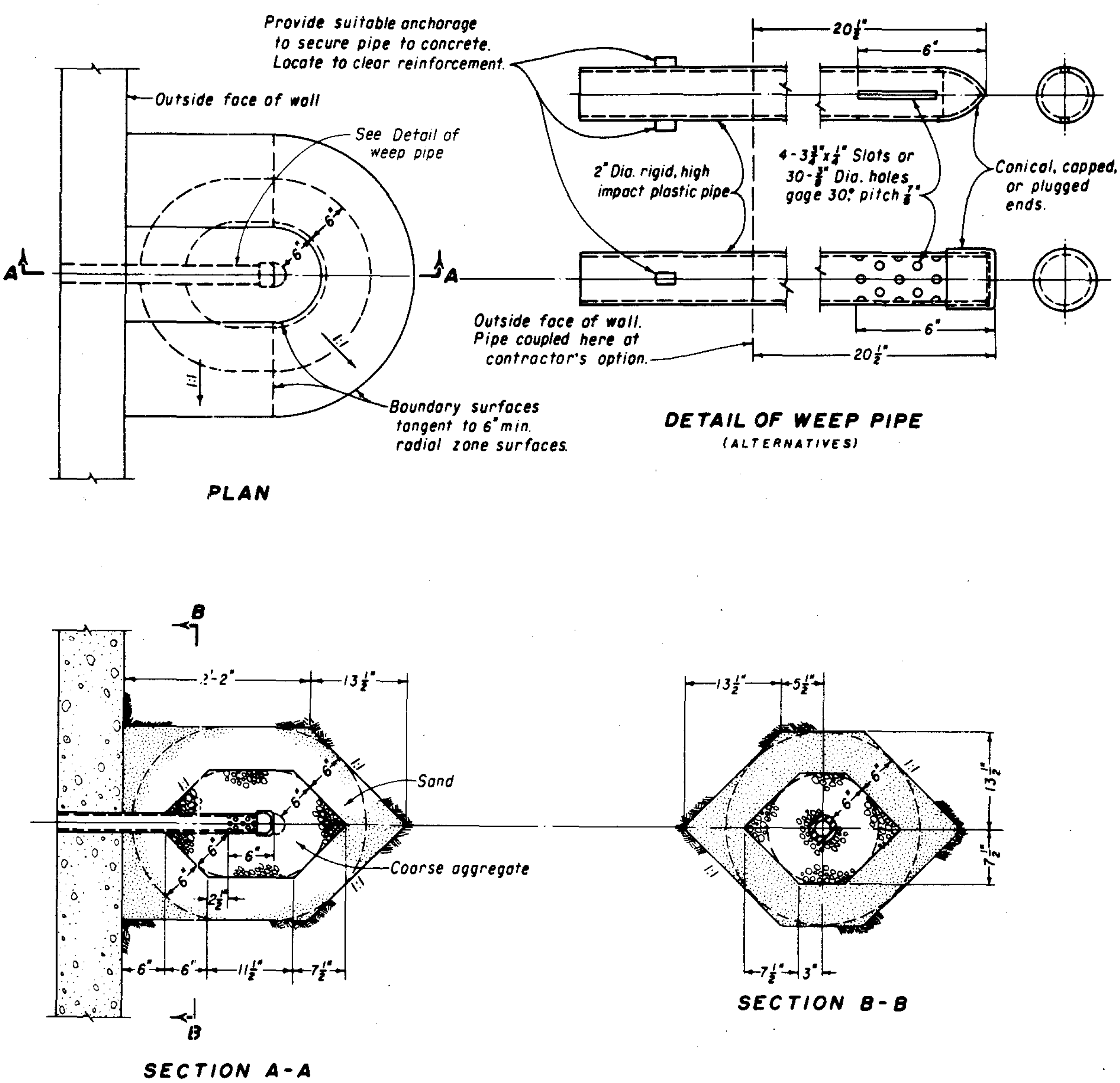


Figure 2-38. **Tailwater and jump elevation curve. 103-D-1275**



GRADATION OF FILTER MATERIALS

FILTER MATERIAL	PERCENT (BY WEIGHT) RETAINED ON STANDARD SEIVE									
	No. 200	No. 100	No. 50	No. 30	No. 16	No. 8	No. 4	3/8"	3/4"	1 1/2"
SAND	97-100	93-97	74-85	38-70	15-45	5-25	0-5			
COARSE AGGREGATE					95-100	75-90	60-80	40-60	20-35	0

NOTE

The configuration of the filter envelope may be changed to suit construction, provided that a minimum radial thickness of 6" is maintained for each zone.

Figure 2-39. Weep hole with gravel pocket. 103-D-1276

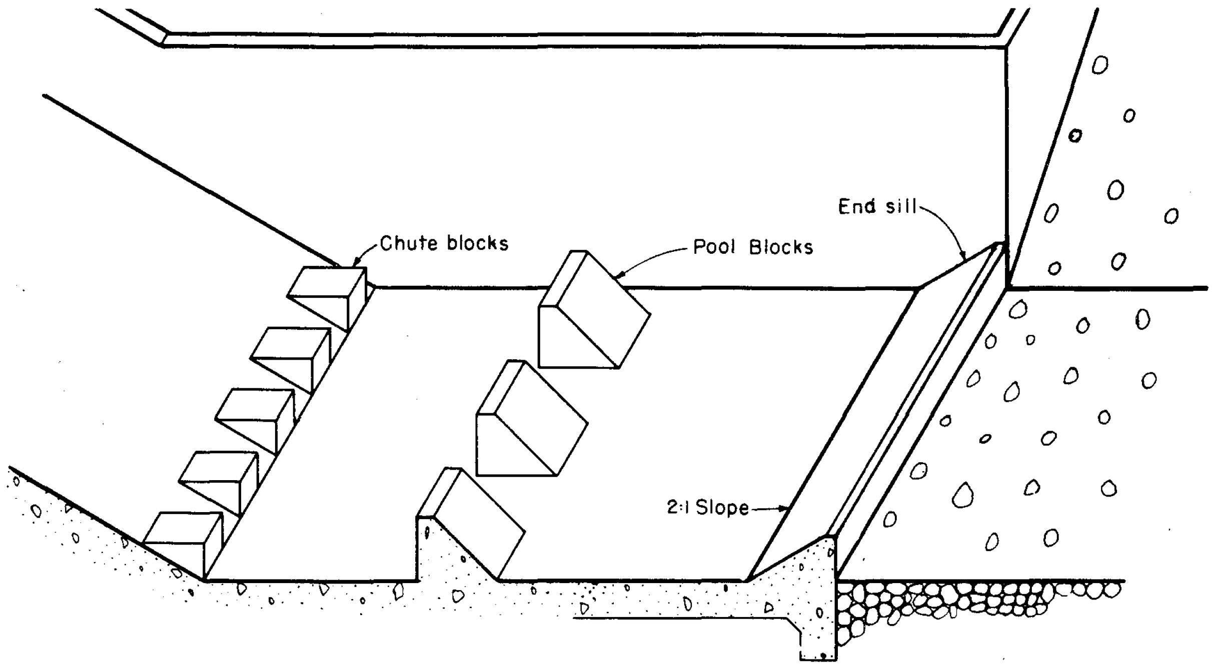


Figure 2-40. **Stilling pool with end sill. 103-D-1277**

Figures 2-45 and 2-46 have been developed to predict the possibility of slug flow occurring in a chute. The use of the figures requires the following operations:

(1) Divide the chute into a number of reaches as shown on figure 2-47. Point 1 should be at the beginning of the chute section.

(2) Using equation (6) or (7), with Q equal to 0.2, 0.5, and 1.0 times design Q, compute the water depth and energy levels at the points along the chute section that are to be checked for slugging. Use Manning's n = 0.010.

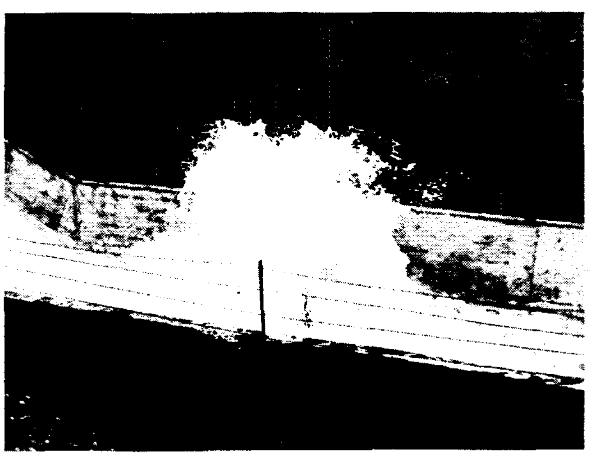


Figure 2-41. **Surging in a stilling pool caused by unstable flow in a chute structure. P222-117-36223**

(3) Determine sL for the point to be checked (fig. 2-47).

(4) At the points to be considered:

a. Compute the Vedernikov number ($\underline{V}$) [10].

$$\underline{V} = \frac{2}{3} \times \frac{b}{wp} \times \frac{V}{\sqrt{gd \cos \theta}} \tag{10}$$

b. Compute the square of the Montuori number ($\underline{M}$) [11].

$$\underline{M}^2 = \frac{V^2}{g\, sL \cos \theta} \tag{11}$$

In equations (10) and (11)

b = bottom width of the chute section

d = average water depth in the chute section:

$$d = \frac{\text{Area of the water prism}}{\text{Top width of the water prism,}}$$

Figure 2-42. **Pulsating flow in steep chute. Flow is 57 cfs. P222-117-36208D**

Figure 2-43. **Unstable flow at shallow depth. H-686-55D**

g = acceleration of gravity,
L = length of reach under consideration,
s = average slope of the energy gradient.

$$s = \tan\theta,$$

V = velocity,
wp = wetted perimeter of the section, and
θ = angle of inclination of the energy gradient.

(5) Plot the computed values on figure 2-45. If the plotted points fall within the slug flow zone, intermediate points may be checked to determine the point at which the waves begin to form.

(6) Compute the shape factor for the chute section.

$$\frac{d}{wp}$$

(7) Plot the computed value of $\frac{d}{wp}$ against the slope of the energy gradient, s, on figure 2-46.

Waves are not likely to be generated in the structure, unless the plotted data falls within the slugging zone on both figures 2-45 and 2-46.

If the charts indicate that slug flow will occur, the design may be modified to reduce the probability of waves being generated or the structure may be adapted to accommodate the

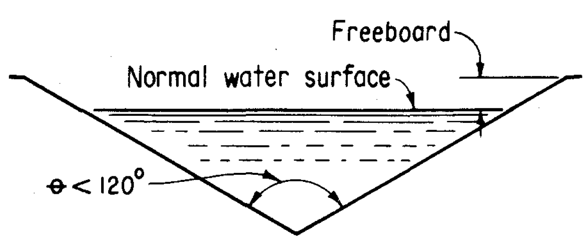


Figure 2-44. **Theoretical waveless chute section. Triangular shape prevents both cross waves and slug flow. 103-D-1278**

surging flow that does occur. Possible design changes include:

(1) Divide the flow in the chute section with a wall down the center of the chute.

(2) Change the shape of the section. The theoretical waveless shapes could be considered.

(3) Reduce the length of the chute. A series of shorter chutes or drops could be considered.

(4) Steepen the chute.

(5) Replace the open channel chute section with pipe.

If these design changes are impractical, the chute section might be adapted to accommodate surging flow by:

(1) Increasing the freeboard of the chute walls.

(2) Providing a cover for the chute section to contain the waves, or

(3) Protecting the backfill around the chute section with riprap or paving.

Adaptations to the stilling pool could include:

(1) Designing the pool to provide for the momentary surge discharge. This would

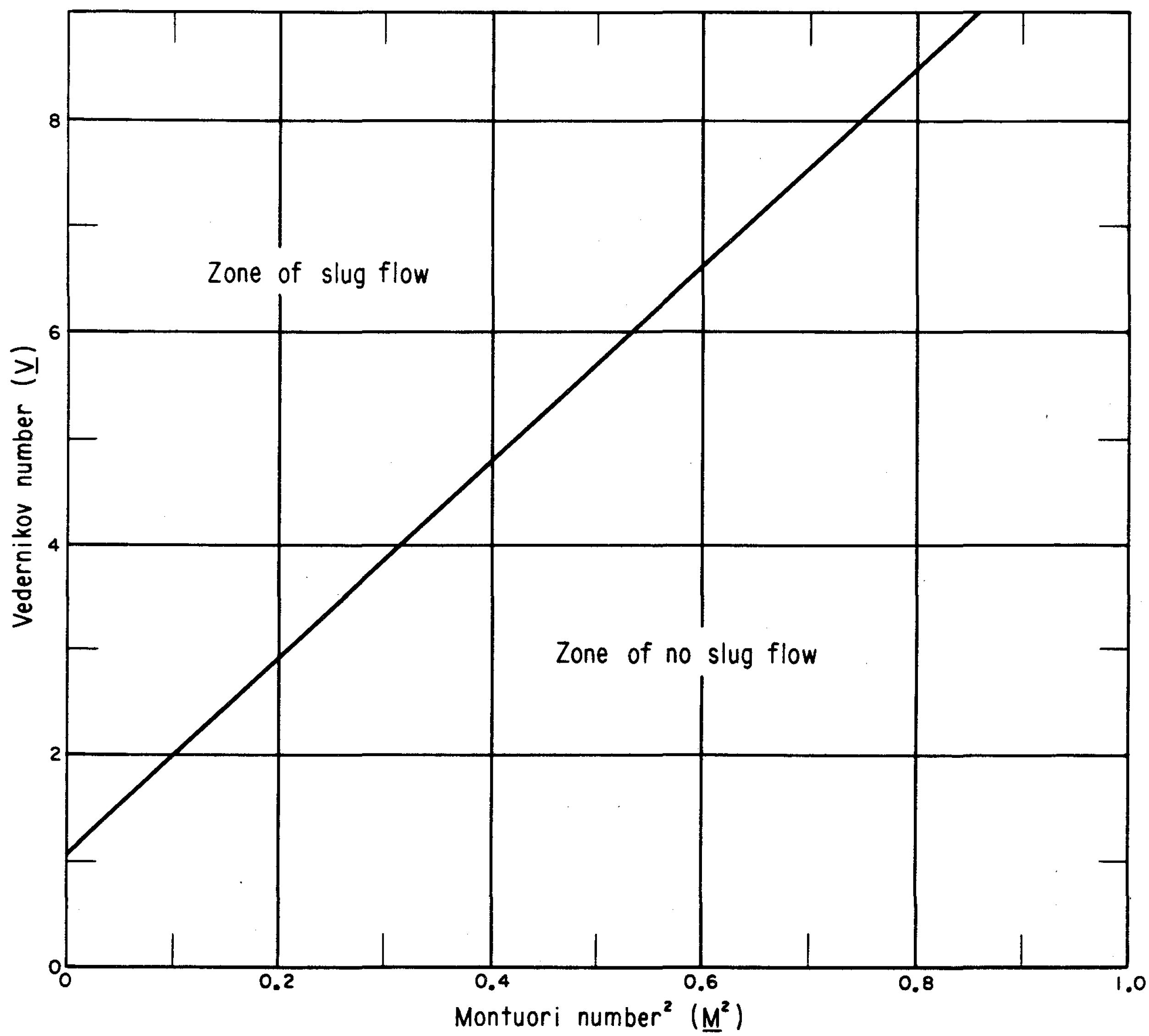


Figure 2-45. **Criteria for slug flow. 103-D-1279**

provide a longer pool and higher pool walls to contain the surge.

(2) Provide additional riprap to protect the channel downstream and the backfill around the pool.

(3) Provide a surge suppressing device in the stilling pool. A baffle or weir wall in the pool could prevent flow from sweeping through the pool and outlet transition (fig. 2-48). Weir walls could also provide tailwater to submerge the surges. Rafts or other floating wave dampeners could be used.

(4) An energy dissipator less sensitive to surging could be used.

The study of wave action in chute structures is based largely on empirical data. If a serious wave problem in a structure is indicated, further studies should be undertaken to verify the extent of the problem and the effectiveness of the proposed solutions.

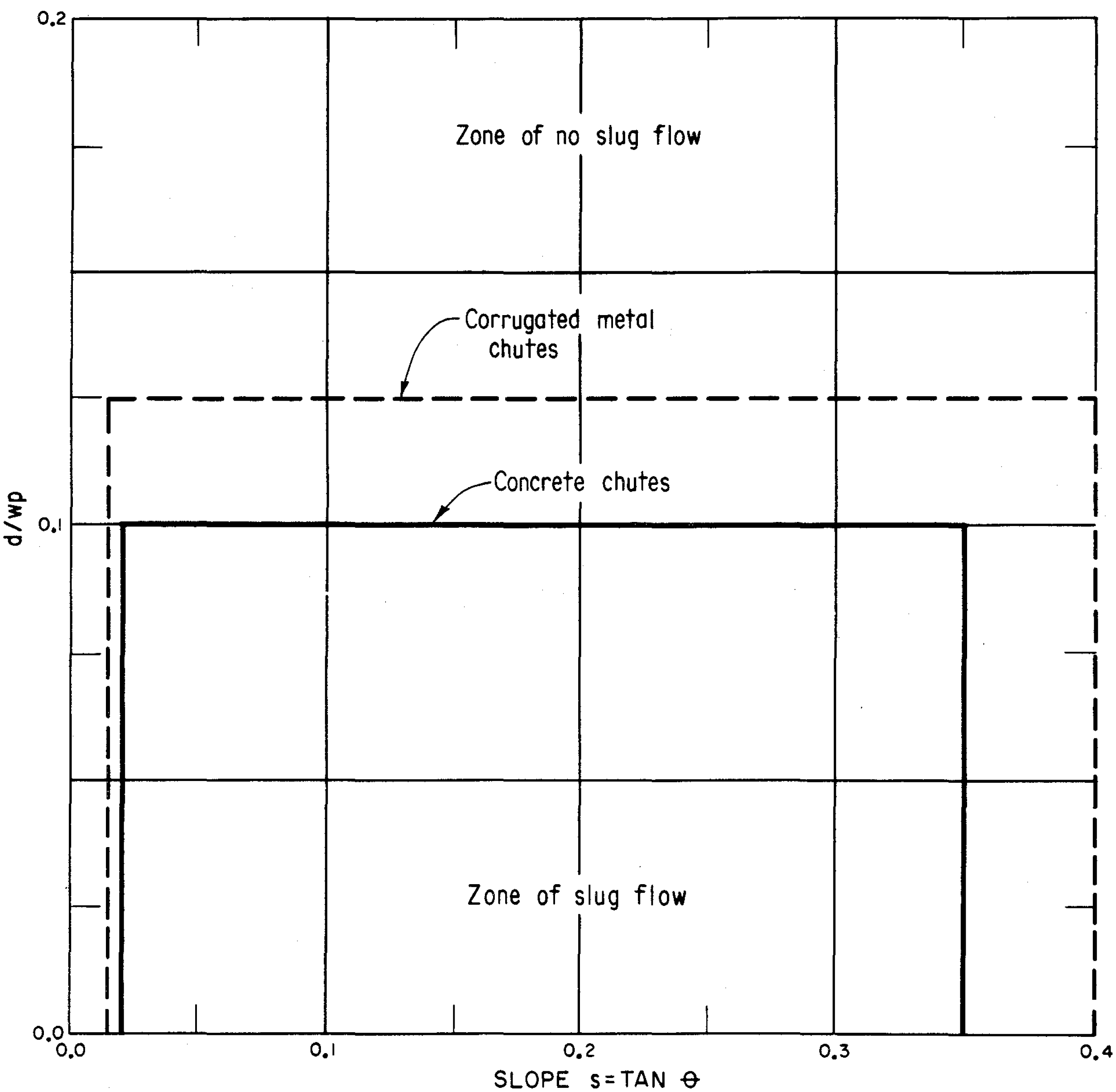


Figure 2-46. **Shape and slope criteria for slug flow. 103-D-1280**

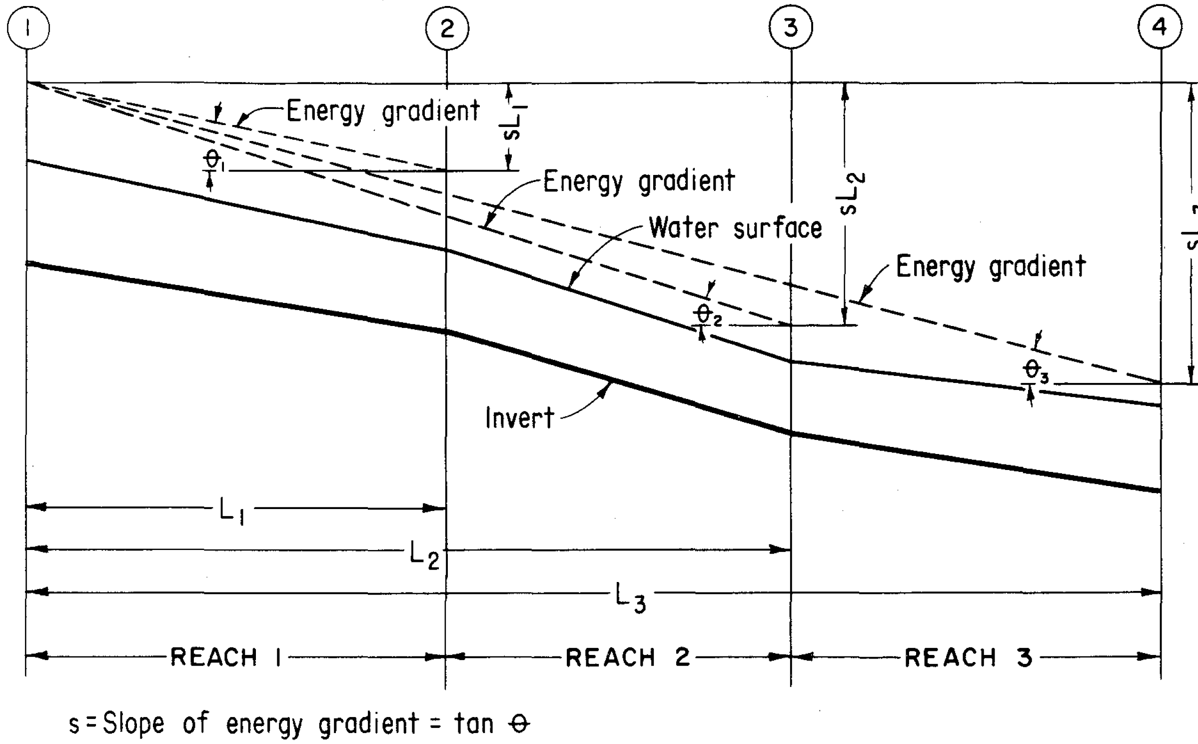


Figure 2-47. **Invert, water surface, and energy profiles of a chute structure. 103-D-1281**

2-35. *Design Procedure.*—

(1) Select and design the type of inlet to be used.

(2) Determine the energy gradient at the beginning of the chute section.

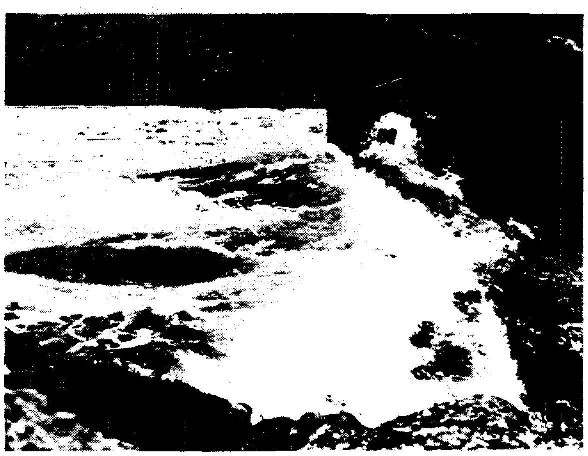


Figure 2-48. **Wave sweeping through the outlet transition of a stilling pool. Flow is 140 cfs. P222-117-36227**

(3) Compute the variables of flow down the chute section.

(4) Design the trajectory and the steep portion of the chute section.

(5) Assume an elevation for the floor of the stilling pool and compute the characteristics of flow upstream of the hydraulic jump.

Determine d_2 and the energy gradient after the hydraulic jump.

(6) Determine the energy gradient in the channel downstream and compare to the energy gradient after the jump.

(7) It may be necessary to assume a new pool invert elevation and to recompute the above values several times before a check in energy gradients is obtained.

(8) Review for proper operation at partial capacity.

(9) Determine the length of the pool and the height of the pool walls.

(10) Design the chute and floor blocks and the end sill or outlet transition as required.

(11) Check the possibility of waves developing on the structure.

(12) Provide protection in the channel downstream as required.

2-36. ***Design Example.***—Design an open channel chute that will convey 35 cfs over the profile shown on figure 2-49. Use a stilling pool to dissipate excess energy at the downstream end of the chute. The structure is to be used on a canal with a section as shown on the drawing.

Inlet Design:

The inlet is designed to provide a control for the channel upstream. Channel properties at point ① taken from figure 2-49 are:

$Q = 35$ cfs
$b = 6.0$ ft.
$d = 2.40$ ft.
$n = 0.025$
$s = 0.00035$ ft./ft., and
$s{:}s = 1.5$ to 1

If channel properties are not given, tables 2-5 and 2-6 may be used to compute flow properties.

The elevation of the energy gradient at ① is computed as follows:

$A_1 = 23.04$ ft.^{2}
$V_1 = 1.52$ f.p.s.
$h_{v_1} = 0.04$ ft.
$E_1 = d_1 + h_{v_1} = 2.40 + 0.04 = 2.44$ ft.

The elevation of the energy gradient in the channel upstream (point ①) equals the invert elevation + E_1 or 3703.18 + 2.44 = 3705.62.

Assume that critical depth will occur at ②. With a discharge of 35 cfs, a chute section width of 3 feet is a reasonable choice. The elevation of the invert at ② will be:

$$d_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{11.7^2}{32.2}} = 1.62 \text{ ft.}$$

$A_c = 4.86$ ft.^{2}
$V_c = 7.20$ ft.
$h_{v_c} = 0.80$ ft.
$R_c = 0.78$ ft.

For $n = 0.010$:

$$s_c = \left(\frac{7.20 \times 0.010}{1.486 \times 0.847}\right)^2 = 0.0033,$$

$$E_c = d_c + h_{v_c} = 1.62 + 0.80 = 2.42 \text{ ft.}$$

Losses in the inlet transition are (1) a convergence loss which is assumed to be 0.2 times the change in velocity head between the beginning of the transition and the end of the transition and (2) a friction loss equal to the average friction slope in the inlet times the length of the inlet.

Inlet losses:
Convergence loss = 0.2 x (0.80 – 0.04) = 0.15 ft.

With a transition 10.0 feet long the friction loss will be:

$$\frac{0.00035 + 0.0033}{2} \times 10 = 0.02 \text{ foot}$$

To balance the energy in the canal upstream, the invert of the inlet at ② must equal:

$3705.62 - E_c -$ Transition losses

or

$3705.62 - 2.42 - 0.15 - 0.02 = 3703.03$

An elevation of 3703.00 at ② will provide a control for flow into the chute channel.

Determine the maximum angle of deflection of the inlet side walls:

From equation (1), cotangent $\alpha = 3.375 \times F$.

$$F = \frac{V}{\sqrt{[(1-K)g]\ d \cos\theta}}$$

$K = 0$, $\cos\theta = 0.99984$

$$F_1 = \frac{2.43}{\sqrt{32.2 \times 2.40 \times 0.99984}} = 0.276$$

$$F_2 = \frac{7.2}{\sqrt{32.2 \times 1.62 \times 0.99984}} = 1.00$$

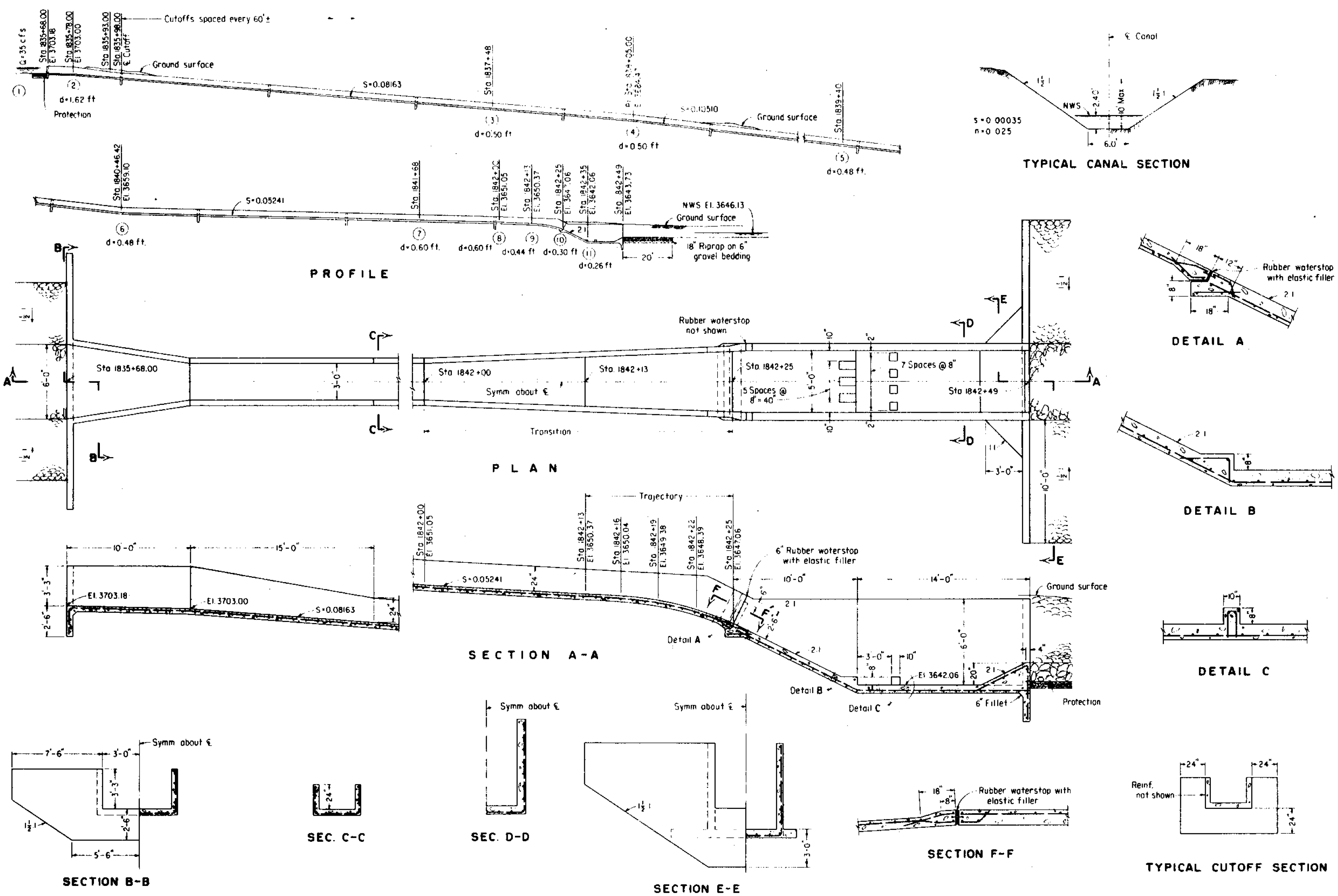


Figure 2-49. **Chute with stilling pool. 103-D-1282**

The mean value of F = 0.64
Cotangent α = 3.375 x 0.64 = 2.16
α = about 25°.

With a transition 10 feet long the deflection angle will be about 8.5° which indicates that waves will not be initiated in the inlet.

Determine the flow in the chute section:

The flow at ② is critical flow. Flow characteristics in the chute section are computed using Bernoulli's equation (6) to balance the energy at various points on the chute section. Uniform flow is at a depth of 0.50 feet on a slope of 0.08163. This depth will be reached at ③, 170.0 feet from ②.

The energy at ② will be:

$E_2 = d_1 + h_{v_1} + Z$
$Z = s \times L = 0.08163 \times 170$
$= 13.88$ ft.
$E_2 = 1.62 + 0.80 + 13.88$
$= 16.30$ ft.

The energy at ③ will be:

$E_3 = d_2 + h_{v_2} + h_f$
h_f = friction loss in the reach,
h_f = average friction slope in the reach times L,
$h_f = s_a \times L$.
$d_3 = 0.50$ ft.
$A_3 = 1.50$ ft.^{2}
$V_3 = 23.33$ f.p.s.
$h_{v_3} = 8.45$ ft.
$s_3 = 0.08163$

$$s_a = \frac{0.08163 + 0.0033}{2} = 0.0425$$

$h_f = 0.0425 \times 170 = 7.23$ ft
$E_3 = 0.50 + 8.45 + 7.23$
$= 16.18$ feet
E_3 balances E_2 for practical purposes.

The flow between ③ and ④ is uniform flow with the loss in elevation Z equal to the friction loss, h_f, in the reach.

For flow between ④ and ⑥:

The normal depth on a slope of 0.10510 is 0.48 foot. This depth is reached at ⑤ and the energy levels at ④ and ⑤ balance. Between points ⑤ and ⑥ the flow is uniform at a depth of 0.48 foot.

For flow between ⑥ and ⑧:

A normal depth of 0.60 foot is reached at point ⑦ and the flow between ⑦ and ⑧ is uniform flow at a depth of 0.60 foot.

For the water depths that will occur on this chute section a minimum wall height of 24 inches will provide the required 12 inches of freeboard.

Design the trajectory:

Flow characteristics on the trajectory and the steep slope section are computed in the same way as those on the chute section. Computed water depths are shown on figure 2-49.

Use a transition 25.0 feet long to flare the bottom width from 3 to 5 feet. Flow characteristics at the beginning of the transition (point ⑧) are:

$d_8 = 0.60$ ft.
$A_8 = 1.80$ ft.^{2}
$V_8 = 19.44$ f.p.s.
$h_{v_8} = 5.86$ ft.
$R_8 = 0.43$ ft., and
$S_8 = 0.05241$ ft./ft.

At the beginning of the trajectory (point ⑨) flow properties are:

$d_9 = 0.44$ ft.
$A_9 = 1.78$ ft.^{2}
$V_9 = 19.66$ f.p.s.
$h_{v_9} = 6.00$ ft.
$R_9 = 0.36$ ft., and
$S_9 = 0.0683$ ft./ft.

The value of K used to compute the trajectory is limited to 0.5. The minimum length of trajectory that will provide a satisfactory value for K is obtained using equation (4):

$$L_T = \frac{(0.5 - 0.0524) \times 2 \times 6.00 \times 0.99863}{0.5}$$

$$L_T = 10.72 \text{ feet}$$

Make the trajectory 12.0 feet long. Coordinates of points on the trajectory are computed using equation (8).

X, feet	Y, feet
3	0.33
6	0.99
9	1.98
12	3.31

At the lower end of the transition and the trajectory (point 10) the characteristics of flow will be:

$$\begin{aligned} d_{10} &= 0.30 \text{ ft.} \\ A_{10} &= 1.50 \text{ ft.}^2 \\ V_{10} &= 23.33 \text{ f.p.s.,} \\ R_{10} &= 0.27 \text{ ft., and} \\ S_{10} &= 0.14107 \end{aligned}$$

The maximum angle of deflection in the sidewalls of the transition is determined from equation (1):

F_8 at 8 with K = 0.0.

$$F_8 = \frac{19.44}{\sqrt{32.2 \times 0.60 \times 0.99863}} = 4.43$$

F_{10} at 10 with K determined from equation (4):

$$K_{10} = \frac{(0.50-0.052) \times 2 \times 6.0 \times 0.999^2}{12}$$
$$= 0.45$$

$$F_{10} = \frac{23.33}{\sqrt{[(1-0.45) \times 32.2] \times 0.30 \times 0.89441}}$$
$$= 10.70$$

$$F_a = \frac{4.43 + 10.70}{2} = 7.56$$

Cotangent α = 3.375 x 7.56 = 25.52
$\alpha = 2^o 15' \pm$

The deflection angle with a transition 25.0 feet long will be:

$$\text{tangent } \alpha = \frac{1}{25} = 0.04$$
$$\alpha = 2^o 15' \pm$$

The deflection angle in the transition sidewall is satisfactory.

Design the Stilling Pool:

An elevation for the invert of the stilling pool must be assumed before the flow properties at the bottom of the steep slope section can be computed. Assume this elevation to be 3642.06. Balancing the energies between the end of the trajectory 10 and the bottom of the steep slope section 11 gives the following flow properties at the bottom of the steep slope section or immediately ahead of the hydraulic jump:

$$\begin{aligned} d_{11} &= 0.26 \text{ ft.,} \\ A_{11} &= 1.30 \text{ ft.}^2, \\ V_{11} &= 26.92 \text{ f.p.s., and} \\ h_{v_{11}} &= 11.25 \text{ ft.} \end{aligned}$$

The Froude number at this point =

$$F = \frac{26.92}{\sqrt{32.2 \times 0.26}} = 9.30$$

The Froude number is within the range in which a stilling pool may be expected to operate effectively as an energy dissipator. The water depth after the jump d_2 is computed from equation (9):

$$d_2 = -\frac{0.26}{2} + \sqrt{\frac{2 \times 26.92^2 \times 0.26}{32.2} + \frac{0.26^2}{4}}$$
$$= 3.29 \text{ feet}$$

The flow characteristics after the jump are:

$$\begin{aligned} A_2 &= 16.45 \text{ ft.}^2 \\ V_2 &= 2.13 \text{ f.p.s.} \\ h_{v_2} &= 0.07 \text{ ft.} \\ E_2 &= 3.29 + 0.07 = 3.36 \text{ ft.} \end{aligned}$$

The elevation of the energy gradient after the jump = 3642.06 + 3.36 = 3645.42. This energy must be balanced by the energy in the channel downstream computed with the n for the channel reduced by 20 percent.

Energy downstream of the structure:

$Q = 35$ cfs
$n = 0.025 \times 0.80 = 0.020$
$b = 6.0$ ft.
$d = 2.16$ ft.
$A = 19.96$ ft.^{2}
$V = 1.75$ f.p.s.
$h_v = 0.05$ ft.
$E = 2.16 + 0.05 = 2.21$ ft.

The minimum elevation of the canal invert required to balance the energy after the jump is:

$$3645.42 - 2.21 = 3643.21$$

The invert elevation shown on figure 2-49 is 3643.73. The energies balance, therefore the elevation assumed for the floor of the stilling pool is satisfactory. Generally several trials with different assumed elevations for the pool floor or with different pool widths must be made before a check in energy levels is obtained.

The length of the stilling pool should be about four times d_2 or 4 x 3.29 or 13.16 feet. Use a length of 14 feet. From figure 2-33 the freeboard allowance for the stilling pool should be about 2.0 feet. This freeboard should be above the maximum downstream energy level. Make the chute walls 6 feet 0 inch high. Chute and pool blocks are sized and positioned as shown on figure 2-33.

Design the Outlet Transition:

When required, a concrete outlet transition is used to transition the flow from the stilling pool to the downstream channel. In this design example an outlet transition is not used. An end sill is provided at the end of the stilling pool and the elevation at the top of the sill is set to provide tailwater for the hydraulic jump. Critical energy at the end of the stilling pool is:

$$d_c = 1.23 \text{ ft.}$$
$$h_{v_c} = 0.50 \text{ ft.}$$
$$E_c = 1.73 \text{ ft.}$$

The minimum height of sill required to provide a control for the downstream flow equals the energy after the jump, E_2, minus the critical energy at the end of the pool, E_c, or $3.36 - 1.73 = 1.63$ feet. A sill height of 1.67 feet is used in the design example.

In parts of the structure where the velocity of the water is high the minimum concrete cover on the top layer of reinforcement steel may be increased according to the velocity as follows:

Velocity, f.p.s.	Increase in the minimum concrete cover, inches
10	0
20	0.5
30	1.0

A transition in the downstream channel and protection for the channel, where they are required, complete the structure.

Check for slug flow:

The possibility of waves forming on the structure should be checked at points ④ , ⑥ , and ⑧ . At ⑧ (Sta. 1842+00) the invert elevation is 3651.05. A check at design discharge will indicate that waves will not form on the structure. The following check at 0.5 times design discharge demonstrates the steps required to check for slug flow.

For Q = 17.5 cfs:

(1) At point ② (Sta. 1835+78 with an invert elevation of 3703.00)

$$d_c = \sqrt[3]{\frac{5.83^2}{32.2}} = 1.02 \text{ ft.}$$
$$A_c = 3.06 \text{ ft.}^2$$
$$V_c = 5.72 \text{ f.p.s.}$$
$$h_{v_c} = 0.51 \text{ ft.}$$

The elevation of the energy gradient at ② is:

$$3703.00 + 1.02 + 0.51 = 3704.53.$$

(2) At point ⑧ with Q = 17.5 cfs.
Using equation (6) the flow characteristics

between ② and ⑧ are computed.

At ⑧ :

$$d = 0.38 \text{ ft.}$$
$$A = 1.14 \text{ ft.}^2$$
$$V = 15.35 \text{ f.p.s.}$$
$$h_v = 3.66 \text{ ft.}$$
$$wp = 3.76 \text{ ft.}$$

The elevation of the energy gradient at ⑧ = 3651.05 + 0.38 + 3.66 = 3655.09.

(3) Determine sL (fig. 2-47):

$$sL = 3704.53 - 3655.09 = 49.44 \text{ ft.}$$

(4) Determine L and s:

$$L = 1842 + 00 - 1835 + 78$$
$$= 622 \text{ ft.}$$
$$s = \tan\theta = \frac{49.44}{622.0} = 0.07949$$
$$\theta = \text{about } 4^{\circ}\ 33'$$

$$\cos\theta = 0.99685$$

(5) Using equation (10), compute $\underline{V}$:

$$\underline{V} = \frac{2}{3} \times \frac{3}{3.76} \times \frac{15.35}{\sqrt{32.2 \times 0.38 \times 0.99685}}$$

$$\underline{V} = 2.34$$

(6) Using equation (11) compute $\underline{M}^2$:

$$\underline{M}^2 = \frac{15.35^2}{32.2 \times 49.44 \times 0.99685} = 0.148$$

(7) Plotted on figure 2-45, the point falls near the slugging zone.

(8) Check shape and slope parameters:

$$\frac{d}{wp} = \frac{0.38}{3.76} = 0.101$$
$$s = 0.07949$$

(9) Plotted on figure 2-46, the point falls near the zone of slug flow.

(10) Both charts indicate that waves may form on this structure at 0.5 times Q. In this example problem with the shallow depths involved, slug flow may not be a problem if it does occur. If slug flow is indicated where the waves would be troublesome, the remedies listed in the text should be considered.

2. *Pipe Chute*

2-37. ***Pipe Chute.***—In a pipe chute the open chute section is replaced by pipe (fig. 2-50). Pipe chutes may be used to provide a crossing

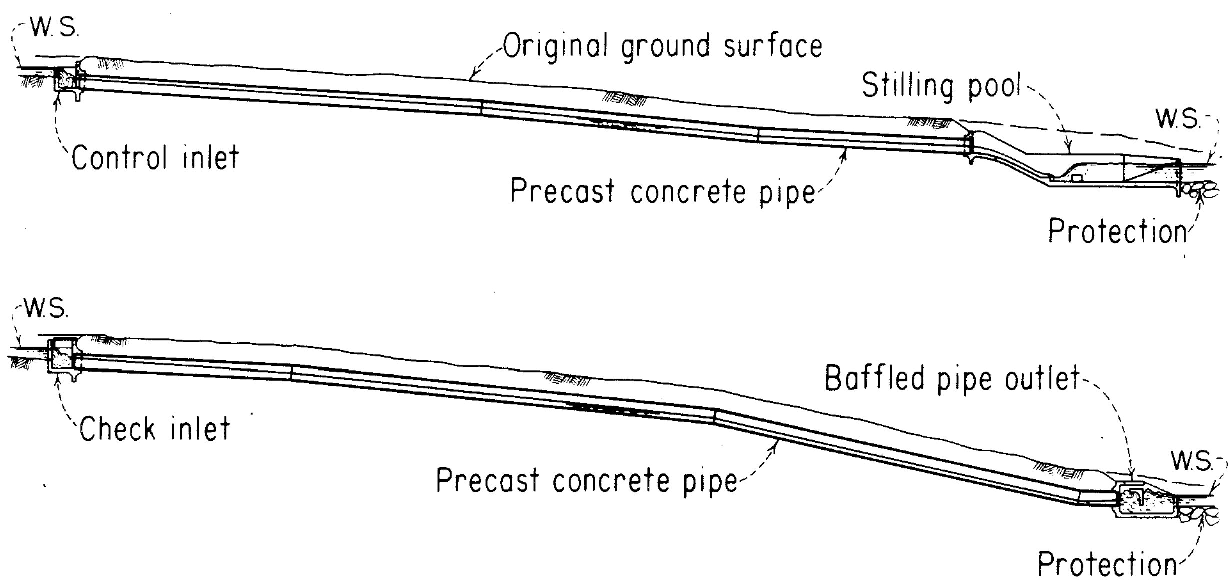


Figure 2-50. **Pipe chutes. 103-D-1283**

or to allow farming or grazing over the structure. A pipe structure is largely underground and may be desirable from an esthetic point of view. Baffled outlets or stilling pools are used as energy dissipators. Figure 2-51 shows a stilling pool at the bottom of a pipe chute. The procedure for designing a pipe chute is similar to that used in designing an open channel chute. The inlet transition should be designed to prevent the flow at the entrance to the pipe from controlling the flow through the structure. Concrete transitions between open channels and pipe sections are discussed in chapter VII. The pipe is sized to allow a maximum full pipe velocity of 12 feet per second. Air in a closed conduit can cause serious problems and care must be exercised in choosing the slopes for the pipe section. The pipe slopes selected should prevent a hydraulic jump from occurring in the pipe. The Bureau of Reclamation requires that for pipe slopes steeper than critical the minimum slope should be two times the critical slope. The pipe slope should not be changed from a steeper slope to a flatter slope. If a change to a flatter slope must be made, the pipe should be transitioned to an open chute channel for a short reach and the change in grade made in a section open to the atmosphere.

Flow variables for circular sections flowing partly full may be computed using table 2-7. For concrete pipe use Manning's n = 0.010.

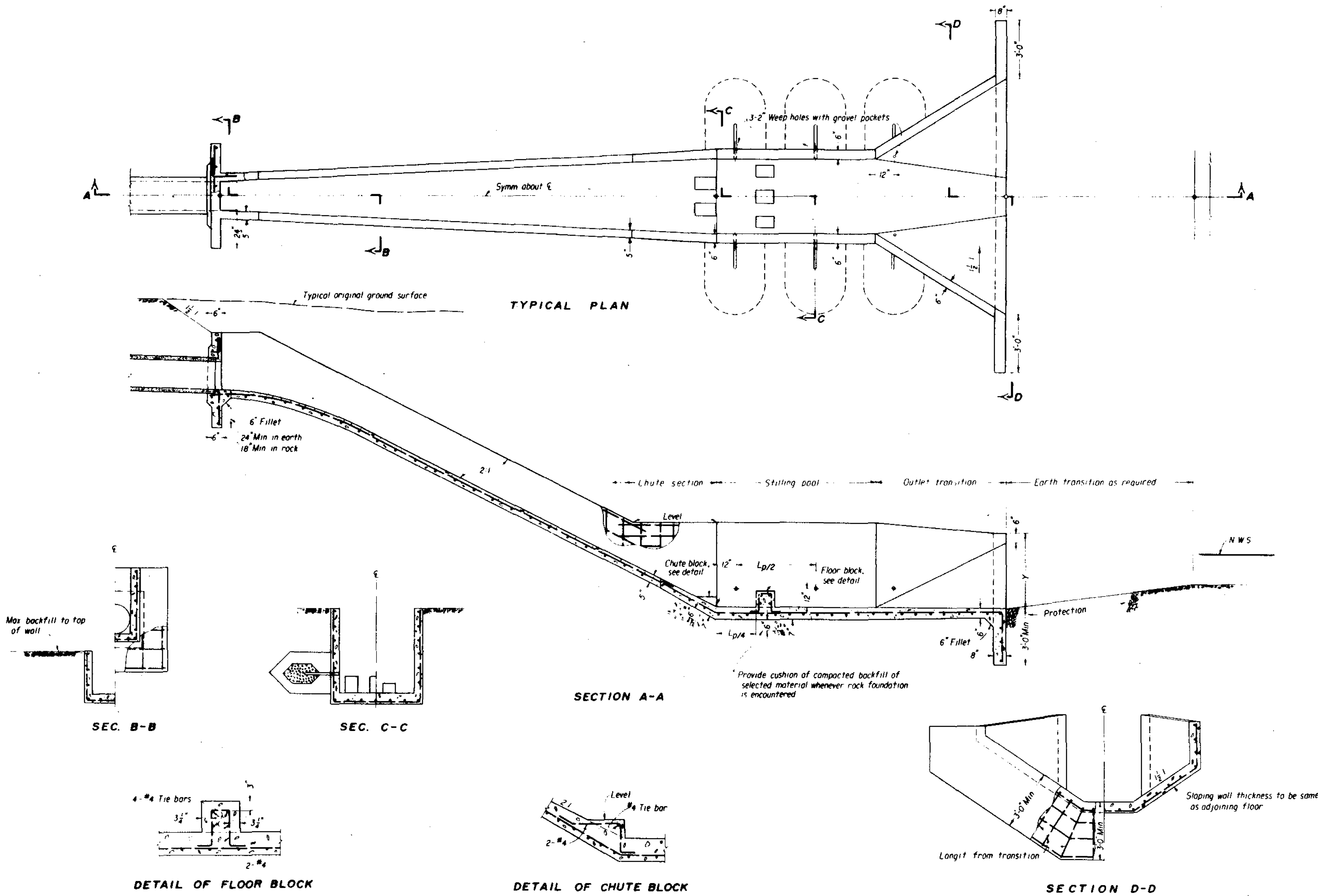
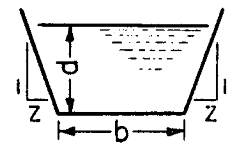


Figure 2-51. **Stilling pool on pipe chute. 103-D-1284**

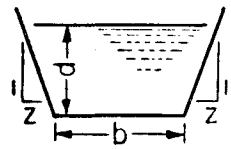
Table 2-5.—*Uniform flow in trapezoidal channels by Manning's formula.* 103-D-1285-1



d/b[1]	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.02	.00213	.00215	.00216	.00217	.00218	.00219	.00220	.00220	.00221	.00223
.03	.00414	.00419	.00423	.00426	.00429	.00431	.00433	.00434	.00437	.00443
.04	.00661	.00670	.00679	.00685	.00690	.00696	.00700	.00704	.00707	.00722
.05	.00947	.00964	.00980	.00991	.0100	.0101	.0102	.0103	.0103	.0106
.06	.0127	.0130	.0132	.0134	.0136	.0137	.0138	.0140	.0141	.0145
.07	.0162	.0166	.0170	.0173	.0176	.0177	.0180	.0182	.0183	.0190
.08	.0200	.0206	.0211	.0215	.0219	.0222	.0225	.0228	.0231	.0240
.09	.0240	.0249	.0256	.0262	.0267	.0271	.0275	.0279	.0282	.0296
.10	.0283	.0294	.0305	.0311	.0318	.0324	.0329	.0334	.0339	.0358
.11	.0329	.0342	.0354	.0364	.0373	.0380	.0387	.0394	.0400	.0424
.12	.0376	.0393	.0408	.0420	.0431	.0441	.0450	.0458	.0466	.0497
.13	.0425	.0446	.0464	.0480	.0493	.0505	.0516	.0527	.0537	.0575
.14	.0476	.0501	.0524	.0542	.0559	.0573	.0587	.0599	.0612	.0659
.15	.0528	.0559	.0585	.0608	.0628	.0645	.0662	.0677	.0692	.0749
.16	.0582	.0619	.0650	.0676	.0699	.0720	.0740	.0759	.0776	.0845
.17	.0638	.0680	.0717	.0748	.0775	.0800	.0823	.0845	.0867	.0947
.18	.0695	.0744	.0786	.0822	.0854	.0883	.0910	.0936	.0961	.105
.19	.0753	.0809	.0857	.0900	.0936	.0970	.100	.103	.106	.117
.20	.0813	.0875	.0932	.0979	.102	.106	.110	.113	.116	.129
.21	.0873	.0944	.101	.106	.111	.115	.120	.123	.127	.142
.22	.0935	.101	.109	.115	.120	.125	.130	.134	.139	.155
.23	.0997	.109	.117	.124	.130	.135	.141	.146	.151	.169
.24	.106	.116	.125	.133	.139	.146	.152	.157	.163	.184
.25	.113	.124	.133	.142	.150	.157	.163	.170	.176	.199
.26	.119	.131	.142	.152	.160	.168	.175	.182	.189	.215
.27	.126	.139	.151	.162	.171	.180	.188	.195	.203	.232
.28	.133	.147	.160	.172	.182	.192	.201	.209	.217	.249
.29	.139	.155	.170	.182	.193	.204	.214	.223	.232	.267
.30	.146	.163	.179	.193	.205	.217	.227	.238	.248	.286
.31	.153	.172	.189	.204	.217	.230	.242	.253	.264	.306
.32	.160	.180	.199	.215	.230	.243	.256	.269	.281	.327
.33	.167	.189	.209	.227	.243	.257	.271	.285	.298	.348
.34	.174	.198	.219	.238	.256	.272	.287	.301	.315	.369
.35	.181	.207	.230	.251	.270	.287	.303	.318	.334	.392
.36	.190	.216	.241	.263	.283	.302	.319	.336	.353	.416
.37	.196	.225	.251	.275	.297	.317	.336	.354	.372	.440
.38	.203	.234	.263	.289	.311	.333	.354	.373	.392	.465
.39	.210	.244	.274	.301	.326	.349	.371	.392	.412	.491
.40	.218	.254	.286	.314	.341	.366	.389	.412	.433	.518
.41	.225	.263	.297	.328	.357	.383	.408	.432	.455	.545
.42	.233	.279	.310	.342	.373	.401	.427	.453	.478	.574
.43	.241	.282	.321	.356	.389	.418	.447	.474	.501	.604
.44	.249	.292	.334	.371	.405	.437	.467	.496	.524	.634

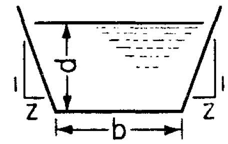
[1] For d/b less than 0.04, use of the assumption R = d is more convenient and more accurate than interpolation in the table.

Table 2-5.–*Uniform flow in trapezoidal channels by Manning's formula.*–Continued. 103-D-1285-2



d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	.256	.303	.346	.385	.422	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.574	.696
.47	.271	.323	.371	.417	.457	.494	.530	.565	.600	.729
.48	.279	.333	.384	.432	.475	.514	.552	.589	.626	.763
.49	.287	.345	.398	.448	.492	.534	.575	.614	.652	.797
.50	.295	.356	.411	.463	.512	.556	.599	.639	.679	.833
.52	.310	.377	.438	.496	.548	.599	.646	.692	.735	.906
.54	.327	.398	.468	.530	.590	.644	.696	.746	.795	.984
.56	.343	.421	.496	.567	.631	.690	.748	.803	.856	1.07
.58	.359	.444	.526	.601	.671	.739	.802	.863	.922	1.15
.60	.375	.468	.556	.640	.717	.789	.858	.924	.988	1.24
.62	.391	.492	.590	.679	.763	.841	.917	.989	1.06	1.33
.64	.408	.516	.620	.718	.809	.894	.976	1.05	1.13	1.43
.66	.424	.541	.653	.759	.858	.951	1.04	1.13	1.21	1.53
.68	.441	.566	.687	.801	.908	1.01	1.10	1.20	1.29	1.64
.70	.457	.591	.722	.842	.958	1.07	1.17	1.27	1.37	1.75
.72	.474	.617	.757	.887	1.01	1.13	1.24	1.35	1.45	1.87
.74	.491	.644	.793	.932	1.07	1.19	1.31	1.43	1.55	1.98
.76	.508	.670	.830	.981	1.12	1.26	1.39	1.51	1.64	2.11
.78	.525	.698	.868	1.03	1.18	1.32	1.46	1.60	1.73	2.24
.80	.542	.725	.906	1.08	1.24	1.40	1.54	1.69	1.83	2.37
.82	.559	.753	.945	1.13	1.30	1.47	1.63	1.78	1.93	2.51
.84	.576	.782	.985	1.18	1.36	1.54	1.71	1.87	2.03	2.65
.86	.593	.810	1.03	1.23	1.43	1.61	1.79	1.97	2.14	2.80
.88	.610	.839	1.07	1.29	1.49	1.69	1.88	2.07	2.25	2.95
.90	.627	.871	1.11	1.34	1.56	1.77	1.98	2.17	2.36	3.11
.92	.645	.898	1.15	1.40	1.63	1.86	2.07	2.28	2.48	3.27
.94	.662	.928	1.20	1.46	1.70	1.94	2.16	2.38	2.60	3.43
.96	.680	.960	1.25	1.52	1.78	2.03	2.27	2.50	2.73	3.61
.98	.697	.991	1.29	1.58	1.85	2.11	2.37	2.61	2.85	3.79
1.00	.714	1.02	1.33	1.64	1.93	2.21	2.47	2.73	2.99	3.97
1.05	.759	1.10	1.46	1.80	2.13	2.44	2.75	3.04	3.33	4.45
1.10	.802	1.19	1.58	1.97	2.34	2.69	3.04	3.37	3.70	4.96
1.15	.846	1.27	1.71	2.14	2.56	2.96	3.34	3.72	4.09	5.52
1.20	.891	1.36	1.85	2.33	2.79	3.24	3.68	4.09	4.50	6.11
1.25	.936	1.45	1.99	2.52	3.04	3.54	4.03	4.49	4.95	6.73
1.30	.980	1.54	2.14	2.73	3.30	3.85	4.39	4.90	5.42	7.39
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.76	5.34	5.90	8.10
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	5.80	6.43	8.83
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.29	6.98	9.62

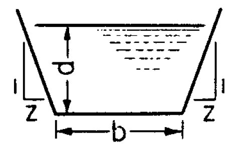
Table 2-5.—*Uniform flow in trapezoidal channels by Manning's formula.*—Continued. 103-D-1285-3



d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	6.81	7.55	10.4
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	7.33	8.14	11.3
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	7.89	8.79	12.2
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	8.47	9.42	13.2
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	9.08	10.1	14.2
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	9.72	10.9	15.2
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	10.4	11.6	16.3
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	11.1	12.4	17.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	12.4	13.2	18.7
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	12.5	14.0	19.9
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	13.3	14.9	21.1
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	15.0	16.8	23.9
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	16.7	18.7	26.8
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	18.6	20.9	30.0
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	20.6	23.1	33.4
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	22.7	25.6	37.0
2.60	2.16	4.90	8.14	11.6	15.0	18.4	21.7	25.0	28.2	40.8
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	27.4	31.0	44.8
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	29.9	33.8	49.1
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	32.6	36.9	53.7
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	35.4	40.1	58.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	41.5	47.1	68.9
3.40	2.90	7.97	14.2	21.0	27.9	34.8	41.5	48.2	54.6	80.2
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	55.5	63.0	92.8
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	63.5	72.4	107
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	72.1	82.2	122
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	96.9	111	164
5.00	4.39	16.7	33.1	51.5	70.3	89.2	108	126	145	216

Table 2-6.—*Uniform flow in trapezoidal channels by Manning's formula.* 103-D-1286-1

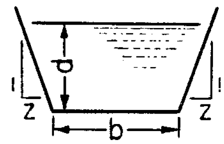
Values of $\frac{Qn}{d^{8/3} S^{1/2}}$



d/b[1]	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.01	146.7	147.2	147.6	148.0	148.3	148.6	148.8	148.9	149.2	149.9
.02	72.4	72.9	73.4	73.7	74.0	74.3	74.5	74.8	74.9	75.6
.03	47.6	48.2	48.6	49.0	49.3	49.5	49.8	50.0	50.2	50.9
.04	35.3	35.8	36.3	36.6	36.9	37.2	37.4	37.6	37.8	38.6
.05	27.9	28.4	28.9	29.2	29.5	29.8	30.0	30.2	30.5	31.2
.06	23.0	23.5	23.9	24.3	24.6	24.8	25.1	25.3	25.5	26.3
.07	19.45	19.97	20.4	20.8	21.1	21.3	21.6	21.8	22.0	22.8
.08	16.82	17.34	17.73	18.13	18.43	18.70	18.95	19.18	19.40	20.2
.09	14.78	15.29	15.72	16.08	16.39	16.66	16.91	17.14	17.36	18.21
.10	13.16	13.66	14.14	14.44	14.75	15.02	15.28	15.51	15.74	16.60
.11	11.83	12.33	12.76	13.11	13.42	13.69	13.94	14.18	14.41	15.28
.12	10.73	11.23	11.65	12.00	12.31	12.59	12.84	13.08	13.31	14.19
.13	9.80	10.29	10.71	11.06	11.37	11.65	11.90	12.14	12.38	13.26
.14	9.00	9.49	9.91	10.26	10.57	10.85	11.10	11.35	11.58	12.48
.15	8.32	8.80	9.21	9.57	9.88	10.16	10.42	10.67	10.89	11.80
.16	7.72	8.20	8.61	8.96	9.27	9.55	9.81	10.06	10.29	11.20
.17	7.19	7.67	8.08	8.43	8.74	9.02	9.28	9.53	9.77	10.68
.18	6.73	7.20	7.61	7.96	8.27	8.55	8.81	9.05	9.30	10.21
.19	6.31	6.78	7.18	7.54	7.85	8.13	8.39	8.64	8.88	9.80
.20	5.94	6.40	6.81	7.16	7.47	7.75	8.01	8.26	8.50	9.43
.21	5.60	6.06	6.47	6.82	7.13	7.41	7.67	7.92	8.16	9.09
.22	5.30	5.75	6.16	6.50	6.82	7.10	7.36	7.61	7.86	8.79
.23	5.02	5.47	5.87	6.22	6.53	6.81	7.08	7.33	7.58	8.51
.24	4.77	5.22	5.62	5.96	6.27	6.56	6.82	7.07	7.32	8.26
.25	4.54	4.98	5.38	5.73	6.04	6.32	6.58	6.84	7.08	8.03
.26	4.32	4.77	5.16	5.51	5.82	6.10	6.37	6.62	6.87	7.81
.27	4.13	4.57	4.96	5.31	5.62	5.90	6.16	6.42	6.67	7.62
.28	3.95	4.38	4.77	5.12	5.43	5.71	5.98	6.23	6.48	7.43
.29	3.78	4.21	4.60	4.95	5.25	5.54	5.81	6.06	6.31	7.26
.30	3.62	4.05	4.44	4.78	5.09	5.38	5.64	5.90	6.15	7.10
.31	3.48	3.90	4.29	4.63	4.94	5.23	5.49	5.75	6.00	6.96
.32	3.34	3.76	4.15	4.49	4.80	5.08	5.35	5.61	5.86	6.82
.33	3.21	3.64	4.02	4.36	4.67	4.95	5.22	5.48	5.73	6.69
.34	3.09	3.51	3.89	4.23	4.54	4.83	5.09	5.35	5.60	6.56
.35	2.98	3.40	3.78	4.12	4.43	4.71	4.98	5.23	5.49	6.45
.36	2.88	3.29	3.67	4.01	4.32	4.60	4.87	5.12	5.38	6.34
.37	2.78	3.19	3.56	3.90	4.21	4.49	4.76	5.02	5.27	6.24
.38	2.68	3.09	3.47	3.81	4.11	4.40	4.67	4.92	5.17	6.14
.39	2.59	3.00	3.37	3.71	4.02	4.30	4.57	4.83	5.08	6.05
.40	2.51	2.92	3.29	3.62	3.93	4.21	4.48	4.74	4.99	5.96
.41	2.43	2.84	3.20	3.54	3.85	4.13	4.40	4.66	4.91	5.88
.42	2.36	2.76	3.13	3.46	3.77	4.05	4.32	4.58	4.83	5.80
.43	2.29	2.68	3.05	3.38	3.69	3.97	4.24	4.50	4.76	5.73
.44	2.22	2.61	2.98	3.31	3.62	3.90	4.17	4.43	4.68	5.66

[1]For d/b less than 0.04, use of the assumption R = d is more convenient and more accurate than interpolation in the table.

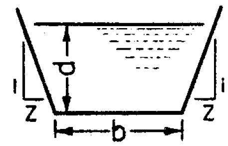
Table 2-6.–*Uniform flow in trapezoidal channels by Manning's formula.*–Continued. 103-D-1286-2



d/b	Values of $\frac{Qn}{d^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	2.15	2.55	2.91	3.24	3.55	3.83	4.10	4.36	4.61	5.59
.46	2.09	2.48	2.85	3.18	3.48	3.77	4.04	4.29	4.55	5.52
.47	2.03	2.42	2.78	3.12	3.42	3.70	3.97	4.23	4.49	5.46
.48	1.977	2.36	2.72	3.06	3.36	3.64	3.91	4.17	4.43	5.40
.49	1.923	2.31	2.67	3.00	3.30	3.58	3.85	4.12	4.37	5.34
.50	1.872	2.26	2.61	2.94	3.25	3.53	3.80	4.06	4.31	5.29
.52	1.777	2.16	2.51	2.84	3.14	3.43	3.70	3.96	4.21	5.19
.54	1.689	2.06	2.42	2.74	3.05	3.33	3.60	3.86	4.11	5.09
.56	1.608	1.977	2.33	2.66	2.96	3.24	3.51	3.77	4.02	5.00
.58	1.533	1.900	2.25	2.57	2.87	3.16	3.43	3.69	3.94	4.92
.60	1.464	1.827	2.17	2.50	2.80	3.08	3.35	3.61	3.86	4.85
.62	1.400	1.760	2.11	2.43	2.73	3.01	3.28	3.54	3.79	4.77
.64	1.340	1.697	2.04	2.36	2.66	2.94	3.21	3.47	3.72	4.71
.66	1.285	1.638	1.979	2.30	2.60	2.88	3.15	3.41	3.66	4.64
.68	1.234	1.583	1.922	2.24	2.54	2.82	3.09	3.35	3.60	4.59
.70	1.184	1.531	1.868	2.18	2.48	2.76	3.03	3.29	3.55	4.53
.72	1.139	1.482	1.818	2.13	2.43	2.71	2.98	3.24	3.49	4.48
.74	1.096	1.437	1.770	2.08	2.38	2.66	2.93	3.19	3.45	4.43
.76	1.056	1.393	1.725	2.04	2.33	2.61	2.88	3.15	3.40	4.38
.78	1.018	1.353	1.683	1.998	2.29	2.57	2.84	3.10	3.35	4.34
.80	.982	1.315	1.642	1.954	2.25	2.53	2.80	3.06	3.31	4.30
.82	.949	1.278	1.604	1.916	2.21	2.49	2.76	3.02	3.27	4.26
.84	.917	1.245	1.568	1.886	2.17	2.45	2.72	2.98	3.23	4.22
.86	.887	1.211	1.534	1.843	2.14	2.41	2.68	2.94	3.20	4.18
.88	.858	1.180	1.501	1.810	2.10	2.38	2.65	2.91	3.16	4.15
.90	.831	1.153	1.470	1.777	2.07	2.35	2.62	2.87	3.13	4.12
.92	.805	1.122	1.441	1.747	2.04	2.32	2.58	2.84	3.10	4.08
.94	.781	1.095	1.413	1.718	2.01	2.29	2.55	2.81	3.07	4.05
.96	.758	1.070	1.396	1.690	1.981	2.26	2.53	2.78	3.04	4.03
.98	.736	1.046	1.360	1.663	1.954	2.23	2.50	2.76	3.01	4.00
1.00	.714	1.022	1.335	1.638	1.928	2.21	2.47	2.73	2.99	3.97
1.05	.666	.969	1.278	1.579	1.871	2.14	2.41	2.67	2.92	3.91
1.10	.622	.920	1.226	1.525	1.813	2.09	2.36	2.61	2.87	3.85
1.15	.583	.876	1.178	1.477	1.763	2.04	2.30	2.56	2.82	3.80
1.20	.548	.836	1.136	1.432	1.717	1.993	2.26	2.51	2.77	3.76
1.25	.516	.800	1.098	1.392	1.676	1.950	2.22	2.47	2.73	3.71
1.30	.487	.767	1.062	1.354	1.638	1.912	2.18	2.43	2.69	3.67
1.35	.460	.736	1.028	1.320	1.603	1.876	2.14	2.40	2.65	3.64
1.40	.436	.708	.998	1.288	1.570	1.843	2.11	2.37	2.62	3.60
1.45	.414	.682	.970	1.259	1.540	1.812	2.08	2.34	2.59	3.57
1.50	.393	.658	.944	1.231	1.512	1.784	2.05	2.31	2.56	3.54
1.55	.374	.636	.920	1.206	1.486	1.757	2.02	2.28	2.53	3.52
1.60	.357	.615	.897	1.182	1.461	1.731	1.995	2.25	2.51	3.49
1.65	.341	.596	.876	1.160	1.438	1.708	1.972	2.23	2.48	3.47
1.70	.325	.578	.856	1.139	1.416	1.686	1.949	2.21	2.46	3.44

Table 2-6.—*Uniform flow in trapezoidal channels by Manning's formula.*—Continued. 103-D-1286-3

Values of $\frac{Qn}{d^{8/3} S^{1/2}}$



d/b	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.75	.312	.561	.838	1.119	1.396	1.666	1.928	2.19	2.44	3.42
1.80	.298	.546	.820	1.101	1.377	1.646	1.905	2.17	2.42	3.40
1.85	.286	.531	.804	1.083	1.359	1.628	1.890	2.15	2.40	3.38
1.90	.275	.517	.788	1.067	1.342	1.610	1.872	2.13	2.38	3.37
1.95	.264	.504	.773	1.051	1.326	1.594	1.856	2.11	2.36	3.35
2.00	.254	.491	.760	1.036	1.310	1.578	1.840	2.10	2.35	3.33
2.10	.236	.469	.734	1.009	1.282	1.549	1.811	2.07	2.32	3.30
2.20	.219	.448	.711	.984	1.256	1.523	1.784	2.04	2.29	3.27
2.30	.205	.430	.690	.962	1.233	1.499	1.760	2.02	2.27	3.25
2.40	.1919	.413	.671	.941	1.212	1.477	1.737	1.993	2.24	3.23
2.50	.1800	.398	.653	.922	1.192	1.457	1.717	1.972	2.22	3.21
2.60	.1693	.383	.637	.905	1.174	1.438	1.698	1.954	2.21	3.19
2.70	.1597	.371	.623	.889	1.157	1.422	1.681	1.937	2.19	3.17
2.80	.1508	.359	.609	.874	1.142	1.406	1.665	1.920	2.17	3.15
2.90	.1427	.348	.596	.861	1.128	1.391	1.650	1.905	2.16	3.14
3.00	.1354	.338	.585	.848	1.114	1.377	1.636	1.891	2.14	3.12
3.20	.1223	.320	.563	.825	1.090	1.353	1.611	1.865	2.12	3.10
3.40	.1111	.305	.545	.805	1.069	1.331	1.589	1.843	2.09	3.07
3.60	.1015	.291	.529	.787	1.050	1.312	1.569	1.823	2.07	3.05
3.80	.0932	.279	.514	.771	1.033	1.294	1.552	1.805	2.06	3.04
4.00	.0859	.268	.501	.757	1.019	1.279	1.536	1.790	2.04	3.02
4.50	.0711	.245	.474	.727	.987	1.246	1.502	1.755	2.01	2.98
5.00	.0601	.228	.453	.704	.962	1.220	1.476	1.729	1.979	2.96

Table 2-7.—*Uniform flow in circular sections flowing partly full.* 103-D-1287

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

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2-38. ***Bibliography.***

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Regulating Structures

A. GENERAL

3-1. ***Regulating Structures.***—A regulating structure in an open irrigation system is used to regulate the flow passing through the structure, to control the elevation of the upstream canal water surface, or to do both. Structures which perform these functions include: (1) checks, (2) check-drops, (3) turnouts, (4) division structures, (5) check inlets, and (6) control inlets.

Regulation is achieved with weirs, control inlets, stoplogs, and slide gates. Control inlets (fig. 3-1) are designed to regulate the canal water surface within particular limits. Stoplogs and slide gates are adjusted as required. Where measurement at the structure is required, weirs may be used. Weirs may or may not be adjustable. Slide gates and weirs may be bolted to the check walls or they may be mounted in frames that are inserted in grooves in the structure. Weirs are discussed later in chapter V.

Figure 3-1. **Control notch maintains water surface above a drop inlet to a pipe structure. Discharge is 10 cfs. PX-D-72615D**

Either stoplogs or slide gates, or a combination of both, are the usual means of flow regulation. Stoplogs are less expensive than gates and are more quickly adjusted but do not control the flow as closely. When stoplogs are used, flow through the structure can be determined from the formula for flow over a rectangular weir:

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

When gates are used the flow through the structure can be determined from the formula for flow through a submerged orifice:

$$Q = CA\sqrt{2gH}$$

The equations will show that the flow over stoplogs is more responsive to changes in water depths in the canal than flow through a gate, therefore the required adjustment of stoplogs is less frequent. Similarly where adjustable weirs are used for control, they require less frequent adjustment than gates.

Floating trash that can pass over stoplogs and adjustable weirs can be a problem in a gated structure. Jetting of flow under gates can pull debris below the water surface and cause the gate opening to become obstructed.

If the canal must be closely regulated or if automatic or remote control of the canal is anticipated, gates must be used. Gates are often manually operated but automation requires motorized operation. Gates using motor-driven hoists can be equipped with control devices that will automatically maintain constant water surface elevations regardless of the discharge [1] [2].[1]

[1]Numbers in brackets refer to items in the bibliography, sec. 3-29.

B. CHECKS

D. L. WINSETT[2]

3-2. ***Purpose and Description.***—Checks are used to regulate the canal water surface upstream of the structure and to control the downstream flow. When a canal is flowing at partial capacity, checks are operated to maintain the canal water surface elevation required for upstream water deliveries. The water surface that the check must maintain is the control water surface. It is generally the same as the normal water surface at the check.

In the event of a break in the canal bank, checks can be used to limit the volume of escaping water to that confined between check structures and prevent the entire canal from being emptied. The use of checks also allows reaches of the canal to be isolated and dewatered for repair or inspection.

Checks have a deck spanning the structure to provide access to the gates or stoplogs. Overflow walls or check walls are provided on both sides of the gates or stoplogs. The tops of these walls are set at or slightly above the control water surface. Excess water in the canal can flow over these walls, the water surface rise in the canal is limited, and the canal banks are less likely to be overtopped. The amount of overflow provided for will depend on the amount of excess water expected in the canal and the degree of safety required for the structure and the canal.

Figure 3-2 shows a typical check structure with riprap downstream to protect the channel.

3-3. ***Combination Structures.***—Checks may be separate structures or they may be combined with the inlets of other structures. Check inlets (figs. 3-3 and 3-4) are often used

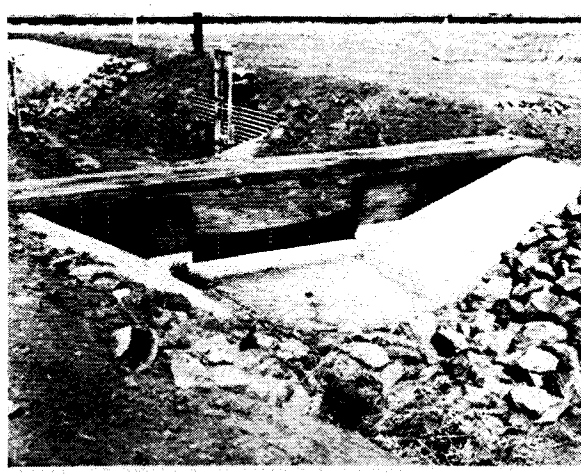


Figure 3-2. **Concrete check structure. Metal turnout structures and outlet for railroad crossing in background. P 796-701-2630**

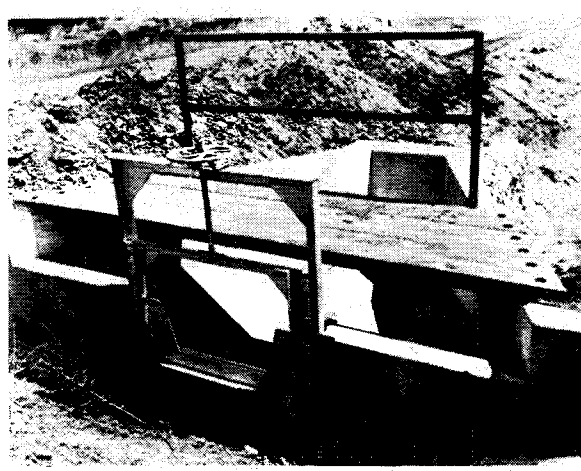


Figure 3-3. **Check inlet to pipe structure. Adjustable weir installed in the stoplog groove. P 372-701-765**

[2]Civil Engineer, Hydraulic Structures Branch, Bureau of Reclamation.

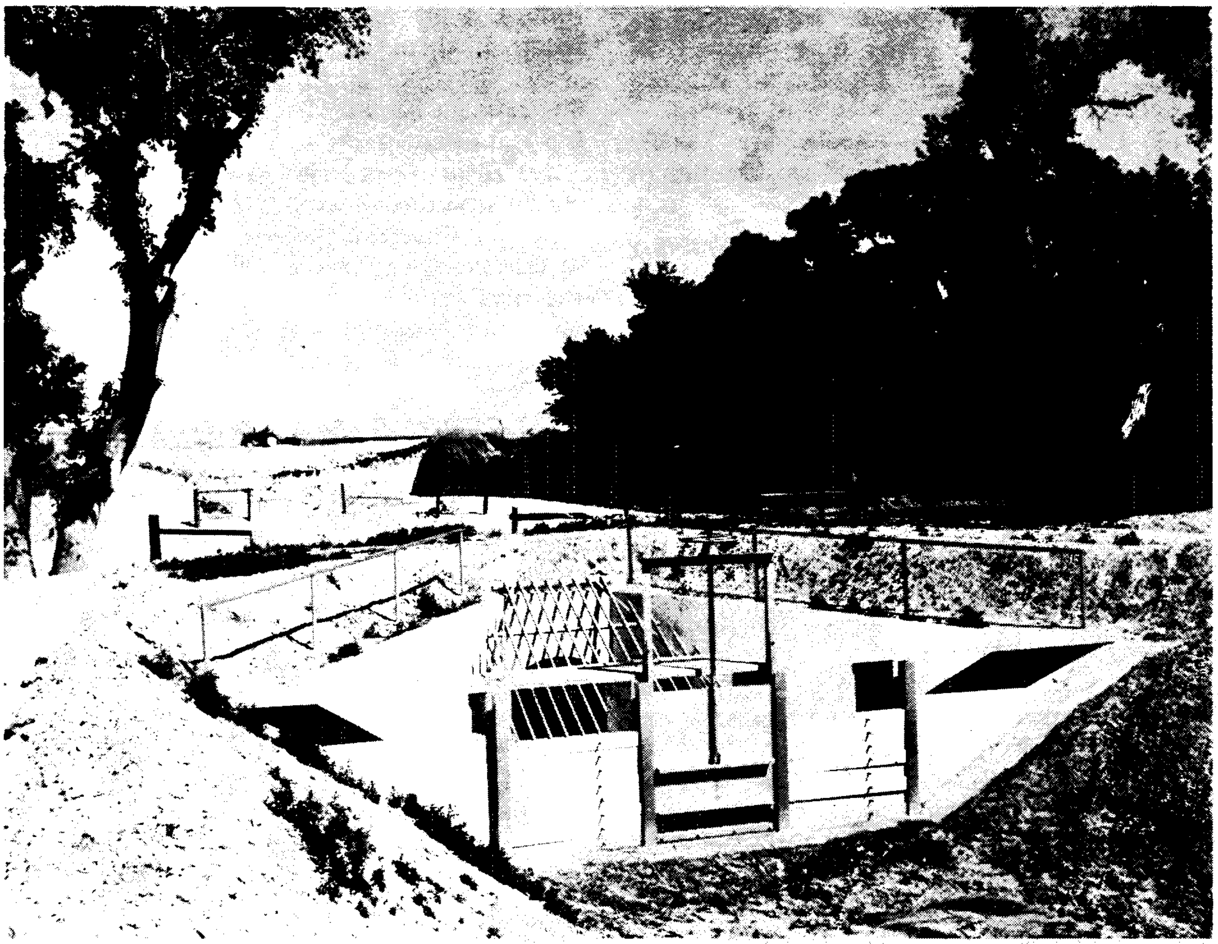


Figure 3-4. **Check inlet to pipe structure. Stoplogs and slide gate installed in grooves. P 328-701-9494**

with such structures as road crossings, inverted siphons, pipe drops, rectangular inclined drops, and chutes.

Checks, when combined with other structures, can prevent water surface drawdown and scour upstream of the structure. Combination structures are generally more economical than separate structures.

3-4. ***Spacing.***—Checks are spaced along a canal primarily to maintain the water surface elevations required by upstream water deliveries. Steeper canal grades usually require closer spacing between the checks. From an initially selected check location, a level line at the control water surface elevation projected upstream will determine the last upstream turnout that can be served.

Check spacing in a concrete-lined canal is often controlled by limiting the drop in normal water surface between check structures. Then, if for any reason, operation of the system must be suddenly suspended the drop in water surface is controlled as the water surface profile changes from the normal water surface to a level checked water surface. Displacement of the concrete lining by unequal hydrostatic pressures is consequently prevented or minimized.

Check spacing may also be influenced by the following considerations:

(1) Canal storage—Locating checks closer together increases the amount of storage in the canal.

(2) Travel time between checks–Spacing may be influenced by the time required for operating personnel to travel between structures.

(3) Canal properties–Checks are usually located where changes in canal cross section or capacity occur.

3-5. ***Design Considerations.***–(a) *Stability and Percolation.*–Checks are designed to withstand the hydraulic forces imposed by water ponded to the top of the check wall on the upstream side and no water on the downstream side. Adequate check length is provided to dampen water turbulence caused by: (1) the sheet of water flowing over stoplogs, or (2) the jet of water flowing through a partially opened gate. The length in conjunction with the cutoff walls also provides a percolation path with sufficient length to prevent the removal of foundation material by water moving adjacent to the structure. For a discussion of percolation see subchapter VIII C. The structure must be stable against sliding and overturning.

(b) *Stoplogs, Gates, and Deck.*–Velocity through check structures using stoplogs should be limited to about 3.5 feet per second for efficient stoplog operation. The velocity through checks using gates may be increased to about 5 feet per second. The use of stoplogs is not recommended if: (1) the flow is greater than 50 cfs, (2) the width of the stoplog opening is greater than 5 feet, or (3) the water depth is greater than 6 feet. Stoplog guides may be vertical if the distance from the floor of the check to the bottom of the operating deck is less than 6 feet. For greater heights, sloping the stoplog guides on a 1 to 4 slope may make the installation or removal of stoplogs less difficult. Where stoplogs are not used operating decks are designed for a live load of 100 pounds per square foot. Where stoplogs are used the live load is increased to 150 pounds per square foot.

For safety, operating decks should be a minimum of 2 feet wide where the height of the deck above the check floor is less than 3.5 feet. Where the height above the floor is 3.5 feet or greater, decks should be a minimum of 3 feet wide. Concrete deck surfaces should be floated or broomed to acquire a skid-resistant surface. A downstream handrail on the deck is required when the deck is 3.5 feet or more above the floor and both an upstream and a downstream handrail are required if the height is 5 feet or more.

(c) *Head Loss.*–The head loss through a check structure is computed as 0.5 Δh_v (0.5 times the difference between the velocity head at the check opening and the velocity head in the canal section upstream of the structure). A minimum loss of 0.1 foot should be allowed for isolated checks in small canals. Except for this head loss, checks normally do not provide for a drop in grade across the structure. At partial canal capacities, with the water upstream checked to the control water surface, a drop in water surface elevation will occur across the structure.

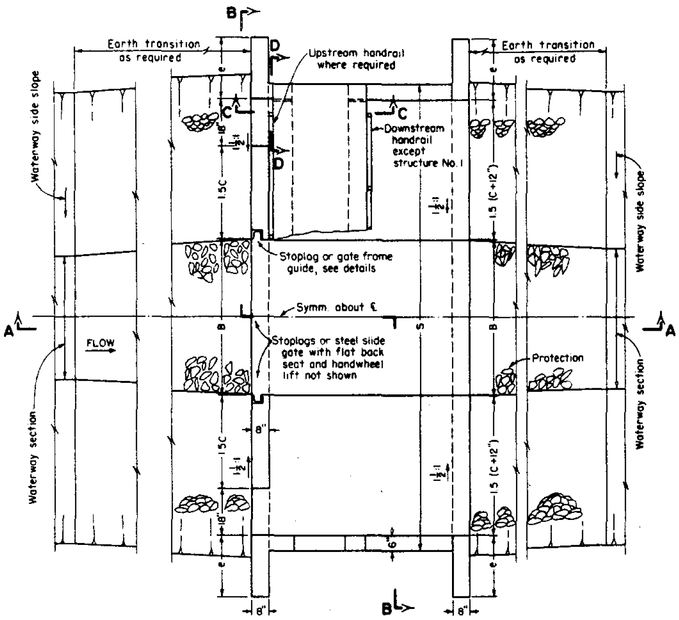
The Bureau of Reclamation has developed standard dimensions for checks with capacities of 100 cfs and less. These checks are shown on figures 3-5 and 3-6. The selection of a check from these standard structures and the determination of the required elevations are shown by the following design example.

3-6. ***Design Example.***–Select a check to be used in a canal with a capacity of 25 cfs and a normal water depth of 1.75 feet. Assume that the canal profile shows elevation A (the bottom grade elevation upstream of the structure) to be 1207.18.

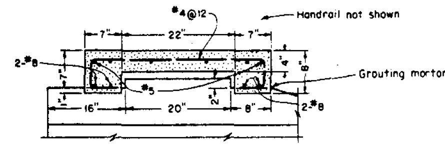
Figure 3-5 shows a number of check structures with standard dimensions. There is no structure listed for a maximum capacity of 25 cfs. From the table on figure 3-5, select structure number 4 with the next largest Q which is 26 cfs. If more than one structure is listed for the required capacity, choose the most economical structure that best fits the canal section. If gates are to be used, the availability of gate sizes may influence the choice of structure.

Determine elevation B:

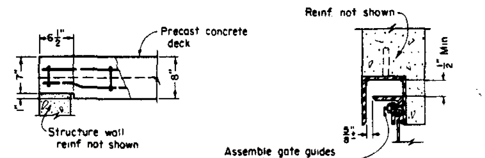
Upstream bottom grade (El. A)	1207.18
+ Upstream normal water depth	+ 1.75
Upstream normal water surface elevation	1208.93



PLAN

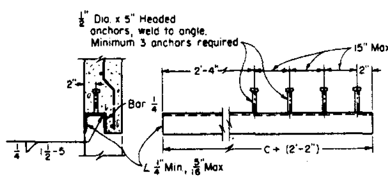


SECTION C-C

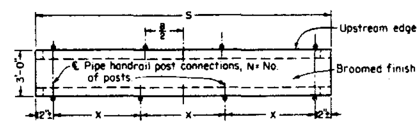


SEC. D-D

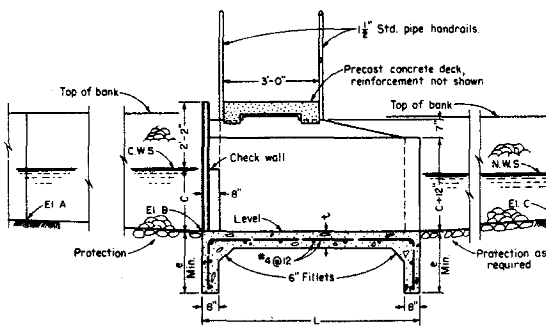
TYPICAL SLIDE GATE ASSEMBLY



STOPLOG OR GATE FRAME GUIDE DETAILS
(TWO REQUIRED)



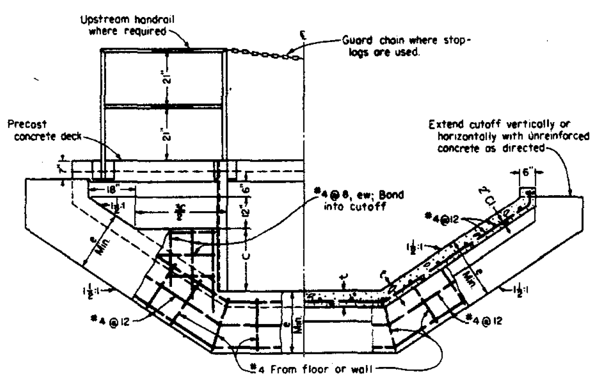
PLAN OF PRECAST CONCRETE DECK



SECTION A-A

NOTES

Gate frame height measured from center of gate opening
Transverse bars to be continuous in walls and floor
Set Elevation B so that top of check wall is at Control Water Surface Elevation (C.W.S.).

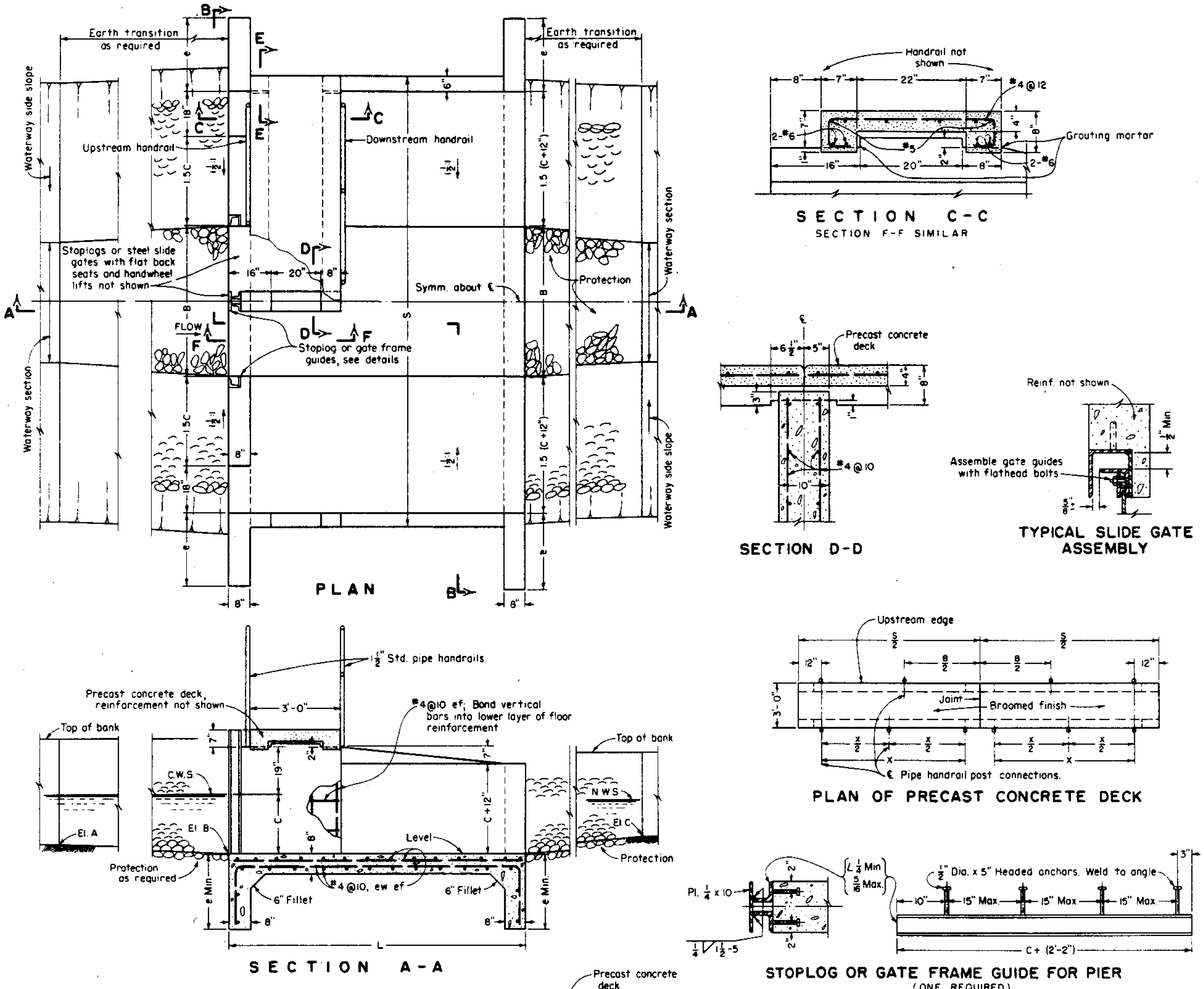


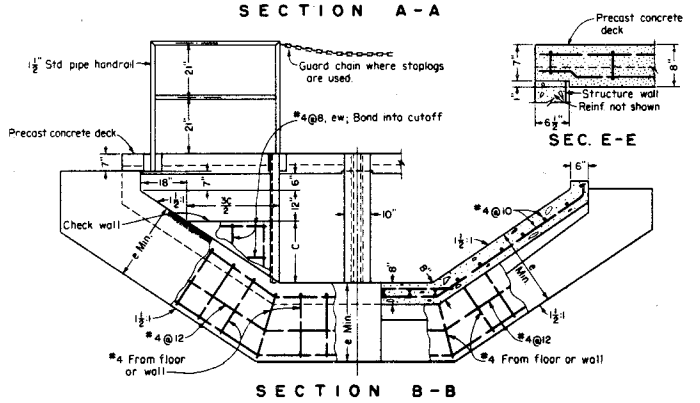
SECTION B-B

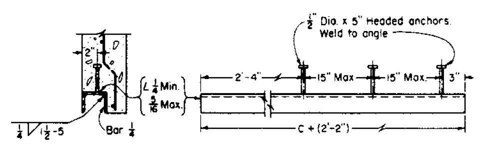
STR. NO.	MAX. Q, CFS	SLIDE GATE WIDTH X HEIGHT*	SLIDE GATE FRM. HT.	B	C	L	S	X	N	e	t	CONC., CU. YDS.	REINF. STEEL, LBS.	MISC. METAL, LBS.
1	10	36"x12"	5'	3'-0"	14"	4'-6"	10'-6"	—	—	24"	6"	2.9	360	80
2	15	36"x18"	6'	3'-0"	20"	4'-6"	12'-0"	5'-0"	3	24"	6"	3.3	420	200
3	21	36"x24"	6'	3'-0"	2'-2"	5'-0"	13'-6"	5'-8"	3	24"	7"	3.9	480	220
4	26	36"x30"	6'	3'-0"	2'-8"	6'-0"	15'-0"	6'-6"	3	24"	8"	5.2	560	240
5	21	48"x18"	6'	4'-0"	21"	4'-6"	13'-3"	5'-7"	3	24"	6"	3.6	460	210
6	28	48"x24"	6'	4'-0"	2'-3"	5'-0"	14'-9"	6'-4"	3	24"	7"	4.4	530	230
7	35	48"x30"	6'	4'-0"	2'-9"	6'-0"	16'-3"	7'-0"	3	24"	8"	5.6	600	250
** 8	42	48"x36"	6'	4'-0"	3'-3"	7'-0"	17'-9"	7'-10"	3	2'-6"	8"	7.2	730	400
9	35	60"x24"	6'	5'-0"	2'-3"	5'-0"	15'-9"	6'-10"	3	24"	7"	4.6	560	240
10	43	60"x30"	6'	5'-0"	2'-9"	6'-0"	17'-3"	7'-7"	3	24"	8"	5.9	640	260
** 11	52	60"x36"	6'	5'-0"	3'-3"	7'-0"	18'-9"	5'-6"	4	2'-6"	8"	7.6	770	440
12	42	72"x24"	6'	6'-0"	2'-3"	5'-0"	16'-9"	7'-4"	3	24"	7"	4.9	590	240
13	53	72"x30"	6'	6'-0"	2'-9"	6'-0"	18'-3"	5'-4"	4	24"	8"	6.1	670	280
** 14	63	72"x36"	6'	6'-0"	3'-3"	7'-0"	19'-9"	5'-10"	4	2'-6"	8"	7.9	800	440

* When a gate of specified height is not available, a gate with next greater available height shall be used with appropriate frame height.
**Upstream handrail required

Figure 3-5. **Standard check structures. Q less than 70 cfs. 103-D-1208**







STR. NO.	MAX. Q, CFS	**SLIDE GATES		STANDARD DIMENSIONS						ESTIMATED QUANTITIES		
		*WIDTH x HEIGHT	FRM. HT.	B	C	L	e	S	X	CONC., CU. YDS.	REINF. STEEL, LBS.	MISC. METAL, LBS.
1	74	42"x 36"	7'-0"	7'-10"	3'-2"	7'-0"	2'-6"	21'-4"	9'-2"	8.9	910	620
2	86	42"x 42"	7'-6"	7'-10"	3'-8"	7'-6"	2'-6"	22'-10"	10'-0"	10.0	1000	660
3	98	42"x 48"	8'-0"	7'-10"	4'-3"	8'-0"	2'-6"	24'-7"	10'-10"	11.3	1100	710
4	84	48"x 36"	7'-0"	8'-10"	3'-3"	7'-0"	2'-6"	22'-7"	9'-10"	9.4	960	630
5	98	48"x 42"	7'-6"	8'-10"	3'-9"	7'-6"	2'-6"	24'-1"	10'-8"	10.5	1050	680
6	105	60"x 36"	7'-0"	10'-10"	3'-3"	7'-6"	2'-6"	24'-7"	10'-10"	10.4	1050	640

* When a gate of specific height is not available, a gate with next greater available height shall be used with appropriate frame height.
** Two gates required for each check.

NOTES
Outer face transverse bars to be continuous in walls and floor.
Set Elevation B so that top of check wall is at Control Water Surface Elevation (C.W.S.).
Gate frame height measured from center of gate opening.

Figure 3-6. **Standard check structures. Q greater than 70 and less than 105 cfs. 103-D-1209**

The top of the check wall is set at or slightly above the elevation of the upstream normal water surface.

From figure 3-5, dimension C for structure number 4 = 2.67 feet.

Elevation of the top of the check wall		1208.93
– Dimension C	–	2.67
Elevation B		1206.26

Elevation B should be equal to or less than elevation A to avoid a hump in the bottom of the canal. If elevation B is higher than elevation A, a structure with a larger dimension C should be selected.

Determine elevation C:

The head loss through the structure must be determined in accordance with subsection 3-5(c) of this chapter. For this example, assume a minimum head loss of 0.1 foot. The downstream bottom grade elevation then equals:

Upstream water surface elevation		1208.93
– Head loss	–	0.10
Downstream water surface elevation		1208.83
– Downstream normal water depth	–	1.75
Elevation C (downstream bottom grade)		1207.08

Transitions are provided as required to fit the structure to the canal section. In unlined canals the channel should be protected above and below the structure to prevent erosion.

Checks for capacities between 70 and 100 cfs are shown on figure 3-6.

C. CHECK-DROPS

D. L. WINSETT [2]

3-7. ***Purpose and Description.***—Check-drops are structures that provide a means of making small changes in canal invert elevations. The structure combines the function of a vertical drop with those of a check. The drop portion of the structure dissipates excess head in the canal while the check portion maintains the required water surface upstream and controls the flow downstream. Figures 3-7 through 3-11 show typical check-drop structures.

The design and operation of the check feature of the check-drop is similar to that of checks and the preceding discussion of checks applies generally to check-drops. Gates, stoplogs, and weirs are used for regulation.

3-8. ***Design Considerations.***—The vertical drop through this type of structure in an unlined canal is generally limited to 3 feet for canal capacities of 70 cfs and less and to 1.5 feet for larger capacities. In lined sections these structures have been used for vertical drops up to about 8 feet. For the larger drops, economic and stability considerations may indicate the use of another type of structure.

Check-drops are designed to meet the same requirements for percolation, stability, velocity, and safety as checks. In addition, the

[2] Op. cit., p. 134.

Figure 3-7. **Operator removing stoplogs from a check-drop structure. Constant-head orifice turnouts in background. P-606-600-66A**

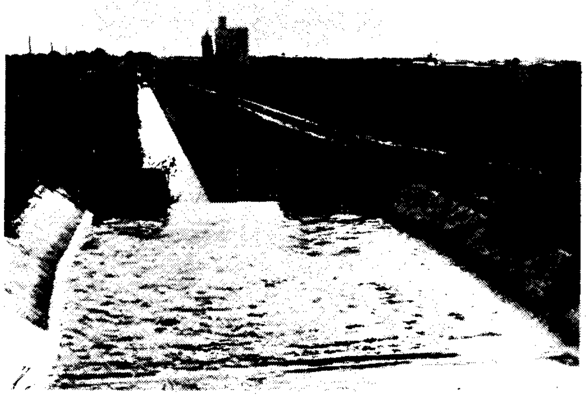


Figure 3-8. Drop structure using a weir for regulation. P 257-500-2967NA

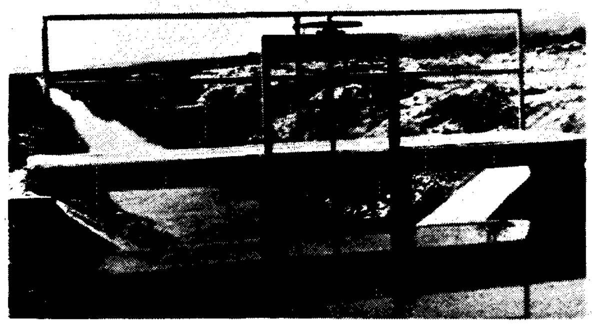


Figure 3-10. Concrete check-drop. Water upstream checked to top of check wall. Discharge is 10 cfs. P 222-116-53587

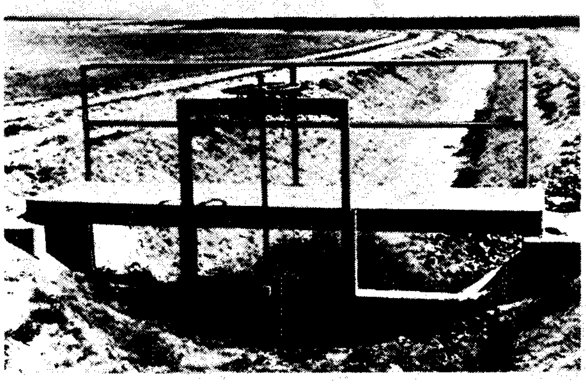


Figure 3-9. Concrete check with slide gate. Design discharge is 24 cfs. P 222-116-53583

Figure 3-11. Check-drop maintaining an upstream water surface. P 17-100-285.

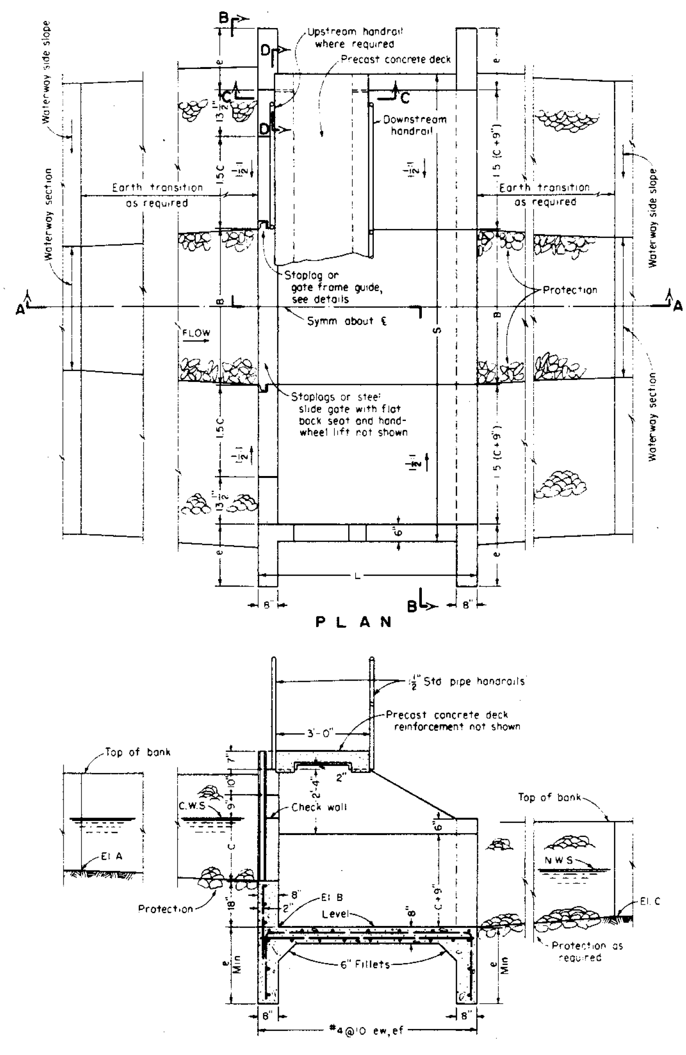
structure must be long enough to contain the falling water and to provide a stilling pool to control turbulence downstream.

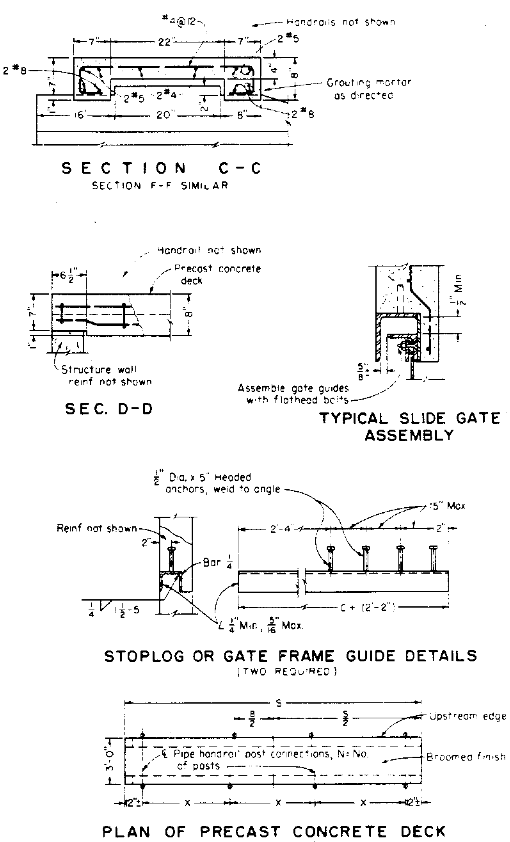
Overflow walls on each side of the check opening allow excess water in the canal to pass through the check-drop in the same manner as through a check.

3-9. ***Application.***—Check-drops are located in a channel where a small change in grade is required. The best use of the dual features of this structure is made when it is placed where a check would normally be located.

The Bureau has developed standard dimensions for check-drop structures with vertical drops of up to 3 feet. These standard structures are shown on figures 3-12, 3-13, and 3-14. The design example demonstrates how a check-drop structure is selected from these standard structures and how the required elevations are determined.

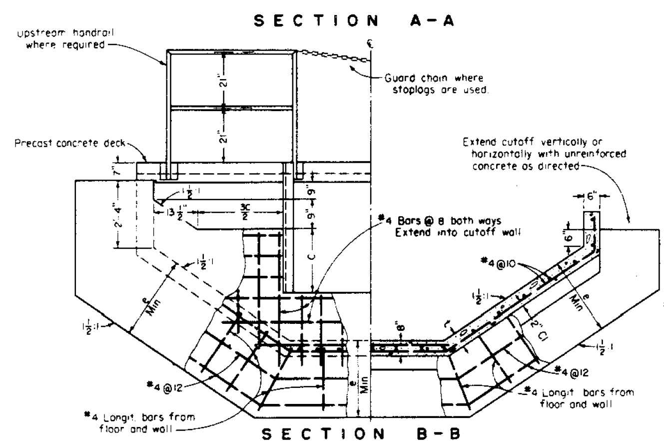
3-10. ***Design Example.***—Select a check-drop structure to be used in a channel with a capacity of 50 cfs and a water depth of 2.75 feet. Assume that the channel profile shows elevation A, the bottom grade at the upstream end of the structure, to be 1207.18 and





NOTES

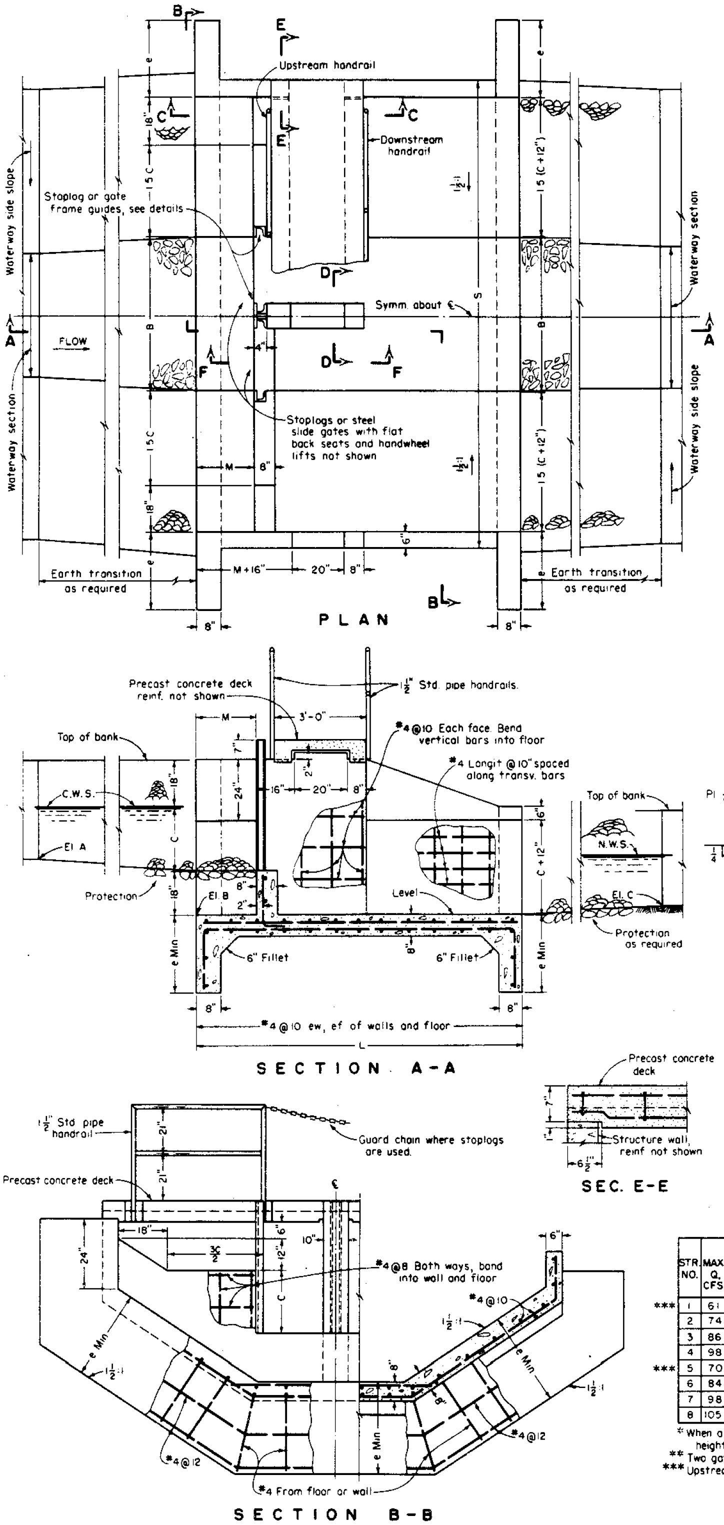
Outer face transverse bars to be continuous in walls and floor
Set Elevation B so that top of check walls is at Control Water Surface Elevation (C.W.S.)
Gate frame height measured from center of gate opening.

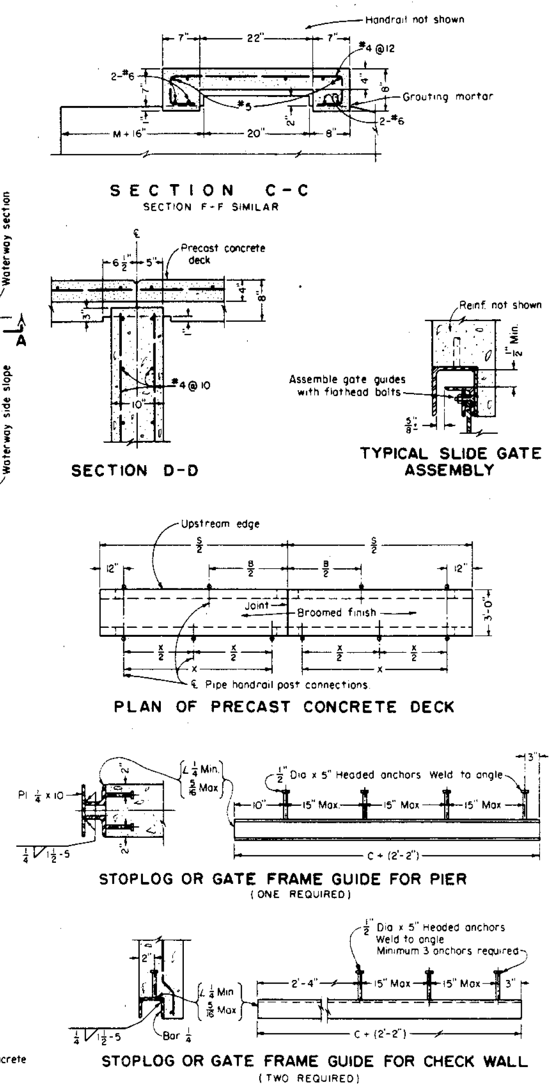


	STR. NO.	MAX. Q, cfs	SLIDE GATE		STANDARD DIMENSIONS								ESTIMATED QUANTITIES		
			WIDTH X HEIGHT*	FRM. HT.	B	C	L	S	X	N	e	t	CONC., CU. YDS.	REINF. STEEL, LBS.	MISC. METAL, LBS.
	1	10	36"x12"	5'	3'-0"	14"	6'-6"	9'-9"	7'-9"	2	24"	6"	3.9	440	160
	2	15	36"x18"	5'	3'-0"	20"	6'-6"	11'-3"	4'-7"	3	24"	6"	4.5	500	200
	3	21	36"x24"	6'	3'-0"	2'-2"	7'-0"	12'-9"	5'-4"	3	24"	7"	5.3	550	220
	4	26	36"x30"	6'	3'-0"	2'-8"	7'-6"	14'-3"	6'-1"	3	2'-6"	7"	7.0	660	240
	5	21	48"x18"	5'	4'-0"	21"	7'-0"	12'-6"	5'-3"	3	24"	6"	5.0	560	210
	6	28	48"x24"	6'	4'-0"	2'-3"	7'-6"	14'-0"	6'-0"	3	24"	7"	6.2	640	230
	7	35	48"x30"	6'	4'-0"	2'-9"	8'-0"	15'-6"	6'-9"	3	2'-6"	7"	7.7	720	250
**	8	42	48"x36"	6'	4'-0"	3'-3"	8'-6"	17'-0"	7'-6"	3	2'-6"	8"	9.0	790	400
	9	35	60"x24"	6'	5'-0"	2'-3"	7'-6"	15'-0"	6'-6"	3	24"	7"	6.3	650	240
	10	43	60"x30"	6'	5'-0"	2'-9"	8'-0"	16'-6"	7'-3"	3	2'-6"	7"	8.1	760	260
**	11	52	60"x36"	6'	5'-0"	3'-3"	8'-6"	18'-0"	8'-0"	3	2'-6"	8"	9.7	850	420
	12	42	72"x24"	6'	6'-0"	2'-3"	7'-6"	16'-0"	7'-0"	3	24"	7"	6.7	690	240
	13	53	72"x30"	6'	6'-0"	2'-9"	8'-0"	17'-6"	7'-9"	3	2'-6"	7"	7.6	800	260
**	14	63	72"x36"	6'	6'-0"	3'-3"	8'-6"	19'-0"	5'-8"	4	2'-6"	8"	10.2	890	430

* When a gate of specified height is not available a gate with next greater available height shall be used with appropriate frame height.
**Upstream handrail required

Figure 3-12. **Standard check-drop structures. Q less than 70 cfs. 103-D-1210**





NOTES

Outer face transverse bars to be continuous in walls and floor
Set Elevation B so that top of check wall is at Control Water Surface Elevation (CWS)
Gate frame height measured from center of gate opening

	STR. NO.	MAX. Q, CFS	** SLIDE GATES: * WIDTH x HEIGHT	** SLIDE GATES: FRM. HT.	STANDARD DIMENSIONS: B	C	L	M	S	e	X	ESTIMATED QUANTITIES: CONC., CU. YDS.	REINF. STEEL, LBS.	MISC. METAL, LBS.
***	1	61	42"x30"	6'	7'-10"	2'-8"	8'-0"	0	19'-10"	24"	8'-6"	9.9	950	450
	2	74	42"x36"	7'	7'-10"	3'-2"	8'-6"	0	21'-4"	2'-6"	9'-4"	11.8	1050	620
	3	86	42"x42"	7'	7'-10"	3'-8"	10'-6"	18"	22'-10"	2'-6"	10'-0"	14.2	1220	670
	4	98	42"x48"	7'	7'-10"	4'-3"	13'-0"	3'-0"	24'-7"	2'-6"	10'-10"	17.2	1440	710
***	5	70	48"x30"	6'	8'-10"	2'-9"	8'-0"	0	21'-1"	24"	9'-2"	10.4	1000	460
	6	84	48"x36"	7'	8'-10"	3'-3"	8'-6"	0	22'-7"	2'-6"	9'-10"	12.5	1100	630
	7	98	48"x42"	7'	8'-10"	3'-9"	10'-6"	18"	24'-1"	2'-6"	10'-8"	14.9	1280	680
	8	105	60"x36"	7'	10'-10"	3'-3"	10'-0"	18"	24'-7"	2'-6"	10'-10"	14.5	1250	650

* When a gate of specific height is not available, a gate with next greater available height shall be used with appropriate frame height.
** Two gates required for each check.
*** Upstream handrail not required

Figure 3-13. **Standard check-drop structures. Q greater than 60 and less than 100 cfs. 103-D-1211** elevation C, the bottom grade at the downstream end of the structure, to be 1205.92. The change in grade then is:

Elevation A	1207.18
– Elevation C	– 1205.92
Change in grade	1.26 feet

Refer to figure 3-12 which shows a number of check-drop structures with standard dimensions for a drop in grade of up to 1.5 feet. There is not a structure listed for a canal capacity of 50 cfs. Therefore, from the table on figure 3-12 select structure number 11 which has the next largest capacity, or 52 cfs. If more than one structure is listed for the required capacity, choose the most economical structure that best fits the channel section. If gates are to be used, the availability of gate sizes may influence the choice of structure.

Determine elevation B:

From figure 3-12, dimension C for structure number 11 is 3.25 feet and the drop in the floor is 1.5 feet.

Elevation A	1207.18
+ Upstream normal water depth	+2.75
Upstream normal water surface elevation	1209.93

The top of the check wall is set at or slightly above the elevation of the upstream normal water surface.

Elevation of the top of the check wall	1209.93
– Dimension C	– 3.25
	1206.68
Drop in floor	– 1.50
Elevation B	1205.18

Elevation B should be equal to or lower than elevation C. If elevation B is higher than elevation C, a structure with a larger dimension C should be selected.

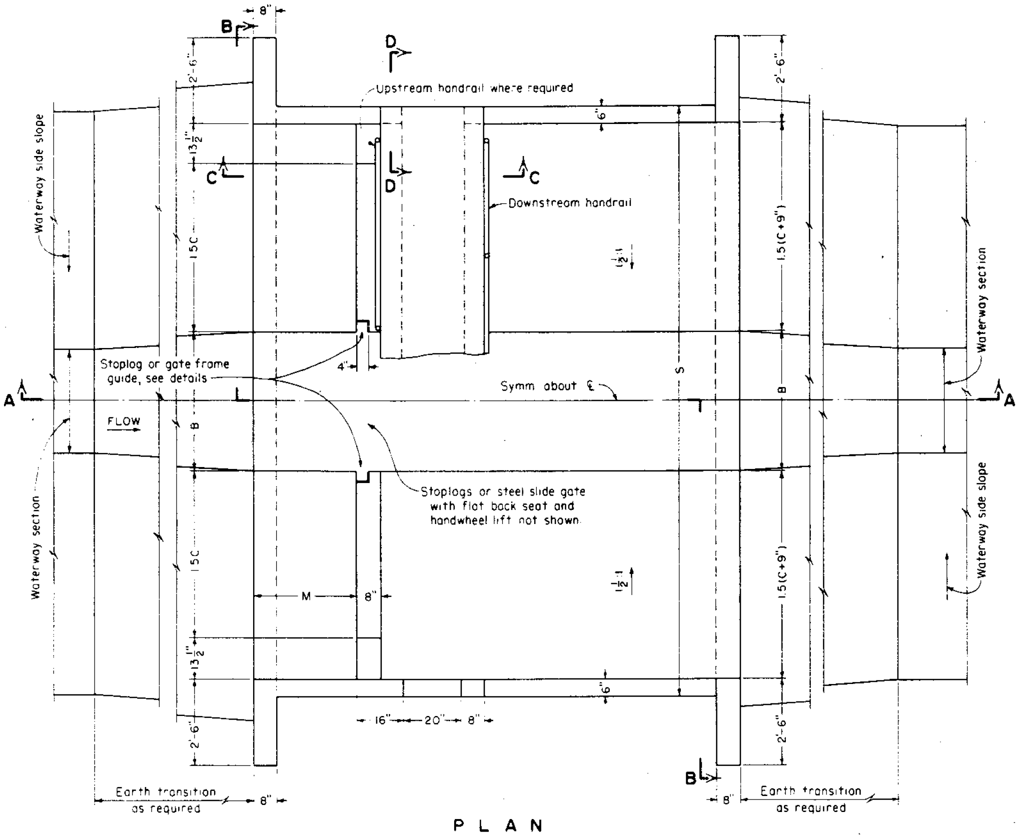
Determine the downstream water surface elevation:

Since there is no change in channel properties at the structure, the downstream normal water depth equals the upstream normal water depth.

Elevation C	1205.92
+ Downstream normal water depth	+ 2.75
Downstream normal water surface elevation	1208.67

Transitions are provided as required to fit the structure to the canal cross section and in unlined canals the channel should be protected above and below the structure to prevent erosion.

Check-drops for capacities between 60 and 100 cubic feet per second are shown on figure 3-13. For structures with a drop in the floor of 3 feet, see figure 3-14.



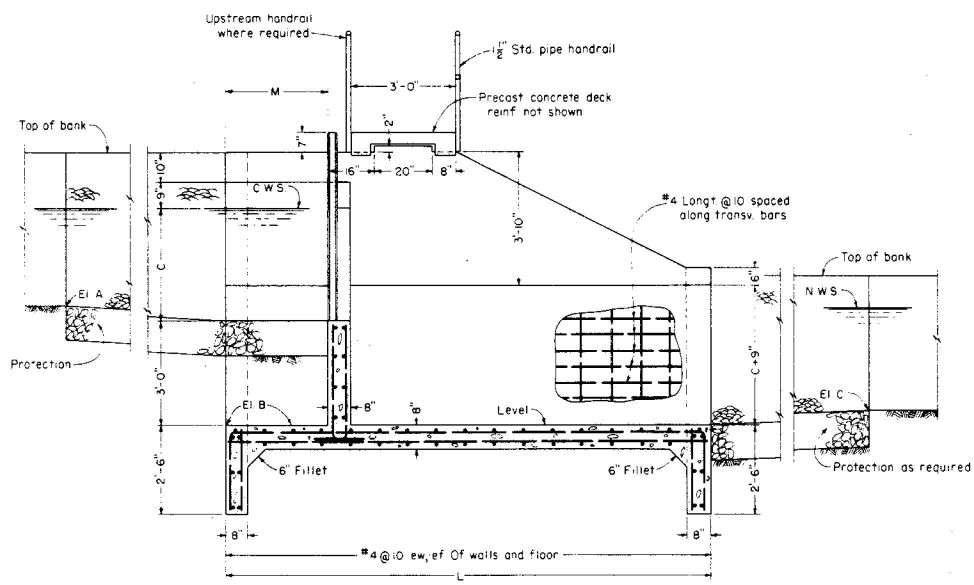
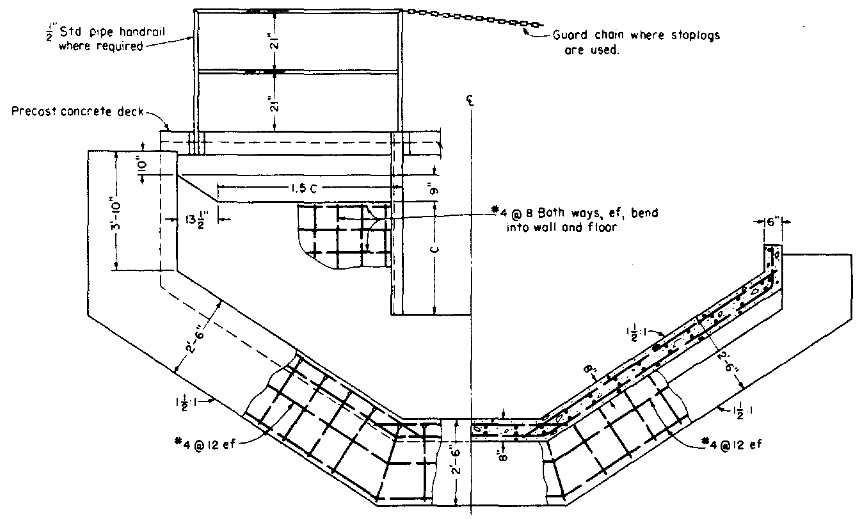
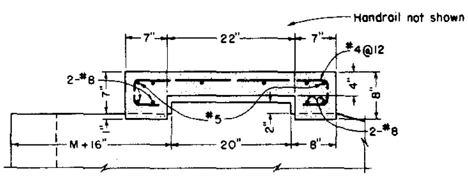


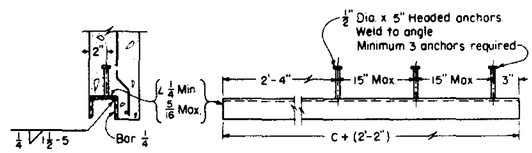
Figure 3-14. **Standard dimensions for a 3-foot check-drop structure. (Sheet 1 of 2.) 103-D-1212**



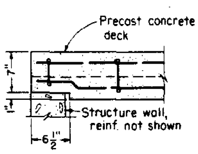
SECTION B-B



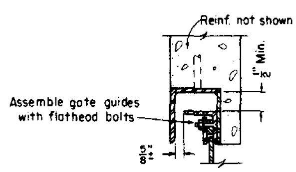
SECTION C-C



STOPLOG OR GATE FRAME GUIDE FOR CHECK WALL
(TWO REQUIRED)



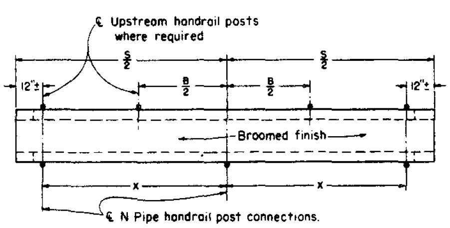
SEC. D-D



TYPICAL SLIDE GATE ASSEMBLY

NOTES

Outer face transverse bars to be continuous in walls and floor
Thickness of concrete to vary uniformly between dimensions shown
Set Elevation B so that top of check wall is at Control Water Surface Elevation
Gate frame height measured from center of gate opening



PLAN OF PRECAST CONCRETE DECK

	STR. NO.	MAX. Q, cfs	SLIDE GATE WIDTH X HEIGHT*	SLIDE GATE FRM. HT.	STANDARD DIMENSIONS B	C	L	M	S	X	N	ESTIMATED QUANTITIES CONC., CU. YDS.	REINF. STEEL, LBS.	MISC. METAL, LBS.
	1	10	36"x12"	5'	3'-0"	14"	8'-6"	12"	9'-9"	7'-9"	2	6.2	780	160
	2	15	36"x18"	6'	3'-0"	20"	9'-6"	2'-0"	11'-3"	4'-7"	3	7.3	900	200
	3	21	36"x24"	6'	3'-0"	2'-2"	9'-6"	2'-0"	12'-9"	5'-4"	3	8.0	1000	220
	4	26	36"x30"	6'	3'-0"	2'-8"	10'-6"	3'-0"	14'-3"	6'-1"	3	9.3	1100	240
	5	21	48"x18"	5'	4'-0"	21"	9'-6"	2'-0"	12'-6"	5'-3"	3	7.9	980	210
	6	28	48"x24"	6'	4'-0"	2'-3"	9'-6"	2'-0"	14'-0"	6'-0"	3	8.6	1050	230
	7	35	48"x30"	6'	4'-0"	2'-9"	10'-6"	3'-0"	15'-6"	6'-9"	3	9.9	1200	250
**	8	42	48"x36"	6'	4'-0"	3'-3"	11'-6"	4'-0"	17'-0"	7'-6"	3	11.2	1350	400
	9	35	60"x24"	6'	5'-0"	2'-3"	9'-6"	2'-0"	15'-0"	6'-6"	3	9.0	1130	240
	10	43	60"x30"	6'	5'-0"	2'-9"	10'-6"	3'-0"	16'-6"	7'-3"	3	10.3	1250	260
**	11	52	60"x36"	6'	5'-0"	3'-3"	11'-6"	4'-0"	18'-0"	8'-0"	3	11.7	1390	420
	12	42	72"x24"	6'	6'-0"	2'-3"	9'-6"	2'-0"	16'-0"	7'-0"	3	9.4	1180	240
	13	53	72"x30"	6'	6'-0"	2'-9"	10'-6"	3'-0"	17'-6"	7'-9"	3	10.7	1300	260
**	14	63	72"x36"	6'	6'-0"	3'-3"	11'-6"	4'-0"	19'-0"	5'-8"	4	12.1	1440	430

* When a gate of specified height is not available a gate with greater available height shall be used with appropriate frame height.
** Upstream handrail required

Figure 3-14. **Standard dimensions for a 3-foot check-drop structure. (Sheet 2 of 2.) 103-D-1212**

D. TURNOUTS

D. L. WINSETT [2]

3-11. ***Purpose and Description.***—Turnouts are used to divert water from a supply channel to a smaller channel. The structure will usually consist of an inlet, a conduit or a means of conveying water through the bank of the supply channel and, where required, an outlet transition. Gates are generally used in the inlet to control the flow. Pipes are generally used to carry the water through the bank of the supply channel. The conduit and outlet sections of the structure may be designed as part of another type of structure such as a siphon or a drop or they may connect to a water measurement structure.

3-12. ***Design Considerations.***—(a) *Velocity.*—For structures not having a concrete outlet transition and discharging into an unlined channel, the maximum velocity in the pipe should be about 3.5 feet per second. If an outlet transition is provided, the maximum velocity in the pipe may be increased to about 5 feet per second. A minimum pipe diameter of 18 inches is generally used.

(b) *Percolation and Stability.*—The structure must provide a percolation path of sufficient length to prevent the removal of foundation materials by seepage water. It may be necessary to attach collars to the conduit to increase the length of the percolation path. For a discussion of percolation and the design of collars see subchapter VI C. Turnout inlets must be stable against flotation and sliding.

(c) *Hydraulics.*—The amount of submergence on the conduit depends on the type of turnout inlet and outlet used. The submergence should be in accordance with the paragraphs dealing with the particular type of structure used. The proper submergence promotes smoothness of flow and the accuracy of water measurement devices where they are incorporated into the turnout structure. The design examples demonstrate the computation of losses through a turnout structure. When there is excessive head across the structure, the conduit and outlet portions of the turnout must be designed as a drop structure. If the design delivery water surface elevation is higher than the maximum elevation to which the turnout can deliver the required flow, some means must be found of either reducing the losses through the structure or raising the control water surface elevation in the supply channel. The use of larger pipe will reduce the losses a small amount. The control water surface may be raised by moving the check that maintains the water surface, closer to the turnout. If the required head cannot be obtained by the above methods, it may be necessary to relocate the turnout or to pump to the required delivery water surface elevation.

(d) *Miscellaneous.*—Turnout inlets should be set so that they do not protrude into the canal prism. The slope of the sidewalls should be such that they will not interfere with cleaning and maintaining the canal.

Cast iron slide gates with machined seats are used as turnout gates to insure watertightness when gates are closed. Steel slide gates are used as orifice gates in constant-head orifice structures.

For watertightness, Reclamation requires rubber gasketed joints in pipe used as a conduit. The air vent shown on figures 3-15, 3-29, and 3-30 is to prevent the formation of a vacuum downstream of the turnout gate and may be omitted when the turnout gate is submerged by the design delivery water surface.

3-13. ***Turnouts Without Water Measurement.***—If water measurement at the turnout is not required, the turnout structure will require only proper sizing and setting to deliver the required flow from the supply channel. Figure 3-15 shows a straight, through-the-bank turnout with neither an outlet transition nor a provision for measurement. The following example shows how this type of structure is set into a canal bank to fulfill a given set of conditions.

3-14. ***Design Example.***—Design a straight, through-the-bank turnout to divert a Q of 25 cfs from a supply channel to an earth ditch. Neither an outlet transition nor water

[2] Op. cit., p. 134.

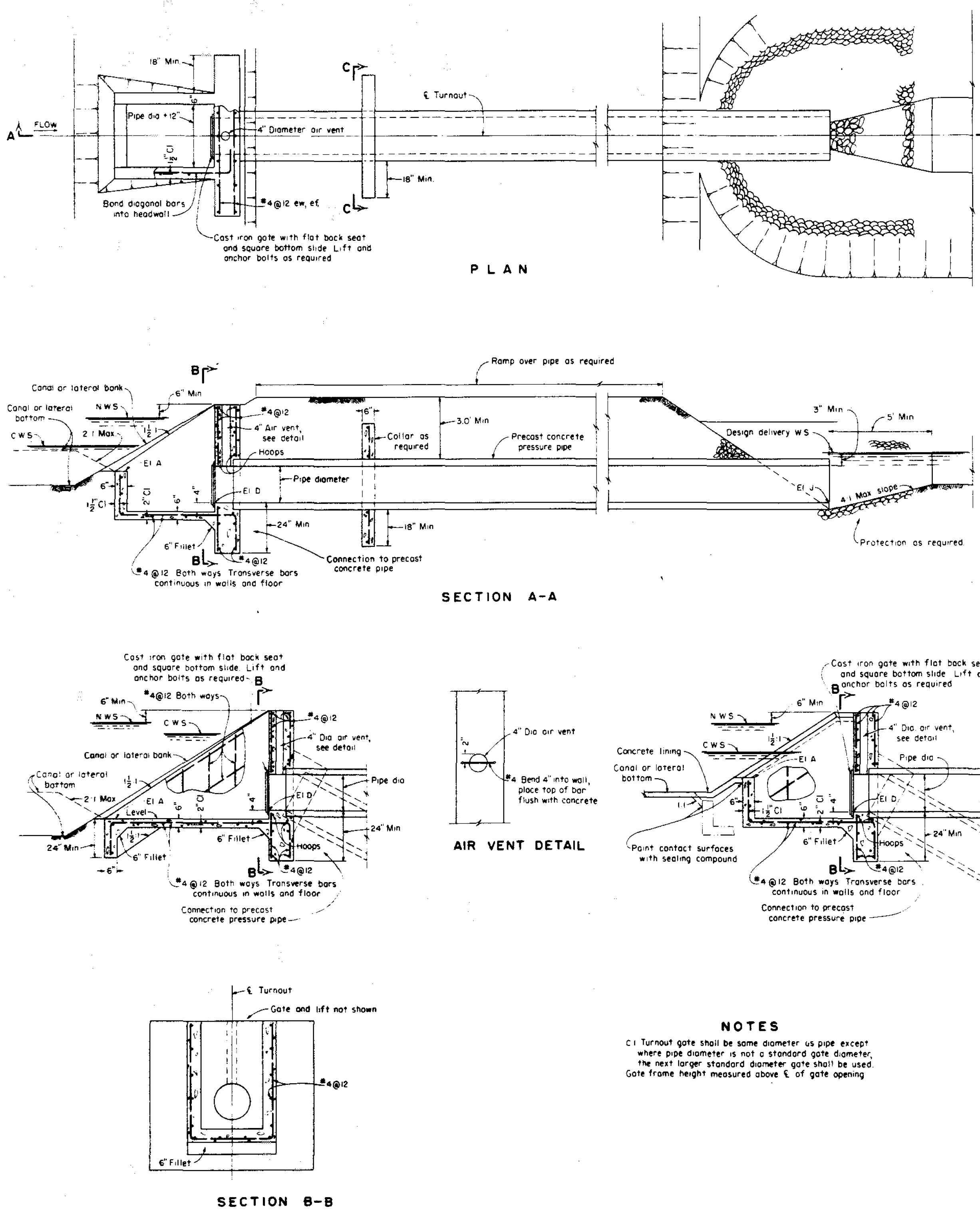


Figure 3-15. **Turnout. 103-D-1213**

measurement is required. The control water surface in the supply channel is at elevation 1217.58. Determine the size of pipe to be used and the maximum elevation to which 25 cfs can be delivered.

Without an outlet transition the velocity in the pipe should be limited to about 3.5 feet per second. Consider a 36-inch-diameter pipe which has an area of 7.07 square feet. Pipe properties may be obtained from tables or computed as below:

Pipe velocity, $V = \frac{Q}{A} = 3.5$ fps

Velocity head in pipe, $h_v = \frac{V^2}{2g} = 0.19$ ft.

Wetted perimeter of pipe, $wp = \pi \times$ diameter $= 9.4$ ft.

Hydraulic radius of pipe, $R = \frac{A}{wp} = 0.76$ ft.

Friction slope of pipe, $s = \left(\frac{nv}{1.486R^{2/3}}\right)^2$

$s = 0.0014$ ft./ft.

Manning's coefficient of roughness for precast concrete pipe, $n = 0.013$

The head loss through this structure is comprised of:

(1) A loss at the entrance to the pipe. This loss is assumed to be 0.78 times h_v where 0.78 is the entrance loss coefficient, or

$$0.78 \times 0.19 = 0.15 \text{ ft.}$$

(2) Pipe losses.–In this example friction is the only pipe loss incurred. The pipe loss equals the length of pipe times the friction slope of the pipe (s). If 50 feet of pipe are required to convey the water through the canal bank, the loss will be:

$$50 \times 0.0014 = 0.07 \text{ ft.}$$

(3) A loss at the exit from the pipe. This loss equals 1 times h_v (chapter VII) or:

$$1 \times 0.19 = 0.19 \text{ ft.}$$

The total loss through the structure equals the sum of the above losses:

$$0.15 + 0.07 + 0.19 = 0.41 \text{ ft.}$$

The highest elevation to which this turnout can deliver 25 cfs is:

$$1217.58 - 0.41 = 1217.17$$

Set elevation D (fig. 3-15) so that the control water surface in the supply channel submerges the top of the pipe at the turnout headwall a distance equal to 1.78 times h_v plus 3 inches minimum:

$$1.78 \times 0.19 + 0.25 = 0.59 \text{ ft.}$$

To determine elevation D:

Control water surface elevation		1217.58
– Submergence on top of pipe	–	0.59
Elevation of the top of pipe		1216.99
– Pipe diameter	–	3.00
Elevation D		1213.99
– 4 inches (fig. 3-15)	–	0.33
Elevation of turnout floor		1213.66

Determine elevation A:

The elevation at the cutoff (El. A) should be set to prevent this point from controlling the flow through the structure.

The width of the structure at the cutoff is equal to the pipe diameter plus 12 inches or in this example, 4 feet.

Assume that the cutoff acts as a suppressed rectangular weir. The head (h) required on a 4-foot weir for a discharge of 25 cfs is obtained from weir tables, or h = 1.6 feet.

Control water surface elevation		1217.58
– h	–	1.60
Maximum elevation A		1215.98

If additional width at the cutoff is required, the sidewalls may be flared 8 to 1.

Choose an elevation A that satisfies both the above criteria and the conditions found in the field. Set elevation J to provide the submergence at the pipe outlet (0.25 ft.) as shown on figure 3-15.

Delivery water surface elevation	1217.17
– Submergence at pipe outlet	– 0.25
Elevation top of pipe at outlet	1216.92
– Pipe diameter	– 3.00
Elevation J	1213.92

Select a class of pipe that will withstand the internal and external loads to which it will be subjected, and provide an earth transition and protection at the outlet as required.

Figures 3-16 and 3-17 show completed precast concrete turnout inlets in the casting yard. Figures 3-18 through 3-21 show typical turnout installations using both precast and cast-in-place concrete transitions.

3-15. ***Turnouts With Measurement.***– (a) *Flow Meters.*–Open-flow meters can be used to measure the amount of water diverted. They are propeller devices that are especially useful where the head available for measurement is limited. They may be attached to the outlet of a structure as shown on figure 3-22. The outlet must be designed to force the conduit to flow full. The flow meter may be detachable allowing it to be used on more than one structure. Line meters are similar to open-flow meters but are installed in pipelines.

Because trash in the water can foul or damage these meters, racks or screens are sometimes installed on the turnout inlet to protect them. See chapter IV for a discussion of flow meters. Figure 3-23 shows the brackets on the outlet headwall that hold the flow meter and the slots into which stoplogs are placed to force the conduit to flow full.

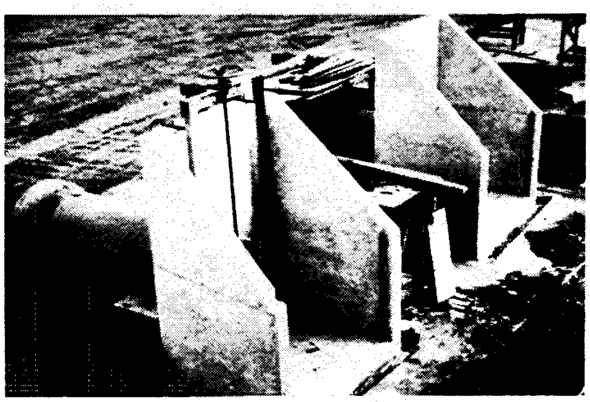


Figure 3-16. **Precast concrete turnout inlets. PX-D-72616 D**

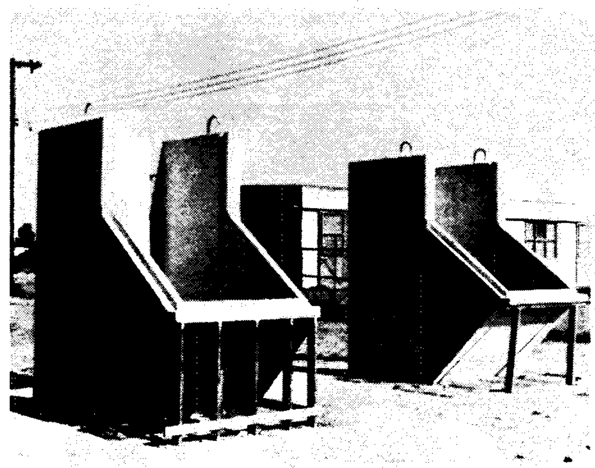


Figure 3-17. **Completed turnout inlets in casting yard. PX 303-602**

Figure 3-18. **Lateral turnout gates. PX-D-72619 D**

Figure 3-19. **Turnout structure in earth channel. P214-D-73783**

(b) *Parshall Flumes and Weirs.*—Parshall flumes and weir structures are sometimes incorporated into turnout designs to measure flows where the required head is available. They should be designed in accordance with the criteria set forth in chapter IV.

(c) *Constant-head Orifice Structures.*—The constant-head orifice structure was developed by Reclamation to both regulate and measure the flow of water. It may be used as an inline structure or as a turnout inlet. Figures 3-24 through 3-26 show various details of the construction and operation of constant-head orifice turnouts.

At least two gates are required for the structure to operate. The first gate, the upstream gate, controls the size of the rectangular orifice. A second gate, the

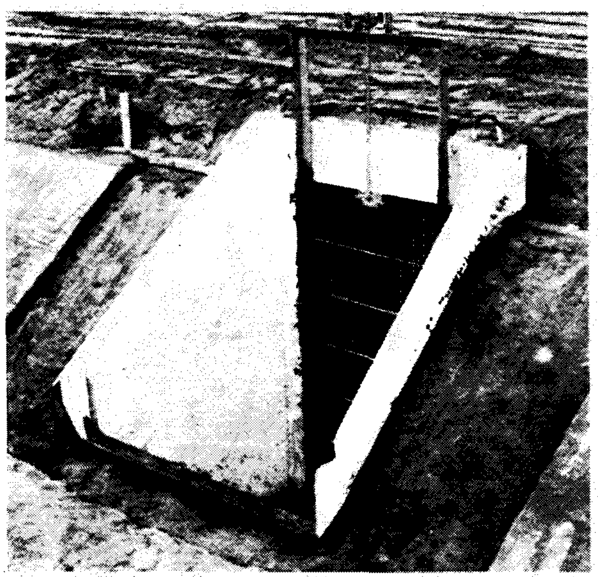


Figure 3-20. **Precast turnout inlet in place in canal bank. Note concrete lining blocked out around structure. PX 303-429 NA**

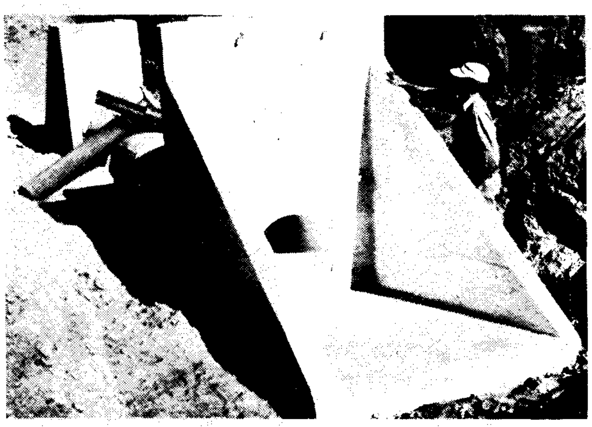


Figure 3-21. **Completed turnout structure ready for backfilling. P 328-701-6580**

downstream gate, controls the water depth below the orifice and is operated to maintain the head across the orifice at a constant value. The flow through the structure is varied by changing the area of the orifice. The approximate flow through the structure can be computed from the equation:

$$Q = CA\sqrt{2gh}$$

The coefficient of discharge (C) is generally considered to be 0.7. Constant-head orifice

Figure 3-22. **Flow through turnout being measured with an open-flow meter. Discharge is 1.7 cfs. P 499-700-615.**

structures are normally operated with a head differential (h) of 0.2 foot but where the head is available a greater differential may be used. Staff gages are used to measure the head across the orifice.

Approximate discharge tables for two orifice gate sizes, 24 by 18 inches and 30 by 24 inches operating under a head of 0.2 foot are shown in tables 3-1 and 3-2. The tables show discharges with both one and two gates in operation. For larger flows, more or larger openings are used.

3-16. ***Design Considerations.***—The following criteria are for constant-head orifice structures with capacities of up to 30 cfs (fig. 3-27).

Area of orifice required:

$$A = \frac{Q}{C\sqrt{2gh}}$$

where:

C = orifice coefficient = 0.7, and
h = differential head = 0.2 ft.

Figure 3-23. **Farm turnout outlet. Brackets on headwall hold open-flow meter. Angles in foreground hold stoplogs to force conduit to flow full. P 499-700-383.**

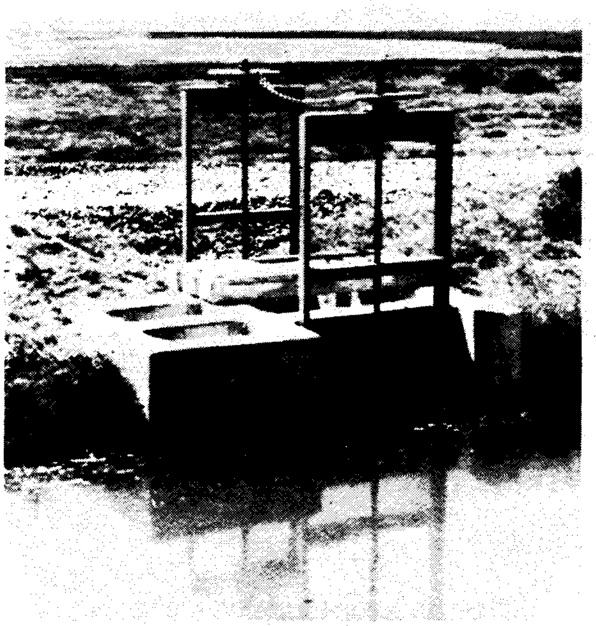


Figure 3-24. **Constant-head orifice turnout with stilling wells. Discharge is 7 cfs. PX-D-72604 D**

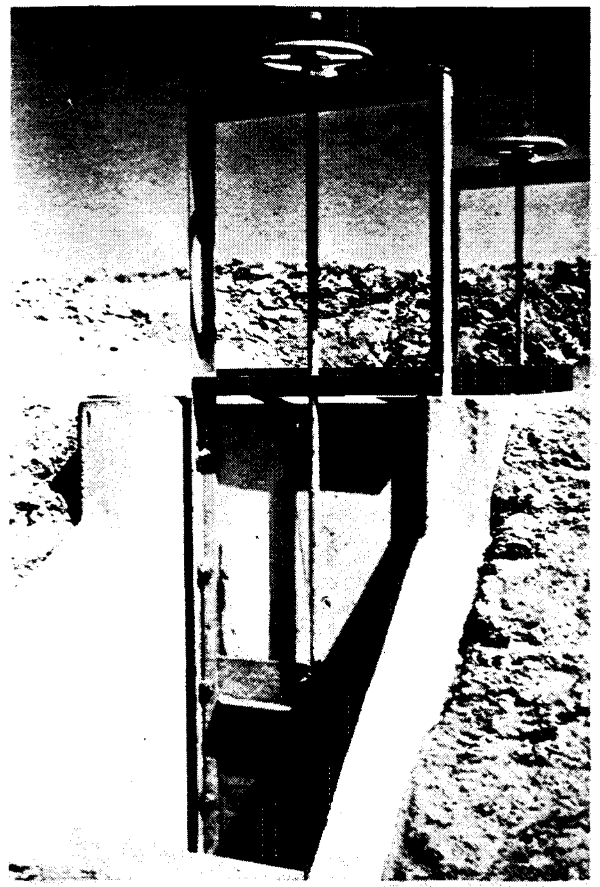


Figure 3-25. **Constant-head orifice turnout inlet. PX-D-72618 D**

This area should be provided in 75 to 80 percent of the gate rise.

The upstream submergence on the orifice gate (s) should be equal to or greater than the orifice gate opening (Y_m) at maximum capacity. The submergence on the turnout gate opening should be 1.78 times the velocity head of the full pipe plus 3 inches minimum.

A level floor length equal to the height of the orifice gate opening (Y_m) should be provided in front of the orifice gates. For structures with capacities up to 10 cfs the minimum distance between the orifice gate and the turnout gate (L) should be 2-1/4 times the orifice gate opening for maximum flow (Y_m) or 1-3/4 times the wall opening (Y_t) whichever is greater (3-foot 6-inch minimum). For structures with capacities greater than 10 cfs, L should be 2-3/4 times the height of the orifice gate opening (Y_m). The width of the structure

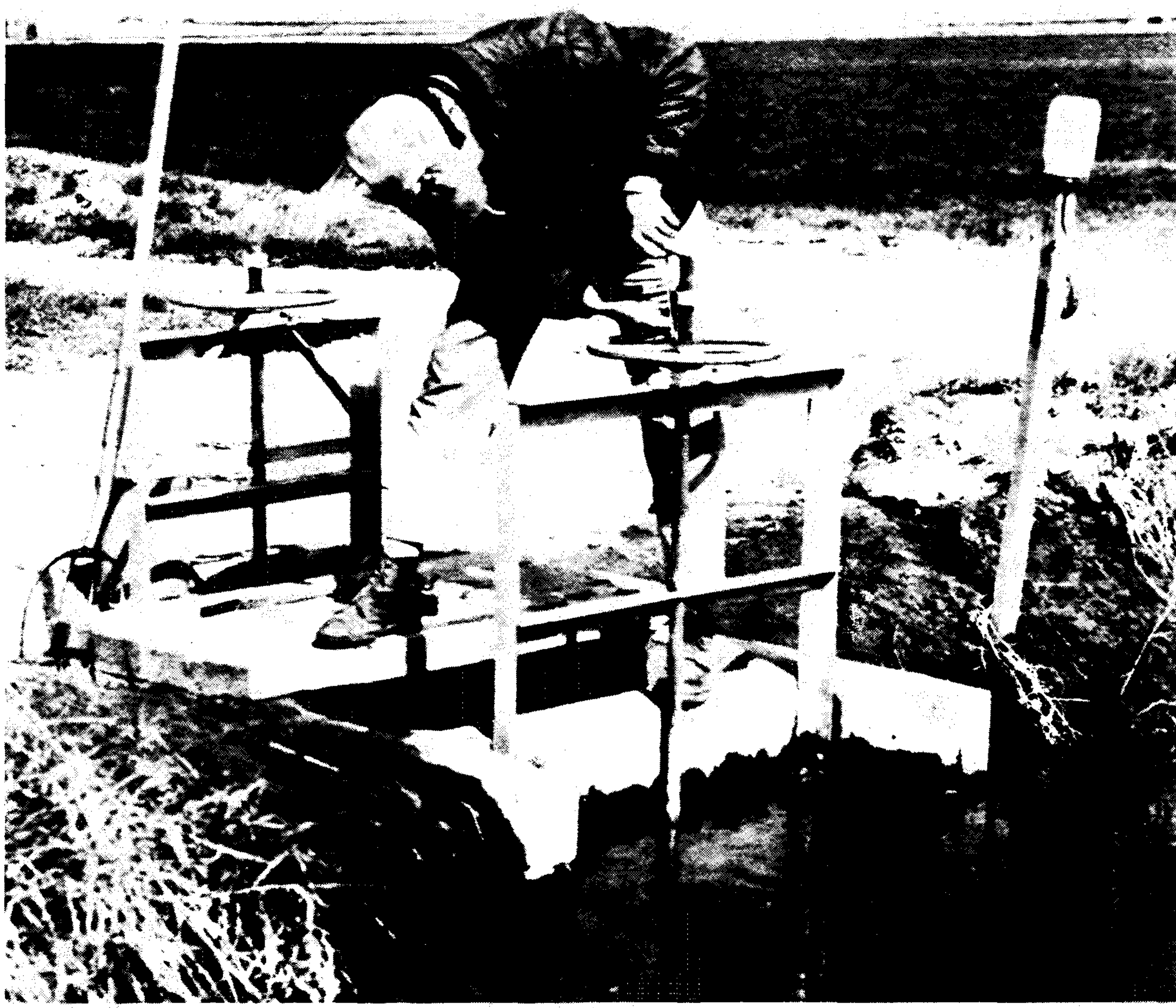


Figure 3-26. **Operator adjusting orifice gate on a constant-head orifice turnout. P222-116-52046 A**

must accommodate the gates used and the width at the cutoff must be great enough to prevent it from becoming a control. The sidewalls may be flared 8 to 1 if additional width at the cutoff is needed.

The conduit and outlet section of the structure should be designed as a siphon or drop if conditions dictate. An air vent behind the turnout gate should be provided when the pipe profile will permit a vacuum to form downstream of the turnout gate.

The above criteria are for structures with capacities up to 30 cfs. For larger flows special structures with multiple gates and conduits are designed to suit conditions.

The accuracy of a constant-head orifice

structure is affected by:

(1) Flow conditions upstream from the orifice gate,
(2) Flow conditions downstream from the gate,
(3) Errors in reading the head differential, and
(4) Obstruction of the orifice by trash in the water.

With the accuracy of the structure dependent upon so many variables, field rating of the structure should be made when accuracies of better than about 10 percent are required.

A constant-head orifice structure built inline can perform checking and measuring functions in the canal (fig. 3-28). One or more orifices with gates are built into a wall that extends across the channel. A second wall with gated openings is built at the downstream end of the structure. These structures also use the principle of a submerged orifice under a known head and are operated in the same manner as smaller constant-head orifice structures. The head differential is measured by either piezometers or staff gages. Generally discharge curves for these larger structures are prepared from field rating data.

The following example demonstrates the procedure in designing a constant-head orifice turnout.

3-17. ***Design Example.***—Refer to figure 3-29. Design a straight, through the bank turnout to divert 20 cfs from a supply channel to an earth ditch. The diversion is to be measured with a constant-head orifice inlet. A broken-back outlet transition is to be provided. The control water surface in the supply channel is at elevation 1217.58. Determine the size of pipe to be used. Determine the maximum elevation to which 20 cfs can be delivered.

With an outlet transition, the full pipe velocity may be about 5 feet per second. Consider a 30-inch-diameter pipe with an area (A) of 4.9 square feet. Pipe properties may be obtained from tables or computed as below:

Pipe velocity $V = \dfrac{Q}{A} = \dfrac{20}{4.9} = 4.1$ feet per second; therefore, a 30-inch-diameter pipe is satisfactory.

Pipe velocity head $h_v = \dfrac{V^2}{2g} = 0.26$ ft.

Wetted perimeter of the pipe (wp) = π x diameter or π x 2.5 = 7.9 ft.

Hydraulic radius of the pipe $R = \dfrac{\text{area}}{\text{wp}}$ or $\dfrac{4.9}{7.9}$ = 0.62 ft.

The coefficient of roughness n for precast concrete pipe is assumed to be 0.013.

Friction slope of the pipe $s = \left(\dfrac{nv}{1.486R^{2/3}}\right)^2$ = 0.0024 ft./ft.

Determine the area of the orifice gate:

$$\text{Area} = \frac{Q}{C\sqrt{2gh}}$$

Assume C = 0.7, h = 0.2 ft.
A = 8 square feet

Consider an orifice gate 42 inches wide by 36 inches high. The area equals 3.5 x 3.0 = 10.5 square feet.

Determine the gate opening to provide an area of 8 square feet:

$$\text{Gate rise} = \frac{8}{3.5} = 2.3 \text{ ft.}$$

$$\frac{2.3}{3} = 0.77 \text{ or } 77 \text{ percent}$$

From above, the gate must be 77 percent open to provide the required area. This meets the requirement that the orifice area must be furnished in 75 to 80 percent of the gate rise. To accommodate a gate 42 inches wide the width of the box must be approximately 5 feet. A 30-inch-diameter cast iron turnout gate will fit both the pipe and the size of box selected.

A level floor length equal to the orifice gate opening (2.3 feet minimum) should be provided in front of the orifice gate.

Because the design capacity is greater than 10 cfs, the length of the measuring well should be 2-3/4 times 2.3 or 6.33 feet minimum. The orifice gate opening should be submerged a distance equal to the orifice gate opening (2.3 feet minimum). The elevations pertaining to the orifice will be:

Table 3-1.–*Discharge of constant-head orifice turnout in cubic feet per second. Capacity 20 cfs, gate size 30 x 24 inches,* Δh *= 0.20 foot.* 103-D-1214

Discharge, cfs	Gate opening in feet		Discharge, cfs	Gate opening in feet	
	2 gates	1 gate		2 gates	1 gate
0.25	0.02	0.04	10.25	0.81	
.50	.04	.08	10.50	.83	
.75	.06	.12	10.75	.85	
1.00	.08	.16	11.00	.87	
1.25	.10	.20	11.25	.89	
1.50	.12	.24	11.50	.91	
1.75	.14	.28	11.75	.93	
2.00	.16	.32	12.00	.95	
2.25	.18	.36	12.25	.97	
2.50	.20	.40	12.50	.99	
2.75	.22	.44	12.75	1.01	
3.00	.24	.48	13.00	1.03	
3.25	.26	.52	13.25	1.05	
3.50	.28	.56	13.50	1.07	
3.75	.30	.60	13.75	1.085	
4.00	.32	.64	14.00	1.10	
4.25	.34	.68	14.25	1.12	
4.50	.36	.72	14.50	1.14	
4.75	.38	.755	14.75	1.16	
5.00	.40	.79	15.00	1.18	
5.25	.42	.83	15.25	1.20	
5.50	.44	.87	15.50	1.22	
5.75	.46	.91	15.75	1.24	
6.00	.48	.95	16.00	1.26	
6.25	.50	.99	16.25	1.28	
6.50	.52	1.03	16.50	1.30	
6.75	.54	1.065	16.75	1.32	
7.00	.56	1.10	17.00	1.34	
7.25	.58	1.14	17.25	1.355	
7.50	.60	1.18	17.50	1.37	
7.75	.62	1.22	17.75	1.39	
8.00	.64	1.26	18.00	1.41	
8.25	.66	1.30	18.25	1.43	
8.50	.68	1.34	18.50	1.45	
8.75	.70	1.375	18.75	1.47	
9.00	.72	1.41	19.00	1.49	
9.25	.74	1.45	19.25	1.51	
9.50	.76	1.49	19.50	1.53	
9.75	.775	1.525	19.75	1.545	
10.00	.79	1.56	20.00	1.56	

Table 3-2.–*Discharge of constant-head orifice turnout in cubic feet per second. Capacity 10 cfs, gate size 24 x 18 inches, Δh = 0.20 foot.* 103-D-1215

Discharge, cfs	Gate opening in feet		Discharge, cfs	Gate opening in feet	
	2 gates	1 gate		2 gates	1 gate
0.25	0.025	0.05	5.25	0.525	
.50	.05	.10	5.50	.55	
.75	.075	.15	5.75	.575	
1.00	.10	.20	6.00	.60	
1.25	.125	.25	6.25	.625	
1.50	.15	.30	6.50	.65	
1.75	.175	.35	6.75	.675	
2.00	.20	.40	7.00	.70	
2.25	.225	.45	7.25	.722	
2.50	.25	.50	7.50	.74	
2.75	.275	.55	7.75	.765	
3.00	.30	.60	8.00	.79	
3.25	.325	.65	8.25	.815	
3.50	.35	.70	8.50	.84	
3.75	.375	.745	8.75	.865	
4.00	.40	.79	9.00	.89	
4.25	.425	.84	9.25	.915	
4.50	.45	.89	9.50	.94	
4.75	.475	.94	9.75	.965	
5.00	.50	.99	10.0	.99	

Control water surface in the canal		1217.58
– Submergence on the orifice gate (S)	–	2.30
Elevation top of orifice gate		1215.28
– Gate opening (Y_m)	–	2.30
Elevation of the turnout floor		1212.98
+ Gate height (Y_t)	+	3.00
Elevation top of orifice gate opening		1215.98

From figure 3-27, X_g = 3.00 – 2.30 = 0.70 ft.

The distance X_g must be greater than or equal to the thickness of the orifice wall (t). Then the maximum thickness of the orifice wall is limited to about 8 inches (0.67 ft.).

Elevations pertaining to the turnout gate are:

Elevation of the turnout floor		1212.98
+ 4 inches (fig. 3-29)	+	0.33
Elevation D		1213.31
+ Pipe diameter	+	2.50
Elevation top of pipe at headwall		1215.81

With a 0.2-foot head differential across the orifice the submergence on the top of the pipe at the headwall equals:

Control water surface in the canal		1217.58
– Head differential	–	0.20
Water surface elevation in the measuring box		1217.38

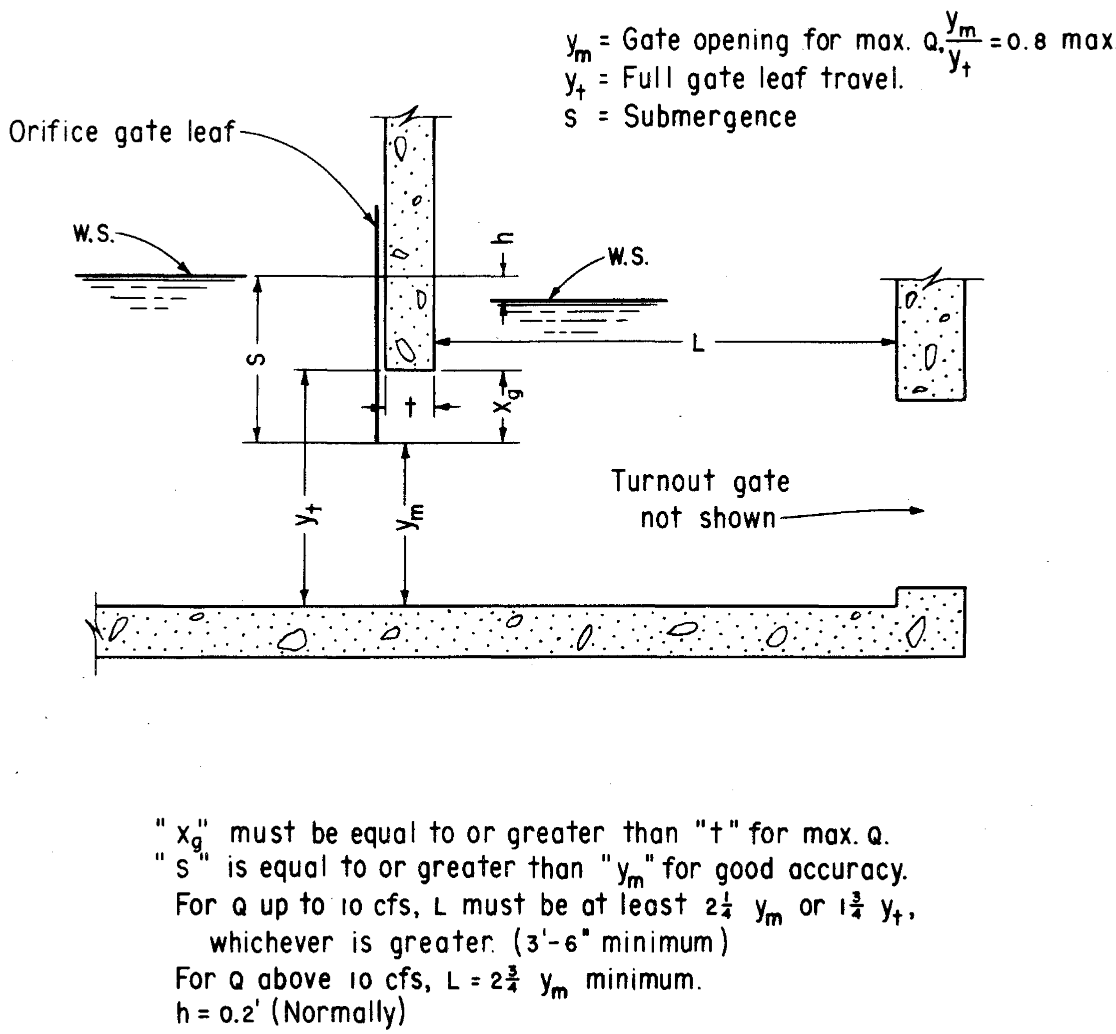


Figure 3-27. **Dimensions for a constant-head orifice. 103-D-1216**

Water surface elevation in the measuring box	1217.38
– Elevation top of pipe at headwall	– 1215.81
Submergence on turnout pipe	1.57 ft.

The required minimum submergence on the turnout pipe equals 1.78 times h_v plus 3 inches minimum = 1.78 x 0.26 + 0.25 = 0.71 ft. The elevations selected meet this requirement.

Determine elevation A:

The elevation at the cutoff (El. A) should be set to prevent this point from controlling the flow through the structure.

The width of the structure at the cutoff in this example is 5 feet. Assume that the cutoff acts as a suppressed rectangular weir. The head (h) required on a 5-foot weir for a discharge of 20 cfs is 1.2 ft. (table 5-6).

Control water surface elevation	1217.58
– h	– 1.20
Maximum elevation A	1216.38

Choose an elevation A that satisfies both the above criteria and the conditions found in the

Figure 3-28. **Constant-head orifice structure used as a check. Structure maintains upstream water surface and measures flow in the canal. PX-D-72605**

field. If additional width at the cutoff is required, the sidewalls may be flared 8 to 1.

The losses through the structure will consist of:

(1) Measurement loss (h) = 0.20 ft.

(2) A loss at the entrance to the turnout pipe equal to 0.78 times h_v or:

$$0.78 \times 0.26 = 0.20 \text{ ft.}$$

(3) Pipe loss. Assuming that the only pipe loss incurred in a 50-foot length of pipe is the friction, the pipe loss equals the length of pipe times the friction slope of the pipe (s):

$$50 \times 0.0024 = 0.12 \text{ ft.}$$

(4) A loss at the exit from the pipe equal to 0.7 times Δh_v (chapter VII) or:

$$0.7 \times 0.26 = 0.18 \text{ ft.}$$

The total loss across the structure is the sum of the above losses:

$$0.20 + 0.20 + 0.12 + 0.18 = 0.70 \text{ ft.}$$

The maximum elevation to which this structure can deliver 20 cfs is:

Control water surface elevation		1217.58
– Head loss in the structure	–	0.70
Maximum delivery water surface		1216.88

Submerge the top of the pipe 0.50 foot at the outlet transition headwall as shown on figure 3-29.

Maximum delivery water surface		1216.88
– Submergence on the top of pipe	–	0.50
Elevation top of pipe at outlet headwall		1216.38
– Pipe diameter	–	2.50
Elevation pipe invert at outlet headwall		1213.88

A class of pipe should be selected that will withstand the external and internal loads to which it will be subjected. The concrete pipe transition provided should fit the structure to the channel, and the channel should be protected to prevent erosion.

Figure 3-30 shows a number of constant-head orifice turnout structures with capacities greater than 30 cfs.

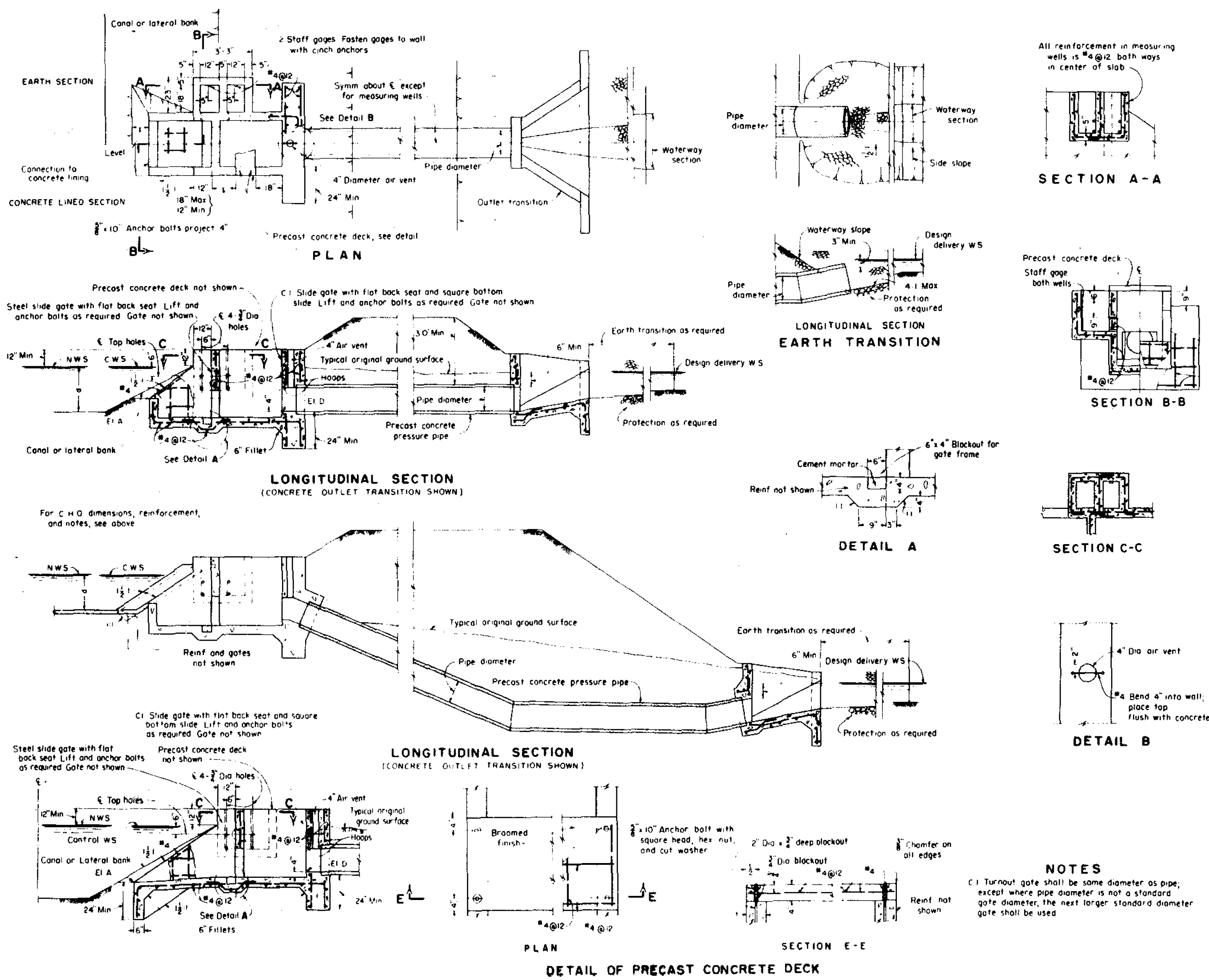
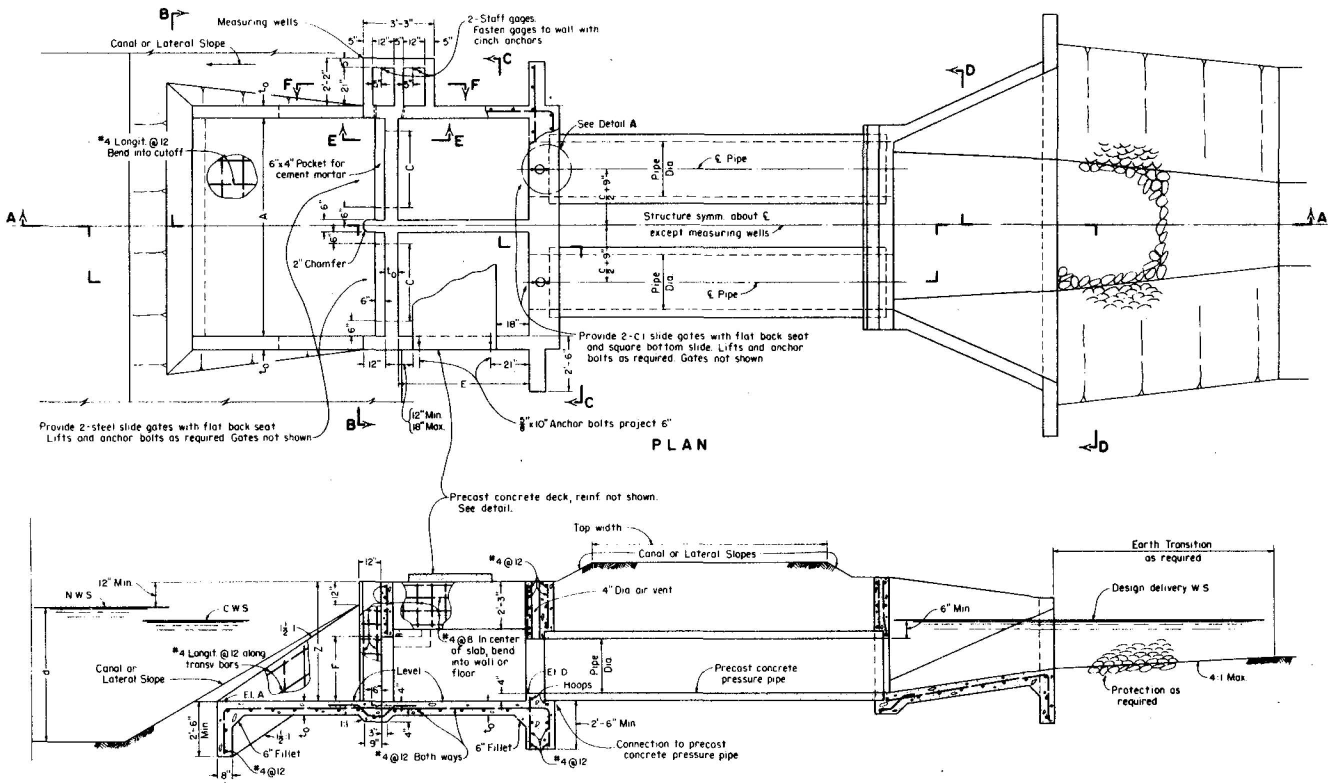


Figure 3-29. **Constant-head orifice turnout. 103-D-1217**



ORIFICE TURNOUT DIMENSIONS AND QUANTITIES											
STR. No.	MAX. Q	DIMENSIONS				STAFF GAGE LENGTH	ORIFICE GATE			TURNOUT GATE	
							SIZE		FRAME HEIGHT	DIA.	FRAME HEIGHT
		Z *	A	E	t_o		C	F			
1	35	5'-6"	8'-6"	5'-6"	6"	2'-6"	36"	30"	6'-0"		6'-0"
2	43	5'-6"	9'-6"	5'-6"	6"	2' 6"	42"	30"	6'-0"		6'-0"
3	51	6' 3"	9'-6"	7'-0"	6"	2'-9"	42"	36"	6'-0"		6'-0"
4	59	6'-3"	10'-6"	7'-0"	6"	2'-9"	48"	36"	6'-0"		6'-0"
5	68	7'-0"	10'-6"	8'-0"	7"	3'-0"	48"	42"	7'-0"		7'-0"
6	86	7'-0"	12'-6"	8'-0"	7"	3'-0"	60"	42"	7'-0"		7'-0"

*Based on a difference between NWS and CWS of approx 0.5 ft.

Figure 3-30. **Two-barrel constant-head orifice turnout. Capacity greater than 30 cfs (Sheet 1 of 2). 103-D-1218**

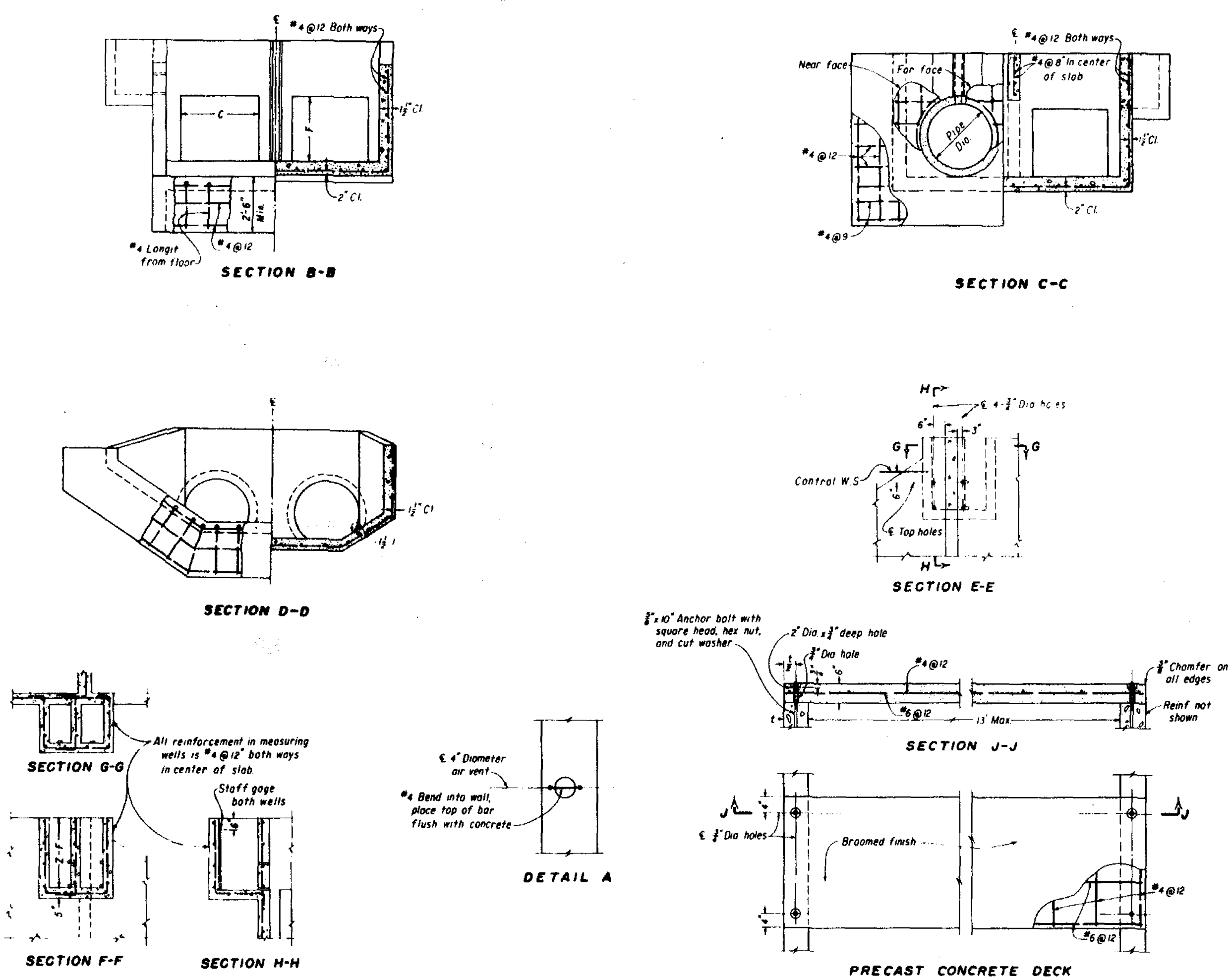


Figure 3-30. **Two-barrel constant-head orifice turnout. Capacity greater than 30 cfs (Sheet 2 of 2). 103-D-1218**

E. DIVISION STRUCTURES

D. L. WINSETT [2]

3-18. ***Purpose and Description.***—Division structures are used to divide the flow from a supply pipe or channel among two or more channels or pipes. The division structure may be a separate structure or it may be the outlet of a siphon, a drop, or a turnout from which further diversion is required. If measurement is not required at the point of division, the flow through the structure may be directed through the various outlets with gates or stoplogs. If the flow must be measured and the required head is available, weirs may be used to proportion the flow. Weirs that are both movable and adjustable are often used in division structures. Adjustable weirs can be adjusted vertically and provide a means of controlling the flow as well as a means of measurement. Movable weirs are mounted in frames that are inserted into grooves in the division structure and may be moved from one structure to another. To avoid reducing the accuracy of the weir measurement, the structure should be designed to provide suitable conditions for the weir as set forth in the discussion of weirs in chapter V. Weir discharges may be obtained from discharge tables or from flow formulas. The head on the weir is generally obtained from staff gages mounted in the structure.

[2] Op. cit., p. 134.

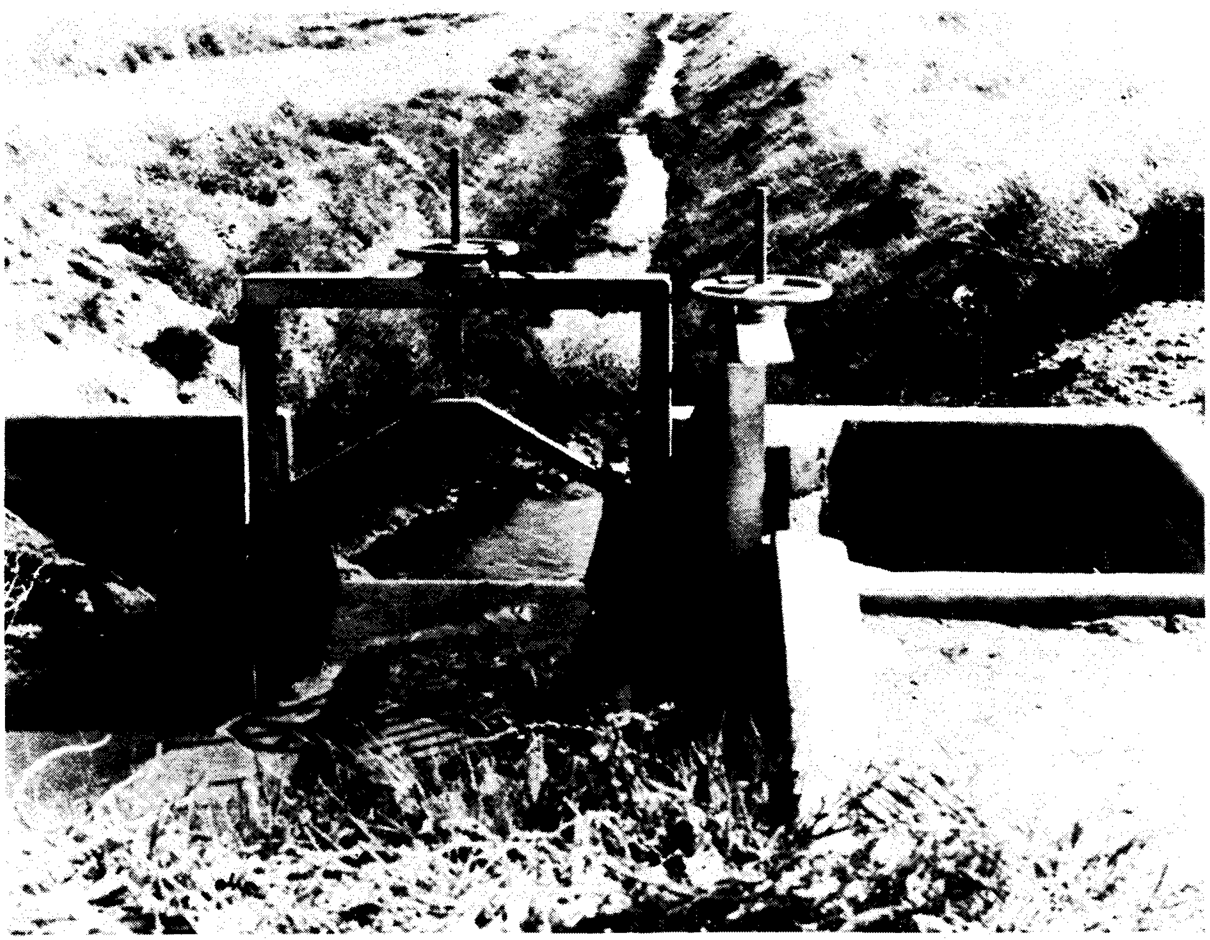


Figure 3-31. **Division structure using 2-foot adjustable weirs. Design Q 3 cfs over each weir. PX-D-72614 D**

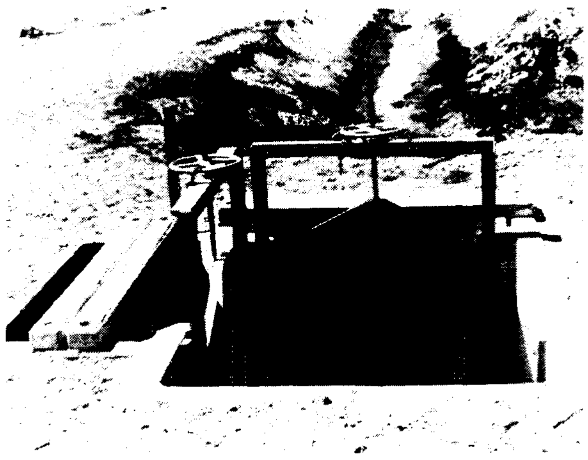


Figure 3-32. **Pipe division box. Note 3-foot adjustable weirs center and left. P222-116-53582**

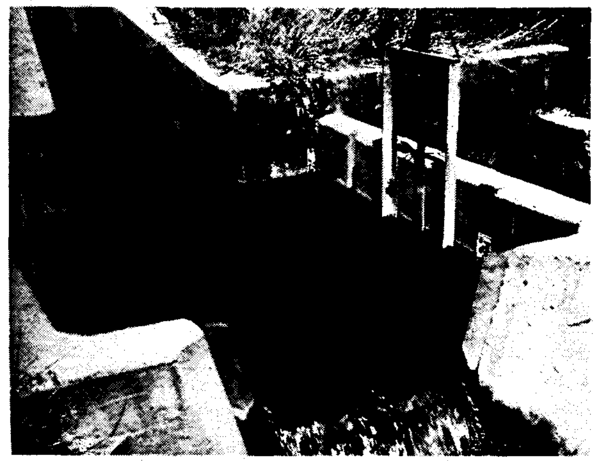


Figure 3-33. **Division structure with flow over 18-inch weir in foreground. P 33-D-33913**

Figures 3-31 and 3-32 show division structures using adjustable weirs to proportion the flow. Figure 3-33 shows a structure with a fixed weir in operation. Figure 3-34 shows the slots which could accept stoplogs, gates in frames, or movable weirs.

Various combinations of supply and delivery are shown on figures 3-35 and 3-36. In practice any of the possible combinations may be encountered. The pool shown ahead of the structures is an attempt to approach standard conditions to increase the accuracy of any weir measurement made in the division structures.

Design considerations.–Water must be delivered to the division structure at an elevation that will provide enough head on the weirs to furnish the required flow at the required delivery water surface elevation.

When delivery to the division structure is through a pipe, the velocity in the pipe is held to about 1.5 feet per second to reduce turbulence ahead of the weirs.

Figure 3-34. **Division structure used as outlet to turnout. P 328-701-7166**

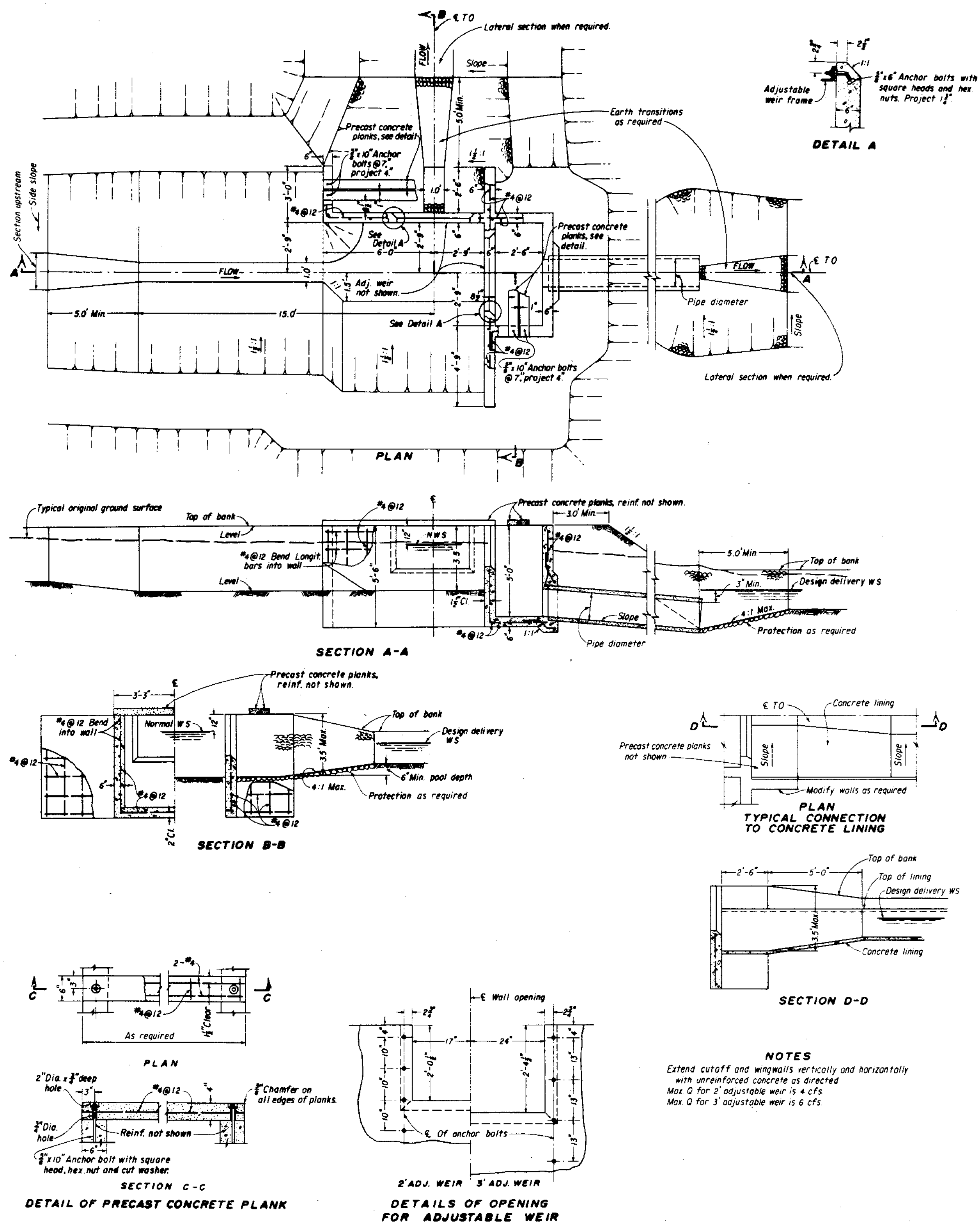


Figure 3-35. **Division structure. 103-D-1219**

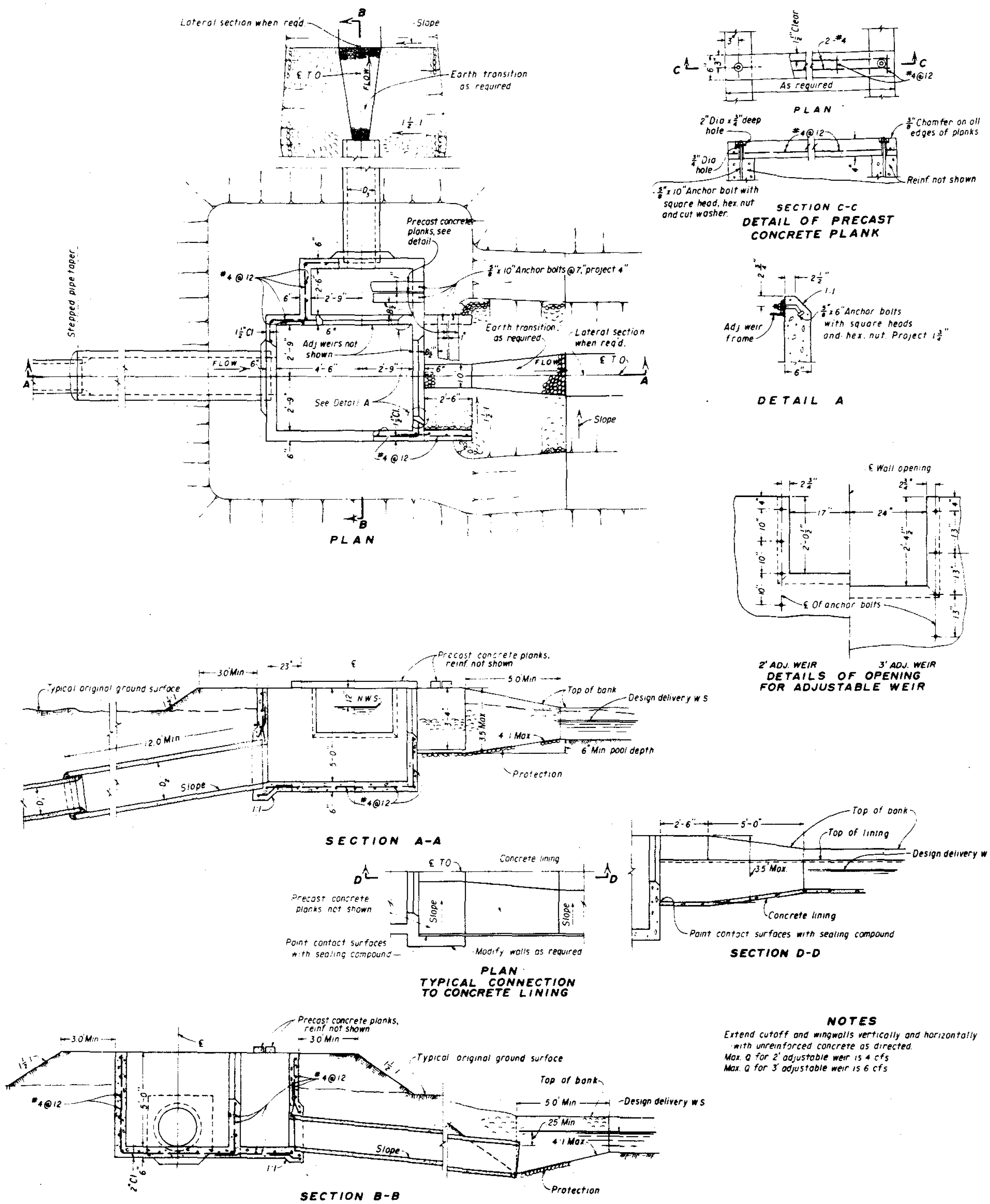


Figure 3-36. **Division structure with pipe inlet. 103-D-1220**

F. CHECK AND PIPE INLETS

R. B. YOUNG[3]

3-19. ***Description and Purpose.***—The check and pipe inlet is a box-type structure on the upstream end of a pipe (see fig. 3-37). The inlet to the box is an opening that provides for installation of either a gate or stoplogs. Walk planks covering the top of the box provide a platform for operation of the gate or stoplogs.

The purpose of a check and pipe inlet is to maintain a water surface at a required elevation for turnout deliveries upstream with partial flows in the canal. By partially closing the inlet with the gate or stoplogs, the canal water surface can be controlled to the desired elevation. The sidewalls provide for emergency overflow into the box if the gate or stoplogs are not appropriately set for a particular flow in the canal; if the inlet becomes obstructed; or if the inlet is closed. This structure can also serve as a shutoff point in the canal. When water delivery downstream from the inlet is not required, or if a canal break should occur, complete closure of the inlet with diversion to a wasteway could isolate the appropriate canal reach.

3-20. ***Application.***—The check and pipe inlet is combined with and used on the upstream end of such structures as road crossings, inverted siphons, pipe drops, and pipe chutes where the capability of providing a checked water surface in the canal is required. Check spacing has been previously discussed in subchapter III B.

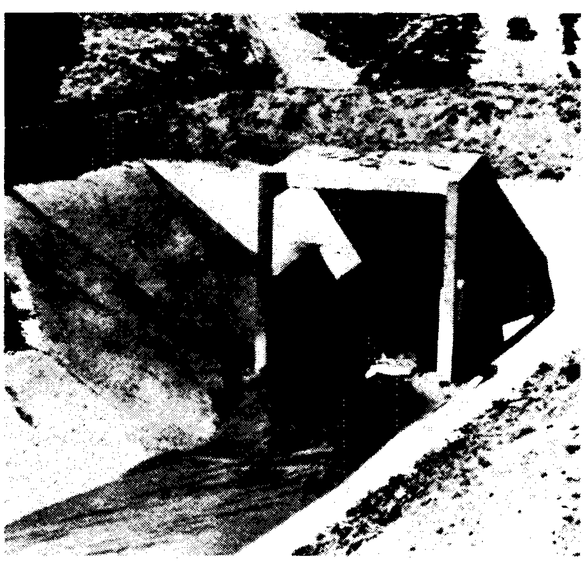


Figure 3-37. **Check inlet to 42-inch-diameter pipe siphon. P 222-116-48458**

3-21. ***Design Considerations.***—(a) *Upstream Transitions.*—The transition from the canal section produces a smooth change of velocity from canal to structure and provides for a gradual change in invert elevation from the canal to the opening in the structure. The invert slope should not be steeper than 4 to 1.

(b) *Protection.*—Protection is often used adjacent to structures in earth-lined canals where erosion may occur. For design of protection see the discussion on Protection in subchapter VII B.

(c) *Inlet Opening.*—The area of the inlet opening should be proportioned to limit the design flow velocity to about 3.5 feet per second. This velocity is considered as the maximum desirable for ease of stoplog handling. The width of the opening is set to reasonably approximate a canal base width for the design flow. The opening has gate frame guides for installation of a gate or for use as stoplog guides. If there is a need to operate the check automatically, a gate should be used. The elevation of the inlet opening should be the same or lower than the invert of the canal.

(d) *Sidewalls.*—The sidewalls should have a top elevation equal to the elevation of the normal canal water surface, irrespective of the control water surface elevation. If the inlet of the structure is inadvertently closed or improperly regulated, the flow in the canal will spill over the sidewalls and inlet closure. Referring to the structure in figure 3-38, assuming the extreme condition of flow at canal design capacity and complete closure of the inlet opening, there will still remain about 0.5 foot of canal bank freeboard provided the freeboard design was based on figure 1-9. The depth of flow over the walls and closure is dependent on the coefficient of discharge, the

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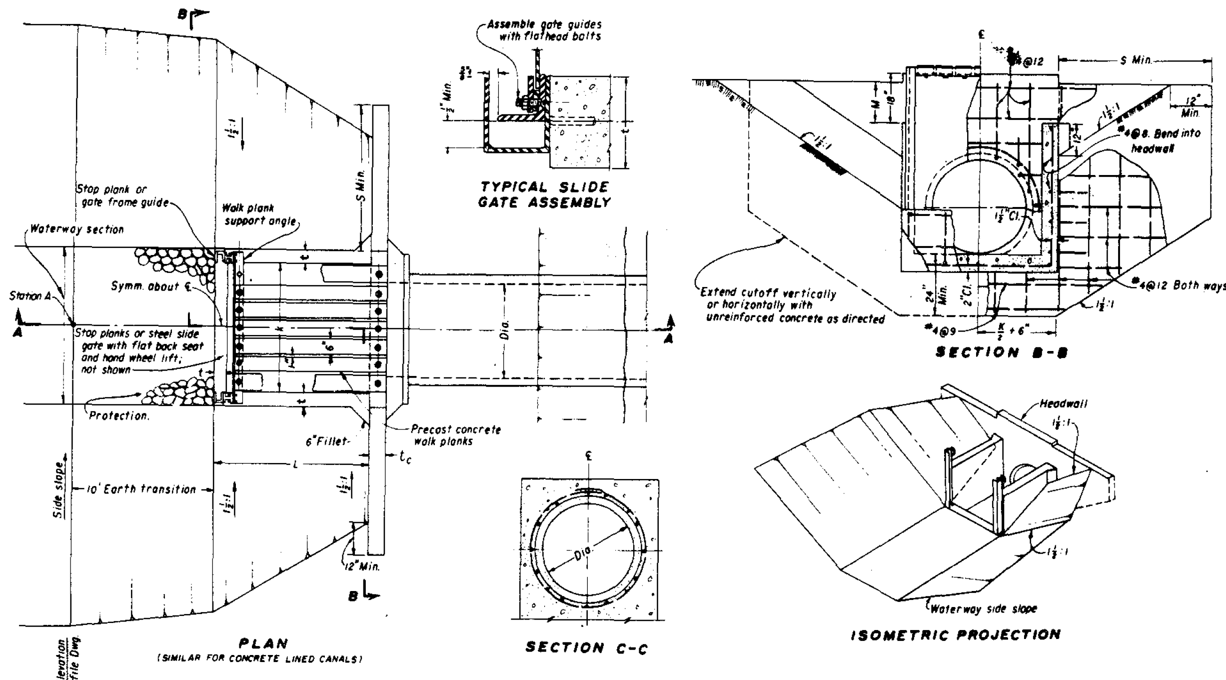
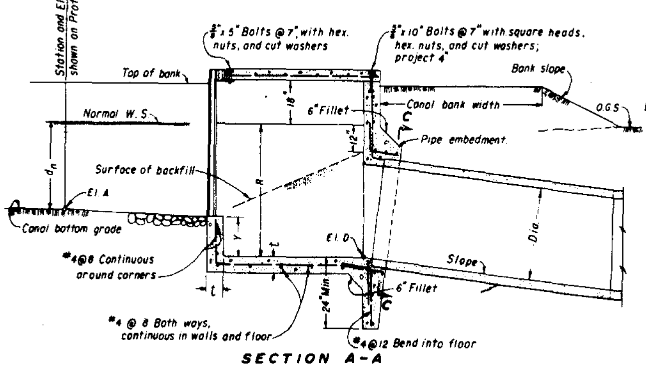
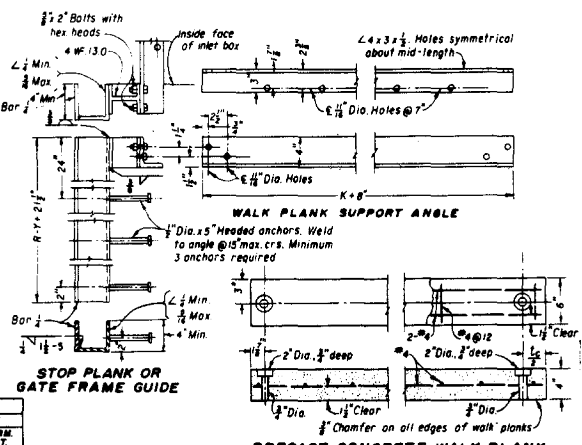






Before entering this table the Structure No. must have previously been determined from velocity criteria established for the pipe structure.

STRUCTURE No.	STANDARD DIMENSIONS: DIA.	K	R	L	Y	t	t_c	M	S	No. OF WALK PLANKS	ESTIMATED QUANTITIES: CONC. CU. YDS.	REINF. STEEL LBS.	MISC. METAL LBS.	SLIDE GATE: WIDTH x HT. *	FRM. HT.
12-1	12"	24"	2'-6"	3'-6"	15"	6"	6"	12"	4'-0"	3	1.6	160	110	24"x15"	5'
12-2	12"	2'-6"	5'-0"	4'-6"	3'-0"	6"	6"	12"	4'-0"	4	2.6	260	130	30"x24"	5'
15-1	15"	24"	2'-6"	3'-6"	15"	6"	6"	12"	4'-0"	3	1.6	160	110	24"x15"	5'
15-2	15"	3'-0"	5'-0"	4'-6"	3'-0"	6"	6"	12"	4'-0"	5	3.0	310	140	36"x24"	5'
18-1	18"	2'-6"	2'-9"	3'-6"	18"	6"	6"	12"	4'-0"	4	1.7	170	120	30"x15"	5'
18-2	18"	3'-0"	5'-0"	4'-6"	3'-0"	6"	6"	12"	4'-0"	5	3.0	310	140	36"x24"	5'
21-1	21"	3'-0"	3'-0"	4'-0"	15"	6"	6"	12"	4'-0"	5	2.1	220	130	36"x21"	5'
21-2	21"	3'-6"	5'-3"	5'-0"	3'-0"	6"	8"	12"	4'-0"	6	4.1	420	150	42"x27"	5'
24-1	24"	3'-0"	3'-3"	4'-0"	15"	6"	6"	12"	4'-0"	5	2.2	230	140	36"x24"	5'
24-2	24"	4'-0"	5'-6"	5'-6"	2'-9"	6"	8"	12"	4'-0"	7	4.5	460	170	48"x33"	6'
27-1	27"	3'-6"	3'-6"	4'-6"	15"	6"	6"	15"	4'-4"	6	2.6	270	150	24"x27"	5'
27-2	27"	5'-0"	5'-9"	6'-0"	2'-9"	6"	8"	15"	4'-4"	9	5.9	600	190	60"x36"	6'
30-1	30"	3'-6"	3'-9"	4'-6"	15"	6"	6"	15"	4'-4"	6	2.9	300	150	42"x30"	6'
30-2	30"	5'-0"	6'-0"	6'-6"	2'-9"	6"	8"	15"	4'-4"	9	5.9	600	200	60"x39"	6'
33-1	33"	4'-0"	4'-0"	4'-6"	15"	6"	6"	18"	4'-9"	7	2.8	290	160	48"x33"	6'
33-2	33"	6'-0"	6'-3"	7'-0"	2'-9"	6"	8"	18"	4'-9"	10	6.9	700	210	72"x42"	6'
36-1	36"	4'-0"	4'-3"	5'-0"	15"	6"	6"	18"	4'-9"	7	3.4	350	180	48"x36"	6'
36-2	36"	6'-0"	6'-6"	7'-6"	2'-6"	6"	8"	18"	4'-9"	10	7.3	740	210	72"x48"	7'
39-1	39"	5'-0"	4'-6"	5'-0"	15"	6"	6"	18"	4'-9"	9	3.8	390	190	60"x39"	6'
39-2	39"	6'-0"	6'-9"	8'-0"	24"	7"	8"	18"	4'-9"	10	7.6	770	240	72"x57"	8'
42-1	42"	5'-0"	4'-9"	5'-6"	15"	6"	6"	18"	4'-9"	9	4.1	420	200	60"x42"	6'
45-1	45"	5'-0"	5'-0"	6'-0"	15"	6"	6"	18"	4'-9"	9	4.2	430	200	60"x45"	6'
48-1	48"	5'-0"	5'-3"	6'-0"	18"	6"	8"	18"	4'-9"	9	5.2	530	200	60"x45"	6'
51-1	51"	5'-6"	5'-6"	6'-6"	15"	6"	8"	18"	4'-9"	9	5.8	590	220	66"x51"	7'
54-1	54"	6'-0"	5'-9"	6'-6"	18"	6"	8"	18"	4'-9"	10	6.1	620	230	72"x51"	7'
57-1	57"	6'-0"	6'-0"	7'-0"	15"	6"	8"	18"	4'-9"	10	6.5	660	240	72"x57"	8'
60-1	60"	6'-0"	6'-3"	7'-0"	18"	6"	8"	18"	4'-9"	10	6.7	680	240	72"x57"	8'

* When a gate with a specified height is not available, a gate with next greater available height shall be used with appropriate frame height.

NOTES

Gate frame height measured from ℄ of gate opening.

Structures are numbered to designate pipe diameter in inches followed by a second number, -1 or -2, which designate maximum allowable pipe velocities of 5 and 12 feet per second respectively. Structures -1 and -2 may also be used for maximum allowable pipe velocities of 3.5 and 10 feet per second respectively. Structure number 30-2 is for a 30 inch diameter pipe having a maximum allowable velocity of either 10 or 12 feet per second

Figure 3-38. **Check and pipe inlet. 103-D-1221** length of walls, the width of structure, and the flow in the canal. When a slide gate is selected from the table of dimensions (fig. 3-38), the top elevation of the gate, when closed, will be at approximately the same elevation as the top of the sidewalls. If the exact selected gate size is not available from the manufacturer and the next greater available height is used, the taller gate when closed with design flow in the canal may cause the canal freeboard to be less than 0.5 foot as the resultant water elevation will have been increased. If the resultant freeboard is much less than 0.5 foot, the sidewalls should be lengthened until the canal freeboard is about 0.5 foot when the gate is closed and normal canal flow is spilling into the structure.

(e) *Headwall.*—The headwall at the pipe end of the box retains earthwork; provides for pipe embedment; and by its horizontal extension into the canal banks and vertical extension below the pipe, serves as a percolation cutoff.

(f) *Floor of Inlet Box and Pipe Invert.*—The invert elevation of the box floor is set at the same elevation as the invert of the pipe. The table of dimensions shown on figure 3-38 was established to set the pipe invert low enough to allow for hydraulic losses through the inlet opening and the pipe entrance, thereby preventing backwater in the canal for its design flow. A minimum loss of 0.5 of the velocity head in the pipe is provided.

(g) *Walk Planks.*—Walk planks are supported by the headwall and the walk plank support angle. The planks provide a platform for operating a gate or handling stoplogs. The 18-inch clearance between the top of sidewalls and bottom of walk planks provides clearance for the passage of floating debris that might otherwise obstruct the flow of water over sidewalls.

(h) *Pipe Diameter.*—The pipe is sized by design steps discussed in respective paragraphs of road crossings, inverted siphons, pipe drops, and pipe chutes. The criteria for determining a pipe size for a structure having a check and pipe inlet is based on providing a pipe with an adequate diameter so that the velocity in the pipe will not exceed the following:

(1) 5 feet per second for a road crossing or a similar structure such as a short inverted siphon having a concrete outlet transition,

(2) 3.5 feet per second for a pipe drop having only an earth transition at the outlet,

(3) 5 feet per second for a pipe drop having a concrete outlet transition,

(4) 10 feet per second for a relatively long inverted siphon having a concrete outlet transition, or

(5) 12 feet per second for a pipe drop or pipe chute having a baffled outlet or a stilling pool outlet.

After the maximum allowable pipe velocity has been determined, the pipe diameter may be selected by using the tables on figures 2-2, 2-4, or 2-31.

(i) *Pipe Vents.*—If a hydraulic jump occurs in the pipe and the inlet is sealed there is a possibility of blowback. A vent at the headwall or immediately upstream from the jump may be required to prevent this blowback. See figures 3-15 and 8-21 for air vent details.

3-22. ***Structure Numbering.***—A combination of the pipe diameter and maximum allowable velocity in the pipe is used to identify a check-and-pipe inlet structure. Thus, structure number 12-1 represents a structure with a 12-inch-diameter pipe having a maximum allowable pipe velocity of 5 feet per second. This structure should also be used with a 12-inch-diameter pipe having a maximum allowable pipe velocity of 3.5 feet per second (also see notes on fig. 3-38). Structure number 12-2 represents a structure with a 12-inch-diameter pipe having a maximum allowable pipe velocity of 12 feet per second. This structure should also be used with a 12-inch-diameter pipe having a maximum allowable pipe velocity of 10 feet per second. The check and pipe inlet structures selected for 3.5 and 10 feet per second will have conservative dimensions, but the difference in cost is not considered excessive. The table in figure 3-38 provides standard dimensions for structures corresponding to structure numbers.

3-23. ***Design Example (see fig. 3-38).***—Assume that a pipe drop is required to convey the canal water to a lower elevation. In addition assume that a check will be required at the inlet of the drop to raise the canal water surface to a required elevation for delivery to a lateral.

(a) *Assume the following design information is given*:

(1) Type of waterway = earth canal.

(2) Type of conveyance structure = pipe drop sumped for energy dissipation with an earth outlet transition.

(3) The conveyance structure would have a maximum allowable pipe velocity of 3.5 feet per second.

(4) Q = 50 cfs.

(5) El. A = 5407.00 (from a profile sheet).

(6) d_n = 2.00 ft. (normal depth at Sta. A).

(7) NWS El. at Sta. A = El. A + d_n = 5407.00 + 2.00 = 5409.00.

(8) El. top of bank = 5410.6.

(9) Check and pipe inlet is required.

(10) Check structure to be automated.

(b) *Determine*:

(1) Need for either gate or stoplog control.–Use gate because of need to automate the check.

(2) Pipe size (see table on fig. 2-31). For a Q of 50 cfs and a maximum allowable velocity of 3.5 feet per second use a 54-inch-diameter pipe.

(3) Structure number.–Structure number 54-1, although proportioned for a maximum allowable pipe velocity of 5 feet per second, can also be used for a maximum allowable pipe velocity of 3.5 feet per second.

(4) Determine standard check and pipe inlet dimensions R, Y, L, and k. These dimensions are read from the standard dimension table for structure number 54-1:

R = 5 ft. 9 in.
Y = 18 in.
L = 6 ft. 6 in.
k = 6 ft.

(5) Pipe velocity head.–For 54-inch-diameter pipe and Q = 50 cfs.

$$A_p = 0.785 \times \text{dia}^2 = 15.90 \text{ ft.}^2$$

$$V_p = \frac{Q}{A_p} = \frac{50}{15.90} = 3.14 \text{ f.p.s.}$$

$$h_{v_p} = \frac{V_p^2}{2g} = \frac{3.14^2}{64.4} = 0.15 \text{ foot}$$

where g = gravitational acceleration

In lieu of performing the above calculation, the hydraulic properties may also be determined by using table 8-1.

(6) Determine elevation D:

$$\begin{aligned} \text{El. D} &= (\text{NWS El. at Sta. A}) - R \\ &= 5409.00 - 5.75 \\ &= 5403.25 \end{aligned}$$

(7) Drop and slope in earth transition.–

$$\begin{aligned} \text{Drop} &= \text{El. A} - (\text{El. D} + Y) \\ &= 5407.00 - (5403.25 + 1.50) \\ &= 5407.00 - 5404.75 \\ &= 2.25 \text{ ft.} \end{aligned}$$

Slope = 10 to 2.25 = 4.6 to 1 (as this is flatter than 4 to 1 it is satisfactory).

(8) Gate size.–The standard dimension table shows a 72- by 51-inch gate for structure number 54-1. Assume for this example problem that this gate is not available and that the next available greater gate height is 60 inches for a 72-inch-wide gate.

(9) Normal canal freeboard = El. canal bank – El. NWS in canal = 5410.60 – 5409.00 = 1.60 ft.

(10) Canal freeboard with check gate closed and normal flow in the canal.–If the 72- by 51-inch gate had been available, then the top of gate when closed would have been at the same elevation as the sidewalls. The head required for discharging the design flow over the sidewalls and gate closure would have been, $H = \left(\frac{Q}{CW}\right)^{2/3} = \left(\frac{50}{3.3 \times 19}\right)^{2/3}$ $= (0.8)^{2/3} = 0.86$ foot.

The above equation is derived from the weir equation:

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

The acceptable coefficient of discharge C is assumed to be 3.3.

W = length of sidewalls plus the width of the structure
= 2 (L) + k = 2 x 6.5 + 6 = 19 ft.

The resulting canal freeboard would then be the El. canal bank – (El. of sidewalls + H) = normal canal freeboard – H = 1.60 – 0.86 = 0.74 foot.

However, use of the 60-inch-high gate results in the top of gate, when closed, being at a higher elevation than the sidewalls, which reduces the canal freeboard to less than 0.74 foot. A canal freeboard of about 0.5 foot is considered satisfactory for the extreme condition of normal Q overflowing the sidewalls and gate closure.

Assume a 0.5-foot canal freeboard and solve for the Q that will overflow into the box using the basic weir equation. For the flow over the sidewalls $Q = C2LH^{3/2}$ and for flow over the top of the gate $Q = CkH_1{}^{3/2}$ therefore, combining these flows we have:

Q = (3.3 x 2 x 6.5 x 1.15) + (3.3 x 6 x 0.01)
= 49.4 + 0.2 = 49.6 cfs.

where

H = (El. canal bank – 0.5 ft. of freeboard) – El. top of sidewalls
= (5410.60 – 0.50) – 5409.00
= 1.1 feet
$H^{3/2}$ = 1.15 feet

and

H_1 = (El. canal bank – 0.5 ft. of freeboard) – El. top of closed gate
= (5410.60 – 0.50) – 5410.04
= 0.06 foot

then

$H_1{}^{3/2}$ = 0.01 foot

and

El. top of closed gate

= El. D + Y + 3-1/2 inches + height of gate
= 5403.25 + 1.50 + 0.29 + 5.00
= 5410.04

The 3-1/2 inches in the above equation is the assumed size of the vertical leg of the gate-frame bottom angle-iron.

The calculated overflow of 49.6 cfs is slightly less than the design flow of 50 cfs. Canal bank freeboard will therefore be just slightly less than 0.5 foot.

G. CONTROL AND PIPE INLETS

R. B. YOUNG[3]

3-24. ***Description and Purpose.***—A control and pipe inlet is a concrete box structure with a trapezoidal-shaped notch for upstream water surface control, overflow sidewalls, and an inlet to a pipe at the box headwall (see fig. 3-1).

The purpose of the control inlet is to prevent drawdown of the canal water surface for a range of flows between design flow and about 0.2 of design flow. In addition, the inlet provides a controlled water surface for turnouts.

3-25. ***Application.***—The control and pipe inlet is sometimes used on the upstream ends of such structures as pipe drops and pipe chutes where the hydraulic control of the upstream canal water surface is at the inlet for all flows. Occasionally it is also used at inlets to inverted siphons to prevent water surface drawdown which may occur for flows less than design flow.

3-26. ***Design Considerations.***—(a) *Upstream Earth Transition.*—The transition is used to effect a gradual change from canal base width

[3]Op. cit., p. 166.

to that of the structure and in so doing produce a more uniform velocity change from the canal to the structure. The transition invert is level since the notch invert is set at the canal invert elevation.

(b) *Erosion Protection.*—Protection is used adjacent to control and pipe inlets in earth canals to prevent erosion resulting from high velocities. For design of protection see the discussion on Protection, subchapter VII B.

(c) *Control Notch.*—The trapezoidal-shaped opening or control notch is designed to control upstream canal water depths and velocities for all flows between design flow and about 0.2 of design flow. For any flow in this range the notch causes the upstream canal water depth to be at or very near normal depth thereby preventing increased velocities which otherwise could cause canal erosion. The elevation of the notch invert should be equal to the upstream canal invert elevation. The depth of the notch should be equal to or slightly more than the depth in the canal for normal flow. The bottom width, top width, and side slopes of the notch may be determined by the procedure shown in the design example (see sec. 3-28).

A control notch which requires slightly more energy than that available from normal flow conditions in the canal will cause a very small rise in water surface elevation in the upstream canal, but this is an acceptable condition. For the critical case of canal flow at or near full design Q, the water surface rise is minimized by a portion of the flow being able to overflow the sidewalls.

(d) *Overflow Sidewalls.*—These walls, with tops at the same elevation as the top of the notch, provide for emergency overflow into the box in the event there is an obstruction in the notch or the flow in the channel is greater than normal. The sidewalls should be of sufficient length to permit design flow to overtop the sidewalls with complete blockage of the notch. Channel banks must have sufficient freeboard for the higher water surface required to overflow these walls. The length of sidewalls (fig. 3-39) are designed to provide about 0.5 foot of channel bank freeboard based on figure 1-9.

(e) *Headwall.*—The headwall retains channel earthwork, embeds the inlet end of the pipe, and by its extension into the earth below and on each side of the pipe, serves as a cutoff to reduce seepage from the channel.

(f) *Width of Inlet Box.*.—The minimum width should be the larger of either 6 inches plus notch width or 6 inches plus inside pipe diameter.

(g) *Inlet Invert Elevation.*—This elevation is the same as the pipe opening invert elevation and should be low enough so the water surface at the pipe opening required to convey the flow into the pipe will not cause the upstream channel water surface elevation to exceed normal. The R values, for commonly used full pipe velocities of 3.5, 5, 10, and 12 feet per second, in the table of dimensions on figure 3-39, were established to prevent the pipe entrance from controlling the channel water surface. A minimum loss of 0.5 pipe velocity head is provided. The dimension R is slightly more than the pipe diameter plus 1.5 pipe velocity head.

(h) *Pipe Diameter.*—The pipe is sized by procedures discussed in paragraphs of road crossings, inverted siphons, drops, and chutes. Generally, the criterion for determining a pipe size for a structure having a control and pipe inlet is to provide a large enough diameter pipe so that the pipe velocity will not exceed the following:

(1) 5 feet per second if a concrete outlet transition is provided on a road crossing or a similar structure such as a short inverted siphon,

(2) 3.5 feet per second for a pipe drop having only an earth transition at the outlet,

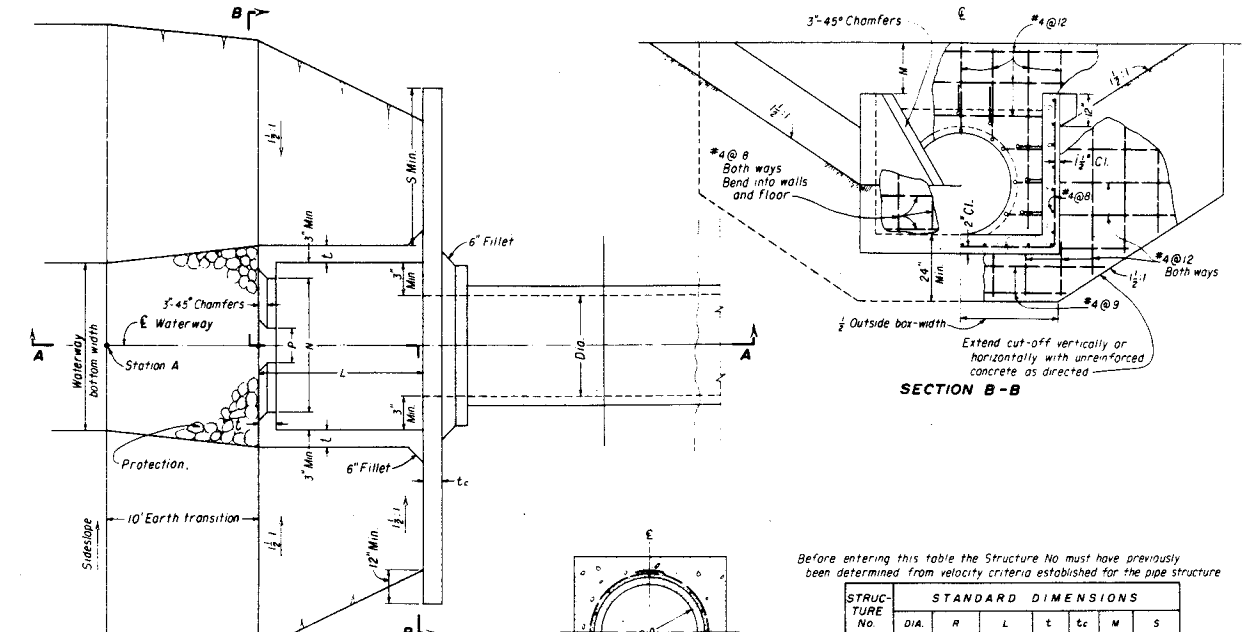
(3) 5 feet per second for a pipe drop having a concrete outlet transition,

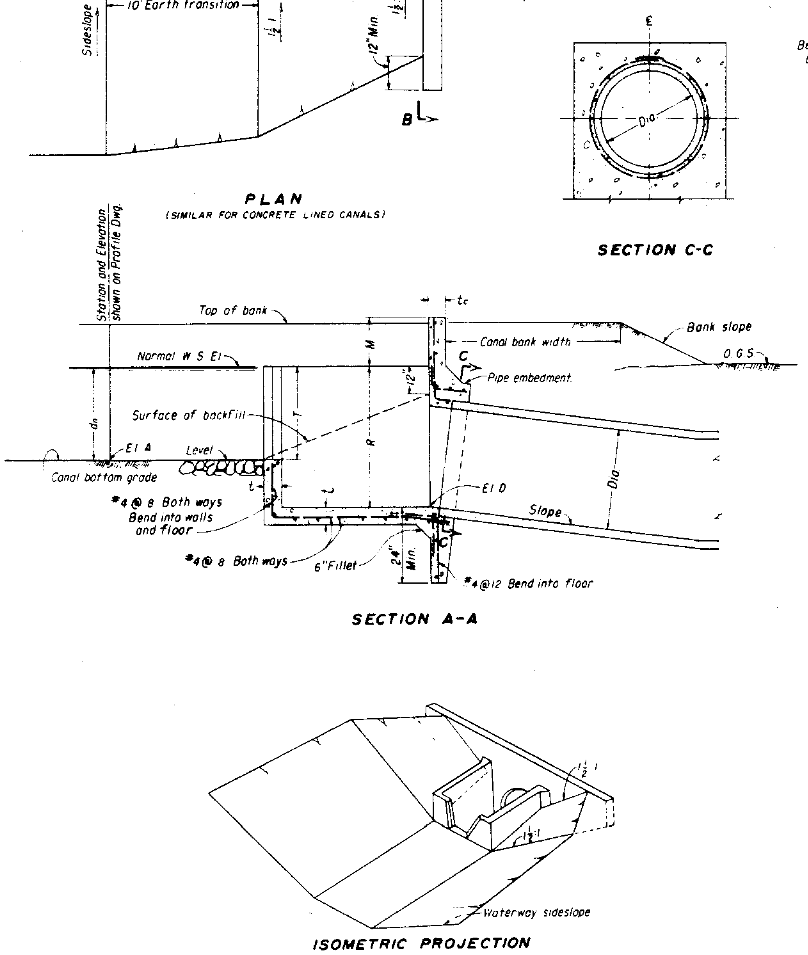
(4) 10 feet per second for a relatively long inverted siphon having a concrete outlet transition, or

(5) 12 feet per second for a pipe drop or a pipe chute having a baffled outlet or a stilling pool.

After the maximum allowable pipe velocity has been determined, the pipe diameter may be selected by using the tables on figures 2-2, 2-4, or 2-31.

3-27. ***Structure Numbering.***—The pipe diameter and maximum allowable full-pipe velocity are used to number the control and pipe inlet structures. Structure number 12-1 represents a structure with a 12-inch-diameter pipe and a maximum allowable full-pipe





Before entering this table the Structure No must have previously been determined from velocity criteria established for the pipe structure

STRUCTURE No.	STANDARD DIMENSIONS						
	DIA.	R	L	t	tc	M	S
12-1	12"	2'-6"	3'-6"	6"	6"	12"	4'-0"
12-2	12"	5'-0"	3'-6"	6"	6"	12"	4'-0"
15-1	15"	2'-6"	3'-6"	6"	6"	12"	4'-0"
15-2	15"	5'-0"	3'-6"	6"	6"	12"	4'-0"
18-1	18"	2'-9"	3'-6"	6"	6"	12"	4'-0"
18-2	18"	5'-0"	3'-6"	6"	6"	12"	4'-0"
21-1	21"	3'-0"	3'-6"	6"	6"	12"	4'-0"
21-2	21"	5'-3"	4'-0"	6"	8"	12"	4'-0"
24-1	24"	3'-3"	3'-6"	6"	6"	12"	4'-0"
24-2	24"	5'-6"	4'-6"	6"	8"	12"	4'-0"
27-1	27"	3'-6"	3'-6"	6"	6"	15"	4'-4"
27-2	27"	5'-9"	5'-0"	6"	8"	15"	4'-4"
30-1	30"	3'-9"	3'-6"	6"	6"	15"	4'-4"
30-2	30"	6'-0"	5'-6"	6"	8"	15"	4'-4"
33-1	33"	4'-0"	3'-6"	6"	6"	18"	4'-9"
33-2	33"	6'-3"	6'-0"	6"	8"	18"	4'-9"
36-1	36"	4'-3"	4'-0"	6"	6"	18"	4'-9"
36-2	36"	6'-6"	6'-6"	6"	8"	18"	4'-9"
39-1	39"	4'-6"	4'-0"	6"	6"	16"	4'-9"
39-2	39"	6'-9"	7'-0"	7"	8"	18"	4'-9"
42-1	42"	4'-9"	4'-6"	6"	6"	18"	4'-9"
45-1	45"	5'-0"	5'-0"	6"	6"	18"	4'-9"
48-1	48"	5'-3"	5'-0"	6"	8"	18"	4'-9"
51-1	51"	5'-6"	5'-6"	6"	8"	18"	4'-9"
54-1	54"	5'-9"	5'-6"	6"	8"	18"	4'-9"
57-1	57"	6'-0"	6'-0"	6"	8"	18"	4'-9"
60-1	60"	6'-3"	6'-0"	6"	8"	18"	4'-9"

NOTES

Structures are numbered to designate pipe diameter in inches followed by a second number, -1 or -2, which designate maximum allowable pipe velocities of 5 and 12 feet per second respectively. Structures -1 and -2 may also be used for maximum allowable pipe velocities of 3.5 and 10 feet per second respectively. Structure number 30-2 is for a 30 inch diameter pipe having a maximum allowable velocity of either 10 or 12 feet per second

Figure 3-39. **Control and pipe inlet. 103-D-1222** velocity of 5 feet per second. This structure should also be used with a 12-inch-diameter pipe and a maximum allowable full-pipe velocity of 3.5 feet per second. Similarly, structure number 12-2 represents a structure with a 12-inch-diameter pipe and a maximum allowable full-pipe velocity of 12 feet per second. This structure should also be used with a 12-inch-diameter pipe and a maximum allowable full-pipe velocity of 10 feet per second. The control and pipe inlet structures selected for 3.5 and 10 cfs will have conservative dimensions, but the difference in cost is not considered excessive. The table on figure 3-39 presents standard dimensions for each structure number.

3-28. ***Design Example (see fig. 3-39).***—Assume that a pipe drop is required to convey the canal water to a lower elevation, and that a control and pipe inlet will be required to control the water surface in the canal upstream from the drop.

(a) *Given*:

(1) Type of waterway = earth canal.

(2) Hydraulic properties of the canal

$Q = 50$ cfs
$b = 5.0$ ft.
$d_n = 2.83$ ft.
$s = 0.0005$
$S{:}S = 1\text{-}1/2{:}1$
$n = 0.025$
$V = 1.91$ f.p.s.
$h_v = 0.06$ ft.
E (energy) = 2.89 ft.

(3) Type of conveyance structure = pipe drop sumped for energy dissipation with an earth outlet transition.

(4) This drop structure would have a maximum allowable velocity of 3.5 feet per second.

(5) El. A = 5406.00 (from a profile sheet).

(6) NWS El. at Sta. A = El. A + d_n = 5406.00 + 2.83 = 5408.83.

(7) El. top of bank = 5410.4.

(8) Control structure at inlet is required to minimize erosion in the canal for flows in the range of about 0.2 design flow to full design flow.

(b) *Determine*:

(1) Pipe size (see table on figure 2-31). For a Q of 50 cfs and a maximum allowable velocity of 3.5 feet per second the table permits the use of a 54-inch-diameter pipe.

(2) Structure number.—A structure with a 54-inch-diameter pipe and a maximum allowable pipe velocity of 3.5 feet per second is numbered 54-1.

(3) Determine control and pipe inlet standard dimensions R and L by entering the structure table for structure number 54-1. Read R = 5 ft. 9 in. and L = 5 ft. 6 in.

(4) Determine A_p, V_p and h_{v_p} for a 54-inch-diameter pipe with a Q = 50 cfs:

$$A_p = 0.785 \times \text{dia}^2 = 15.90 \text{ ft.}^2$$

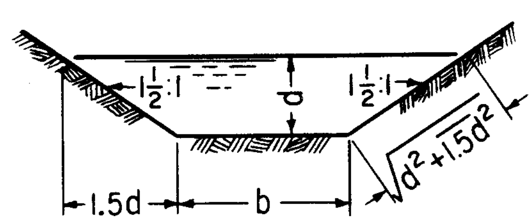
$$V_p = \frac{Q}{A_p} = \frac{50}{15.90} = 3.14 \text{ f.p.s.}$$

$$h_{v_p} = \frac{V_p^2}{2g} = \frac{3.14^2}{64.4} = 0.15 \text{ ft.}$$

where g = gravitational acceleration.

In lieu of performing the above calculations, the hydraulic properties may also be determined by using table 8-1.

(5) Determine the canal hydraulic properties for a flow of 0.2 the normal Q, so that E (energy) may be calculated for use on the control notch graphs (fig. 3-40). The area, wetted perimeter, and hydraulic radius equations and associated sketch are as follows:



CANAL SECTION

wp = wetted perimeter

$= b + 2\sqrt{d^2 + (1.5d)^2}$
$= b + 2\sqrt{3.25d^2}$
$= b + 2(1.803d)$
$= b + 3.606d$

and

A = area
$= db + d(1.5d)$
$= db + 1.5d^2$

also

$$R = \text{hydraulic radius} = \frac{A}{wp}$$

Then to calculate hydraulic properties for flow of 0.2Q

$0.2Q = 0.2 \times 50 = 10$ cfs

and

$s^{1/2} = 0.0005^{1/2} = 0.02235$

From Manning's equation [3]

$$AR^{2/3} = \frac{0.2\ Qn}{1.486s^{1/2}} = \frac{10 \times 0.025}{1.486 \times .02235} = 7.53$$

Assume various depths, d, in the canal and solve for $AR^{2/3}$ until it equals 7.53.

From the tabulation below, a d value of 1.2 feet results in $AR^{2/3} = 7.51$ which is close to the desired 7.53. The area A, corresponding to a depth of 1.2 feet is 8.16 sq. ft.

Knowing d and A, we can now compute the remaining hydraulic properties:

$$V = \frac{0.2\ Q}{A} = \frac{10}{8.16} = 1.23 \text{ f.p.s.}$$

$$h_v = \frac{V^2}{2g} = \frac{1.23^2}{64.4} = 0.02 \text{ ft.}$$

E (energy) depth of water + velocity head $= d + h_v = 1.20 + 0.02 = 1.22$ ft.

Assume d, ft.	d^2, sq. ft.	$1.5d^2$, sq. ft.	b, ft.	db, sq. ft.	A = $db+1.5d^2$, sq. ft.	3.606d, ft.	wp = b+3.606d, ft.	R = A/wp	$R^{2/3}$	$AR^{2/3}$
1.1	1.21	1.82	5.0	5.5	7.32	3.97	8.97	0.816	0.87	6.36
1.2	1.44	2.16	5.0	6.0	8.16	4.33	9.33	0.875	0.92	7.51

An alternative method may be used to compute normal depth of water as follows: use table 7-11, Handbook of Hydraulics [3] as shown below, to find D:

$$K^1 = \frac{0.2\ Qn}{s^{1/2}\ b^{8/3}} = \frac{10 \times 0.025}{0.02235 \times 73.1}$$

$$= 0.153$$

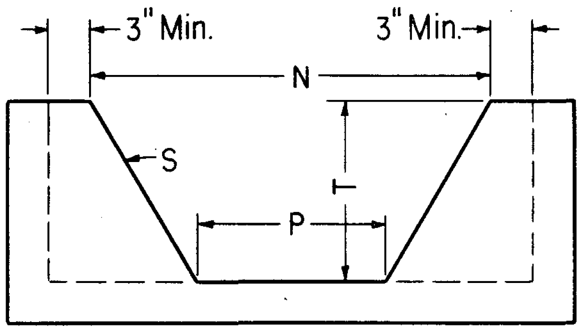
Enter table with $K^1 = 0.153$ and side slopes of 1.5 to 1 and read a $\frac{D}{b}$ ratio of

$$0.24 = \frac{\text{depth of water}}{\text{canal base width}}$$

So D = depth of water = 0.24 x 5 = 1.20 ft.

Values of A, V, h_v, and E are then determined as before.

(6) Determine control notch dimensions using sketch below:



CONTROL NOTCH

$N = P + 2\ (ST)$
P = bottom width of control notch
$S = \text{side slopes} = \frac{\text{Horizontal}}{\text{Vertical}}$
T = Height of notch $\geq$ d
d = Normal water depth in canal

For a given condition, there is usually more than one notch that can provide approximately the same control for the range of flows to be controlled. In this example, the control notch is to act as a control between the limits of 10 and 50 cfs.

From given values and previous computations the hydraulic properties for the canal with full Q and 0.2Q are:

Flow	b, ft.	d, ft.	s	S:S	n	V,fps	h_v, ft.	E, ft.
Full Q = 50 cfs	5.0	2.83	0.0005	1.5:1	0.025	1.91	0.06	2.89
0.2 Q = 10 cfs	5.0	1.20	0.0005	1.5:1	0.025	1.23	0.02	1.22

To begin the process of selecting a control notch, select the graph from figure 3-40 having the smallest P value which furnishes the full range of discharge from 10 through 50 cfs. In this example the control notch graph with P = 12 inches is first selected. Enter the graph at E (energy) = 2.89 ft.; go vertically to the intersection with the horizontal line for Q = 50 cfs. Read the value of S for the slope curve which lies immediately to the **RIGHT** of the point. This curve is for slope of S = 0.75. Check in the same manner to see if the same curve will also control for 10 cfs (0.2Q). Enter the graph at E (energy) = 1.22 ft.; go vertically to the intersection with the horizontal line for Q = 10 cfs. The curve immediately to the **RIGHT** of this point shows a slope, S = 1.00. The slope curve should have been the same for both discharges so that the notch will act as a control for both Q = 50 cfs and 10 cfs; however, this did not occur.

It is necessary then to try the graph with the next greater P value = 16 inches. Repeating the previous procedure the notch slope is determined to be S = 0.50 for Q = 50 cfs and s = 0.75 for Q = 10 cfs. Therefore, try the graph with the next greater value of P = 20 inches. Repeat the previous procedure from which the notch slope is determined to be S = 0.50 for Q = 50 cfs and S = 0.50 for Q = 10 cfs. This control notch is satisfactory as it will act as a control for all discharges between 10 and 50 cfs.

P = 20 inches, S = 0.50, T = d_n = 2.83 feet = 2 ft. 10 in.
N = P + 2 ST = 20 in. + 2 (0.5)(2 ft. 10 in.) = 4 ft. 6 in.

By using the same procedure a control notch with P = 16 or 12 inches could have been used if a lesser range of control (50 cfs to something greater than 10 cfs) had been determined adequate.

(7) Determine inlet box inside width, figure 3-39.

min. width = pipe dia. + 6 in. = (4 ft. 6 in.) + 6 in. = 5 ft.

or

min. width = N + 6 in. = (4 ft. 6 in.) + 6 in. = 5 ft.

use width = 5 feet.

(8) Determine El. top of notch and sidewalls

= El. A + T = 5406.00 + 2.83
= 5408.83

(9) Determine El. D

= El. top of sidewalls
 – R (from structure table)
= 5408.83 – 5.75 = 5403.08

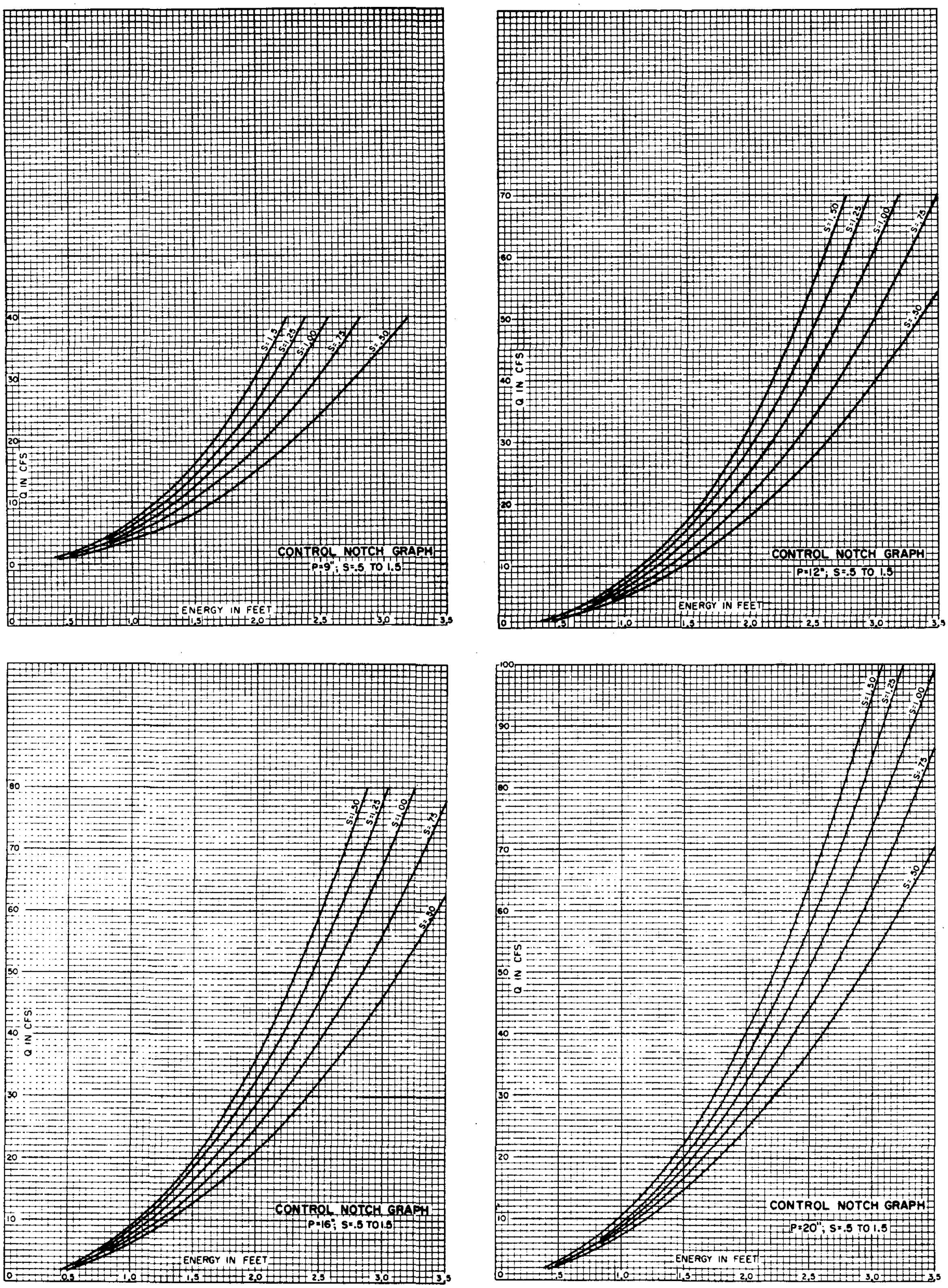


Figure 3-40. **Control notch graphs (Sheet 1 of 2). 103-D-1223**

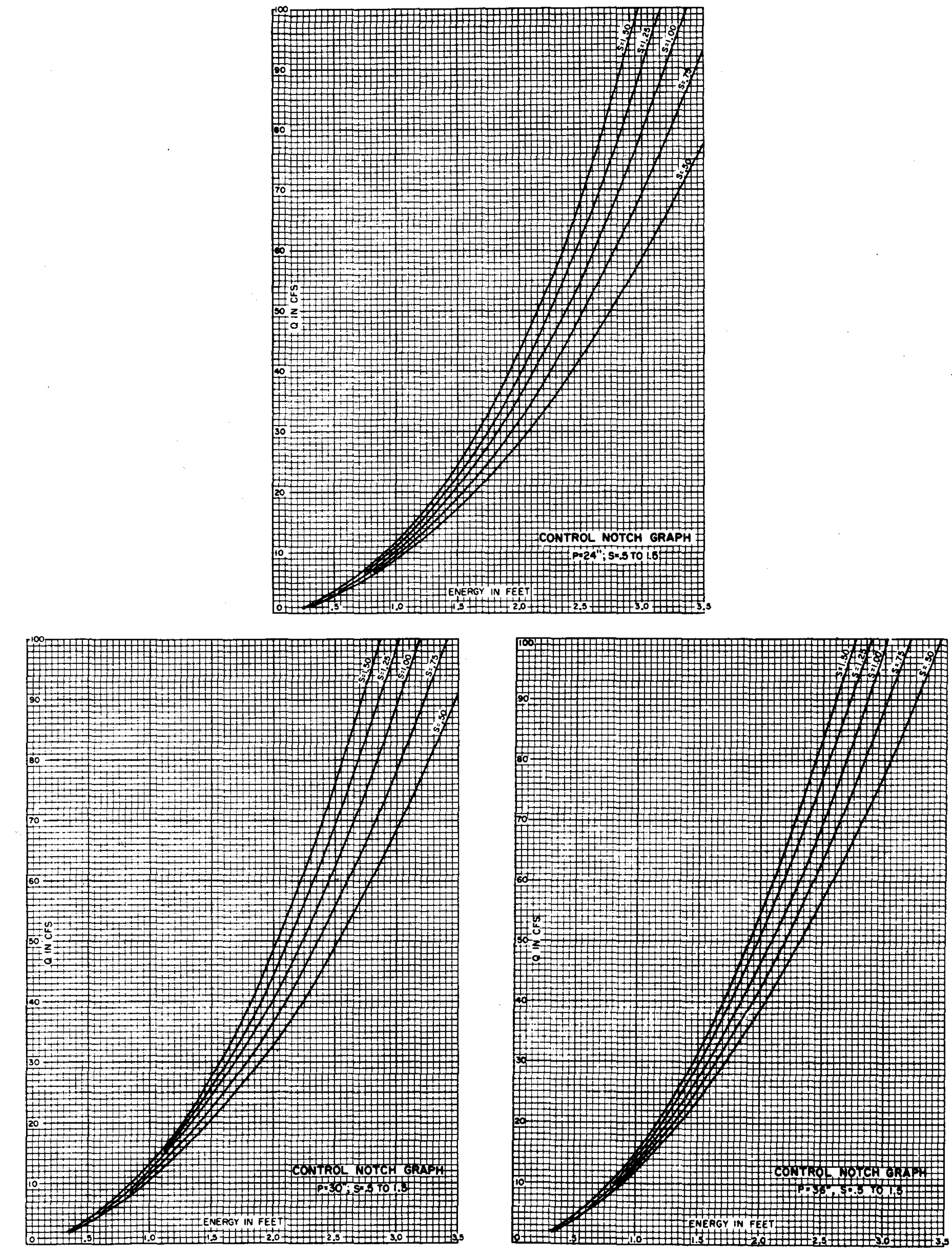


Figure 3-40. **Control notch graphs (Sheet 2 of 2). 103-D-1223**

H. BIBLIOGRAPHY

3-29. *Bibliography*.

[1] Operation and Maintenance, Equipment and Procedures, Release No. 20, April, May, and June 1957, Bureau of Reclamation.

[2] Operation and Maintenance, Equipment and Procedures, Release No. 32, April, May, and June 1960, Bureau of Reclamation.

[3] King, H. W. and Brater, E. F., "Handbook of Hydraulics," Fifth Edition, McGraw Hill Book Co., New York, 1963.

Protective Structures

R. B. HAYES[1]

A. GENERAL

4-1. *Purpose.*—Protective structures protect the canal system and adjacent property from damage which would result from uncontrolled storm runoff or drainage water, or an uncontrolled excess of flow within the canal.

Storm and drainage water must be controlled to prevent erosion of the uphill canal bank, and accompanying silting of the canal prism. Storm or drainage water must have either: (1) controlled entrance into the canal through a drain inlet; (2) controlled conveyance over the canal in an overchute; (3) controlled conveyance under the canal through a culvert; or (4) the canal must be routed under the cross-drainage channel in a siphon.

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Excess flows within a canal must be controlled to prevent damage from overtopping of the bank, or deterioration of the bank resulting from operation at an excessive depth. Wasteways are provided to remove excess water which may result from: (1) entrance of cross-drainage flows at drain inlets; (2) operational mismatch of water supply and demand due to upstream gate operation; or (3) shutdown of a pumping plant or powerplant downstream from a wasteway.

Wasteways are also used to empty the canal for inspection, maintenance, seasonal shutdown, or an emergency such as a canal bank failure. A terminal wasteway, located at the end of a canal, provides controlled discharge of unused water to a channel or reservoir.

B. WASTEWAYS

4-2. *Description.*—Basically, a wasteway is comprised of an overflow or gate structure, in combination with a drop or chute structure and a wasteway channel. The overflow and gate structures are frequently combined as shown on figure 4-1, to provide for the various conditions of canal operation.

(a) *Wasteway Inlets.*—An overflow structure, such as a side channel spillway or a siphon spillway, provides automatic release of excess canal water. A wasteway turnout gate provides the capability of emptying the canal. The alinement of most wasteway inlets is normal to the canal, but an inline structure is more effective in removing trash or floating ice.

A check and pipe inlet structure provides both an overflow for automatic control of excess water and a gated or stoplogged section for emptying the canal (see fig. 4-2). It is used primarily as an inline terminal wasteway structure at the end of the canal to dispose of the unused canal flow. However, it is also used at other locations along the canal, by providing a short channel from the canal to the check

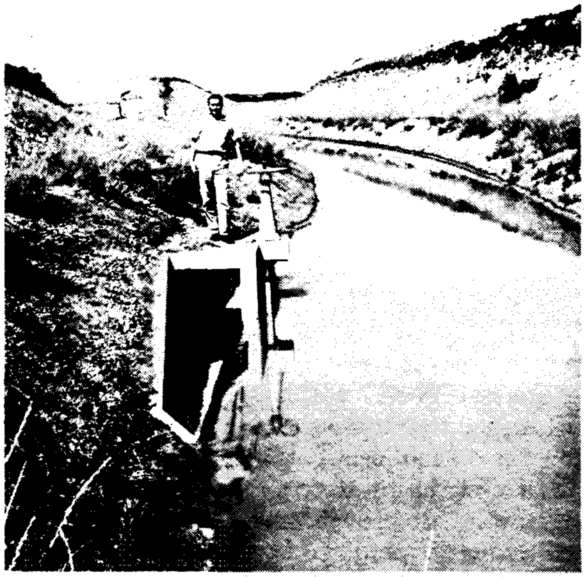


Figure 4-1. **Wasteway turnout with side channel spillway and slide gate. P-271-701-3736**

Figure 4-2. **Check and pipe inlet in operation. P-222-116-53579D**

and pipe inlet.

A check drop is equally suitable as a wasteway structure in providing both the overflow and the gated or stoplogged section (see fig. 3-10).

Wasteway turnout gates may be operated manually, with or without power hoists, or by automatic or remote control.

(b) *Wasteway Outlets.*—The wasteway outlet usually performs the function of dissipating excess energy, to protect the wasteway structure and channel from erosion. Where the vertical drop from the canal to the wasteway channel is minimal, a transition will suffice. For greater drops in elevation, either a baffled outlet, baffled apron, or stilling pool, in conjunction with a drop or chute structure is required.

Baffled outlets are effective energy dissipators for pipe discharges, but should not be used where a heavy weed load exists.

Baffled apron drops are best suited for use with open-flow discharges, but have been used downstream from closed conduits by providing a long transition to the baffled apron drop. The presence of a heavy weed load requires cleaning of the baffle blocks to insure optimum operation. Where floating debris such as brush is probable, baffled apron drops should not be used. Baffled apron drops do not require a downstream water surface for satisfactory operation. They can operate with a wide variation of downstream water surface elevations, and are therefore often used as terminal structures discharging into reservoirs.

Stilling pools are ideally suited for use below rectangular inclined drops or chutes, but have been used downstream from closed conduit by providing a long transition to the pool. They are not adversely affected by normal weed loads, but do require a controlled downstream water surface to insure a hydraulic jump. Where the natural channel does not provide a downstream control, a concrete sill, stoplog sill, or a control notch can be incorporated in the stilling pool, as described in subchapter II F. Downstream control can also be provided by sloping the transition invert, as shown in figure 4-32.

(c) *Wasteway Channel and Channel Structures.*—A natural channel is utilized where practicable (see sec. 4-3), to transport the wasteway discharge to a disposal point, such as a river or reservoir. However, it is usually necessary to construct a channel to connect the wasteway structure to the natural drainage channel. Such connections may be only a few feet in length, or may extend thousands of feet. The cross section of such connecting channels is similar to that for the canal, but with a lesser emphasis on protective features.

Wasteway channel structures are sometimes required to control flow and prevent excessive erosion of the channel. These structures include rectangular inclined (R.I.) drops or chutes, baffled apron drops, control and pipe inlets, pipe drops or chutes and road crossings.

(d) *Appurtenant Canal Structures.*—Check structures while not a part of the wasteway itself, are usually provided in the canal a short distance downstream from the wasteway, so that the entire canal flow can be diverted to the wasteway.

4-3. ***Location and Number.***—Many factors must be considered in determining the optimum number and location of wasteways. An evaluation of the following factors serves as a guide:

(a) The cost of the canal system and wasteways,

(b) The presence of natural drainage channels of adequate capacity, at locations suitable for wasteways,

(c) The volume and rate of storm or drainage water that can enter the canal through drainage inlets,

(d) The volume available for safe storage in the canal or in regulating reservoirs,

(e) The extent of automatic or remote control of the canal system, or the estimated time required to operate manually, and

(f) The magnitude of the economic loss that could result from a canal bank failure.

Wasteways may not be required for short laterals having fewer than four or five farm turnouts. Longer laterals should have a terminal wasteway at or near the end, located near a natural channel if possible. The number of turnouts downstream from the last wasteway should be limited to avoid overloading that reach of canal in the event that it is necessary to close all the turnouts.

4-4. ***Wasteway Capacity Considerations.***—The variety of situations and needs for which wasteways are provided requires that each wasteway be given individual consideration in determining its design capacity.

(a) *Spillway.*—The required automatic overflow capacity is dependent upon inflows from drain inlets, additional flows resulting from upstream gate operation, and deliveries to pumping plants or powerplants, as follows:

(1) *Inflows from drain inlets.*—The spillway design capacity should be adequate to permit wasting of the cumulative design inflows from drain inlets upstream from the wasteway. Generally, this inflow should be limited to about 20 percent of the canal design capacity. See subsection 4-41(c) for other limitations on inflows from drain inlets.

(2) *Upstream gate operation.*—Closure of upstream turnout gates, without a corresponding adjustment of the canal headgate will result in additional flow in the canal. The required spillway capacity is sometimes controlled by the capacity of the largest upstream turnout.

(3) *Pumping plants and powerplants.*—A canal that delivers water to a pumping plant or powerplant usually requires provision for automatic spillway capacity equal to the design capacity of the plant. This requirement arises when a pumping plant experiences a power failure, or when a powerplant must shut down suddenly. Since plant shutdowns are likely to occur during a storm, while the drain inlets are flowing at full capacity, it may be desirable to provide a spillway capacity equal to the plant capacity plus the design capacity of the drain inlets. This could be about 120 percent of the canal design capacity.

(b) *Wasteway Turnout Gate.*—The wasteway turnout gate capacity should equal or exceed the canal design capacity to divert the entire canal flow to the wasteway in the event of a canal break, or other downstream emergency. To protect the endangered reach of canal, the wasteway turnout gate should be opened and the adjacent check gate closed.

Sizing the wasteway turnout gate for the canal design capacity provides more than adequate capacity for unwatering the canal for inspection, maintenance, or end of irrigation season shutdown.

(c) *Wasteway Outlet.*—The wasteway outlet should be capable of discharging the simultaneous flows of the wasteway turnout gate and overflow feature (see fig. 4-4).

(d) *Natural Drainage Channels.*—The capacity of the wasteway turnout or spillway features should not exceed the safe capacity of the natural channel into which it discharges. Thus, the capacity of the natural channel may control the design capacity of the wasteway, unless the channel is improved.

4-5. ***Limitations.***—Operation of the wasteway turnout gate should be such that the canal water surface drawdown rate is within the limits imposed by the structural stability of the canal bank, lining, and appurtenant structures. Generally, to avoid excessive hydrostatic pressures in the bank or behind the lining, the canal water surface should not be lowered more than 6 inches in 1 hour, and not more than 1 foot in 24 hours.

I. Side Channel Spillways

4-6. ***General.***—Side channel spillways are located along the canal bank with the spillway crest parallel to the canal alinement (see figure 4-1). As the canal water surface rises above the crest, the excess water is automatically discharged into a side channel. From the side channel, the water drops into a pool from which the flow is directed through a pipe or open section, into the wasteway channel.

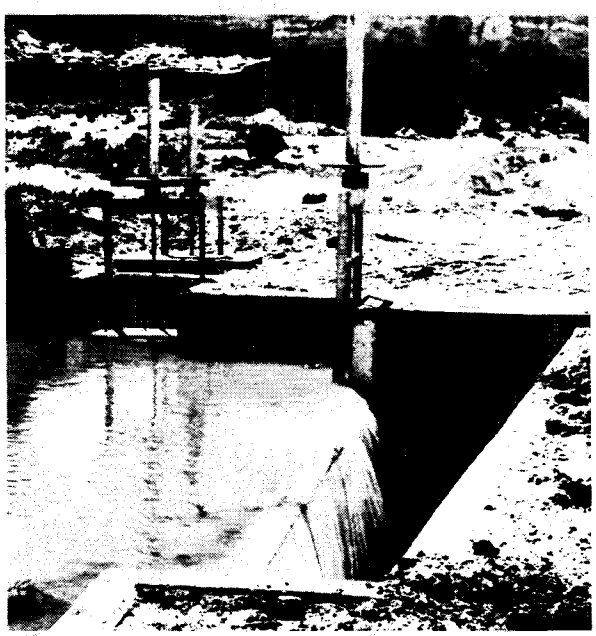


***Figure 4-3.* Side channel spillway in operation in foreground, and wasteway gate at far end of spillway crest. Constant head orifice turnout at left side of side channel spillway. PX-D-72608D**

The structure is generally used in conjunction with a slide gate which provides for complete drainage of the canal. Such a combination is shown in figure 4-3. A check structure is usually provided in the canal a short distance downstream from the wasteway, to permit diversion of the entire canal flow into the wasteway.

Overflow spillways have been used effectively for removal of weeds, trash, or ice from the canal. The water surface can be raised by partial closure of the downstream check, until weeds or ice overflow the spillway crest. A floating boom is sometimes placed diagonally across the canal to divert weeds, trash, or ice to the wasteway. Baffled outlets should not be used where such a weed, trash, or ice load is anticipated.

4-7. ***Design Considerations.***—The following design considerations and recommendations are considered appropriate for side channel spillways on small canals. The design capacity should be based upon the requirements of section 4-4.

(a) *Overflow Crest.*—The spillway crest should be level and set about 0.2 foot above the normal water surface to prevent unnecessary waste of water from normal wave action. To provide a reasonable safety factor against overtopping, the maximum water surface encroachment on freeboard is normally limited to:

- 50 percent of the canal lining freeboard, or
- 25 percent of the canal bank freeboard in an unlined canal section.

However, during emergency operation of the spillway flowing at full design capacity, a maximum encroachment of 50 percent of the canal bank freeboard may be permitted in the vicinity of the wasteway.

The crest length should be computed using the formula [1]:[2]

$$Q = 3.33\, L_c\, H^{3/2}$$

[2]Numbers in brackets refer to items in the bibliography, sec. 4-46.

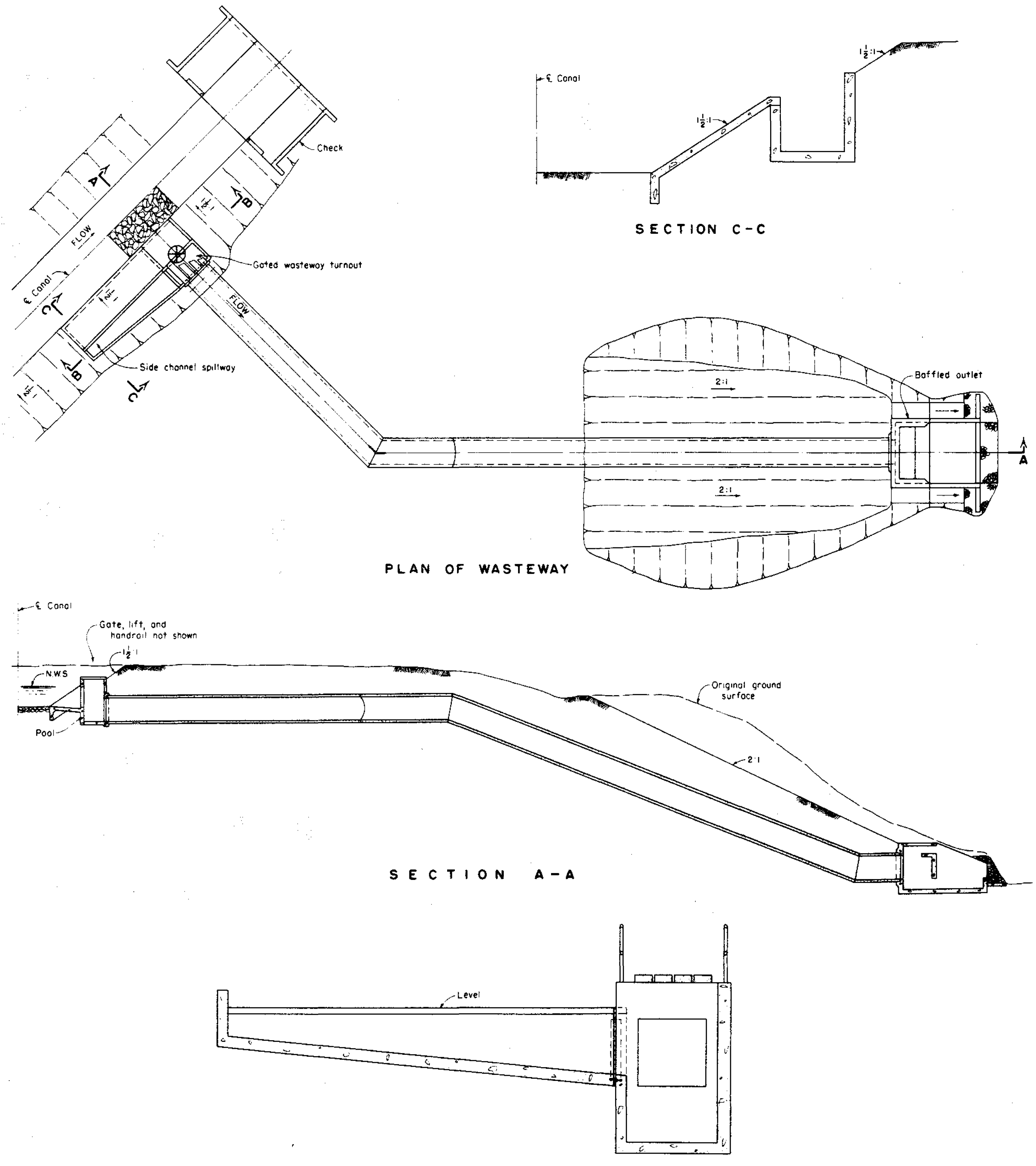


Figure 4-4. **Side channel spillway used as a wasteway turnout. 103-D-1294**

(b) *Side Channel.*—The side channel usually has a rectangular cross section. The width, depth, and slope should be adequate to provide free flow while carrying a weed, trash, or ice load, and should provide adequate dimensions to facilitate construction. A reasonable width should vary uniformly from about 2 feet at the upstream end, to about 4 feet at the downstream end, except that a downstream width of 3 feet is sufficient for discharges of 50 cubic feet per second (cfs) or less. A constant channel width may be more practical under some conditions.

To insure free spillway discharge into the side channel, the side channel water surface must be below the spillway crest. This is accomplished by setting the side channel invert, at the downstream end, below the crest a distance equal to the specific energy plus 1 foot, based upon total design flow. Assuming critical flow, the specific energy is equal to the critical depth plus velocity head ($d_c + h_{v_c}$), which for a rectangular cross section, is equal to $1.5d_c$. The minimum downstream wall height is thus equal to $1.5d_c$ plus 1 foot.

The invert slope should be sufficient to provide supercritical flow to the pool. However, the transverse flow of incoming water over the crest impedes the flow in the side channel, causing bulking (increased volume due to entrained air). Therefore, the invert should be set considerably steeper than the critical slope. An arbitrary slope of about 0.05 is a conservative value.

The upstream wall height, h_1, is then equal to the downstream wall height, h_2, minus 0.05 times the crest length.

(c) *Wasteway Turnout Gate.*—The wasteway turnout gate is ordinarily used when the canal must be emptied. To empty the downstream reach, the design discharge of the canal must flow through the wasteway gate without overflowing the downstream check gate or the side channel spillway. As the need to pass the canal design discharge through the wasteway gate will be infrequent, a velocity of 10 cfs is used in determining the gate area. With this gate area, it is necessary to provide sufficient head, as shown by the orifice equation[2]:

$$Q = CA\sqrt{2gh}$$

which may be expressed:

$$h = \frac{Q^2}{2gA^2C^2}, \text{ or } h = \frac{V^2}{2gC^2}$$

Using a value of 0.6 for the coefficient C,[3] and a velocity of 10 feet per second, the required head must be equal to 4.32 feet. The centerline of the gate should therefore be set 4.32 feet below the normal water surface of the canal. However, to avoid an unnecessarily deep structure, and to minimize the problem of silt removal, the gate centerline should not be set below the canal invert. The head is thus limited to a maximum equal to the normal depth of the canal. Where this limitation is applied, the required gate area should be recomputed for the reduced head (see fig. 4-6C). If a gate of the size required is not available, the next larger standard size should be selected, and the head requirement recomputed.

(d) *Pool.*—To approach free-flow conditions through the gate or from the side channel, the pool invert should be set low enough that the pool water surface is no higher than the centerline of the gate [9], and not appreciably higher than the side channel invert.

If the pool discharges into a pipe, the top of the pipe requires a minimum submergence of 1.5 h_{v_p} (see fig. 4-6B), to provide for an entrance loss of 0.5 h_{v_p} and a pipe velocity head, h_{v_p}.

(e) *Pipe.*—Typically, the wasteway includes a pipe from the pool, extending under the operating road to the outlet structure. A diameter of 24 inches is considered to be a minimum where the passage of weeds, trash, or ice may be required.

If the outlet structure consists of a concrete transition, the pipe diameter may be based upon a maximum velocity of 10 feet per second flowing full. The invert slope should be about 0.001 to facilitate complete drainage, without inducing supercritical flow.

If the pipe terminates in a baffled outlet, the pipe diameter should be based upon a maximum velocity of 12 feet per second, flowing full. The pipe slope may be as steep as 1-1/2 to 1, provided other design considerations are satisfied.

(f) *Wasteway Outlet Structure.*—The type of outlet structure is dependent upon the topography. Where excess energy is minimal, an outlet transition as shown on figure 7-4 should be used to protect the wasteway channel from erosion (see chapter VII). Where a considerable drop is required, the excess energy should be dissipated by use of a pipe drop, baffled outlet, baffled apron drop, or stilling pool. For design of these energy dissipators, see chapter VI.

(g) *Percolation.*—As the canal water surface is usually considerably higher than the channel invert at the wasteway outlet (see dimension F, figure 4-6), there will be a tendency for the water to percolate through the soil, or along the structure, toward the outlet. If the percolation path is too short, the percolation rate will be sufficient to remove soil particles by piping which may cause failure of the bank or structure. To prevent piping, the percolation path should be increased by providing collars on the pipe (see sec. 4-27).

(h) *Protection.*—Figure 7-8 may be used as a guide for determining protection requirements for unlined or earth-lined canals or wasteway channels. It indicates that type 2 or type 3 protection should be used at the outlet. Type 1 protection should be used on the canal invert adjacent to the wasteway turnout gate inlet, extending to the canal centerline and along the canal for a length equal to twice the gate width (see fig. 4-4).

(i) *Operation.*—For limitation on drawdown rate, see section 4-5.

4-8. *Design Example.*—Utilizing a side channel spillway and a slide gate, design a wasteway turnout for the canal features shown in figure 4-5.

(a) *Assumptions.*—

(1) Assume an earth-lined canal with the following properties:

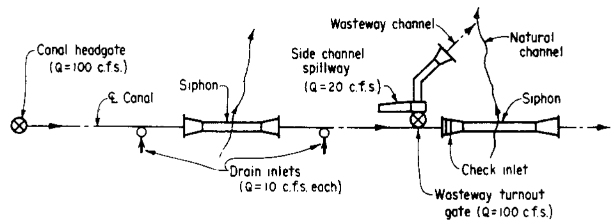


Figure 4-5. **General plan of typical canal with side channel spillway. 103-D-1295**

Q = 100 cfs	b = 8.0 ft.	R = 2.0
A = 37.5 sq. ft.	d_n = 3.0 ft.	n = 0.025
V = 2.67 f.p.s.	f_b = 2.0 ft.	s = 0.0008
ss = 1-1/2:1	h_B = 5.0 ft.	h_L = 3.5 ft.

(2) The head from the pool to the wasteway channel is sufficient to discharge the design flow, but not great enough to require a structure to dissipate excess energy.

(3) A wasteway channel having dimensions approximately equal to those of the canal is required to connect the wasteway structure to the natural channel.

(4) The natural channel has sufficient capacity to safely discharge the wasteway design flow without damage to the wasteway or adjacent property.

(b) *Solution:* (see fig. 4-6).—

(1) Set spillway crest 0.2 foot above NWS. Find maximum head, H, on spillway crest, allowing 50 percent encroachment on normal canal bank freeboard (see subsec. 4-7(a)):

$$H = 0.5 \times 2.0 - 0.2 = 0.8 \text{ ft.}$$

(2) Find the required crest length from the standard suppressed rectangular weir formula, for a discharge equal to the design inflow from upstream drain inlets. Assume inflow Q = 20 cfs.

$$L_c = \frac{Q}{3.33H^{3/2}} = \frac{20}{3.33 \times 0.8^{3/2}} = 8.4$$

Use L_c = 9 feet

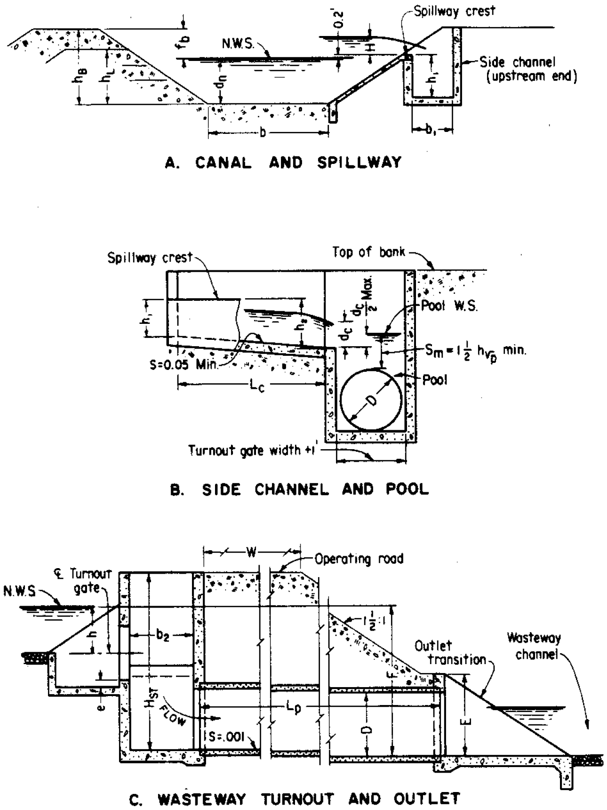


Figure 4-6. **Side channel spillway–typical sections. 103-D-1296**

(3) Design side channel.

Bottom width.–

Upstream end: Use $b_1 = 2$ ft. min.
Downstream end: Use $b_2 = 3$ ft., since $Q < 50$ cfs.*

Wall height.–

$$q_2 = \frac{Q}{b_2} = \frac{20}{3} = 6.67 \text{ cfs}$$

$$d_c = 0.314q^{2/3}$$
$= 1.1$ ft. (see table 17, [4])

$$h_2 = 1.5\ d_c + 1.0 = 1.5 \times 1.1 + 1.0$$
$= 2.65$ ft.
Use 2 ft. 8 in.

$$h_1 = h_2 - 0.05\ L_c = 2.67 - 0.05 \times 9$$
$= 2.22$ ft.
Use 2 ft. 2 in.

(4) Determine wasteway turnout gate size and head for normal canal Q = 100 cfs, maximum gate velocity of 10 feet per second, and a coefficient of discharge, C = 0.6.

Gate area, $A = \frac{100}{10} = 10$ sq. ft. Try a 42- by 36-in. gate with an area of 10.5 sq. ft. Required head,

$$h = \frac{Q^2}{2gA^2C^2} = \frac{(100)^2}{64.4(10.5)^2(0.6)^2} = 3.92 \text{ ft.}$$

As this would require setting the gate centerline below the canal invert, set the centerline of gate at canal invert elevation, and recompute gate size for resultant head of 3.0 feet:

$$A = \frac{Q}{C\sqrt{2gh}} = \frac{100}{0.6\text{x}8.02\sqrt{3.0}}$$
$= 12.0$ sq. ft. required

Therefore, use 42- by 42-inch gate. (A = 12.2 sq. ft.). Refer to gate catalog to determine adequate structural dimensions for mounting gate frame, and find e = 7 inches.

(5) For optimum flow, assume pool WS at gate centerline, or at the side channel invert plus $\frac{d_c}{2}$, whichever WS is lower. Pool WS at gate centerline places pool WS at canal invert. Pool WS at side channel invert plus $\frac{d_c}{2}$ places pool WS 1.08 feet above canal invert. Therefore, pool WS at gate centerline will be used.

(6) Determine pipe diameter and crown submergence for the normal canal capacity of 100 cfs plus the drain inlet capacity of 20 cfs, and with a pipe velocity of 10 feet per second.

The pipe area, $A = \frac{Q}{V} = \frac{120}{10} = 12$ sq. ft. min.

---

*$<$ = Less than
$>$ = Greater than

Then the diameter,

$$D = \sqrt{\frac{4A}{\pi}} = 3.91 \text{ ft. min.}$$

Use D = 4 ft.; A = 12.56 sq. ft.; V = 9.55 f.p.s.; h_{v_p} = 1.42 ft.

Set pipe crown 1.5 h_{v_p} below the pool WS. Submergence,

$$Sm = 1.5\, h_{v_p} = 1.5\,(1.42) = 2.13 \text{ ft.}$$

Set pipe invert on a slope of 0.001.

(7) Find structure height:

$$H_{ST} = h_B + Sm + D = 5.0 + 2.13 + 4.0 = 11.13 \text{ ft.}$$

(8) Select outlet transition from figure 7-4:

D = 4.0 ft., L = 9.0 ft., E = 6.0 ft.

(9) Determine pipe length, L_p (see fig. 4-6):

$$L_p = W + 1.5\,(H_{ST} - E) + 1 = 12 + 1.5\,(11.13 - 6.0) + 1 = 20.70$$

Use 21 ft.

(10) Use type 1 protection at the inlet, extending to the canal centerline, for a length equal to twice the gate width. Use type 3 protection at the outlet, in accordance with figure 7-8. As the maximum velocity from the pipe is less than 15 feet per second, the standard protection provision is adequate.

(11) Check the percolation path according to the design parameters included in chapter VIII, using the appropriate soil type and head, F, shown in figure 4-6C, and by the criteria given in section 4-27.

(c) *Check of Hydraulics (see fig. 4-7).*—Check pool WS (assumed in design) by the Bernoulli Theorem [2], to verify that a submergence of 1.5 h_{v_p} is adequate. Assume L_p is total length of conduit.

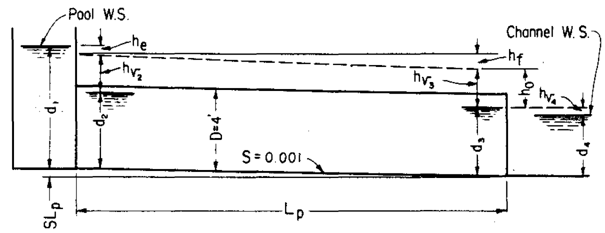


Figure 4-7. **Side channel spillway–outlet hydraulics. 103-D-1297**

(1) Assume wasteway channel as follows:

Q = 120 cfs	b = 8 ft.	R = 1.96
A = 36.65 sq. ft.	d_4 = 2.95 ft.	n = 0.025
V = 3.28 f.p.s.	f_b = 2.05 ft.	s = 0.00125
h_{v_4} = 0.17 ft.	h_B = 5.00 ft.	ss = 1-1/2:1

Specific energy in wasteway channel,
$E_{s_4} = d_4 + h_{v_4} = 3.12$ ft.

(2) Compute pipe flow at critical flow (minimum energy), using tables 21, 22, 58, and 60 [10]. For D = 4 feet,

$D^2 = 16$; $D^{5/2} = 32.0$; and $D^{8/3} = 40.32$.

Then, $\frac{Q}{D^{5/2}} = 3.75$, and $\frac{d}{D} = 0.825$, from which d_c = 3.30 feet, and

$$\frac{A}{D^2} = 0.693, \text{ giving}$$

A = 11.1 sq. ft,, $V_c = \frac{Q}{A} = 10.82$ feet per second, and $h_{v_c} = \frac{V_c^2}{2g} = 1.82$ feet.

Minimum specific energy,
$E_{s_c} = d_c + h_{v_c} = 5.12$ feet

From $\frac{Qn}{D^{8/3} s^{1/2}} = 0.466$, with Q = 120 cfs and n = 0.013, $s_c^{1/2} = 0.083$ and s_c = 0.0069.

(3) As the pipe slope of 0.001 is flatter than that required for critical flow, 0.0069 (0.0041 if n = 0.010), subcritical flow in the pipe is assured.

(4) According to the Bernoulli theorem, the energy in the downstream channel controls the flow in the pipe if a balance of

energy exists. That is, if the energy required in the pipe, E_{s_3} is equal to the energy required in the outlet channel plus outlet losses, the outlet channel controls the flow, as indicated by the equation:

$$E_{s_3} = E_{s_4} + h_o$$

Assume the outlet loss, $h_o = 0.8\Delta h_v$, where Δh_v is the difference in velocity heads in the pipe and in the channel. Thus, the outlet channel controls if

$$E_{s_3} = 3.12 + 0.8\ (1.82 - 0.17)$$
$$= 3.12 + 1.32 = 4.44 \text{ ft.}$$

Since this is less than the minimum energy of 5.12 feet required to sustain a flow of 120 cfs in the pipe, the channel does not control the pipe flow. As the excess energy of 0.68 foot is small, an energy dissipator is not considered necessary; especially since the maximum discharge of 120 cfs occurs very infrequently.

(5) It should be noted that a depth of flow in the pipe, either greater than or less than d_c, would require more energy, resulting in an even greater imbalance of energy. Therefore, critical flow exists at the pipe outlet.

(6) Determine specific energy, E_{s_2}, in the upstream end of the pipe, using the Bernoulli theorem relationship:

$$E_{s_2} + s_o\ L_p = E_{s_3} + h_f$$

where

$$h_f = \text{friction loss} = L_p \left(\frac{s_2 + s_3}{2}\right)$$

Try $d_2 = 3.73$ ft.,

which results in $\dfrac{d}{D} = 0.933$, $\dfrac{A}{D^2} = 0.7627$,

and $\dfrac{Qn}{D^{8/3} s^{1/2}} = 0.498$, from which A = 12.20 sq. ft., $V_2 = 9.84$, and $h_{v_2} = 1.51$ ft.

With Q = 120 c.f.s. and n = 0.013,

$s^{1/2} = 0.0777$, and s = 0.0060.

From the Bernoulli equation,

$$E_{s_2} = E_{s_3} + h_f - s_o\ L_p,$$

$$d_2 + h_{v_2} = 5.12 + L_p \left(\frac{s_2 + s_3}{2}\right) - s_o\ L_p$$

$$3.73 + 1.51 = 5.12 + 21\left(\frac{0.0060 + 0.0069}{2}\right)$$
$$- 0.001 \times 21$$

$$5.24 = 5.12 + 0.14 - 0.02$$

$$5.24 = 5.24$$

Therefore,

$d_2 = 3.73$ ft., $h_{v_2} = 1.51$ ft., and $E_{s_2} = 5.24$ feet.

(7) Determine specific energy, E_{s_1}, in pool. Assume a zero velocity in the pool, with respect to the direction of flow in the pipe. Assume pipe entrance loss, $h_e = 0.5\ \Delta h_v$.

Then $h_e = 0.5\ (1.51 - 0) = 0.76$ ft.

$$E_{s_1} = E_{s_2} + h_e = 5.24 + 0.76 = 6.0 \text{ ft.}$$

As $h_{v_1} = 0$, the pool water depth, $d_1 = 6.0$ ft.

Required submergence, $Sm = d_1 - D$
$= 6.0 - 4.0$
$= 2.0$ ft.

(8) Conclusion.—The submergence of 2.13 feet provided in part (6) of the solution is ample to sustain a discharge of 120 cfs.

(9) If the required submergence had been greater than the 2.13 feet provided, the outlet design would have to be modified by lowering the pipe, or increasing the pipe diameter, and possibly improving downstream channel hydraulics.

2. *Gated Wasteway Turnouts*

4-9. *General.*—(a) *Usage.*—While wasteway gates are usually combined with side channel spillways or siphon spillways to provide both the automatic control feature and the capability of emptying the canal, some wasteway turnouts do not have provision for overflow. Such wasteways, which depend upon timely gate operation, are best used in canals that do not have drain inlets, or are equipped with automatic or remote control operation, or in canals that can, for a short time, store or accommodate excess flows without endangering the system.

This type of wasteway turnout provides the means of removing all or part of the canal water to safeguard the canal and adjacent property in the event of a canal bank failure or threatened failure. To perform this function, it is usually necessary to provide a check structure on the canal, immediately downstream from the wasteway turnout (see fig. 4-8). The threatened reach of canal may be isolated by closing the check gate and opening the wasteway turnout gate to divert the canal flow to the wasteway.

The wasteway turnout is also used to empty the canal for inspection or maintenance, or at the end of the irrigation season. Stoplogs and stoplog slots are usually provided just upstream from the gate, to permit maintenance of the gate without draining the canal. The gates may be operated manually, remotely, or automatically.

Figure 4-8. **Wasteway turnout at left, and radial gate check structure at right side of photograph. Backfill not completed. PX-D-72606D**

(b) *Wasteway Capacity.*—Most wasteway turnouts should be capable of discharging the entire canal capacity, including inflows from drain inlets. The check overflow wall usually has a freeboard of 0.2 foot maximum above the normal water surface. Therefore, the maximum head available for this wasteway turnout gate without overflowing the check walls is equal to the normal depth plus 0.2 foot.

(c) *Types of Gates.*—

(1) Slide gates are readily available in many types to meet the varying head and operational requirements. A low-head slide gate with no back-pressure requirement is usually satisfactory for a wasteway turnout from a canal. However, to prevent gate leakage, it should be of cast iron construction. An air vent is sometimes required behind the gate if the gate is attached to a closed conduit (see chapter VIII).

As design considerations for slide gates are discussed in subsections 4-7(c) and 4-10(b), the following discussion places an emphasis on radial gate wasteway turnouts. Slide gate wasteway turnouts do not differ essentially from the standard turnouts discussed in chapter III.

(2) Radial gates (see fig. 4-9).—Radial gates are not often used for small discharges. However, they have an advantage over slide gates in that they can be lifted clear of the water surface, allowing floating debris and ice to be flushed into the wasteway channel. This, combined with excellent flow characteristics give the radial gate wasteway turnout a definite advantage in some locations. With a gate freeboard of about 0.2 foot, to prevent waste of water from minor wave action, the gate can accommodate some overflow before it is opened.

Radial gates used for wasteway turnouts are equipped with rubber seals which bear against metal wall and floor plates for better watertightness. The floor and wall plates may be equipped with heating cables or tapes, to insure free movement of the gate during the winter, when ice might otherwise

Figure 4-9. **Radial gate used in combination with overflow spillway. P245-700-1405D**

render the gate inoperable.

4-10. ***Hydraulics.***—(a) *Inlet.*—The wasteway turnout structure usually includes an inlet transition which converges the flow to the gate bay. The invert at the gate is usually slightly lower than the canal invert to facilitate complete drainage of the canal.

The entrance loss is a function of the differential velocity heads, Δh_v, and is usually assumed to be $0.5\Delta h_v$ (see chapter VII). The differential velocity head at an inline transition is equal to $h_{v_2} - h_{v_1}$. However, a transition oriented in a position normal to the canal alinement, as shown in figure 4-11, will have a differential velocity head equal to h_{v_2}, as the canal velocity is equal to zero in the direction of the turnout flow. Thus, assuming a transition loss of $0.5\ h_{v_2}$, the specific energy available in the gate bay,

$$E_{s_2} = E_{s_1} + \Delta El. - 0.5\ h_{v_2}$$

$$d_2 + h_{v_2} = d_1 + h_{v_1} + \Delta El. - 0.5\ h_{v_2}$$

Having assumed a zero velocity in the canal in the direction of the wasteway flow, the velocity head, $h_{v_1} = 0$. If the velocity in the gate bay, $V_2 = 10$ feet per second, the velocity head,

$$h_{v_2} = \frac{V_2{}^2}{2g} = \frac{10^2}{64.4} = 1.56 \text{ feet}$$

Then, using an invert drop, $\Delta El. = 0.1$ foot (considered to be a minimum drop),

$$d_2 + h_{v_2} = d_1 + 0 + 0.1 - 0.5\ h_{v_2}$$

$$d_2 = d_1 + 0.1 - 1.5\ h_{v_2}$$

$$d_2 = d_1 + 0.1 - 2.34$$

$$d_2 = d_1 - 2.24 \text{ ft.}$$

(b) *Slide Gates.*—The slide gate installation is usually designed for an upstream water surface above the top of the gate opening. Under this condition, the discharge is determined by the orifice equation [2],

$$Q = CA\sqrt{2gh}$$

where

Q is the discharge in cubic feet per second,
C is the discharge coefficient,
A is the area of the gate opening in square feet,
h is head available at the gate in feet, and
g is the acceleration of gravity in feet per second per second

If free flow exists downstream from the gate, as shown in figure 4-12, the head, h, is measured from the upstream water surface elevation to the centerline of the gate opening. If, however, the bottom of the gate is submerged on the downstream side also, the head, h, is a measure of the difference in water surface elevations, upstream and downstream from the gate. A conservative value of the coefficient of discharge, C, for the slide gate [3] is equal to about 0.60.

As the wasteway is not often operated at the maximum design discharge, the gate size is determined on the basis of a maximum velocity of 10 feet per second through the fully opened gate. However, if sufficient head is not available to attain this velocity, a larger gate may be required to obtain the desired discharge.

(c) *Radial Gates.*—When the radial gate is operated partially open, the discharge is determined by the orifice equation [2],

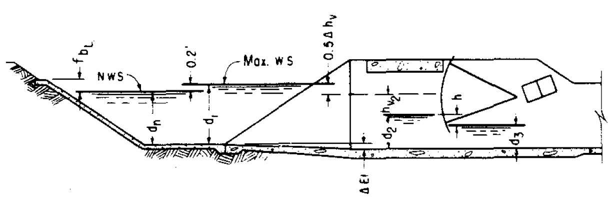
$$Q = CA\sqrt{2gh}$$

Where free flow exists downstream from the gate, as shown in figure 4-12, the head, h, is measured from the upstream water surface elevation to the centerline of the gate opening. If the downstream water depth submerges the bottom of the gate, as shown in figure 4-10A, the head is a measure of the difference in water surface elevations, upstream and downstream from the gate. The coefficient of discharge, C, for the radial gate [3] is equal to about 0.72.

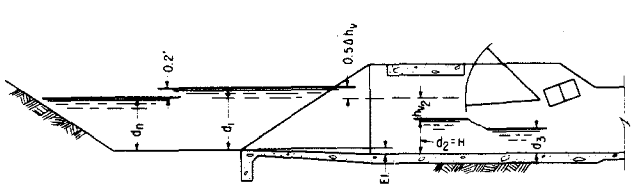
The radial gate, when operated for maximum discharge, is fully open, with the bottom of the gate above the upstream water surface, as shown in figure 4-10B. Under this condition, the discharge is determined from the weir equation [2],

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

where H is the upstream water depth. A discharge coefficient [2], C, of 3.09, is often



A. Submerged orifice flow
$Q=CA\sqrt{2gh}$



B. Submerged flow over broad crested weir
$Q=CL(nH)^{3/2}$

Figure 4-10. **Radial gate wasteway turnout–submerged flow (for free flow at outlet see fig. 4-12). 103-D-1298**

used for the broad crested weir with free flow.

Where the weir flow is partially submerged by the downstream depth, a correction factor, n, is used:

$$Q= CL\,(nH)^{3/2}$$

Values of n may be taken from table 37 [4].

As the wasteway is not often operated at the maximum design discharge, the size of the radial gate is determined on the basis of a maximum velocity of 10 feet per second through the gate bay (with the gate fully opened). The minimum gate width, equal to $\frac{A}{d_2}$, is then equal to $\frac{Q}{10d_2}$. An excessively wide radial gate would be uneconomical, and may also present binding problems in opening and closing. The gate width should not be greater than $3h_g$, where h_g is the gate height (equal to d_n plus e plus a freeboard of 0.2 foot minimum, as shown in figure 4-12).

(d) *Trajectory.*—When the gate is open slightly, a jet of water will flow under the gate at a high velocity. Where a drop or chute is situated downstream from the gate, the flow should be transitioned to the steeper slope. Without such a transition, the high velocity flow may spring free from the invert at the beginning of the steeper slope, resulting in negative pressure, and unstable flow conditions.

To avert such a condition, a trajectory could be used to transition the flow to the steeper slope. The length of the trajectory should be determined from the equation [5],

$$X^2 = 4HY$$

where

X is the horizontal distance from the beginning of the curve to a point on the curve,
Y is the vertical distance from the beginning of the curve to a point on the curve, and
H is the head measured from the maximum upstream water surface to the gate bay invert, assuming no inlet head loss.

The required length and profile of the

trajectory may be determined by plotting points on the curve, ending where the slope of the curve is equal to the slope of the chute or drop. The length may also be determined by solving the above equation by differential calculus, for the distance, X, at which point the slope of the curve is equal to the slope of the downstream drop.

Where both the upstream and downstream inverts are on a slope, a better solution is described in chapter II.

(e) *Outlet.*—For a discussion of considerations for wasteway outlets, see subsection 4-7(f).

4-11. *Lifts and Hoists.*—(a) *Manually Operated Lifts.*—Lifts, consisting of a hand crank or wheel and gears are required to open and close the gates. The gear ratio should be consistent with the gate weight and the frictional force imparted by the hydrostatic load on the slide gate, or by the rubber seals of the radial gate. An unnecessarily high gear ratio should be avoided, as it would retard the rate of opening, which may be critical.

(b) *Power-operated Lifts or Hoists.*—Slide gates and radial gates may be controlled automatically, remotely, or by a local on-off switch, where electrical power is available or where a motor-generator is used. The water level is monitored by floats or probes, which actuate the hoist controls. One such device for providing automatic control of the water surface level is the "Little Man" [6], designed and used by the Bureau of Reclamation.

(c) *Hydraulically Controlled.*—Automatic radial gates have been used without an external power source, by utilizing the increased hydrostatic head which accompanies increased depth. To provide upstream water level control, the gate leaf includes a horizontal projection, upon which the uplift pressure imparts a force to open the gate. With an adjustable counterweight to provide sensitivity, any rise in the upstream water level causes an equivalent rise in gate position.

This type of automatic gate, if the installation provides enough clearance to move freely, has a tendency to leak, and should be used only where some leakage is acceptable. It is also susceptible to erratic behavior if weeds or trash lodge under the gate, preventing complete closure.

4-12. *Miscellaneous Considerations.*—

(a) *Protection.*—Erosion protection for the canal at the inlet, and for the outlet channel should be in accordance with the requirements of figure 7-8.

(b) *Percolation.*—Excessive percolation of water from the canal to the outlet channel should be avoided. See section 4-27 and subchapter VIII C for design considerations for alleviation of percolation.

(c) *Operation.*—For limitations on drawdown rate, see section 4-5.

4-13. *Design Example.*—Utilizing a radial gate, design a wasteway for the canal features shown in figure 4-11. (A design example using a slide gate is included in section 4-8.)

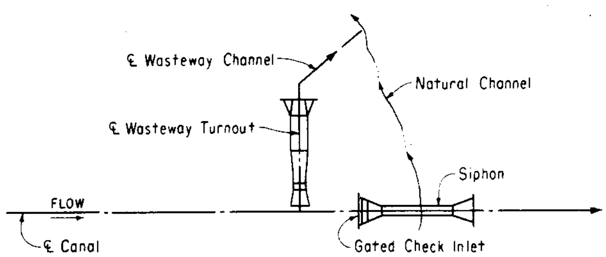


***Figure 4-11.* General plan of typical canal with wasteway turnout. 103-D-1299**

(a) *Assumptions.*—

(1) Assume an earth-lined canal with the following properties:

Q = 100 cfs	h_L = 5.0 ft.	n = 0.025
b = 8.0 ft.	h_B = 6.3 ft.	s = 0.0002
d_n = 4.3 ft.	f_b = 2.0 ft.	ss = 1-1/2:1
V = 1.61 f.p.s.		

(2) Assume that the entire canal flow must be diverted to the wasteway to prevent failure of the canal bank downstream. As there are no drain inlets upstream, the maximum wasteway discharge will be equal to the normal canal capacity of 100 cfs.

(3) Assume that a check structure is located immediately downstream from the wasteway, as shown in figure 4-11, and that an overflow wall is provided with a freeboard of 0.2 foot above the normal water surface.

(4) Assume that the drop in the wasteway profile provides free flow downstream from the wasteway gate, and enough fall (see dimension F, fig. 4-12) to require energy dissipation.

(5) Assume an invert drop, e = 0.5 foot, is desired in the inlet transition.

(6) Assume that a natural channel is available, having sufficient capacity to take the maximum canal flow without damage to adjacent property.

(b) *Solution (see fig. 4-12).*–

(1) Inlet dimensions:

Length, L_1: Assuming the top of the structure at about the elevation of the top of the bank,

$$L_1 = 1.5\ h_B$$
$$= 1.5 \times 6.3 = 9.45 \text{ feet}$$

Use

$$L_1 = 9 \text{ ft. } 6 \text{ in.}$$

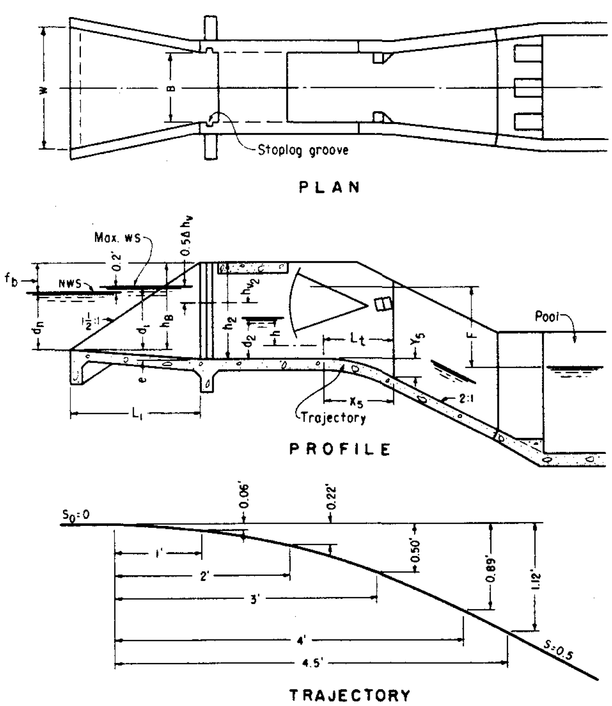


Figure 4-12. **Radial gate wasteway turnout–typical sections. 103-D-1300**

Wall height, h_2:

$$h_2 = \frac{L_1}{ss} + e$$

$$= \frac{9.50}{1.5} + 0.5$$

$$= 6.33 + 0.5 = 6.83 \text{ feet}$$

Use $h_2 = 6$ ft. 10 in.

(2) Determine required gate height for normal depth. Assume gate freeboard, f_{b_g} = 0.20 foot

Gate height,

$$h_g = d_n + e + f_{b_g}$$

$$= 4.3 + 0.5 + 0.20 = 5.00 \text{ feet}$$

Use 5 ft. 0 in.

(3) Determine maximum canal depth, d_1, without overflowing the check overflow wall.

$$d_1 = d_n + 0.2$$

$$= 4.3 + 0.2 = 4.5 \text{ feet}$$

(4) Determine depth, d_2, in gate bay, assuming a velocity, V_2 = 10 feet per second in the gate bay, and a velocity, $V_1 = 0$ in the canal (in the direction of the turnout flow).

The specific energy,

$$E_{s_2} = E_{s_1} + e - 0.5\ \Delta h_v$$

$$d_2 + h_{v_2} = d_1 + h_{v_1} + e - 0.5\ (h_{v_2} - h_{v_1})$$

$$d_2 + 1.56 = 4.5 + 0 + 0.50 - 0.5\ (1.56)$$

and

$$d_2 = 5.00 - 1.5\ (1.56) = 2.66 \text{ ft.}$$

(5) Determine the required gate bay

width, B, using a maximum velocity of 10 feet per second.

$$A = \frac{Q}{V} = \frac{100}{10} = 10 \text{ sq. ft. minimum}$$

$$B = \frac{A}{d_2} = \frac{10}{2.66} = 3.76 \text{ feet minimum}$$

(6) Determine the required gate bay width, B, using the available head, ($d_2 + h_{v_2}$ = 2.66 + 1.56 = 4.22 feet) in the weir equation [2] (assuming the radial gate is fully opened),

$$Q = CB\,H^{3/2}$$

$$B = \frac{Q}{CH^{3/2}} = \frac{100}{3.09(4.22)^{3/2}}$$

$$= 3.73 \text{ feet}$$

$$< 3.76 \text{ feet}$$

Use

$$B = 4 \text{ ft. } 0 \text{ in.}$$

(7) Design a trajectory to transition the flow from the level invert at the gate to the 2 to 1 invert slope ($\tan \phi = 0.5$) which precedes the stilling pool. The minimum length of the trajectory is determined from the equation,

$$X^2 = 4HY$$

where $H = d_1 = 4.5$ feet

Then $X^2 = 18Y$

and $Y = X^2/18$

The slope of the tangent to the curve at any point is represented by the first derivative,

$$\frac{d}{dX}\frac{X^2}{18} = \frac{2X}{18} = \frac{X}{9}$$

Setting this slope equal to the downstream slope of 0.5,

$$\frac{X}{9} = 0.5$$

$$X = 0.5 \times 9 = 4.5 \text{ feet}$$

Solving for the vertical distance, Y, at 1-foot intervals of X, using the relationship,

$$Y = \frac{X^2}{18}$$

The following tabulation in feet can be made:

X	Y
0	0
1	0.06
2	0.22
3	0.50
4	0.89
4.5	1.12

(8) Check of turnout capacity.—To insure the capability of discharging the design flow without exceeding the maximum canal water surface, the hydraulics should be checked by the Bernoulli equation [2].

Assume that the gate is fully open, and that critical depth occurs at the beginning of the trajectory, as the downstream slope is obviously steeper than critical (see fig. 4-12). Use an entrance loss of 0.5 Δh_v, and disregard the minimal effect of friction.

In the rectangular section, critical depth

$$d_c = \frac{q^{2/3}}{g^{1/3}} = 0.314\,q^{2/3}$$

$$= 0.314 \left(\frac{Q}{B}\right)^{2/3}$$

$$= 0.314 \left(\frac{100}{4}\right)^{2/3}$$

$$= 0.314\,(8.55) = 2.68 \text{ feet}$$

$$\text{and } h_{v_c} = \frac{d_c}{2} = \frac{2.68}{2} = 1.34 \text{ feet}$$

From the Bernoulli equation,

$$d_1 = d_c + h_{v_c} + 0.5\,\Delta h_v - e$$

$$= 2.68 + 1.34 + 0.5\,(1.34) - 0.5$$

$$= 4.19 \text{ feet}$$

As the maximum canal water depth is 4.50 feet, the structure, as designed, has sufficient capacity.

(9) Design the stilling pool and outlet protection in conformance with the design requirements discussed in chapters II and VII.

(10) Protection.—Inlet protection should consist of 6 inches of coarse gravel, and should extend to the canal centerline for a canal length equal to twice the gate width, or 8 feet minimum.

(11) Percolation.—Percolation should be checked by the Lane weighted-creep method, as discussed in subchapter VIII C.

3. *Siphon Spillways*

4-14. *General.*—(a) *Usage.*—Siphon spillways, as discussed in this text, are limited to hydrostatic heads no greater than the atmospheric pressure equivalent at the site. For this reason, they are sometimes referred to as "low-head siphon spillways."

As distinguished from the inverted siphon (discussed in chapter II), which operates under positive pressure, the siphon spillway, which operates under negative pressure, is sometimes called a true siphon. Once primed, the siphon spillway has the capability of lifting the water over the crest, and discharging it at a lower elevation (see fig. 4-17).

Siphon spillways are very effective in rapidly removing a large volume of water from a canal, but may be more expensive to construct than other wasteway turnouts, because of the formwork required to construct the structure. However, the narrow crest width required for the siphon spillway favors its use where space is inadequate for structures utilizing free-flow crests.

The automatic operation of the siphon spillway eliminates the need for frequent manual manipulation of control structures along the canal. A distinct advantage is its ability to discharge a large volume of water with a small rise in water surface.

(b) *Description.*—The siphon spillway consists of a closed conduit, having an upper leg extending into the upstream pool, and a lower leg extending down from the crest into the lower pool (see fig. 4-17). A siphon breaker, consisting of an air vent pipe, extends upward from the siphon crown, and down into the upstream pool (see fig. 4-13). The downstream pool provides dissipation of excess energy.

(c) *Function.*—Small excesses of flow from the inflow of storm water at drain inlets will result in a slowly rising water surface in the canal, initially causing the siphon to act as a weir. If the canal water surface continues to rise, the discharge through the siphon will attain a velocity sufficient to prime the siphon.

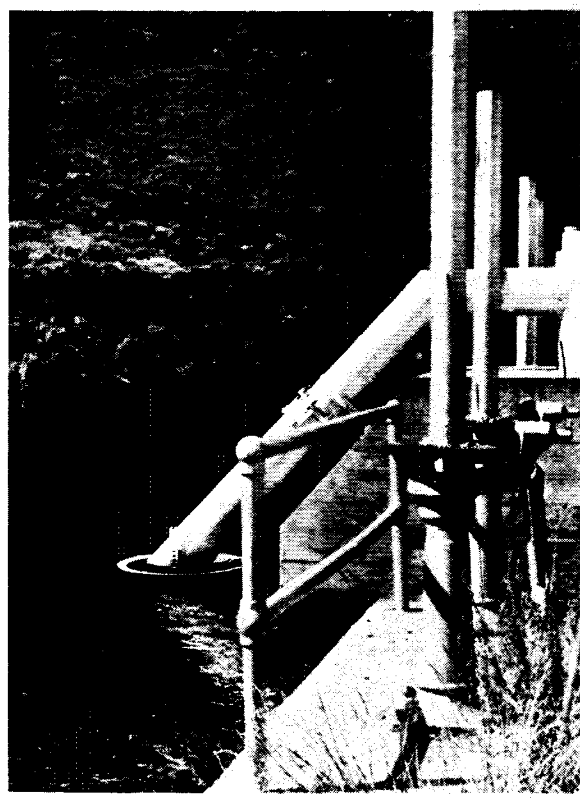


Figure 4-13. **Siphon spillway with siphon breaker pan. Gate hoist in foreground. P17-D-62105**

The resulting siphonic flow will quickly lower the canal water surface and break the siphonic action.

It is necessary that the siphon be fully primed, to discharge its full design capacity. That is, all the air must have been removed from the rectangular siphon barrel. Only then is the full area of the barrel effective for flow. Priming is initiated as the water flowing over the crest and into the downstream leg attains a velocity sufficient to carry an air-water mixture out of the barrel. To facilitate the air-water mixing process, a deflector is included on the sloping invert, positioned to cause the trajectory of water to leave the invert and intercept the top side of the barrel. A full prime occurs when evacuation of air from the barrel is complete, and pressure over the crest becomes subatmospheric.

Once fully primed, siphonic action continues until the canal water surface drops below the inlet elevation of the siphon breaker pipe, allowing an inflow of air to restore atmospheric pressure at the crest, ending the flow abruptly.

One disadvantage to the use of a siphon spillway in a small canal is the abrupt drawdown and the attendant surges or bore waves which may accompany the sudden action of priming and breaking. See Design Considerations, section 4-15(h). This may disturb briefly the fine adjustment of turnout gates, and may also cause erosion in the downstream channel.

(d) *Appurtenances.*—Slide gates or radial gates are generally used in conjunction with siphon spillways to provide complete drainage of the canal. Such a combination is shown in figure 4-14. A check structure is usually provided in the canal, a short distance downstream from the wasteway, to permit diversion of the entire canal flow into the wasteway.

4-15. *Design Considerations.*— (a) *General.*—While many types of siphon spillways have been used in the past, this discussion will be limited to the configuration shown in figure 4-17, which features a straight leg on the downstream side of the crest. The advantages [7] of this type over the S-curve type structure are as follows:

(1) Greater consistency of discharge capacity,

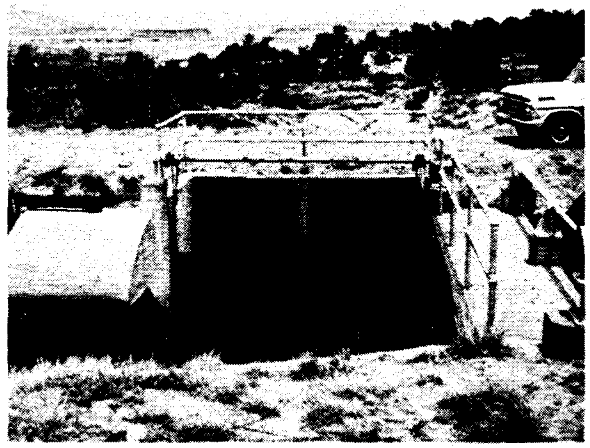


Figure 4-14. **Siphon spillway hood at left, radial gate for wasteway turnout in center, and hoist for radial gate check at right side of photograph. PX-D-72609**

(2) Reduction in priming time,

(3) Attaining a prime with a smaller crest head, and

(4) Simplicity of construction resulting in greater economy.

(b) *Capacity.*—The siphon spillway utilizes a large differential head to discharge large flows through a siphon barrel of small cross-sectional area, as computed from the following orifice and vortex flow equations [8]:

$$q = CD\sqrt{2gH} \leq R_c\sqrt{0.7h\,(2g)}\,\log_e \frac{R_s}{R_c}$$

where

q is the discharge per foot of width;

D, H, R_c, and R_s are dimensions shown in figure 4-17;

C is the coefficient of discharge in the orifice equation (a conservative value being 0.6 for the design illustrated);

h, in the vortex flow equation is the atmospheric pressure at the site, in feet of water; and the constant, 0.7 provides a conservative limitation on the subatmospheric pressure allowed [8].

(c) *Head.*—The atmospheric pressure head, h, is based upon the elevation at the site. The operating head, H, should not exceed the atmospheric pressure head.

The concrete structure should be capable of withstanding a negative internal pressure equal to the head, H, plus any other loads imposed upon the structure.

(d) *Entrance.*—The inlet orifice should provide an area equal to at least twice the throat area. The normal canal water surface should submerge the top of the inlet orifice at least 1.5 h_{v_o} plus 6 inches, or at least 1 foot, whichever is greater (where h_{v_o} is the velocity head through the orifice).

(e) *Crest.*—The siphon crest should be set about 0.2 foot above the normal water surface. An adjustable crest is sometimes used, but an adverse effect upon priming may result. To provide a satisfactory discharge coefficient, a ratio, $\frac{R_{℄}}{D} = 2.0$ is recommended (see fig. 4-17).

(f) *Throat.*—To facilitate construction, and to allow passage of some debris, the throat opening, D, as shown in figure 4-17, should be at least 2 feet, and the crest width, b, should be at least 3 feet. It is often desirable to use two or more adjacent siphons in lieu of one having a wide throat, to avoid long spans subjected to heavy loads.

(g) *Downstream Leg.*—The straight leg design has several advantages over the S-curve design as itemized above (see subsection (a) above). A deflector is positioned on the sloping invert, as shown on figure 4-17, at a location designed to deflect the flow to a trajectory extending across the tube to the top side. Impact with the top side, and turbulence in the pool serve to mix air and water. The turbulence thus created and the forward component of velocity combine to produce immediate air evacuation to initiate the prime.

The top side of the downstream leg terminates in the pool. The terminal end should be pointed as shown in figure 4-17, to facilitate an exodus of air bubbles from the siphon tube.

(h) *Siphon Breaker.*—The location of the siphon breaker should be determined precisely. It opens into the crown area at a point 15° downstream from the vertical axis through the crest, as shown in figure 4-17. It has been found that maximum negative pressures occur at or near this point. The cross-sectional area of the siphon breaker pipe should be about one twenty-fourth of the throat area.

The upstream end of the siphon breaker pipe should be set at the normal water surface elevation in the canal. An adjustable sleeve permits a fine adjustment to compensate for operational changes. It should be airtight to insure negative pressure in the siphon barrel.

To prevent the siphon from lowering the canal water surface below the normal water surface elevation, a shallow pan is mounted a short distance below the siphon breaker pipe to break the prime (see fig. 4-17, detail A). Without the pan, some siphon breaker pipes act as siphons themselves, lifting a free column of water, allowing the siphon spillway to continue to operate while the canal water surface is drawn down well below the siphon crest, before siphonic action is broken [9]. With the pan in place, as shown in figure 4-15, the magnitude of water surface fluctuations in the canal should be very slight.

(i) *Pool.*—The downstream pool submerges the siphon tube outlet to prevent re-entry of air. The top of the sloping sill should be at the same elevation as the crown of the siphon tube outlet.

The roof of the pool section should be of sufficient distance above the pool water surface to permit free escape of air from the pool. Assuming that the outlet channel does not materially submerge the flow over the sill, a wall height above the sill should be equal to the critical depth plus velocity head, plus 1 foot of freeboard, as shown in figure 4-17.

The width of the pool should be equal to the throat width. A minimum width of 3 feet is recommended for capacities of 100 cfs and less.

(j) *Outlet Transition.*—The outlet transition, usually in a well defined channel, should be designed on the same basis as broken-back transitions for stilling pools (see subchapter II F). An angle of divergence of 25° on each side may be used.

(k) *Protection.*—Erosion protection should conform to the requirements of figure 7-8 for cross drainage. Inlet protection in an unlined canal, should extend to the canal centerline for a canal length equal to twice the siphon throat

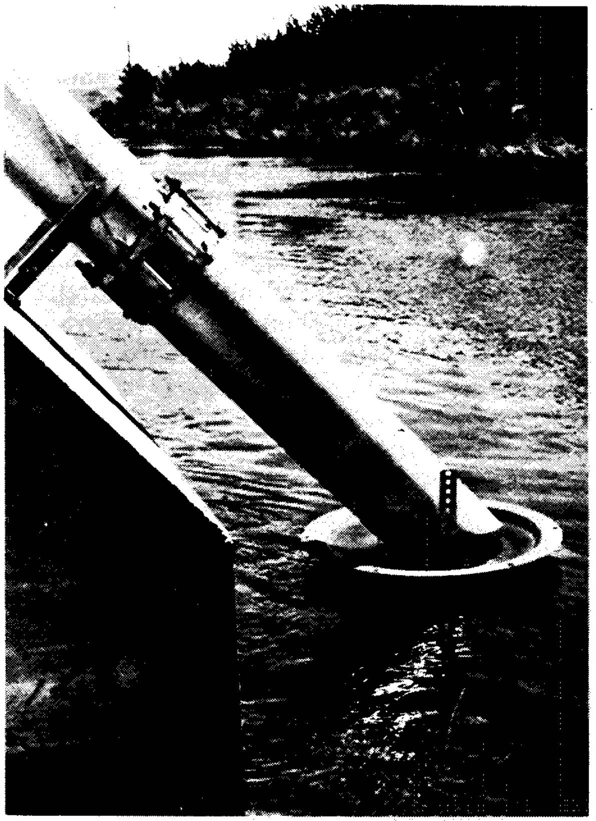


Figure 4-15. **Siphon spillway with siphon breaker pan in place. P-17-D-62107**

width, or 8 feet minimum.

(l) *Percolation.*—To control percolation of water from the canal to the outlet channel, a percolation study should be made in accordance with chapter VIII, and cutoff walls provided where necessary. A cutoff wall is often located in line with the bottom of the pool sill as shown in figure 4-17.

4-16. ***Design Example.***—Design a siphon spillway to be used for discharging excess canal flows into a wasteway channel, as shown in figure 4-16. A slide gate, designed separately, provides for emptying the canal.

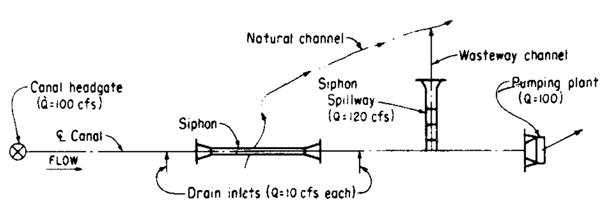


Figure 4-16. **General plan of typical canal with siphon spillway. 103-D-1301**

(a) *Assumptions:*

(1) Canal properties are as follows:

Q_n = 100 cfs	b = 8 feet	f_b = 2.0 feet
d_n = 3.7 feet	h_B = 5.7 feet	ss = 1-1/2:1

(2) The canal flows to a pumping plant downstream from the wasteway. A power failure occurs at the plant while the canal is flowing at design capacity plus drain inlet capacity, resulting in a total discharge of 120 cfs.

(3) The available head, H, as shown in figure 4-17, is equal to 6 feet.

(4) A wasteway channel, having dimensions approximately equal to those of the canal, is required to connect the wasteway structure to the natural channel.

(5) The natural channel has sufficient capacity to safely discharge the wasteway design flow without damage to the wasteway or adjacent property.

(6) The wasteway structure is located at an elevation of 6,000 feet, with an atmospheric pressure, h, equal to 11.8 pounds per square inch, or 27.3 feet of water.

(b) *Solution (see fig. 4-17).*—

(1) Given:

$$Q = 120 \text{ cfs}$$
$$H = 6 \text{ ft.}$$
$$h = 27 \text{ ft.}$$

(2) Use:

$$C = 0.6$$
$$\frac{R_{\text{℄}}}{D} = 2.0$$

(3) Try:

$$D = 2 \text{ ft.}$$

(4) Crest dimensions:

$$R_{\text{℄}} = 2D = 2 \times 2 = 4 \text{ ft.}$$

$$R_C = R_{\text{℄}} - \frac{D}{2} = 4 - 1 = 3 \text{ ft.}$$

$$R_s = R_{\text{℄}} + \frac{D}{2} = 4 + 1 = 5 \text{ ft.}$$

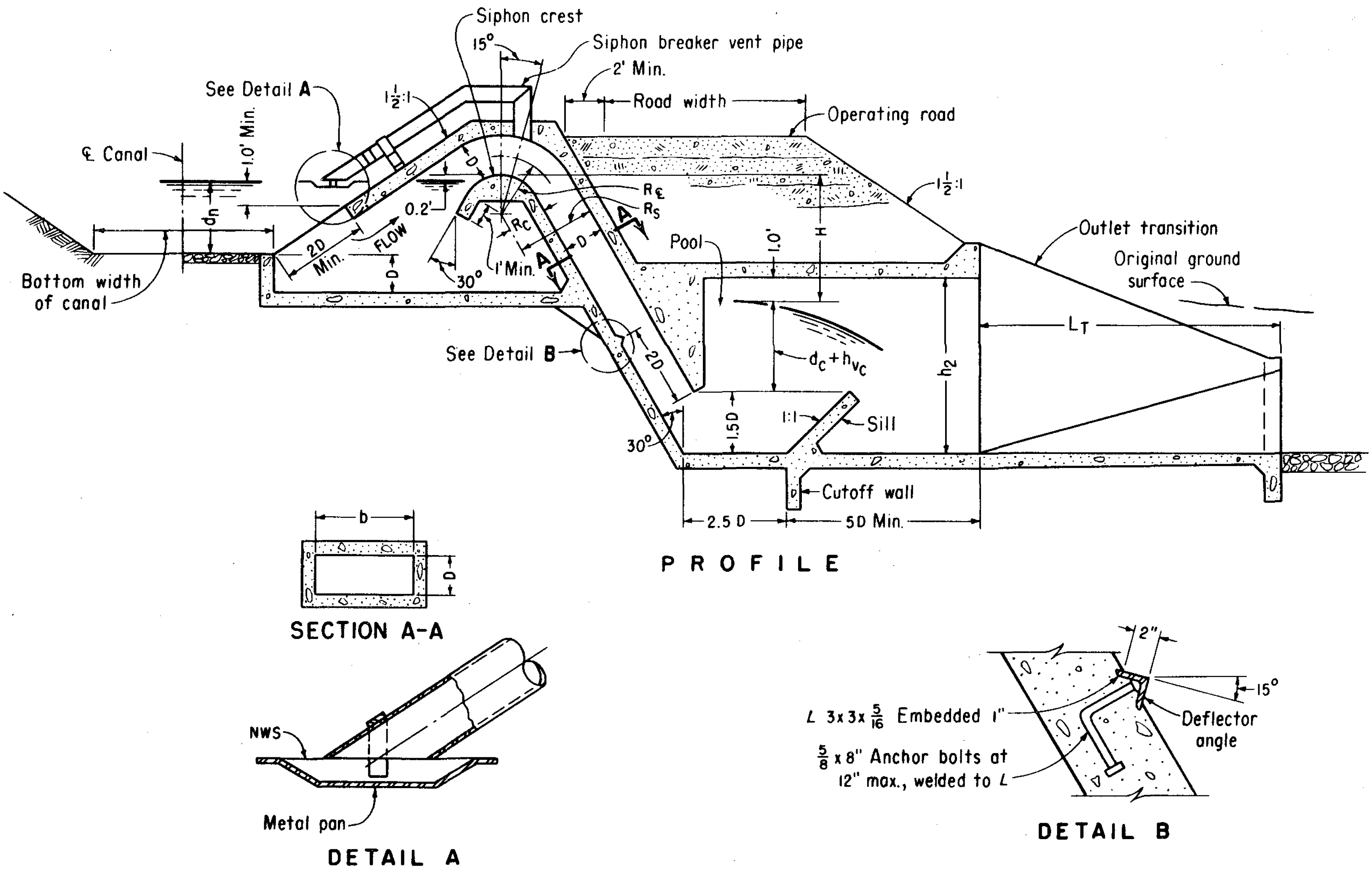


Figure 4-17. **Siphon spillway–typical sections. 103-D-1302**

(5) Determine discharge by the orifice equation:

$$\begin{aligned} q &= CD\sqrt{2gH} \\ &= 0.6 \times 2\sqrt{64.4 \times 6} \\ &= 0.6 \times 2 \times 8.02\sqrt{6} \\ &= 23.6 \text{ cfs per foot of width} \end{aligned}$$

(6) Determine discharge by the vortex equation:

$$\begin{aligned} q_{max.} &= R_C \sqrt{0.7h\ 2g}\ \mathrm{Log}_e \frac{R_s}{R_C} \\ &= 3\sqrt{0.7 \times 27 \times 64.4}\ \mathrm{Log}_e \frac{5}{3} \\ &= 3\sqrt{1216}\ \mathrm{Log}_e\ 1.667 \\ &= 3 \times 34.9 \times 0.511 \\ &= 53.5 \text{ cfs per foot of width} \end{aligned}$$

(7) As q does not exceed q_{max}, the design of the siphon for q = 23.6 cfs per foot of width is valid. If q had been greater than $q_{max.}$, the head, H, should be decreased; or, had the dimensions D or R_c been greater than the minimum permitted, they could be reduced to provide a discharge $q < q_{max}$.

(8) Determine the throat width,b.

$$b = \frac{Q}{q} = \frac{120}{23.6} = 5.08 \text{ ft.}$$

Use b = 5 feet

Had the width, b, resulted in an excessively wide span, a greater dimension, D, should be used to reduce the span.

(9) Determine siphon breaker pipe diameter, D_p, to provide a pipe area, A_p, equal to or greater than one twenty-fourth the throat area, A_t.

Thus,

$$A_p = \frac{A_t}{24} = \frac{2 \times 5}{24} = 0.417 \text{ sq. ft.}$$

Then, the pipe diameter,

$$D_p = 0.73 \text{ ft.}$$

Use

$$D_p = 9 \text{ inches}$$

(10) Pool sill height,

$$\begin{aligned} h_s &= 1.5D \\ &= 1.5 \times 2 = 3 \text{ ft.} \end{aligned}$$

(11) Pool box height (see fig. 4-17),

$$h_2 = 1.5D + d_c + h_{v_c} + 1$$

where the pool width, W_p = 5.0 feet,

$$q = \frac{120}{5} = 24.0 \text{ cfs}$$

d_c = 2.62, and

$$h_{v_c} = \frac{2.62}{2} = 1.31 \text{ (in rectangular section)}$$

Then,

$$\begin{aligned} h_2 &= 1.5 \times 2 + 2.62 + 1.31 + 1 \\ &= 7.93 \text{ feet} \end{aligned}$$

Use

$$h_2 = 8 \text{ ft. } 0 \text{ in.}$$

(12) *Outlet Transition.*—Assuming a well defined outlet channel with a bottom width of 8 feet and a water depth, d = 4 feet (for Q = 120 cfs), use a broken-back transition with a uniform invert width equal to the pool width, W_p = 5.0 feet.

The water surface width at the cutoff,

$$\begin{aligned} T_c &= 5.0 + 2 \times 1.5\ d \\ &= 5.0 + 3 \times 4 = 17.0 \text{ feet} \end{aligned}$$

The water surface divergence is equal to

$$T_c - W_p = 17 - 5 = 12.0$$

with 6 feet of divergence on each side.

The transition length,

$$L_T = \frac{6}{\text{Tan } 25^\circ} = \frac{6}{0.4663} = 12.9 \text{ feet}$$

Use

$$L_T = 13 \text{ feet}$$

The remaining divergence to the channel having a bottom width of 8 feet, should be accomplished in an earth transition.

(13) Determine entrance hydraulics, using an orifice slope length of 2D (see fig. 4-17).

$$V = \frac{Q}{A} = \frac{120}{2Db}$$

$$= \frac{120}{2 \times 2 \times 5.0} = \frac{120}{20} = 6.0 \text{ f.p.s.}$$

$$h_v = \frac{V^2}{2g} = 0.56 \text{ feet}$$

(14) Check entrance submergence (minimum of 1 foot) for NWS, to insure a submergence,

$$Sm = 1.5\ h_v + 0.5 \text{ min.}$$
$$= 1.5 \times 0.56 + 0.5 = 1.34 \text{ ft. minimum}$$
$$Sm = d_n - y$$

where

$$y = 2D \sin\ \phi$$

and ϕ is the angle of the canal side slope.

Assume the bottom of the orifice at the canal invert.

Then,

$$Sm = d_n - 2\ D \sin 33^\circ 41'$$
$$= 3.7 - 4 \times 0.5546$$
$$= 3.7 - 2.22 = 1.48 \text{ feet}$$
$$> 1.34 \text{ feet minimum}$$

(15) *Protection.*—As indicated in figure 7-8 for cross-drainage flows, inlet protection should consist of 6 inches of coarse gravel, and should extend to the canal centerline, for a canal length equal to twice the throat width. Outlet protection should consist of 12 inches of riprap on 6 inches of sand and gravel bedding, extending 16 feet downstream from the structure.

(16) *Percolation.*—Percolation should be checked in accordance with the requirements of subsection 4-15(l).

4. *Check Inlet Wasteway Turnouts*

4-17. *General.*—The check and pipe inlet, and the rectangular inclined (R.I.) drop with a check inlet are ideally suited as turnouts to small terminal wasteways. They include an overflow (see fig. 4-18) to spill (automatically) the unused water resulting from operational mismatch of water supply and demand, or excess water from storm or drainage inflows. Complete drainage of the canal can be accomplished by removal of stoplogs or opening of the gate.

As the need to empty the canal is infrequent, stoplogs are usually satisfactory for use with the check and pipe inlet. However, they should be sealed with a waterproofing material if leakage is objectionable. Slide gates should be used where remote control or frequent adjustment is desired. To minimize

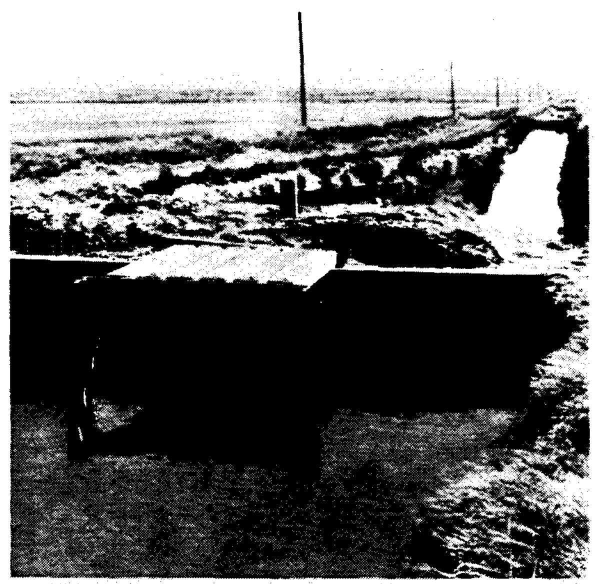


Figure 4-18. **Check and pipe inlet spilling over crest. P222-116-53588**

leakage, the gate should be of cast iron. Where the gate frame is mounted in stoplog guides, it should be sealed in the guides. See section 4-11 for lift requirements.

The check and pipe inlet provides a connection to a pipe drop, as shown in figure 2-31, or to a pipe chute. A check inlet with R.I. drop is shown in figure 2-26. The check inlet can also be used with a baffled apron drop, making an excellent combination for a terminal wasteway to a reservoir having a variable tailwater elevation.

4-18. ***Design Considerations.***—A check and pipe inlet, or a check inlet for an R.I. drop should be designed for an overflow capacity (with stoplogs or gates closed) equal to the design capacity of the canal. When used as a wasteway turnout, the overflow crest should be sufficient to accommodate the design discharge, encroaching no more than 50 percent on the normal canal bank freeboard.

For other design considerations and a design example, see subchapters II E and III F.

C. CROSS-DRAINAGE STRUCTURES

4-19. ***General.***—The need for cross-drainage structures results from the flow of drainage or storm runoff water from the high side of the canal to the low side. To protect the canal from such flows, cross-drainage structures are provided at locations best suited for handling them.

While the alinement of a canal usually follows the natural ground contours, in the interest of economy it is often necessary to take shortcuts across natural drainages or through ridges. In crossing natural drainage channels, the canal flows may go under the channel in a siphon, or the channel flows may go under the canal in a culvert. Where natural channels are not available, or where economy dictates, the cross-drainage flows may be carried over the canal in an overchute, or small flows may be taken into the canal through a drain inlet.

Cross-drainage flows are sometimes collected in open drain channels which parallel the canal on the uphill side. These drain channels may carry the water to a natural channel, where it is conducted under the canal in a culvert, or to a collection point where the water can be carried over the canal in an overchute, or into the canal through a drain inlet, or over a siphon crossing.

(a) *Siphon Crossings.*—(See subchapter II C.) Where a small canal crosses a large drainage channel, it is usually more economical to carry the canal water under the channel in an inverted siphon than to carry the drainage water under the canal through a culvert. Siphons provide excellent reliability, as the accuracy of the cross-drainage flow prediction is less critical where siphons are used. However, the use of a siphon is contingent upon availability of head for siphon losses. Other factors which will affect the results of a cost comparison are the width and depth of the drainage channel.

(b) *Cross-drainage Alternatives.*—Where the capacity of the canal is large in comparison to the channel capacity, it is usually more economical to carry the drainage water under or over the canal prism, or into the canal. In addition to the original cost of a structure, the maintenance cost should be included in the cost comparison. Another factor, sometimes difficult to evaluate, is the degree of reliability offered by the various types of cross-drainage structures. Overchutes are better able than culverts to handle flows where sand, gravel, or debris are present. Neither culverts nor overchutes impose an additional head requirement on the canal.

Other factors affecting the best choice of cross-drainage structures are as follows:

(1) *Culverts.*—Where the canal section is primarily in fill as it crosses a drainage channel, a culvert is the logical structure to carry the drainage water under the canal. Small culverts may become plugged with trash, particularly if the drainage area has a cover of brush. Weed screens have been used, but sometimes increase the susceptibility to plugging.

(2) *Overchutes.*—Where the drainage

channel invert, on the uphill side of the canal, is higher than the normal water surface of the canal, an overchute may be the logical choice of structure, carrying the drainage water over the canal. The rectangular concrete overchute is able to pass a considerable amount of debris, whereas the pipe overchute, like the culvert, is subject to plugging.

(3) *Drain inlets.*—If the design flow for a drainage area is so small that the canal can accommodate the additional flow, it may be desirable to take the drainage water into the canal through a drain inlet. If an economic study is made, it should include the cost of additional sediment removal from the canal, and the added cost for greater wasteway capacity, if required.

(c) *Cross-drainage Capacity.*—For the determination of the required design capacity for cross-drainage structures, based upon expected storm runoff, see Design of Small Dams [8]. Generally, for small irrigation systems, cross-drainage structures are sized on the basis of storm runoff for a 25-year flood frequency.

1. Culverts

4-20. ***General.***—Culverts, as discussed in this text, carry storm runoff or drainage water under the canal (see fig. 4-19). Thorough consideration should be given to the culvert alinement, profile, conduit, inlet, and outlet, with special attention given to the hydraulic design.

4-21. ***Alinement.***—A primary rule in locating a culvert is to utilize the natural channel with as little disturbance as possible to the natural runoff pattern. Thus, if a canal crosses a natural channel on a skew, it is usually better to locate the culvert on a skew with the canal, rather than to realine the inlet or outlet channel. If the natural channel changes direction between the inlet and outlet of the culvert, a horizontal bend may be required in the culvert conduit.

Open drains paralleling the canal on the uphill side to intercept runoff water, should flow, if possible, to a natural channel, where conditions are best for locating a culvert.

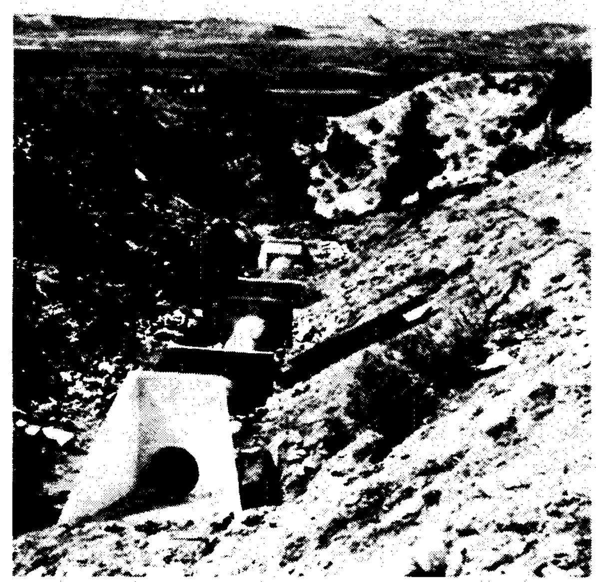


Figure 4-19. **Culvert, constructed prior to construction of canal. P514-417-366**

However, where natural channels are not available, such flows should be diverted across the canal at locations based upon economic culvert size and spacing.

4-22. ***Profile.***—The profile of the culvert is usually determined by the invert profile of the natural channel and the cross section of the canal. The inlet invert should be located near the existing ground surface, or at the bottom grade of the inlet channel, if provided, to prevent upstream degradation. Hydraulic requirements are discussed in section 4-26.

Where the conduit is on a uniform grade, it should be steep enough to prevent sedimentation in the conduit but not steep enough to require an energy dissipator. In practice, it has been found satisfactory to use a minimum slope of 0.005, and a maximum slope slightly steeper than the critical slope, s_c. For determination of critical slope, see subsection 4-26(e).

Where a uniform slope would greatly exceed the critical slope, requiring an energy dissipator, it is usually preferable to include a vertical bend and two invert slopes, s_1 and s_2, as shown in figure 4-28. The upstream slope, s_1, should be much steeper than critical, resulting in free flow at the inlet, and *inlet control*. The downstream slope, s_2, is usually set on the flat slope of 0.005 to facilitate dissipation of excess energy by a hydraulic jump in the pipe, without being flat enough to permit sedimentation in the pipe. To obtain a downstream leg of sufficient length to insure a hydraulic jump in the pipe, or to reduce the velocity in the pipe, it may be necessary to steepen the upstream slope, s_1.

Two slopes, s_1 and s_2, are also required where the culvert inlet is relatively high with respect to the canal invert elevation. The pipe should cross under the canal prism with at least 2 feet of clearance below the invert of an earth section canal, and at least 0.50 foot below the lining of a concrete-lined canal.

To minimize the "daylighting" length of the outlet channel, the culvert outlet should be no lower than necessary to extend beyond the canal bank and provide adequate protection to the canal.

4-23. ***Conduit.***—Culverts may be single or multibarreled as shown in figure 4-20, and may consist of any of the following types of conduit:

(a) Precast reinforced concrete pressure pipe (PCP),
(b) Precast reinforced concrete culvert pipe (RCCP),
(c) Asbestos-cement pressure pipe (AC),
(d) Reinforced plastic mortar pressure pipe (RPM), or
(e) Rectangular concrete box section.

Subsequent references to pipe, pipe velocity, etc., are not intended to exclude the concrete box section. The type of conduit selected for a particular site is dependent upon the project life, the life expectancy of the pipe, the loading conditions to be imposed upon the pipe, the relative cost of each type, and its availability at the site. For a detailed discussion of the various types of pipe, see subchapter VIII B. Generally, the concrete box section, as shown in figure 4-21, is used for flows larger than those considered applicable to this text.

Considering the above factors, precast concrete pipe is the usual choice for culverts under canals, providing excellent strength, durability, watertightness, and flow characteristics. Rubber gasket joints are used to prevent leakage and to provide flexibility.

Determination of the required diameter of pipe, based upon hydraulic considerations, is included in subsection 4-26(c). To avoid

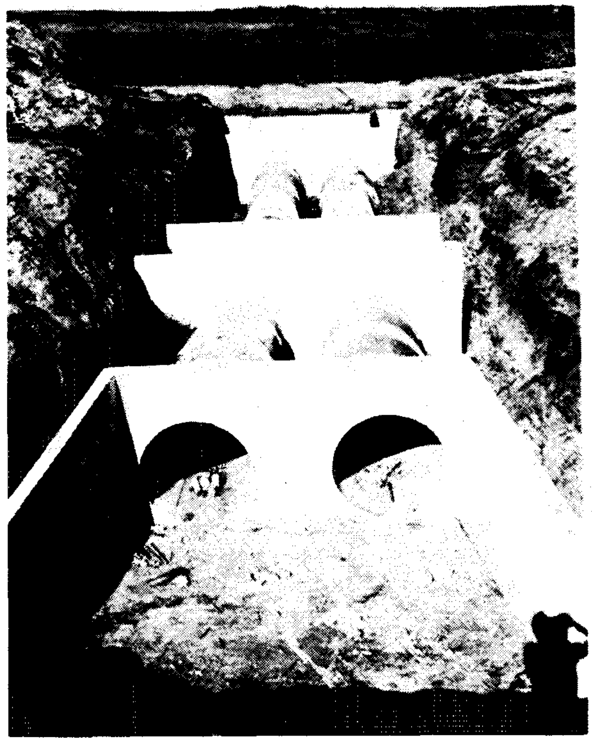


Figure 4-20. **Double-barreled pipe culvert with concrete percolation collars. P-328-701-2021**

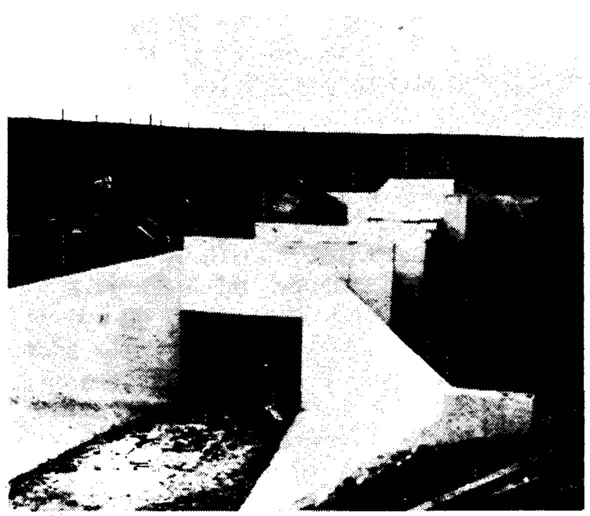


Figure 4-21. **Concrete box culvert with concrete percolation collars. P328-701-3743**

plugging with debris, the minimum diameter usually considered for culverts is 24 inches. However, this should not be considered inflexible, as the nature and quantity of debris in the drainage area, together with the extent of damage that could result from plugging, should be evaluated in establishing a minimum diameter.

4-24. ***Inlet.***—Several types of transitions are used as culvert inlets. The best choice for any particular situation is dependent upon the hydraulics, the topographic character of the site, and the relative elevations of the canal and drainage channel. Suitability of each of the basic types of transitions to use under various conditions is given as follows:

(a) *Type 1.*—The broken-back transition, as shown in figures 4-21 and 7-2, is best suited to a well defined inlet channel, where the channel banks can be shaped to conform to the sides of the transition.

(b) *Type 2.*—The type 2 transition is well suited to use in a wide, poorly defined channel, and combines the economy of simple lines with good flow characteristics (see figs. 4-19 and 7-4).

(c) *Type 3.*—The type 3 transition (see figs. 4-22 and 7-5), like type 2, is suitable to use in a poorly defined channel. By extending the pool beyond the end of the sloping sidewalls, a longer crest results. By lowering the invert, the headwall opening is lowered, permitting a lower water surface elevation for the inlet pool (provided the control is upstream).

(d) *Type 4.*—Except for its sloping floor, and the omission of the headwall cutoff, the type 4 transition, as shown in figure 7-6, is similar to the type 3. The sloping floor permits a lower pipe invert at the inlet headwall with the type 4 transition.

For a more detailed discussion of these transitions, see subchapter VII A.

(e) *Precast Concrete Transitions.*—Precast concrete transitions, as shown in figure 4-23 are usually quite satisfactory, and may be more economical in some areas, particuarly if a large number of transitions of the same size are required.

(f) *Earth Transitions.*—The designer may find that a particular culvert does not require a concrete transition due to the character of the earth or rock material in which it is constructed. An earth transition is permitted if the design flow can be carried with a maximum full pipe velocity of 5 feet per second with no encroachment on the freeboard requirement. In such a case, a concrete headwall may be used to shorten the length of pipe required, or the pipe may be extended completely through the canal bank.

Concrete transitions provide the following benefits:

(1) A greater capacity is provided by good transitioning.

(2) The required length of pipe may be shortened by the length of the concrete transitions.

Figure 4-22. **Precast concrete pipe culvert with type 3 inlet transition. P482-417-824**

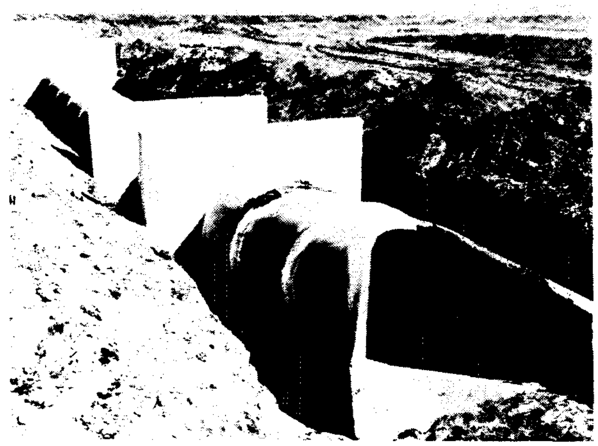


Figure 4-23. **Precast concrete pipe culvert with precast concrete inlet transition. P-328-701-6507**

(3) The potential reduction in erosion reduces the threat to the canal bank.

(4) The cutoff walls used with concrete transitions augment the function of the pipe collars, and reduce the threat of piping by percolation.

To prevent erosion, concrete inlet transitions should be set with the inlet sill or lip at or near the elevation of the channel invert. The sill may be lowered slightly to permit complete drainage of the inlet area.

Freeboard from the inlet water surface to the top of the bank should be at least 2 feet. Freeboard to the top of the concrete headwall varies with the type of transition. Where the normal bank height does not provide the required inlet freeboard, the bank should be raised for such a distance that the top will intersect the natural ground surface at the required elevation.

4-25. ***Outlet.***—The culvert outlet performs the basic function of releasing water to the outlet channel without excessive erosion. Depending upon the amount of excess energy to be dissipated, this function may be performed by a concrete transition, baffled outlet (as shown in fig. 4-24), baffled apron drop (as shown in fig. 4-25), or some other form of energy dissipator (see chapter VI). The concrete transition should be used where possible in preference to most other types of outlets which are more susceptible to problems with weeds, trash, and sediment.

(a) *Concrete Transitions.*—Concrete transitions may be used at culvert outlets provided: the conduit is sized on the basis of a maximum full pipe velocity of 10 feet per second; standard outlet protection is used if the pipe velocity at the outlet is equal to, or less than 15 feet per second; and the next higher class of protection is required if the pipe velocity at the outlet is greater than 15 feet per second.

Two basic types of concrete outlet transitions are used as follows:

(1) *Type 1 concrete transition.*—The type 1 transition as shown in figure 4-26A, is best suited to a well defined outlet channel, where the banks can be shaped to conform to the sides of the transition. Thus, it is generally used where it is necessary to construct a channel with its invert considerably below the original ground surface.

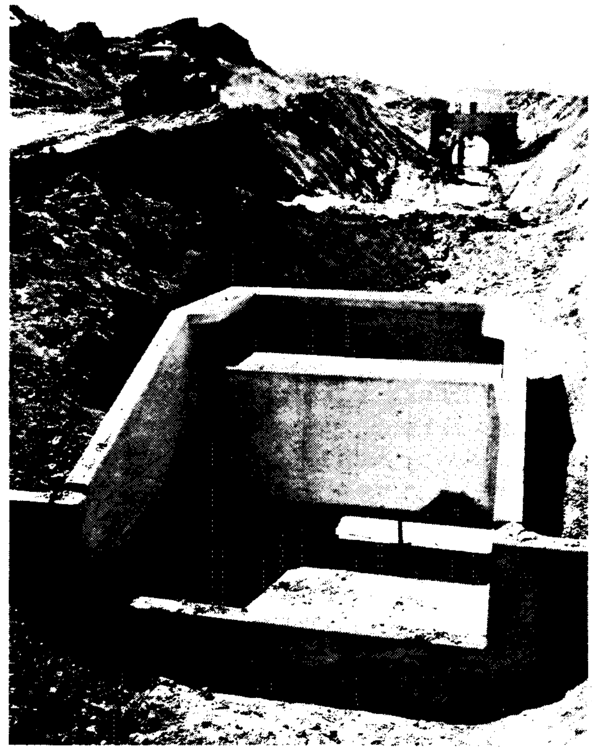


Figure 4-24. **Baffled outlet at end of a culvert. P328-701-8219**

(2) *Type 2 concrete transition.*—The type 2 transition as shown in figure 4-19 is well suited to use in a poorly defined channel. For this reason, it is the best choice where the outlet elevation is not appreciably lower than the original ground surface.

(b) *Outlet Energy Dissipators.*—The need for energy dissipation structures at culvert outlets should be avoided if possible, by dissipating excess energy in the pipe. In addition to a higher construction cost for energy dissipators, their maintenance cost is also greater. The pipe velocity may be reduced by diminishing the slope, s_2 but to not less than a slope of 0.005.

Excess energy at culvert outlets may be dissipated by the following structures:

(1) *Baffled outlets.*—Baffled outlets, as shown in figure 4-24 perform well in dissipating excess energy, provided clogging by weeds or other debris can be avoided.

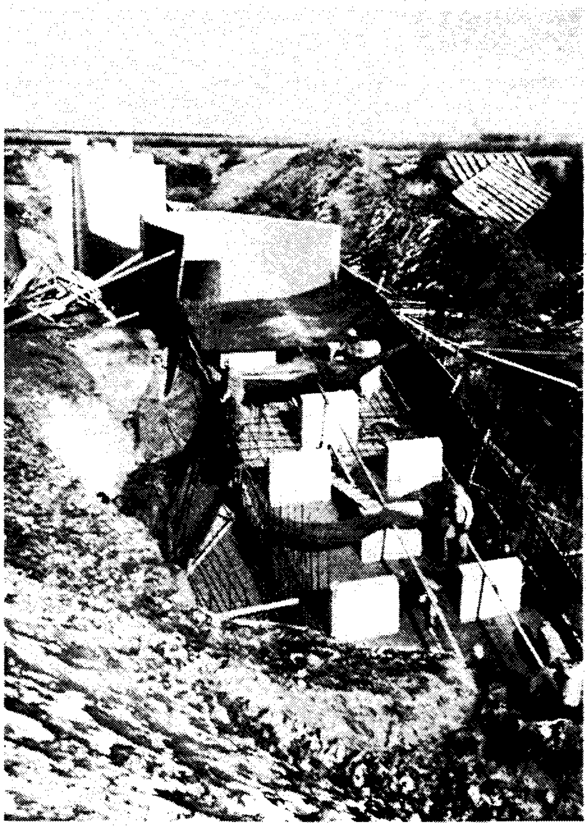


Figure 4-25. Concrete box culvert with baffled apron drop at outlet. P-707-729-1947

When using a baffled outlet, the culvert pipe should be sized on the basis of a full pipe velocity of 12 feet per second. The theoretical velocity, $V_t = \sqrt{2gh}$, should not exceed 50 feet per second. For a detailed discussion of baffled outlet design, see subchapter VI C.

The potential damage from plugging of the culvert cannot be overemphasized, as failure of the canal banks and the culvert could occur.

(2) *Other types of energy dissipators.*—Drops and chutes with stilling pools have been used at culvert outlets, as have baffled apron drops (see fig. 4-25). The stilling pool requires sufficient tailwater to produce a hydraulic jump whereas the effectiveness of the baffled apron drop is independent of tailwater. See chapter VI for design requirements of these structures.

4-26. *Hydraulics.*—(a) *Design Capacity.*—For determination of the required design capacity, see comments in subsection 4-19(c).

(b) *Pipe Velocity.*—The culvert should be designed for a maximum full pipe velocity of 10 feet per second if a concrete transition is used at the outlet, and for a maximum full pipe velocity of 12 feet per second if an energy dissipator is used. In the rare cases that concrete inlets and outlets are not considered necessary, the pipe should be designed for a maximum full pipe velocity of 5 feet per second.

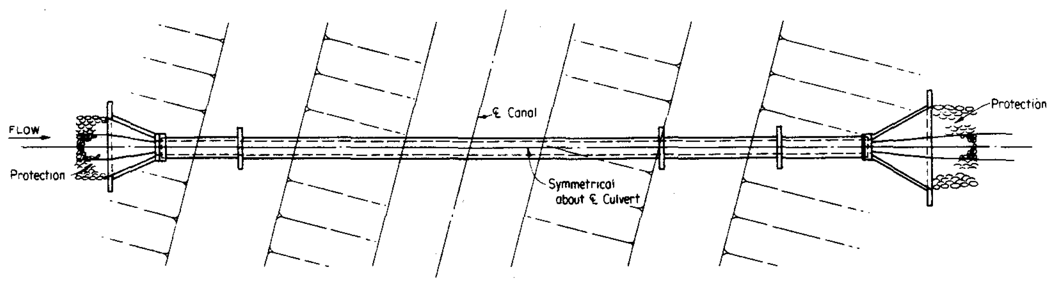
The actual full pipe velocity will usually be less than the maximum design velocity, due to the availability of pipe in 3-inch increments of diameter, only. However, the pipe will not necessarily flow full, and the actual velocity may be considerably faster than the full pipe velocity. If the exit velocity in the pipe exceeds 20 feet per second, an energy dissipator should be provided.

(c) *Pipe Diameter.*—The diameter of the pipe is determined from the basic equation, $Q = AV$ when related to a pipe flowing full. This reduces to: $D = 1.13\sqrt{\frac{Q}{V}}$. The minimum diameter usually permitted for culvert pipe is 24 inches. Precast concrete pipe is widely supplied in increments of 3 inches of diameter.

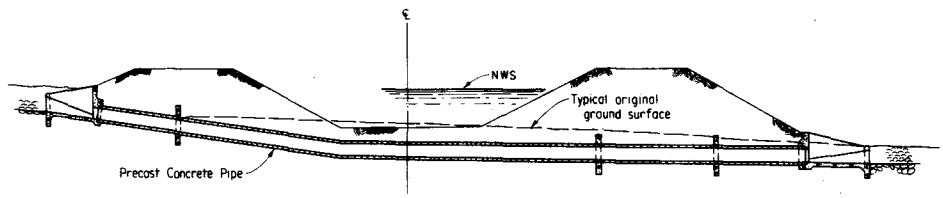
(d) *Hydraulic Control.*—The upstream water surface will be controlled by the head that is required to satisfy: inlet conditions or outlet conditions (such as a high tailwater, or pipe losses). To insure the validity of other hydraulic computations, it must be determined whether the hydraulic control is located at the inlet or outlet.

(1) *Inlet control.*—If the upstream water surface is not influenced by flow conditions downstream from the inlet, it is said to have inlet control. Inlet control results from the following conditions:

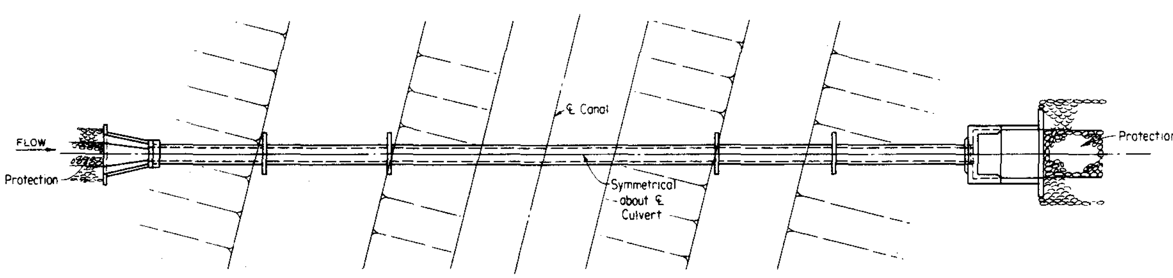
A downstream water surface that is low enough with respect to the inlet, that it does not influence the upstream water surface, combined with an upstream pipe slope that is steeper than the critical slope.



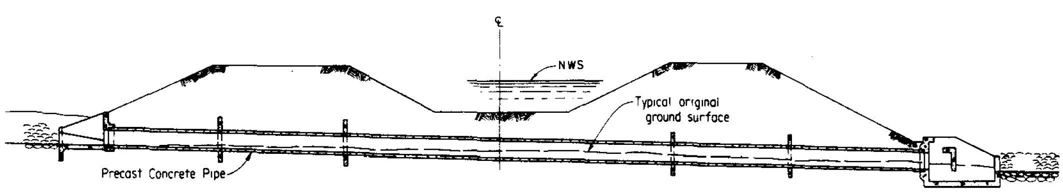
A. TYPICAL PLAN WITH TYPE I OUTLET



PROFILE



B. TYPICAL PLAN WITH BAFFLED OUTLET



PROFILE

Figure 4-26. Plan and profile of typical culverts. 103-D-1303

(2) *Outlet control.*—If the upstream water surface is influenced by downstream conditions, it is said to have outlet control. Outlet control results from the following conditions:

A downstream water surface that is high enough with respect to the inlet, that it influences the upstream water surface; or pipe losses that cause pipe flow at a depth greater than the critical depth.

(e) *Determination of Inlet or Outlet Control.*—

(1) *Examination of profile.*—The type of control can sometimes be determined by examination of the culvert profile. For example, *inlet control* is suggested if the downstream channel is wide, poorly defined, and considerably lower than the inlet. *Outlet control* is suggested if a high tailwater (relative to the inlet invert) is assured by a well defined channel with a flat slope. Such conclusions should be supported by performing minimal hydraulic computations. The critical slope for the design discharge, pipe diameter, and roughness coefficient can be determined from tables 21, 22, 58, and 60 of bibliography reference [4], using the equation:

$$\frac{Qn}{D^{8/3} s^{1/2}} = k$$

or

$$s = \left(\frac{Qn}{D^{8/3} k}\right)^2$$

where k is the tabulated value in table 21, n is the coefficient of roughness of the pipe, D is the pipe diameter in feet, and Q is the design discharge in cubic feet per second.

Among other requirements, inlet control cannot be assured unless the upstream pipe slope is greater than the critical slope.

(2) *Bernoulli Theorem.*—Where the location of the hydraulic control cannot be definitely established by examination of the profile, the Bernoulli theorem [2] should be used. By the Bernoulli process, it should be determined if an energy balance exists throughout the length of the culvert as shown in figure 4-29. That is, the specific energy, E_{s_2} at one point, plus the drop in invert elevation to a downstream point is equal to the specific energy, E_{s_3} (at the downstream point), plus intermediate hydraulic losses,

or $$E_{s_2} + \Delta El. = E_{s_3} + \text{losses}$$

Beginning with the water surface and energy established for the outlet, a balance of energy, if obtainable, should be verified between pairs of points proceeding upstream.

If a balance of energy results between each pair of points, outlet control is assured. If a balance cannot be achieved between any two points, this indicates that a hydraulic jump occurs between the two points, resulting in additional head loss. This inability to achieve a balance of energy confirms *inlet control* of the upstream water surface, except that such an imbalance occurring between the outlet channel and a point just inside the pipe indicates *outlet control* if pipe friction and bend losses produce subcritical flow in the pipe, but a low tailwater in the channel allows the flow to pass through critical depth as it emerges from the pipe.

(f) *Inlet Control Hydraulics.*—Where inlet control exists, the head required at the culvert inlet is computed from the orifice equation [2], $Q = CA\sqrt{2gh}$, where C is the coefficient of discharge, A is the area of the pipe, and h is the head of water. Under inlet control, the head, h, is measured from the upstream water surface to the centerline of the pipe at the headwall. To determine the required head, the orifice equation may be written

$$h = \frac{Q^2}{2g\,C^2 A^2}$$

With C = 0.6, and substituting V^2 for $\frac{Q^2}{A^2}$

$$h = 0.0433\ V^2$$

where V is the velocity with the pipe flowing full (see subchapter VII A).

(g) *Outlet Control Hydraulics.*—Where outlet control exists, the head required to produce the design discharge is a function of the losses in the system as follows:

(1) *Inlet losses.*—The inlet loss,

$$h_i = K_i \Delta h_v$$

(2) *Pipe losses.*—The pipe losses consist of friction and bend losses. The friction loss should be computed from the Manning equation [2],

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

where s is the friction slope of the pipe, and n is the roughness coefficient, equal to 0.013 for precast concrete pipe. Pipe bend losses may be determined as shown in subchapter VIII A.

(3) *Outlet losses.*—The outlet loss,

$$h_o = K_o \Delta h_v.$$

See subchapter VII for explanation of inlet and outlet losses.

4-27. ***Pipe Collars.***—Percolation of water from the canal to the drainage channel could result in piping, or the gradual removal of material along the percolation path, allowing a gradually increasing flow. Failure of the canal bank could result unless proper precautions are taken in the design and construction of the culvert.

Pipe collars, as shown in figure 4-23, are effective in reducing the percolation along culverts. Unless the percolation study indicates otherwise, it is customary to locate collars on the pipe as follows:

One collar below the centerline of the uphill canal bank; two collars under the downhill bank, with one under the inside edge, and one located 2 feet downstream from the outside edge (see fig. 4-28).

A "short path" between collars will occur if the resistance to percolation through the soil is less than the resistance to percolation along the concrete surface of the pipe and collars. To evaluate these resistances, Lane's weighted-creep method [8] uses the following weighted resistance values:

K_1 = 1.0 – For flow along a vertical concrete surface, such as that of the collar.

K_2 = 0.33 – For flow along a horizontal or nearly horizontal concrete surface, such as that of the culvert pipe (primarily along the under side).

K_3 = 2.0 – For flow through the soil.

By this method, where X is the distance between collars, and Y is the vertical projection of the collar from the pipe (see fig. 4-27), it can be shown that a short path occurs if the distance, X, is less than the following:

$$K_3 X = K_2 X + 2K_1\ Y,$$

or

$$2X = 0.33\ X + 2\ (1 \times Y)$$

Solving for X in terms of Y,

$$2X - 0.33X = 2Y$$

or

$$1.67\ X = 2Y$$

and

$$X\ (\text{min.}) = 1.2Y$$

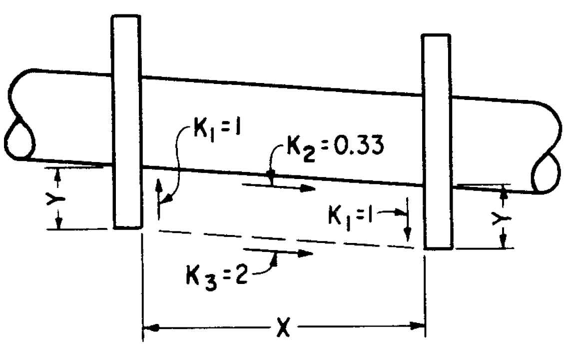


Figure 4-27. **Weighted-creep method. 103-D-1304**

To determine the adequacy of the customary three collars, in providing sufficient path to prevent piping, use Lane's weighted-creep method as discussed in subchapter VIII C. To assure the stability of the culvert and adjacent earth material, a high quality of construction must be achieved.

4-28. ***Protection.***—Protection for the culvert should meet the requirements of figure 7-8, chapter VII. If the outlet velocity in the pipe exceeds 15 feet per second, the outlet protection should be based upon the next higher discharge class.

4-29. ***Design Example.***—Assume that a precast concrete pipe culvert is needed to convey storm runoff water under a canal at its intersection with a natural channel.

(a) *Assumptions.*–

(1) Properties of the canal section are as follows:

$b = 8$ ft., $h_b = 6$ ft., ss = 1-1/2:1, $d_n = 4$ ft., $W_1 = 6$ ft., and $W_2 = 12$ ft. to accommodate an operating road.

(2) The outside bank heights, measured at the toe of the outside bank slopes, are:

$h_{b_1} = 4$ ft., and $h_{b_2} = 9$ ft., as shown in figure 4-28.

(3) The cross-drainage channel is wide, shallow, and poorly defined, both at the inlet and outlet. It is estimated that the outlet channel will support a depth of 1 foot and a velocity of 1 foot per second for the design flow.

(4) It has been determined that the 25-year flood could yield a discharge of 45 cfs.

(b) *Solution.*–

(1) *Pipe velocity.*–To use a standard outlet transition, the pipe diameter is determined on the basis of a maximum full pipe velocity of 10 feet per second.

(2) *Pipe diameter and class.*–The required pipe diameter (see section 4-26(c)),

$$D = 1.13\sqrt{\frac{Q}{V}}$$

$$= 1.13\sqrt{\frac{45}{10}}$$

$$= 2.38 \text{ ft. (min.)}$$

Use a 30-inch diameter, which provides an area of 4.91 square feet, resulting in a full pipe velocity of 9.17 feet per second.

For pipe class, see subchapter VIII B.

(3) *Hydraulic control.*–From a preliminary profile, using, tentatively, a type 3 inlet transition and a type 2 outlet transition in the poorly defined channel (see section 4-24), it is seen that the culvert can clear the canal invert by the required 2 feet, and emerge beyond the lower canal bank with an outlet channel of minimal depth, requiring only minor excavation to daylight.

As the depth of cut in the poorly defined outlet channel is minimal, the tailwater will spread as permitted by the natural topography, and will probably not influence the upstream water surface. Further, to provide 2 feet of clearance beneath the canal invert, the pipe must descend on a slope which is presumed to be steeper than the critical slope, resulting in free flow at the pipe inlet, assuring inlet control. This will be verified.

(4) *Inlet type.*–As the drainage channel is not well defined, a type 1 transition is not suitable. With the upper canal bank only 4 feet higher than the channel invert, a type 3 or 4 inlet transition would probably be better than a type 2, as they require less water depth at the inlet. The type 3 inlet will be used, provided the head is adequate to discharge the design flow without encroaching on the required bank freeboard.

(5) *Pipe friction slope (for pipe flowing full).*–Where inlet control is certain, determination of the friction slope is not necessary. However, a better understanding of the culvert hydraulics is gained by an

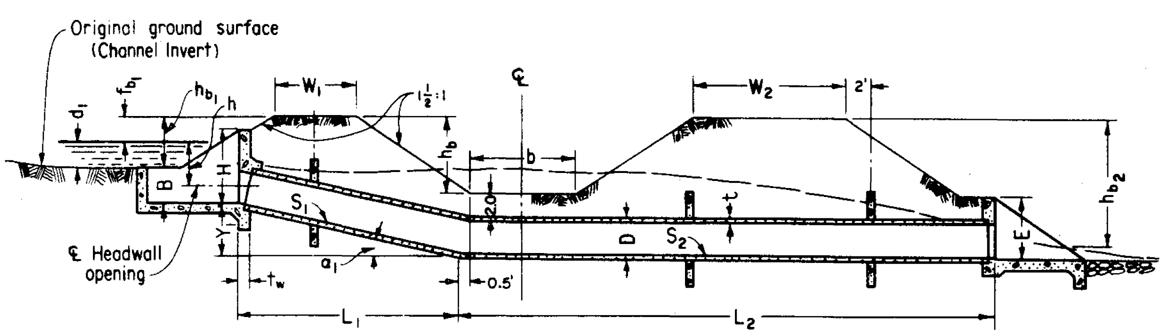


Figure 4-28, **Culvert–typical profile. 103-D-1305** awareness of the relative slopes. From the Manning equation [2], the pipe velocity is:

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

Using a coefficient of roughness, n = 0.013, and a hydraulic radius,

$$R = \frac{D}{4} = \frac{2.5}{4} = 0.625$$
$$R^{2/3} = 0.731$$

the friction slope,

$$s_f = \left[\frac{Vn}{1.486\ R^{2/3}}\right]^2$$
$$= \left[\frac{9.17 \times 0.013}{1.486 \times 0.731}\right]^2$$
$$= 0.0120$$

This friction slope can also be determined from table 8-1, sheet 2 of 9, chapter VIII:

$$s_f = \frac{H}{1000} = \frac{12.04}{1000} = 0.0120$$

(6) *Critical slope.*—Determine the critical slope (for comparison with the invert slope, s_1) for Q = 45 cfs and D = 2.5 feet, using tables 21, 22, 58, and 60 [4].

Find $D^{5/2}$ = 9.882 from table 58

and $D^{8/3}$ = 11.51 from table 60

Then $\frac{Q}{D^{5/2}} = \frac{45}{9.882} = 4.55$

and $\frac{d_c}{D} = 0.89$ from table 22

Also $\frac{Qn}{D^{8/3}\ s^{1/2}} = 0.491$ from table 21

Then $s_c^{1/2} = \frac{Qn}{0.491\ D^{8/3}}$

$$s_c^{1/2} = \frac{45 \times 0.013}{0.491 \times 11.51}$$
$$s_c^{1/2} = 0.1034$$

and $s_c = (0.1034)^2 = 0.0107$

(7) *Invert slope, s_1.*—From figure 4-28, determine the required pipe invert slope to provide a minimum of 2 feet of clearance between the canal invert and the concrete pipe. Assume class B-25 pipe (see subchapter II C) will be used, with a wall thickness, t = 3-1/2 inches.

$$y_1 = D + t + 2 + h_b - h_{b_1} - B$$
$$= 2.5 + 0.3 + 2 + 6 - 4 - 3 = 3.8 \text{ ft.}$$

$$L_1 = 1.5h_b + w_1 + 1.5(h_{b_1} + B - H) + t_w - 0.5$$
$$= 1.5 \times 6 + 6 + 1.5\ (4 + 3 - 6) + 0.5 - 0.5$$
$$= 9 + 6 + 1.5 = 16.5 \text{ ft.}$$

$$s_1 = y_1/L_1 = 3.8/16.5 = 0.23$$
(= tan α_1 in figure 4-28)

Therefore $s_1 > s_c$, permitting free flow at the pipe inlet, with inlet control; providing the downstream water surface does not control (see subsection 4-29(c)).

(8) *Invert slope, s_2.*—Since s_1 is too steep to be extended to the outlet, without an outlet channel of excessive depth and length, a bend and a second slope, s_2 are required. Let s_2 = 0.005, as indicated in section 4-22.

(9) *Length, L_2.*—The length, L_2 must be sufficient to locate the outlet headwall beyond the canal bank at its intersection with the top elevation of the headwall, which should be consistent with an invert elevation providing an outlet of minimal daylighting length and depth (see fig. 4-28). Determine L_2 by scaling the preliminary profile (see step 3). L_2 = 41 feet.

(10) *Outlet type.*—As the outlet channel is poorly defined, the type 1 transition would not be suitable. Therefore, the type 2 transition is selected. From figure 7-4, chapter VII, with D = 30 inches, dimensions L and E are found to be 6 feet 9 inches and 4 feet 6 inches, respectively.

(11) *Inlet hydraulics.*—Proceeding with the assumption that inlet control exists, the ability to discharge the design flow is a function of the inlet hydraulics, without regard to the culvert pipe and outlet conditions. The type 3 inlet transition dimensions provide hydraulic control at the headwall opening. Therefore the head

required to discharge 45 cfs can be determined from the orifice equation [2],

$$Q = CA\sqrt{2gh}$$

which, with the coefficient, C = 0.6, can be written (see subchapter VII A):

$$h = 0.0433\ V^2$$

or $$h = 0.0433\,(9.17)^2 = 3.64 \text{ ft.}$$

(12) *Inlet freeboard.*—Referring to figure 7-5, the type 3 transition would require ponding the channel water surface to a depth,

$$d_1 = h - \left[B - \frac{D'}{2}\right]$$

where $D' = \dfrac{D}{\cos \alpha_1}$ (see fig. 4-28)

$$= \frac{2.5}{0.975} = 2.56 \text{ ft.}$$

Then, $d_1 = 3.64 - [3 - 1.28] = 1.9$ ft.

The bank freeboard, f_b, is equal to the bank height above the channel invert minus the required ponding depth, or

$$f_{b_1} = h_{b_1} - d_1$$
$$f_{b_1} = 4 - 1.9$$
$$= 2.1 \text{ ft., which exceeds}$$

the required freeboard of 2 feet minimum.

(13) *Collars.*—Unless percolation studies show that more concrete collars are required, one collar should be located under the upper bank of the canal, and two under the lower bank, as shown in figure 4-28. For a detailed treatment of percolation, see subchapter VIII C.

(14) *Pipe bend.*—For pipe bend requirements, see chapter VIII.

(15) *Protection.*—From figure 7-8, chapter VII, it is determined that erosion protection is not required at the culvert inlet. Type 2 protection is indicated for the outlet. However, if the pipe velocity at the outlet should exceed 15 feet per second, as determined by the following Bernoulli solution of hydraulics, the outlet protection should be increased to that required for the next greater discharge, resulting in type 3 in lieu of type 2 protection.

(c) *Verification of Hydraulics by the Bernoulli Method (see fig. 4-29).*—

(1) *Energy balance at outlet.*—The specific energy in the outlet channel is determined as follows:

$$E_{S_4} = d_4 + h_{v_4}$$
$$= d_4 + \frac{V_4^2}{2g}$$
$$= 1.0 + \frac{(1)^2}{2 \times 32.2} = 1.02 \text{ ft.}$$

The specific energy in the outlet end of the pipe can not be less than E_{s_c}, as minimum energy accompanies critical flow. From step (b)(6), above,

$$\frac{d_c}{D} = 0.89$$

Then

$$d_c = 0.89\,(2.5) = 2.22 \text{ ft.}$$

From table 21 [4],

$$\frac{A_c}{D^2} = 0.7384$$

and

$$A_c = 0.7384\,(2.5)^2 = 4.62 \text{ sq. ft.}$$

giving

$$V_c = \frac{Q}{A_c} = \frac{45}{4.62} = 9.74 \text{ f.p.s.}$$

and

$$h_{v_c} = \frac{V^2}{2g} = 1.48 \text{ ft.}$$

Finally,

$$E_{s_c} = d_c + h_{v_c}$$

$$= 2.22 + 1.48 = 3.70 \text{ ft.}$$

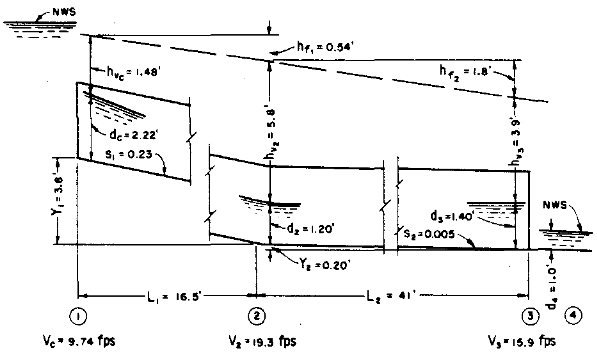


Figure 4-29. **Culvert hydraulics. 103-D-1306**

As $E_{s_c} > E_{s_4}$, and E_{s_c} represents minimum energy, a balance of energy is unobtainable at the outlet. Therefore, the tailwater cannot provide outlet control.

(2) *Specific energy at point 1 (just inside the pipe at the inlet).*—As s_1 is steeper than s_c, and the tailwater does not control, supercritical flow will occur from point 1 to point 2, and critical flow (flow at minimum energy) will occur at point 1. From step 1 above, the specific energy for critical flow,

$$E_{s_c} = 3.70 \text{ ft.}$$

(3) *Specific energy at point 2.*—Find d_2 and h_{v_2} yielding a specific energy, E_{s_2} such that

$$E_{s_2} = E_{s_c} + y_1 - h_{f_1}$$

where h_{f_1} is the pipe friction loss from point 1 to point 2.

Try

$$d_2 = 1.20 \text{ ft.}; \frac{d_2}{D} = 0.48$$

Then, from table 21 [4],

$$\frac{A_2}{D^2} = 0.3727$$

$$A_2 = 0.3727\,(2.5)^2 = 2.33 \text{ sq. ft.}$$

$$V_2 = \frac{Q}{A_2} = \frac{45}{2.33} = 19.3 \text{ f.p.s.}$$

$$h_{v_2} = 5.8 \text{ ft.}$$

$$E_{s_2} = d_2 + h_{v_2} = 1.2 + 5.8 = 7.0 \text{ ft.}$$

$$\frac{Qn}{D^{8/3}\left(s_{f_2}\right)^{1/2}} = 0.216$$

$$\left(s_{f_2}\right)^{1/2} = \frac{Qn/D^{8/3}}{0.216}$$

$$\left(s_{f_2}\right)^{1/2} = \frac{45(0.013)/11.51}{0.216} = \frac{0.0508}{0.216}$$
$$= 0.235$$

$$s_{f_2} = (0.235)^2 = 0.0554$$

Friction loss in the length, L_1, is

$$h_{f_1} = \left[\frac{s_c + s_{f_2}}{2}\right] L_1$$

$$= \left[\frac{.0107 + .0554}{2}\right] 16.5 = 0.54 \text{ ft.}$$

Finally, the assumption that d_2 = 1.20 feet is correct if

$$E_{s_2} = E_{s_c} + y_1 - h_{f_1}$$

$$7.0 = 3.7 + 3.8 - 0.5$$
$$7.0 = 7.5 - 0.5$$
$$7.0 = 7.0 \text{ (check)}$$

Therefore, E_{s_2} = 7.0 ft.

(4) *Specific energy and velocity at point 3 (just inside the pipe).*—Find d_3 and h_{v_3} yielding a specific energy, E_{s_3} such that

$$E_{s_3} = E_{s_2} + y_2 - h_{f_2}$$

where

$$y_2 = s_2\ L_2$$

$$y_2 = 0.005 \times 41 = 0.2 \text{ ft.}$$

Try $d_3 = 1.40$ ft.; $d_3/D = 0.56$.

Then from table 21 [4],

$$\frac{A_3}{D^2} = 0.4526;$$

$$A_3 = 0.4526\,(2.5)^2 = 2.83 \text{ sq. ft.}$$

$$V_3 = \frac{Q}{A_3} = \frac{45}{2.83} = 15.9 \text{ f.p.s.};$$

$$h_{v_3} = 3.94 \text{ ft.}$$

$$E_{s_3} = d_3 + h_{v_3} = 1.4 + 3.9 = 5.3 \text{ ft.}$$

$$\frac{Qn}{D^{8/3} s^{1/2}} = 0.279$$

$$\left(s_{f_3}\right)^{1/2} = \frac{Qn/D^{8/3}}{0.279}$$

$$= \frac{45 \times 0.013/11.51}{0.279} = 0.182$$

$$s_{f_3} = (0.182)^2 = 0.033$$

Friction loss,

$$h_{f_2} = \left[\frac{s_{f_2} + s_{f_3}}{2}\right] L_2$$

$$= \left[\frac{0.055 + 0.033}{2}\right] 41 = 1.8 \text{ ft.}$$

Finally, the assumption that $d_3 = 1.40$ feet is correct if

$$E_{s_3} = E_{s_2} + y_2 - h_{f_2}$$

$$5.3 = 7.0 + 0.2 - 1.8$$

$$5.3 \neq 5.4; \text{ close enough for design}$$

Therefore,

$$E_{s_3} = 5.3 \text{ ft., and } V_3 = 15.9 \text{ f.p.s.}$$

(5) *Outlet protection.*—Since the velocity in the outlet end of the pipe is 15.9 feet per second, exceeding the velocity of 15 feet per second permitted with normal protection, type 3 protection should be used in lieu of type 2, as indicated in step (b)15, above.

2. *Overchutes*

4-30. *General.*—Overchutes are structures used to carry storm runoff or drainage water over a canal. They may consist of a rectangular concrete flume section, supported on piers as shown in figure 4-30, or a closed conduit such as the steel pipe shown in figure 4-31. The concrete flume section is used primarily for large cross-drainage flows, or in areas where a

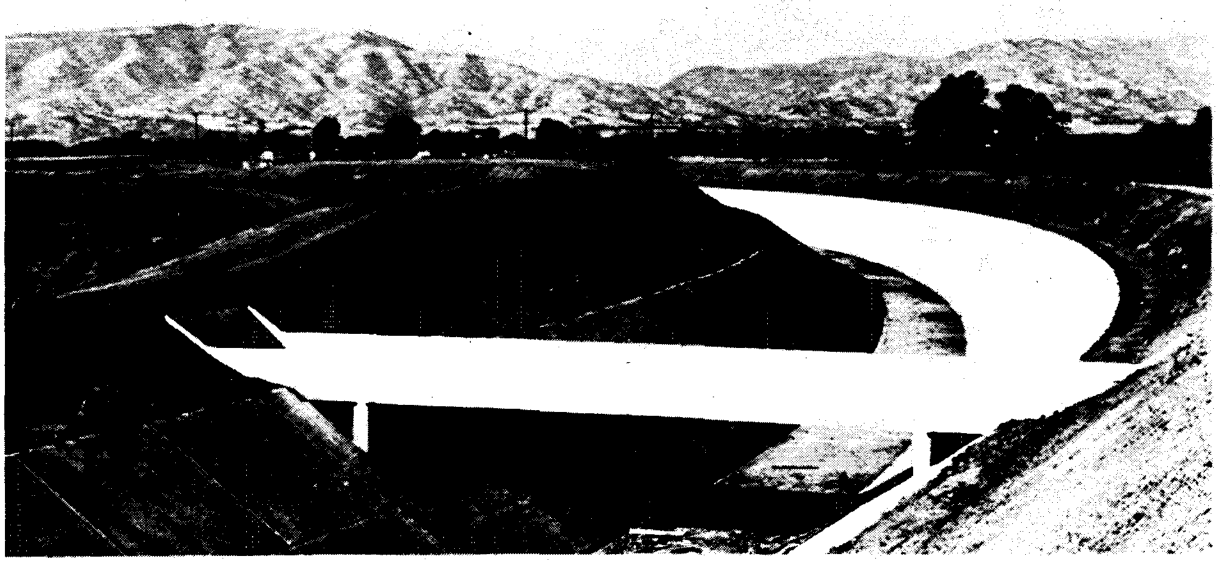


Figure 4-30. **Rectangular concrete overchute. SO-2217-R2.**

Figure 4-31. **Steel pipe overchute on the Colorado-Big Thompson Project. P-245-713-300**

pipe would be susceptible to plugging with trash.

The inlet can be any of the standard types of concrete transitions described in chapter VII. The outlet may be a standard transition, but sometimes consists of an energy dissipator, such as a stilling pool, a baffled apron drop, or a baffled outlet. The outlet may also include a rectangular concrete box section through the downhill canal bank, to accommodate traffic on the operating road. A similar provision may be made for the uphill bank if required, as shown in figure 4-32.

4-31. *Alinement.*—The alinement of overchutes usually follows natural drainage channels. While an alinement normal to the canal is shorter and more economical, a skewed alinement is sometimes used, as the natural channel should be disturbed as little as possible. An overchute may also be located at the end of a drain which parallels the canal, to provide a crossing over the canal. If a natural channel is not available at such a location, a downstream channel should be constructed.

4-32. *Profile.*—To permit complete drainage of the inlet channel and the structure, overchutes should slope enough to compensate for a slight amount of unequal settlement of the piers. Overchutes are best suited to use where the canal is in thorough cut, or where the ground surface on the uphill side is well above the canal water surface. A minimum clearance of 1 foot should be maintained between the canal water surface and the overchute section, which should also clear the top of the concrete lining in a lined canal section. If the uphill ground surface is not sufficiently above the canal water surface to provide the required clearance, a culvert should be used under the canal in lieu of the overchute.

A flat slope of the cross-drainage channel profile is usually accompanied by a flat slope and slow velocity in the overchute, permitting the use of a standard outlet transition.

A steep slope of the cross-drainage channel profile may be accompanied by a steep slope and fast velocity in the overchute. If the overchute has an exit velocity exceeding 20 feet per second, an energy dissipator is required at the outlet to dissipate the excess head. Where weeds or trash are not prevalent, a baffled outlet may be used in conjunction with a pipe overchute.

A concrete overchute, having an energy dissipator such as a stilling pool, may be set on a supercritical slope to minimize the required cross section. After crossing the canal water prism, the overchute often drops on a 2 to 1 slope to the pool, or vertically to a pool as shown in figure 4-35.

Where a baffled apron drop is used with a concrete overchute, the slope should be less than critical (see chapter VI).

4-33. *Capacity.*—For determination of the required design capacity, refer to subsection 4-19(c).

4-34. *Conduit.*—An overchute should have a rectangular concrete section, or a circular pipe section, based upon a cost comparison for the particular span and drainage capacity required, or upon inlet conditions. Generally, the rectangular concrete section is used for larger capacities, and the pipe section for smaller capacities.

Concrete piers are usually required, and should preferably be located just outside the canal water prism to allow unobstructed flow in the canal.

(a) *Rectangular Conduit.*—The conduit for the rectangular concrete overchute will be referred to, hereinafter, as a flume or chute. It may span from one bank of the canal to the other, but it is often jointed over the piers to accommodate differential settlement of the

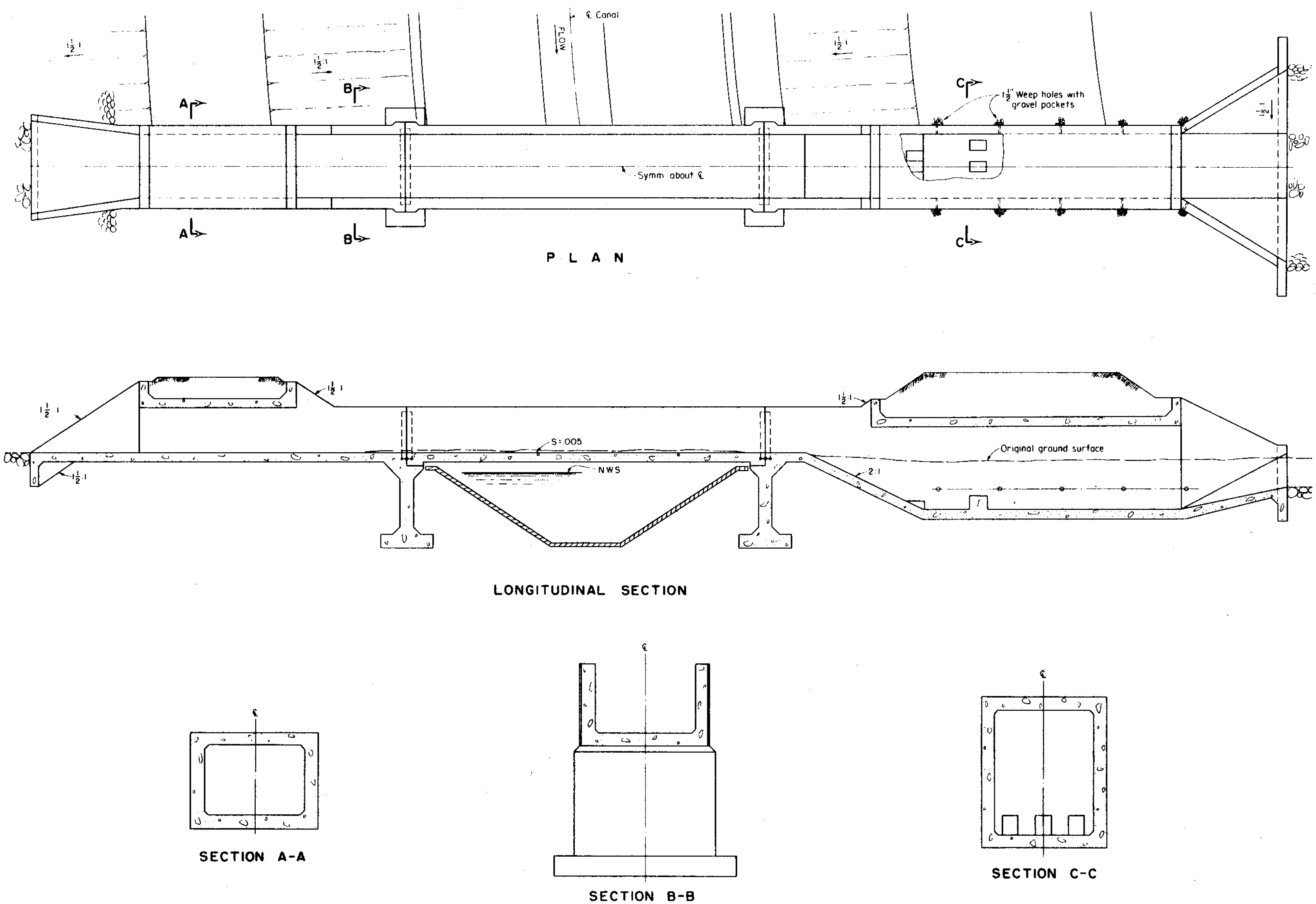


Figure 4-32. **Overchute—typical plan and sections. 103-D-1307**

piers without resulting in undue stress in the overchute. The size of the flume section can sometimes be minimized by setting the flume invert on a supercritical slope (see sec. 4-38).

(b) *Pipe Conduit.*—Pipe overchutes usually consist of steel pipe supported on two piers, as shown in figure 4-33. The steel pipe is usually terminated within the canal prism, and extension of the pipe through the canal banks is made with precast concrete pipe, coupled to the steel pipe.

The required gage of the steel pipe for beam strength depends on the span and pipe diameter. As shown in table 61 [10], a pipe of a given gage and diameter can span a much greater distance if reinforced with stiffeners at the supports.

4-35. ***Inlet Types.***—With slight modification, the standard types of concrete transitions, as discussed in chapter VII, may be used for transitioning to a rectangular section. For recommended usage of the various types, see section 4-24.

The concrete transition invert should be set at or slightly below the elevation of the channel invert, to permit complete drainage of the upstream pool without excessive upstream erosion. The canal bank height should be sufficient to provide a freeboard of 2 feet above the inlet water surface. An inlet dike may be required where the normal bank height is insufficient.

4-36. ***Outlet Types.***—The overchute outlet, like the culvert outlet, should release water to the outlet channel without excessive erosion. The energy to be dissipated in a rectangular concrete overchute is often sufficient to require an energy dissipator such as a stilling pool or a baffled apron drop. As pipe overchutes are usually designed for a smaller capacity, a concrete outlet transition is often satisfactory. However, an energy dissipator such as a baffled outlet may be required if the drop in elevation is considerable. The concrete transition should be used where possible in preference to other types of outlets which are susceptible to problems with weeds, trash, and sediment. The standard outlet transitions may be used for exit velocities of 20 feet per second or less, provided outlet protection conforms to the requirements discussed in section 4-25.

Overchute outlets should preferably be free draining, to eliminate ponding of water. However, if this is impracticable, ponding is sometimes permitted.

(a) *Concrete Transitions.*—The comments concerning concrete transitions as culvert outlets (see subsec. 42-5(a)) apply equally to their use as overchute outlets.

Generally, a concrete outlet transition can be used with pipe overchutes if the pipe is sized on the basis of a full-pipe velocity of 10 feet per second or less and the slope of the downstream leg of the pipe is not greater than the critical slope, s_c. For proper use of the type 1 and type 2 transitions, see section 4-24.

(b) *Outlet Energy Dissipators.*—Rectangular concrete overchutes often require an energy dissipator such as a stilling pool or baffled apron drop. Pipe overchutes require an energy dissipator, such as a baffled outlet, only if the exit velocity from the pipe exceeds 20 feet per second.

The various types of energy dissipators are used as follows:

(1) *Stilling pool.*—Stilling pools are often used to dissipate the excess energy at the outlet of concrete overchutes. They are dependent upon a sufficient tailwater depth in the outlet channel to insure a hydraulic jump. See chapter VI for design of the standard stilling pool.

Stilling pools usually follow a chute or drop on a 2 to 1 slope, but may follow a vertical drop, as shown in figure 4-35. This arrangement is particularly useful if an operating road must cross over the stilling pool structure.

(2) *Baffled apron drops.*—Baffled apron drops may be used for concrete overchute outlets without dependence upon tailwater. See chapter VI for design of baffled aprons.

(3) *Baffled outlets.*—Where weeds and trash are not prevalent, and where excess energy cannot be satisfactorily dissipated in the pipe, baffled outlets may be considered for pipe overchutes. They have been adapted to use at the end of an open chute section, as well as with pipe. See chapter VI for design of the standard baffled outlet.

4-37. ***Pipe Overchute Hydraulics.***—(a) *Inlet Transition.*—The elevation of the overchute

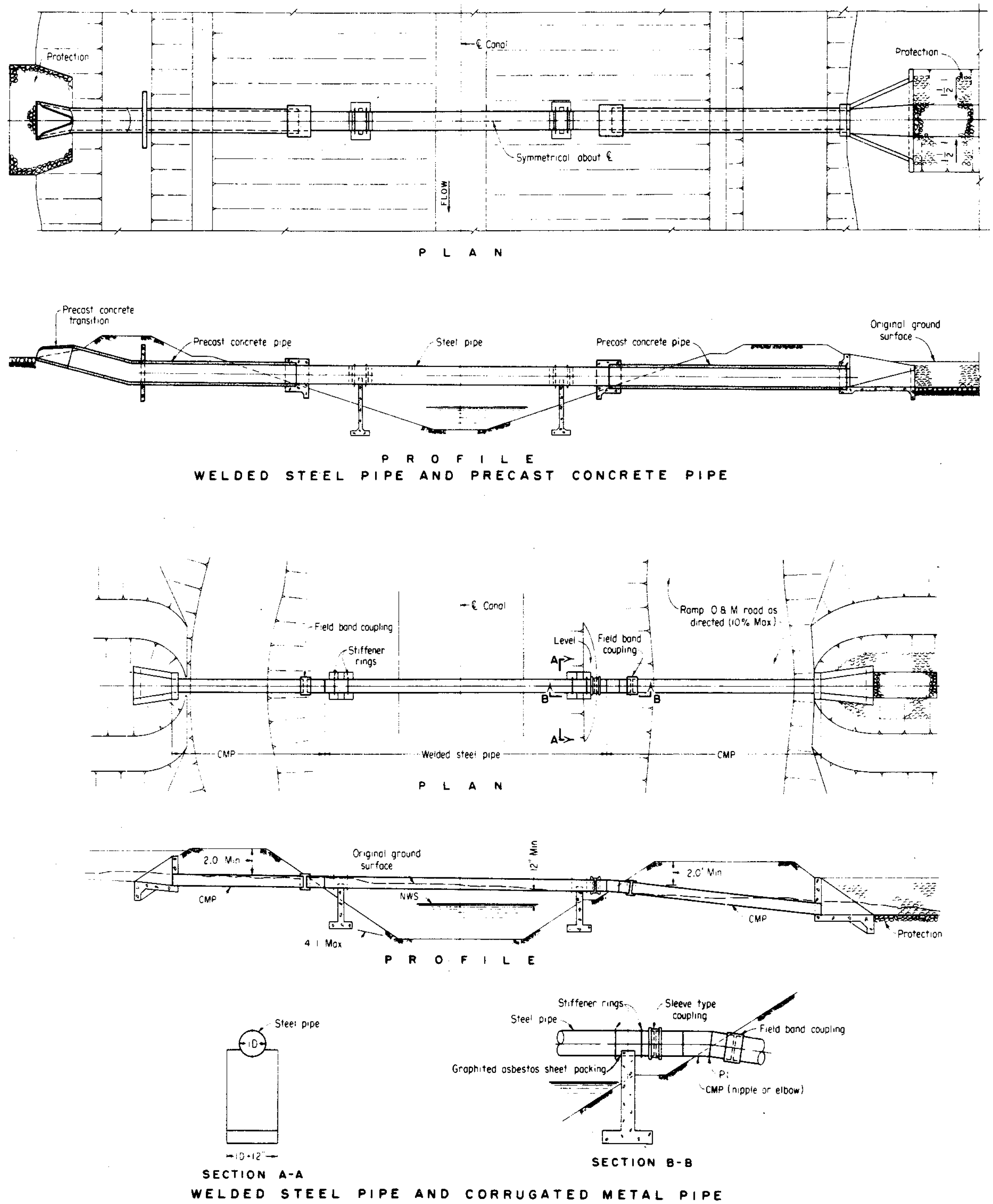


Figure 4-33. **Overchutes–typical plan and sections. 103-D-1308**

inlet is limited by three factors as follows:

(1) *Canal water surface.*—The inlet must be high enough to provide flow across the canal above the canal water surface, as discussed in section 4-32.

(2) *Pipe submergence.*—To fully utilize the capacity of the pipe, the water surface in the inlet channel should submerge the top of the pipe opening by at least 1.5 h_{v_p}. This provides for an inlet loss of 0.5 h_{v_p} and for a velocity head of 1.0 h_{v_p} in the pipe.

(3) *Bank freeboard.*—The maximum water surface for the design flow in the inlet channel should be at least 2 feet lower than the top of the uphill canal bank, or inlet dike as shown in figure 4-44.

In a well-defined cross-drainage channel, a type 1 inlet transition is ordinarily used, and the above limitations can usually be met without inundation of an excessively large area. However, where there is no channel, or where the channel is poorly defined, a large area may be inundated. To minimize this area, the pipe inlet may be lowered by using a type 3 or type 4 inlet transition (in lieu of the type 2 transition), provided that it does not infringe on the required pipe clearance above the canal water surface. This capability can be facilitated, if necessary, by using an oversized pipe, with little or no submergence of the inlet end.

A complete hydraulic analysis should be made to determine the location of the hydraulic control (see subsec. 4-26(d)), and the elevation of the inlet water surface. Where the channel is well defined, the flow to and from the overchute should coincide, as much as possible, with the natural channel flow.

With *inlet control*, the upstream water surface is controlled by the head required for entrance into the pipe. Thus, the water surface will rise to a height above the inlet centerline,

$$h = \frac{Q^2}{2gC^2A^2} \text{ (see subsec. 4-26(f))}$$

where the coefficient of discharge, $C = 0.6$.

Where *outlet control* exists, the head required to produce the design discharge is a function of the losses in the system as follows:

(1) *Inlet losses.*—The inlet loss,

$$h_i = K_i \Delta h_v$$

(2) *Pipe losses.*—The pipe losses consist of friction and bend losses. The friction loss should be computed from the Manning equation [2],

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

where s is the friction slope of the pipe, and n is the roughness coefficient, equal to 0.013 for precast concrete pipe. Pipe bend losses may be determined as shown in subchapter VIII A.

(3) *Outlet losses.*—The outlet loss,

$$h_o = K_o \Delta h_v.$$

See subchapter VII for explanation of inlet and outlet losses.

(b) *Pipe Velocity.*—Pipe overchutes should be sized for a maximum full pipe velocity of 10 feet per second if a concrete transition is used at the outlet, and for a maximum full pipe velocity of 12 feet per second if a baffled outlet is used.

(c) *Pipe Diameter.*—The diameter of the pipe is determined from the basic equation, Q = AV, when related to a pipe flowing full. This equation reduces to:

$$D = 1.13 \sqrt{\frac{Q}{V}}$$

The minimum diameter usually permitted for pipe overchutes is 24 inches for cross-drainage water which could carry weeds or trash. A minimum diameter of 12 inches may be permitted if the overchute carries canal irrigation flows that are relatively clean. In this case the overchute is sometimes called an irrigation crossing.

The pipe diameter may be increased from the value determined by the above equation to reduce the area of inundation in the upstream channel, or to provide a greater area for passage of weeds.

(d) *Pipe Outlet.*—Hydraulics in the outlet end of the pipe should be determined by the Bernoulli process, as illustrated in the design example for side channel spillway, subsection 4-8(c).

4-38. ***Concrete Overchute Hydraulics.***—(a) *Rectangular Section.*—As overchutes are fairly short, the most economical section is not necessarily the most desirable. A wider and shallower section will permit a shallower inlet pool depth, resulting in less inundation of the channel. Thus, the freeboard requirement can more easily be satisfied without excessively raising the uphill bank of the canal. The wide, shallow overchute section is particularly applicable to a shallow or poorly defined natural channel.

Where a stilling pool is used as an outlet energy dissipator, the width and depth of the flume section may be minimized by setting the chute on a supercritical slope, provided such a slope can be used within the limits imposed by the ground surface and the required clearance above the canal water surface. Critical flow may then be assumed to occur at the beginning of the chute section. Caution should be used to avoid a slope at or near the critical slope, as critical flow is rather unstable. A small change in energy will cause a relatively large change in water depth. To avoid excessive water surface undulations from unstable flow, a supercritical slope should be used, exceeding the critical slope by about 20 percent, with computations based upon a "rough" n (e.g., 0.015 in lieu of 0.013 in the concrete section).

If a baffled apron drop is used as an outlet energy dissipator, the chute velocity should be less than critical, as required for entrance to a baffled apron drop (see chapter VI).

The wall height in the chute section should provide a freeboard of at least 1 foot at the inlet, and for the full length of the chute section, based upon a "rough" n in the chute.

(b) *Critical Flow.*—Where the chute is set on a supercritical slope, critical depth at the beginning of the chute should be determined from table 17 [4], or the equation:

$$d_c = \frac{q^{2/3}}{g^{1/3}} = 0.314q^{2/3}$$

Then, the area, $A_c = bd_c$, where b is the width of the chute section. The velocity,

$$V_c = \frac{Q}{A_c}$$

and the velocity head,

$$h_{v_c} = \frac{V_c^2}{2g}$$

Specific energy,

$$E_{s_c} = d_c + h_{v_c}$$

Finally, the critical slope, s_c, should be determined from the Manning equation [2]:

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

or

$$s_c = \left[\frac{nV}{1.486\ R^{2/3}}\right]^2$$

where n is the roughness coefficient and R is the hydraulic radius, which is equal to the area divided by the wetted perimeter, wp.

(c) *Inlet Transition.*—An inlet transition is required to provide entrance from this channel to the chute. The upstream water surface elevation must be equal to the energy gradient in the chute, plus inlet losses. That is, it should be equal to the invert elevation at the beginning of the chute,

$$\text{plus } d_c + h_{v_c} + 0.3\ \Delta h_v$$

As the velocity in the inlet pool is near zero, and the velocity at the beginning of the chute is V_c, the inlet loss, $h_i = 0.3\ h_{v_c}$.

(d) *Chute Hydraulics.*—After determining critical flow conditions at the upstream end of the chute section, the hydraulics can be determined at the downstream end by using the Bernoulli process [2]:

$$E_{s_2} + h_f = E_{s_1} + s_o L \text{ (see fig. 4-36)}$$

where

E_{s_1} is the known specific energy;

E_{s_2} is the specific energy to be determined;
h_f is the friction loss between points 1 and 2; and
$s_o L$ is the drop in invert elevation between points 1 and 2.

Various values of d_2 are assumed until the associated velocity head and friction loss (from the Manning equation) provide an energy balance in the foregoing equation.

The chute walls usually have a uniform height, except that a greater height is often used through the upper bank of the canal (see fig. 4-35). The wall height through the bank, h_1, should include a minimum freeboard of 1 foot above the upstream water surface. The remaining portion of the chute should have a wall height, h_2, which includes a minimum freeboard of 1 foot above the maximum water depth in the chute. With the chute set on a supercritical slope, the maximum depth in the chute would be d_c, occurring at the inlet end.

(e) *Stilling Pool.*—Two types of stilling pools are used to dissipate the excess energy of cross-drainage flows in an overchute. With either type, a well-defined outlet channel is required to provide a reliable tailwater which is an essential factor in forming a hydraulic jump in the pool. Where the natural channel does not provide adequate tailwater, stoplogs, or a concrete curb have been used in the downstream end of the pool to assure the jump (see fig. 2-36). The standard stilling pool should be designed in accordance with the requirements shown in chapter II.

The vertical drop and pool requires less length than does the standard drop and pool, giving it an advantage where right-of-way is a consideration. However, in the interest of economy, its use should be limited to a vertical drop, D, that does not extend much below the canal invert. Where greater drops are required, other types of energy dissipators, or a culvert should be considered.

The design procedure for the vertical drop and pool is as follows:

(1) *Vertical drop.*—The magnitude of the vertical drop, D, should permit a free-draining pool to discharge into an outlet channel that can provide a reliable tailwater, capable of insuring a hydraulic jump in the pool. Thus, the invert of the pool must be set far enough below the downstream channel water surface to insure a pool depth, d_2, that is sufficient to cause a hydraulic jump as shown in figure 4-35. As the required depth, d_2, is dependent upon the depth, d_1, which is dependent upon the vertical drop, D, and it is dependent upon the available tailwater surface, a trial-and-error solution is necessary to determine the optimum dimension, D. After assuming a drop, D, the depths d_1 and d_2 are determined as shown in the following paragraphs.

The tailwater elevation is established using a conservative (smoother than expected) roughness coefficient for the downstream channel, to insure a hydraulic jump under extreme conditions. A reduction of approximately 20 percent in the normal value of n is generally used to give a conservative value for the earth section.

The downstream water surface should not submerge the crest of the drop by more than $0.6d_L$, as shown in figure 4-34, as greater submergence would prevent a hydraulic jump and excessive waves would continue into the downstream channel.

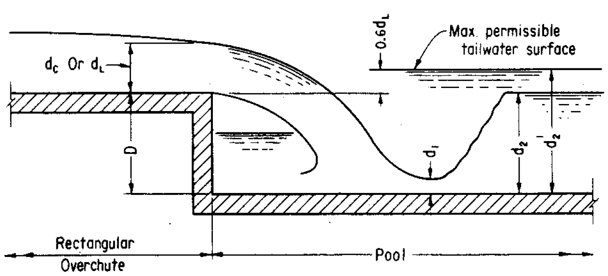


Figure 4-34. **Overchute—vertical drop hydraulics. 103-D-1309.**

(2) *Depth, d_1.*—The depth, d_1, can be obtained from the equation [11]

$$d_1 = \frac{\sqrt{2}\, d_c}{1.06 + \sqrt{\frac{D}{d_c} + 1.5}}$$

Even though the rectangular chute is set on a supercritical slope, the water depth, d_L, at the crest of the drop, will not be appreciably less than d_c, as the relatively short length required to span the canal is not sufficient to attain normal depth for a steep slope. Therefore, the substitution of critical depth for the actual depth can be made without a significant error.

(3) *Depth, d_2.*—The depth, d_2, can be obtained from the standard equation for depth in stilling pools (see chapter II),

$$d_2 = \frac{-d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2V_1^2 d_1}{g}}$$

(4) *Pool freeboard.*—The required pool freeboard is normally determined from the curve in figure 2-33, chapter II. Entering along the ordinate with the value of

$$\frac{QV_1 d_1}{A}, \text{ or } V_1^2 d_1$$

the freeboard can be read along the abscissa. If the pool has a concrete top (for a road crossing, e.g.) a freeboard of 1 foot is sufficient.

(5) *Pool length (see fig. 4-35).*—Length L_3: The *minimum* length for L_3 should be great enough to provide clearance between the headwall and the upper nappe of the flow trajectory. The profile of the upper nappe can be determined as shown for L_4, following. The *maximum* length should preferably not exceed the horizontal component of: the bank height above the canal lining height; or of the bank height above the water surface of an unlined canal. However, the stilling pool is sometimes permitted to encroach slightly upon the edge of the water prism in an unlined canal. Further, the headwall at the downstream end of L_3 is often permitted to encroach slightly upon the alinement of the normal canal bank. This is essential where the bank is raised to permit the operating road to cross the pool. Between the minimum and maximum values, the length, L_3, may be rounded as desired.

Length L_4: The length, L_4, is designed to locate the first row of pool blocks immediately downstream from the intersection of the upper nappe surface with the pool floor. The flow trajectory can be determined from the equations [2]:

$$y = (1/2)g\,t^2$$

and

$$x = v\,t$$

where

y is the vertical drop in water surface in t seconds,
x is the horizontal distance traveled in t seconds,
v is the velocity of flow at the beginning of the trajectory, and
g is the acceleration of gravity, equal to 32.2 feet per second per second.

Thus,

$$L_4 = x = vt$$

where t is the time required to drop a distance y, with

$$y = d_L + D$$

Length L_5: From the standard dimensions for stilling pools (see chapter VI),

$$L_5 = (3/4)\,d_2$$

Length L_6:

$$L_6 = 3\,d_2 \text{ minimum}$$

The total length, $L_4 + L_6 - L_3$ must also be sufficient to accommodate a crossing for the operating road, if required (see fig. 4-35).

(6) *Pool blocks.*—From the standard dimensions for pool blocks (see chapter II), the block height should be equal to $\frac{d_2}{4}$, and the block length should be equal to $1.25\,\frac{d_2}{4}$.

The block width (8 inches minimum) should conform to the requirement for equal block and space widths. Two rows of blocks should be located as shown in figure 4-35, with blocks in each row staggered with those in the other. One row will then consist of one block less than the other row, but will include a half block at each end, in contact with the sidewall. These half blocks are sometimes omitted. In either row, the block width,

$$W_b = \frac{\text{pool width}}{2\,N}$$

where N is the number of whole blocks plus one-half for each half block.

In the standard stilling pool, the first row of blocks are referred to as chute blocks, and are smaller than the floor blocks (see fig. 2-33). However, in the pool with a vertical drop, the blocks in each row should have the same dimensions.

Where other considerations require a length, L_6, equal to or greater than $5d_2$, the blocks may be omitted.

(7) *Outlet transition.*—The outlet transition should be of the broken-back type, as a well-defined outlet channel is required below stilling pools. A level invert is desirable to provide a free-draining pool. However, it is sometimes necessary to slope the invert up to the outlet channel, to provide an adequate tailwater depth, d_2, thereby insuring a hydraulic jump in the pool. See chapter VII for general requirements for transitions.

4-39. ***Other Considerations.***—(a) *Bank Heights.*—

(1) *Upper canal bank.*—The upper bank of the canal should provide at least 2 feet of freeboard above the inlet pool water surface. Where this would increase the normal bank height, the bank should be raised for such a distance that the top will intersect the natural ground surface at the required elevation. An inlet dike, as shown in figure 4-44, may be preferable to raising a long reach of the canal bank.

(2) *Lower canal bank.*—The normal height of the lower bank may not be great enough to permit the operating road (if required) to cross the stilling pool on its normal grade. Where such a crossing is required, the road should be ramped up as necessary, to provide adequate pool freeboard and concrete thickness, plus 2 feet of earth cover to reduce vehicular impact loads (see fig. 4-35). The grade of the ramp should not exceed 10 percent, and any change in alinement should be gradual to avoid introducing a roadway hazard. Where an earth cover over the box structure is not desired, the box should be designed to withstand the impact load.

(b) *Protection.*—The outlet channel should be protected from erosion by providing riprap or gravel protection in accordance with figure 7-8.

(c) *Percolation.*—Excessive percolation of water from the canal to the outlet channel, or from the inlet pool to the canal, should be avoided. See subchapter VIII C for design consideration for percolation.

4-40. *Design Example.*—Assume a canal, crossing a natural drainage channel, has insufficient head to accommodate a siphon under the channel. Therefore a cross-drainage structure is required to carry drainage water across the canal. As the profile of the channel invert is above the canal water surface, a culvert would require the construction of a long, deep outlet channel. An overchute is selected, as it can carry the drainage flows over the canal with little channelization required. Note that an overchute may have been selected on the basis of economics even if the canal had sufficient head for a siphon.

(a) *Assumptions.*—

(1) Properties of the canal section are as follows:

d_n = 3.6 feet	b = 6 feet
h_L = 4.3 feet	ss = 1-1/2:1
h_B = 5.6 feet	W_1 = 6 feet
	W_2 = 12 feet

(2) The drainage channel has a well-defined section at the inlet, having properties approximately as follows:

b = 8 feet, h_o = 6 feet, ss = 1-1/2:1, and a downstream slope s = 0.0002, with n = 0.030.

The alinement of the natural channel is approximately normal to the canal alinement, permitting a normal crossing for the overchute. The invert elevation of the natural channel drops about 4 feet as it crosses the canal, and then becomes rather level beyond the canal, as shown in figure 4-35.

(3) Weeds and other debris can be expected to accompany the storm runoff.

(4) It has been determined that the 25-year flood yields a discharge of 200 cfs in the channel.

(5) A 12-foot-wide operating road is located along the lower bank of the canal, and should be ramped up, if necessary, to cross over the overchute pool.

(b) *Solution.*–

(1) *Type of overchute.*–Assume that a cost comparison has indicated that a rectangular concrete overchute is competitive with a pipe overchute. As it is better able to carry the trash load, the rectangular concrete overchute should be used.

(2) *Overchute width.*–An economic study indicates that a 6-foot width gives the most economical cross section. However, a wider, shallower section, 8 feet wide, is selected to minimize the required depth of the inlet pool, and to increase the trash-carrying capability of the structure.

(3) *Critical flow hydraulics.*–By setting the chute invert on a supercritical slope, critical depth, d_c, will occur at the upstream end of the chute, as shown in figure 4-35. The discharge per foot of width,

$$q = \frac{Q}{b} = \frac{200}{8} = 25 \text{ cfs}$$

$$d_c = 2.69 \text{ feet from table 17 [4]}$$

$$h_{v_c} = 0.5d_c = 1.35 \text{ feet}$$

Then

$$E_{s_c} = 1.5d_c = 4.04 \text{ feet}$$

$$A_c = b\, d_c = 8 \times 2.69 = 21.5 \text{ sq. ft.}$$

$$V_c = \frac{Q}{A_c} = \frac{200}{21.5} = 9.3 \text{ f.p.s.}$$

Wetted perimeter, $wp_c = b + 2d_c = 8 + 5.38 = 13.38$ ft.

$$\text{Hydraulic radius, } R = \frac{A_c}{wp_c} = \frac{21.5}{13.38} = 1.61$$

From the Manning formula [2], with a roughness coefficient, n = 0.014,

$$s_c = \left[\frac{Vn}{1.486R^{2/3}}\right]^2 = \left[\frac{9.3 \times 0.014}{1.486 \times 1.37}\right]^2 = 0.0041.$$

(4) *Invert slope.*–To insure supercritical flow in the flume, the invert is set on a slope of 0.005, which is steeper than the critical slope of 0.0041 by about 20 percent.

(5) *Inlet depth.*–The required inlet depth, d_o (see fig. 4-35), is equal to the specific energy at the beginning of the chute, plus transition losses.

Thus,

$$\begin{aligned} d_o &= E_{s_c} + 0.3\Delta h_v \\ &= d_c + h_{v_c} + 0.3\,(1.35\text{-}0) \\ &= 2.69 + 1.35 + 0.41 \\ &= 4.45 \text{ feet} \end{aligned}$$

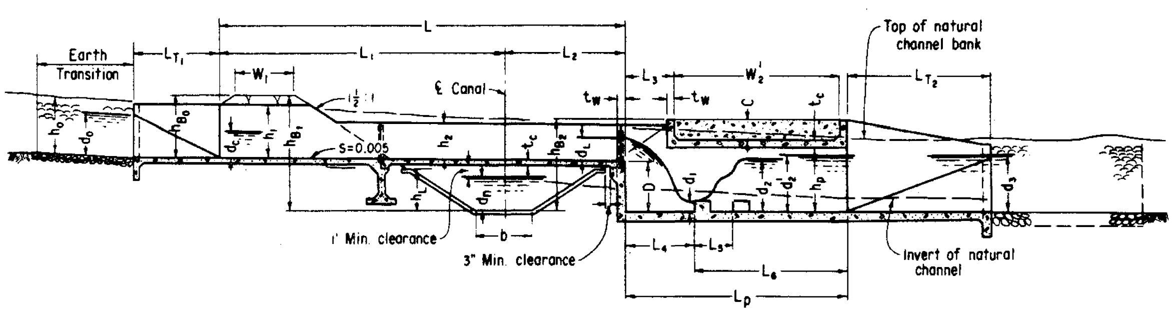


Figure 4-35. **Overchute–typical profile. 103-D-1310**

(6) *Inlet bank height.*—To provide 2 feet of freeboard above the inlet pool water surface, the upper bank of the canal should be raised to an elevation equal to

$$\text{El. } A + d_o + 2$$

Then

$$h_{B_o} = 4.45 + 2$$
$$= 6.45 \text{ feet}$$

(7) *Inlet transition.*—To transition the flow from the well-defined natural channel to the 8-foot-wide rectangular flume section, a broken-back transition, similar to the type 1 transition described in chapter VII, is best suited. As the length of the transition is dependent upon the convergence required, a part of the convergence will be accomplished in a 10-foot (minimum) earth transition, reducing the convergence and length required for the concrete transition. The length of the earth transition should be extended, as required, to the length of the inlet protection.

An invert width (b_c) of 4 feet will be used at the cutoff to reduce the width of the concrete transition. This will result in a water surface width at the cutoff,

$$T_c = b_c + 2(1.5\ d_o)$$

$$= 4 + 3 \times 4.45 = 17.35 \text{ ft.}$$

If the trash problem is sufficiently severe, the bottom width should not be reduced.

The concrete transition must accomplish the remaining required convergence to the 8-foot-wide flume (T_f = 8 feet). Conforming to the maximum angle of convergence (27.5° on each side), the concrete transition must have a length,

$$L_{t_1} = \frac{T_c - T_f}{2 \tan 27.5^\circ} = \frac{17.35 - 8}{2 \times 0.52} = 9 \text{ ft.}$$

The profile indicates that it is unnecessary to slope the invert of the concrete transition, and a level, free-draining inlet will be used, with the invert elevation at the cutoff set at the elevation of the channel thalweg.

(8) *Chute wall height, h_1 (at inlet).*—The wall height, h_1, should include a minimum freeboard of 1 foot above the upstream water surface. As the invert of the inlet transition is level, the wall height,

$$h_1 = d_o + 1.0 \text{ minimum}$$
$$= 4.45 + 1 \text{ minimum}$$
$$= 5.45 \text{ ft. minimum}$$
$$\text{Use } h_1 = 5 \text{ ft. } 6 \text{ in.}$$

(9) *Chute wall height, h_2.*—The height of the chute wall should be equal to the maximum water depth in the chute plus a freeboard of 1 foot minimum. As the invert slope is steeper than critical, the maximum depth in the chute is d_c, which occurs at the inlet end. Then, the minimum wall height in the chute

$$h_2 = d_{max.} + 1.0$$
$$= 2.69 + 1.0 = 3.69$$
$$\text{Use } h_2 = 3 \text{ ft. } 9 \text{ in.}$$

(10) *Chute length.*—Determine lengths L_1 and L_2 to clear the canal prism.

Use L_1 = 28 ft. 7 in.
L_2 = 11 ft. 1 in.
and L = 39 ft. 8 in.

(11) *Depth, d_L.*—Determine the depth, d_L, at distance L from the inlet transition (see fig. 4-36), using the Bernoulli method and a "slick" n.

$$E_{s_L} + h_f = E_{s_c} + s_o L$$

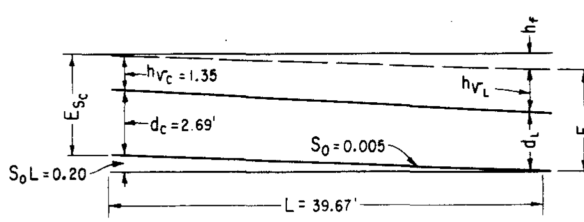


Figure 4-36. **Overchute–chute hydraulics. 103-D-1311**

Using a "slick" n = 0.011,

$$s_c = \left[\frac{Vn}{1.486R^{2/3}}\right]^2 = \left[\frac{9.3 \times 0.011}{1.486 \times 1.37}\right]^2$$

$$= (0.050)^2 = 0.0025$$

Try

$d_L = 2.4$ ft., $A = 19.2$ sq. ft., $V = 10.41$ f.p.s.
$h_v = 1.69$ ft., $wp = 12.8$ ft., $R = 1.5$
$E_{s_L} = 4.09$ ft., $R^{2/3} = 1.31$

Then

$$s_f^{\ 1/2} = \frac{Vn}{1.486R^{2/3}} = \frac{0.011 \times 10.41}{1.486 \times 1.31} = 0.059$$

$$s_f = 0.0035$$

$$h_f = \frac{s_L + s_c}{2} L = \left(\frac{0.0035 + 0.0025}{2}\right) \times 39.67$$

$$= 0.12$$

and, from

$$E_{s_L} + h_f = E_{s_c} + s_o L$$

$$4.09 + 0.12 = 4.04 + 0.20$$

$$4.21 \neq 4.24 \text{ (close enough for design)}$$

Therefore, $d_L = 2.4$ ft.

(12) *Vertical Drop, D.*—From the profile of the natural channel, it is seen that a downstream channel requiring a minimum of excavation, can probably support the required tailwater to insure a hydraulic jump in the pool if the invert is set about the same elevation as the canal invert. Thus, in the trial-and-error solution for the optimum drop, D (see fig. 4-35), try a value

$$D = d_n + 1.0 + t_c$$

$$= 3.6 + 1.0 + 0.67 = 5.27 \text{ ft.}$$

say 5.25 ft.

Proceed with the trial-and-error computations by using D = 5.25 feet to determine d_1 and d_2 in the pool.

(13) *Stilling pool depths, d_1 and d_2.*—From the equation in bibliography reference [11]

$$d_1 = \frac{\sqrt{2}\ d_L}{1.06 + \sqrt{\frac{D}{d_L} + 1.5}} = \text{(See fig. 4-35)}$$

$$= \frac{1.414\,(2.4)}{1.06 + \sqrt{\frac{5.25}{2.4} + 1.5}} = \frac{3.393}{1.06 + 1.92}$$

$$= 1.14 \text{ feet}$$

and $V_1 = \frac{Q}{A_1} = \frac{Q}{bd_1} = \frac{200}{8 \times 1.14} = 22$ f.p.s.

From the design of the standard stilling pool (chapter II) it is seen that

$$d_2 = \frac{-d_1}{2} + \sqrt{\frac{d_1^{\ 2}}{4} + \frac{2(V_1)^2\ d_1}{g}}$$

$$= \frac{-1.14}{2} + \sqrt{\frac{(1.14)^2}{4} + \frac{2\,(22)^2 \times 1.14}{32.2}}$$

$$= -0.57 + \sqrt{0.324 + 34.3}$$

$$= 5.31 \text{ feet}$$

Then, $A_2 = 8 \times 5.31 = 42.48$ sq. ft.,

$$V_2 = \frac{Q}{A} = \frac{200}{42.48} = 4.71,$$

$$h_{v_2} = \frac{V^2}{2g} = \frac{(4.71)^2}{64.4} = 0.35,$$

and $E_{s_2} = d_2 + h_{v_2}$

$$= 5.31 + 0.35 = 5.66 \text{ feet}$$

(14) *Tailwater depth, d_3.*—The depth, d_3, in the downstream channel, should be determined on the basis of a "slick" n, equal to about 80 percent of the estimated roughness coefficient in the earth section:

$$n_s = 0.80 \times 0.030 = 0.024$$

As the dimensions of the channel are only approximate, an additional safety factor is used by rounding to the next lower standard value,

$$n_s = 0.0225.$$

The tailwater depth, d_3, is determined by the trial-and-error method as follows, for the channel with dimensions as shown in assumption, subsection (a)(2) above:

$$\begin{aligned}
\text{Try } d_3 &= 5.8 \text{ feet} \\
\text{Then } A &= bd + 1.5d^2 \\
&= (8 \times 5.8) + 1.5\,(5.8)^2 \\
&= 97.0 \text{ sq. ft.} \\
wp &= b + 2d \csc \phi \text{ (see table 16 [4])} \\
&= b + 3.606d \\
&= 8 + 20.90 = 28.9 \text{ ft.}
\end{aligned}$$

$$\text{and } R = \frac{A}{wp} = \frac{97.0}{28.9} = 3.355$$

$$R^{2/3} = 2.24$$

$$\text{Finally,} \quad V = \frac{1.486\, R^{2/3} s^{1/2}}{\text{slick } n} = \frac{1.486 \times 2.24\,(0.0002)^{1/2}}{0.0225} = 2.09 \text{ f.p.s.}$$

$$\text{and} \quad Q = AV = 97.0 \times 2.09 = 202.7 \text{ cfs} > 200 \text{ cfs}$$

Therefore, d_3 is approximately 5.8 feet.

(15) *Outlet energy balance.*—To support the pool depth, d_2, which is required to produce a hydraulic jump in the pool, the tailwater depth, d_3, must be sufficient to provide a downstream specific energy, E_s, that is equal to or greater than the specific energy in the pool, minus the outlet transition loss. Neglecting the friction loss, a Bernoulli balance is checked as follows:

$$\begin{aligned}
E_{s_3} &\geqq E_{s_2} - 0.5\Delta h_v \\
5.87 &\geqq 5.66 - 0.5\,(h_{v_2} - h_{v_3}) \\
&\geqq 5.66 - 0.5\,(0.35 - 0.07) \\
&\geqq 5.66 - 0.14 \\
5.87 &> 5.52
\end{aligned}$$

The tailwater is more than adequate to support the jump in the pool. The vertical drop, D, could therefore be reduced slightly, to attain optimum economy at the expense of the safety factor provided by the excess downstream energy. However, as the permissible reduction in D is small, the pool design will be continued using a drop of 5.25 feet as assumed above.

If the net energy in the tailwater channel had not been equal to or greater than the energy in the stilling pool, E_{s_2}, the vertical drop, D, should be increased, to lower the energy gradient in the pool. The resulting energy balance should again be checked to insure adequate tailwater energy to insure a hydraulic jump in the pool.

(16) *Backwater depth in pool.*—The stilling pool depth, d_2, as computed from d_1, is valid only if an exact energy balance exists from d_2 to d_3. As the Bernoulli equation indicated an excess of energy, E_{s_3}, the resulting backwater depth in the pool,

$$d_2' > d_2$$

The depth, d_2', should be determined by a Bernoulli balance as follows:

Neglecting friction losses,

$$\begin{aligned}
E'_{s_2} &= E_{s_3} + 0.5\Delta h_v \\
&= d_3 + h_{v_3} + 0.5\Delta h_v \\
&= 5.8 + 0.07 + 0.5h'_{v_2} - 0.5\,h_{v_3}
\end{aligned}$$

$$\text{Then } d_2' + h'_{v_2} - 0.5h'_{v_2} = 5.87 - 0.03$$

$$\text{and} \quad d_2' + 0.5h'_{v_2} = 5.84.$$

$$\begin{aligned}
\text{Try} \quad d_2' &= 5.69 \\
\text{Then, } A_2' &= b\,d_2' = 8 \times 5.69 = 45.50 \text{ sq. ft.} \\
V_2' &= \frac{Q}{A_2'} = \frac{200}{45.50} = 4.4 \text{ f.p.s.} \\
\text{and } h'_{v_2} &= \frac{(V_2')^2}{2g} = \frac{(4.4)^2}{64.4} = 0.30 \text{ ft.}
\end{aligned}$$

$$\begin{aligned}
\text{Check: } d_2' + 0.5h'_{v_2} &= 5.84 \text{ (from above)} \\
5.69 + (0.5 \times 0.30) &= 5.84 \\
5.84 &= 5.84
\end{aligned}$$

Therefore, $d_2' = 5.69$ feet as assumed.

(17) *Crest submergence.*—The backwater depth in the pool should not be permitted to submerge the chute crest more than $0.6d_L$ (see fig. 4-34). Therefore, the submergence,

$$Sm \leqq 0.6\ d_L$$
$$d_2' - D \leqq 0.6 \times 2.4$$
$$5.69 - 5.25 \leqq 1.44$$
$$Sm = 0.44 < 1.44 \text{ ft.}$$

(18) *Pool wall height.*—The pool wall height, h_p, should be equal to the depth, d_2', plus a freeboard (or clearance below the top slab of the box) of 1 foot. Thus,

$$h_p = 5.69 + 1 = 6.69 \text{ ft.}$$
Use $h_p = 6$ ft. 9 in.

(19) *Bank height, h_{B_2}.*—The lower bank must be raised, locally, to a height suitable for crossing over the pool. As the pool invert was set at the same elevation as the canal invert, the height of the lower bank,

$$h_{B_2} = h_p + t_c + C$$
$$= 6.75 + 0.75 + 2$$
$$= 9.5 \text{ ft.}$$

assuming the slab thickness, t_c = 9 inches, and minimum earth cover, C = 2.0 feet. As the normal bank height, h_B is only 5.6 feet, the ramp should provide the additional height, 9.5 – 5.6 = 3.9 feet.

The ramp height may be reduced by eliminating the earth cover, provided that the concrete box is designed to withstand the full vehicular impact load. The minimum bank height, h_{B_2}, is then equal to 9.5 – 2.0, or 7.5 feet.

(20) *Pool length (see fig. 4-35).*—Length L_3: The minimum permissible length, L_3 minimum, should provide clearance between the headwall and the upper nappe of the flow trajectory.

From the relationship,

$$y = (1/2)\, gt^2,$$
with
$$y = (d_L + D) - h_p$$
$$= 7.65 - 6.75$$
$$= 0.90 \text{ ft.}$$

the time, $t = \sqrt{\dfrac{2y}{g}} = \sqrt{\dfrac{2(0.9)}{32.2}}$
$= 0.24$ sec.,

and from $L_{3\,min.} = vt + t_w$
$$L_{3\,min.} = 2.5 + 0.67 = 3.17 \text{ ft.}$$

The maximum length,

$$L_{3\,max.} = 1.5\,(h_{B_2} - h_L) - t_w - \frac{f_L}{2} - cl.$$
$$= 1.5(9.5-4.3)-0.67-0.25-0.25$$
$$= 7.8 - 1.17 = 6.63 \text{ ft.}$$

where

h_{B_2} is the raised bank height,
h_L is the canal lining height,
t_w is the wall thickness,
$\frac{f_L}{2}$ is half the lining flange width, and
cl. is the minimum clearance.

Let L_3 = 5.0 feet, resulting in a clearance of 1.88 feet.

Length L_4: From the relationship,

$$y = (1/2)gt^2$$
with $y = d_L + D$
$$= 2.4 + 5.25$$
$$= 7.65 \text{ ft.}$$

the time, $t = \sqrt{\dfrac{2y}{g}} = \sqrt{\dfrac{2 \times 7.65}{32.2}}$
$= 0.69$ sec.

and $L_4 = v\,t$
$$= 10.41 \times 0.69 = 7.18 \text{ ft.}$$
Use L_4 = 7 ft. 2 in.

Length L_5:

$$L_5 = (3/4)\, d_2 = (3/4) \times 5.31 = 3.98 \text{ ft.}$$
Use L_5 = 4 ft. 0 in.

Length L_6:

$$L_6 = 3d_2 \text{ (min.)} = 3 \times 5.31$$
$$= 15.93 \text{ ft. (min.)}$$
Use L_6 = 16 ft.

Length L_p: The pool length,

$$L_p = L_4 + L_6$$
$$= 7 \text{ ft. } 2 \text{ in.} + 16 \text{ ft. } 0 \text{ in.}$$
$$= 23 \text{ ft. } 2 \text{ in.}$$

This would accommodate a maximum roadway width,

$$W_2' = L_p - L_3 - t_w = 23.17 - 5 - 0.75$$
$$= 17.42 \text{ ft.}$$

(Required roadway width, W_2 = 12.0 feet.)

(21) *Pool blocks.*–

Block height $= \frac{d_2}{4} = \frac{5.31}{4} = 1.33$ ft. Use 16 inches.

Block length $= 1.25 \frac{d_2}{4} = 1.67$ ft. Use 20 inches.

Block width, W_b = 8 in. minimum.

Try N = 4 blocks in second row.

Then $W_b = \frac{\text{pool width}}{2N} = \frac{8}{2 \times 4} = 1.0$ ft.

Use three 12-inch blocks in first row, staggered with the second row blocks.

(22) *Outlet transition.*–A broken-back transition is the best type to use between the 8-foot-wide rectangular stilling pool and the well-defined trapezoidal channel (see chapter VII).

As the length of the transition is dependent upon the divergence required, a part of the divergence will be accomplished in a 10-foot (minimum) earth transition, reducing the divergence and length required for the concrete transition. The length of the earth transition should be extended, as required, to the length of the outlet protection. An invert width b_c of 4 feet will be used at the cutoff to reduce the width of the concrete transition. This will result in a water surface width at the cutoff,

$$T_c = b_c + 2(1.5d_3)$$
$$= 4 + 3 \times 5.8 = 21.4 \text{ ft.}$$

The concrete transition must accomplish the remaining divergence from the 8-foot-wide pool (T_p = 8 feet). As conservation of head is not a consideration, a water surface divergence, $\phi = 25^o$ (on each side) will be used in lieu of the usual 22.5^o. The concrete transition will then have a length,

$$L_{t_2} = \frac{T_c - T_p}{2 \tan \phi} = \frac{21.4 - 8}{2 \times 0.47}$$
$$= 14.26 \text{ ft.}$$

Use L_{t_2} = 15 ft.

(23) *Protection.*–From figure 7-8, inlet protection should be type 1, extending 1 foot above the design water surface and 2.5 d_o upstream from the cutoff. Thus, the length of the upstream protection,

$$L_u = 2.5\, d_o = 2.5 \times 4.45$$
$$= 11.10 \text{ ft.}$$

Use L_u = 12 ft.

Type 3 outlet protection is indicated, extending 16 feet downstream from the cutoff. The protection should extend to an elevation 1 foot above the design water surface in the channel.

(24) *Percolation.*–Percolation should be checked by the Lane method, as discussed in subchapter VIII C.

3. Drain Inlets

4-41. ***General.***–Drain inlets are structures used to carry relatively small amounts of storm runoff or drainage water into the canal. For larger flows, it is usually preferable to take the cross-drainage water over or under the canal in an overchute or culvert to be discharged beyond the canal. This is particularly true for flows that are expected to carry an appreciable amount of silt, sand, or debris. However, it is sometimes possible to bring relatively clean water into the canal more economically than diverting it to the other side of the canal.

(a) *Location.*–The drain inlet may be

located in a natural drainage channel, or at the terminal end of an interceptor drain which parallels the canal. As the inlet end must be above the canal water surface, the drain inlet is best suited where the canal prism is below the original ground surface. In a fill section (requiring embankment of earth material), ponding of undrained water may result unless the inlet channel is filled with earth material to the invert elevation of the conduit.

(b) *Types.*—Drain inlets are of three basic types as follows:

(1) *Pipe section.*—A pipe section (see fig. 4-37) is generally used where the design flow is small, and it is therefore the type most commonly used for the range of discharges covered by this text. Use of the pipe section is desirable if an operating road crosses the drain inlet. Corrugated-metal pipe is often used, as it usually provides a moderately long life at a relatively low cost. Replacement of the pipe does not require shutdown of the canal.

The outlet of the pipe discharges above the canal water surface, with dissipation of energy being accomplished as this inflow mixes with the canal flow.

(2) *Rectangular concrete section.*—The rectangular concrete section is used primarily where the flow is relatively large, and it is therefore used infrequently with small canals. However, it is particularly well suited where the drainage channel is wide, shallow, or where a pipe inlet would be subject to clogging with weeds.

A box section should be provided, as shown in figure 4-38, where roadway traffic is required over the drain inlet. In an earth section, the concrete chute terminates in a rectangular concrete pool extending below the canal invert, and the excess energy is dissipated in the pool. In a concrete-lined canal, the concrete chute may extend only to the concrete lining, as shown in figure 4-39.

(3) *Gravel blanket.*—Where the inflow is very small, a coarse gravel blanket may be used to cover the top and sides of the upper canal bank, as shown in figure 4-42. The entrance is shaped to confine the flow to a relatively narrow section. Again, excess energy is dissipated as the inflow mixes with the canal flow.

The gravel blanket drain inlet should be used only at locations which provide a suitable subgrade, as erosion or excessive percolation through the bank could cause bank failure.

(c) *Design Capacity.*—Drain inlets should be designed to accommodate the storm runoff

Figure 4-37. **Drain inlet on far bank of earth section canal, and motor-operated gate hoist for turnout gate in foreground. P-222-116-50929**

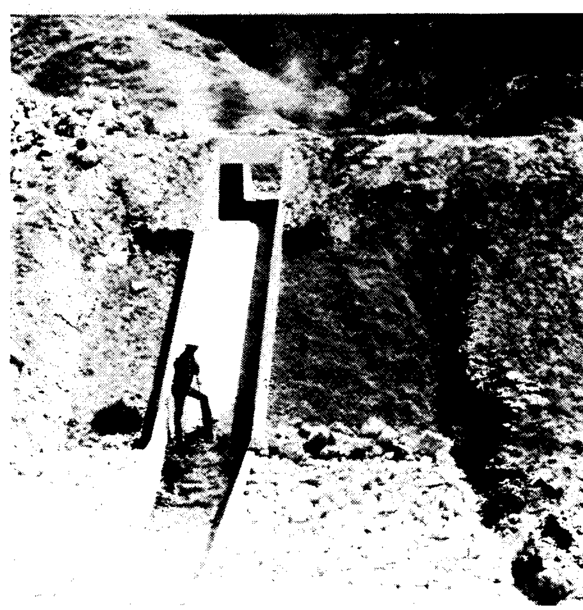


Figure 4-38. **Rectangular inclined drain inlet to earth section. Erosion protection incomplete. P-707-729-1550**

from the drainage area, based upon the design storm frequency (usually 25 years).

Where overflow wasteway facilities are not provided for a reach of canal, the total design capacity of all drain inlets within that reach should be limited to 10 percent of the normal design capacity of the canal.

Where an overflow wasteway facility *is* provided, the design capacity of an individual drain inlet should not exceed 10 percent of the normal design capacity of the canal, and the cumulative inflows from all drain inlets within that reach should not exceed 20 percent of the normal design capacity of the canal.

Further, inflows from drain inlets should not cause the canal water surface to rise above its normal depth by more than one-half the normal lining freeboard, or by more than one-fourth the normal bank freeboard.

Exceptions may be made to the foregoing limitations for the short duration of the maximum design flow provided an *automatic* wasteway feature, such as a side channel spillway, is located within a short distance of the drain inlet. However, the resulting canal bank freeboard should never be less than 1 foot. Where drain inlets cannot accommodate the design discharge of the drainage area within the foregoing limitations, culverts, overchutes, or canal siphons should be used.

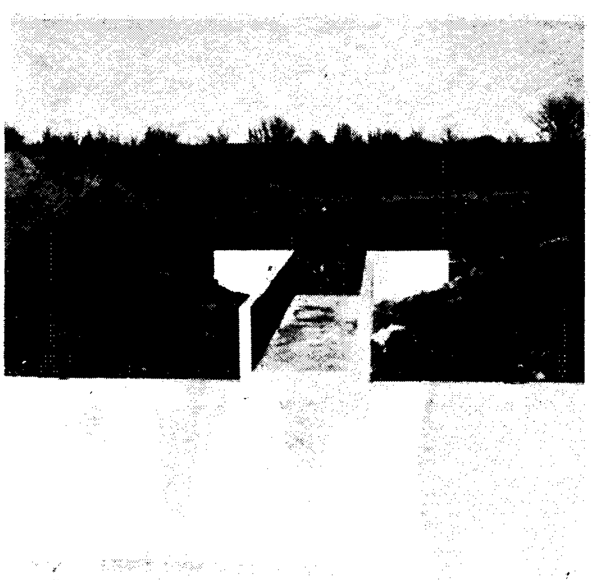


Figure 4-39. **Rectangular concrete drain inlet to a concrete-lined canal section. P-430-500-383**

(d) *Canal Bank.*—The upper canal bank adjacent to the drain inlet should be raised if necessary, to provide adequate freeboard above the inlet water surface, or to provide adequate cover over a pipe drain inlet. When the requirement for pipe cover is the controlling criterion, this increase in bank height should extend for a minimum distance of 3 pipe diameters. Where an operating road crosses the drain inlet, the roadway should be ramped up as required on a slope not exceeding 10 percent.

If inlet freeboard is the controlling criterion, the raised bank should be extended as required. Where the upper bank of the canal is in cut (excavation), the drain inlet dike should be extended to intersect the natural ground surface at the top elevation of the dike (see fig. 4-44).

4-42. ***Pipe Drain Inlets (see fig. 4-41).***—(a) *Inlet.*—Earth inlet transitions are usually satisfactory for drain inlets. If a particular situation requires a concrete transition, its type should be determined on the basis of the conditions discussed in section 4-24.

To prevent degradation of the inlet channel and to provide complete drainage of the channel, the pipe invert (assuming an earth transition) should be set at or slightly below the original ground surface of the channel invert. The pipe inlet should be high enough above the canal water surface to prevent reverse flow. Reverse flow, or failure to effect complete drainage of the inlet channel, could result in hydrostatic pressure in the canal bank, causing bank failure; or behind the canal lining, causing lining failure. A flap gate, as shown in figure 4-40, is sometimes included to prevent reverse flow.

The inlet water surface should provide pipe submergence as follows:

(1) *Subcritical pipe slope.*—Where the slope of the pipe is less than the critical slope (see sec. 4-26), the top of the pipe inlet should be submerged at least 1.5 h_{v_p}, where h_{v_p} is the velocity head in the pipe at the inlet. See chapter VII for a detailed discussion of inlet losses.

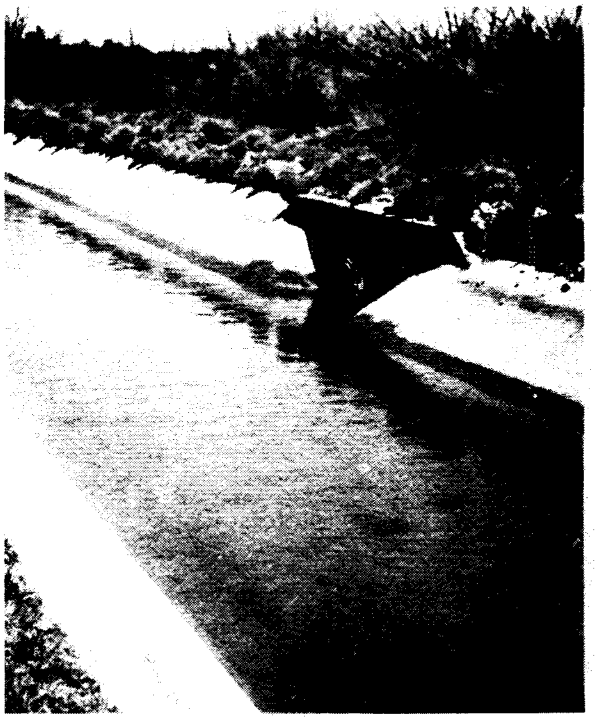


Figure 4-40. **Drain inlet with flap gate to prevent reverse flow. PX-D-73101**

(2) *Supercritical pipe slope.*—Where the slope of the pipe is equal to or greater than the critical slope, the invert of the pipe inlet should be submerged at least 1.5 h_{v_c} + d_c, where d_c is the critical depth in the pipe.

Freeboard from the inlet water surface to the top of the canal bank, or drain inlet dike, should be at least 2 feet.

(b) *Conduit.*—Pipe drain inlets usually consist of corrugated-metal pipe (CMP). Other types of pipe could be used, but probably not as economically. The pipe should be designed as follows:

(1) *Diameter.*—The pipe diameter should be determined on the basis of a maximum full pipe velocity of 10 feet per second if a concrete inlet is used, and 5 feet per second if an earth transition is used. However, a minimum diameter of 18 inches should be used where weeds or debris can be expected, and 12 inches where relatively clean water is assured.

The pipe diameter is determined from the basic equation, Q = AV, which reduces to

$$D = 1.128\sqrt{\frac{Q}{V}}$$

(2) *Slope.*—The slope of the pipe should be at least 0.005 to permit complete drainage. No maximum slope is established; however, it is sometimes desirable to include a pipe bend and two slopes, s_1 and s_2, as shown in figure 4-28, to reduce the slope of the outlet pipe. This is particularly appropriate where a considerable drop in grade occurs, or where an intermediate berm is used.

(3) *Gage or shell thickness.*—For the required gage of corrugated-metal pipe, or the shell thickness of other types of pipe for various loading conditions, see the pipe manufacturer's recommendations (see bibliography references [4] and [5] in chapter VIII).

(4) *Pipe cover.*—The drain inlet pipe should have a minimum cover that provides 6 inches of earth over the percolation collar, as shown in figure 4-41. Where a collar is not provided, the pipe should have a cover of at least 1 foot. However, the inlet freeboard requirement of 2 feet will often control the bank height, resulting in a pipe cover of at least 2 feet.

(c) *Outlet.*—The drain inlet pipe discharges directly into the canal prism, having its invert preferably 6 inches, but at least 4 inches, above the normal water surface in an earth canal; or 1 inch above the maximum water surface, whichever results in the higher elevation. Where the canal section has concrete lining, the pipe invert should be above the top of the lining. The end of the pipe usually extends about 1 pipe diameter (24 inches minimum) into the canal prism, as shown in figure 4-41. However, they are sometimes cut flush with the canal bank. The relative advantages of each type are as follows:

(1) *Uncut projection.*—The pipe discharge enters the canal water prism far enough from the bank to minimize erosion at the bank.

(2) *Flush cut.*—A flush end facilitates weed cutting along the bank, and may present a better appearance. While a saving in pipe length results, it may be offset by additional labor cost and poorer energy dissipation.

(d) *Percolation.*—To reduce percolation

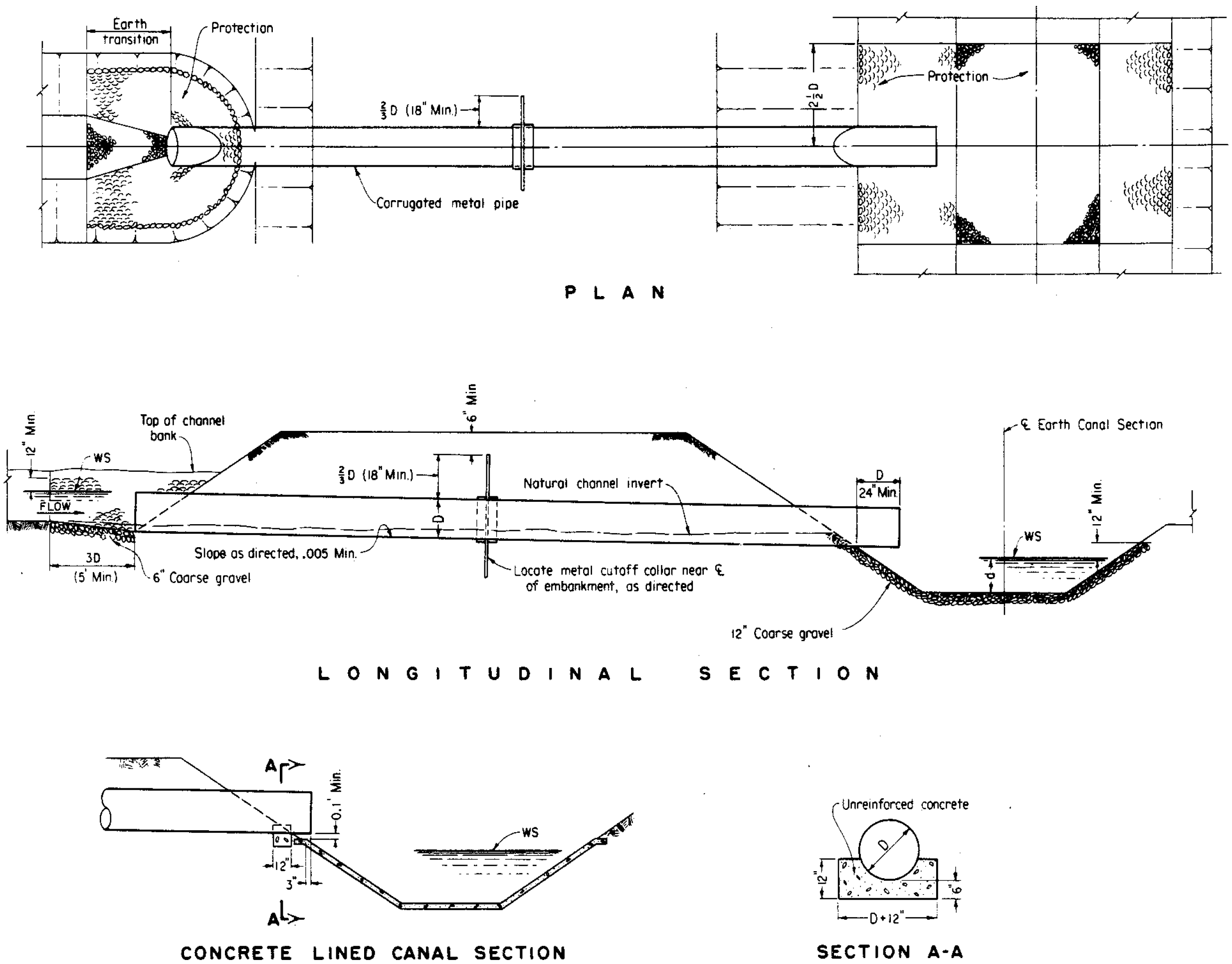


Figure 4-41. **Drain inlet–plan and sections. 103-D-1312**

from the inlet channel to the canal, a collar should be placed around the pipe as shown in figure 4-41. Some drain inlet pipes may require two or more collars. The requirement for collars should be determined by an analysis of percolation as discussed in section 4-27 and in chapter VIII. The standard collar for corrugated-metal-pipe drain inlets extends radially two-thirds D (18 inches minimum) from the periphery of the pipe.

(e) *Protection.*—Inlet protection should consist of 6 inches of coarse gravel, extending at least 12 inches above the normal or ponded water surface. The length of protection should be at least 3 pipe diameters (5 feet minimum) if an earth transition is used, and at least 2.5 pipe diameters (5 feet minimum) if a concrete transition is used. Where the inlet channel alinement is perpendicular to that of the pipe, or if the inlet channel is poorly defined, the protection should extend at least 2-1/2 pipe diameters each side of the pipe centerline.

Outlet protection, except in a concrete-lined canal section, should consist of a minimum of 12 inches of coarse gravel, extending 12 inches above the normal water surface of the canal, and at least 2-1/2 diameters upstream and downstream from the pipe centerline.

4-43. ***Rectangular Concrete Drain Inlet to Earth Section (see figure 4-42).***—(a) *Inlet.*—The rectangular concrete flume usually requires no transitioning other than connecting the earth channel to the rectangular opening. Wingwalls are provided to

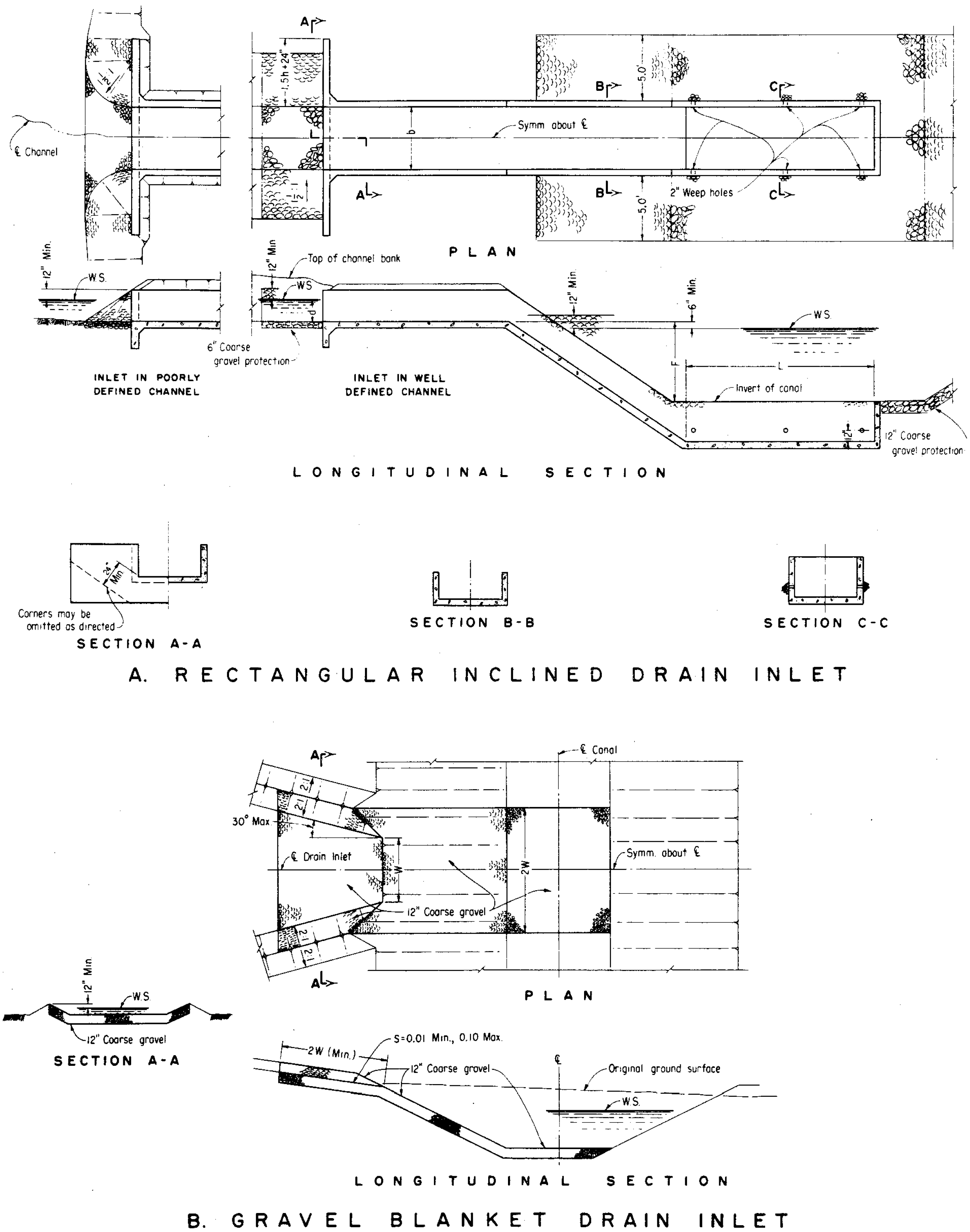
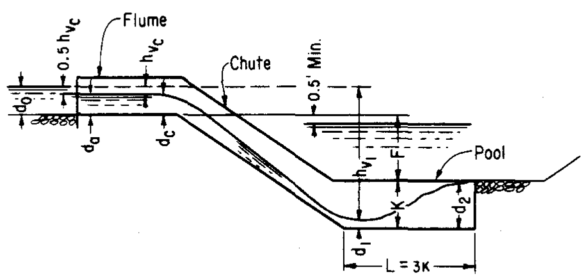


Figure 4-42. **Drain inlets–plan and sections. 103-D-1313**



DROP F, FT	Q, CFS:	5	10	15	20
	b, FT:	2'-0"	3'-0'	3'-6"	4'-0"
2	K:	18"	21"	24"	2'-3"
	L:	4'-6"	5'-3"	6'-0"	6'-9"
4	K:	20"	24"	2'-3"	2'-6"
	L:	5'-0"	6'-0"	6'-9"	7'-6"
6	K:	22"	2'-3"	2'-6"	2'-9"
	L:	5'-6"	6'-9"	7'-6"	8'-3"
8	K:	24"	2'-6"	2'-9"	3'-0"
	L:	6'-0"	7'-6"	8'-3"	9'-0"
10	K:	2'-3"	2'-9"	3'-0"	3'-3"
	L:	6'-9"	8'-3"	9'-0"	9'-9"

Figure 4-43. **Concrete drain inlet dimensions. 103-D-1314**

retain the bank or dike slope, and a cutoff to reduce percolation. The concrete invert should be set at or slightly below the channel invert, to permit complete drainage of the channel, and at least 6 inches above the canal water surface to prevent reverse flow.

The canal bank should have 2 feet of freeboard above the water surface in the channel.

(b) *Flume.*—The rectangular concrete flume extends through the canal bank on a subcritical slope (usually 0.005) assuring a velocity less than critical, until the flow reaches the steep slope of the chute section, at which point critical depth occurs (see fig. 4-43). Critical depth, d_c, can be determined from table 17 [4], or from the equation:

$$d_c = \frac{q^{2/3}}{g^{1/3}} = 0.3143\, q^{2/3}$$

where q is the discharge per foot of width and g is the acceleration of gravity, equal to 32.2 feet per second per second.

The flume width, b, is selected so that the specific energy ($d + h_v$) in the flume does not exceed the specific energy in the inlet channel. As the flume section is short, friction losses may be disregarded, and it can be assumed that d_a is equal to d_c. Then, assuming an inlet loss of 0.5 Δh_v and using the Bernoulli theorem [2]:

$$E_{s_o} = E_{s_c} + 0.5\, \Delta h_v$$

or

$$d_o + h_{v_o} = d_c + h_{v_c} + 0.5\, \Delta h_v$$

Assuming a zero velocity head in the inlet channel,

$$d_o = d_c + h_{v_c} + 0.5\, h_{v_c}$$

In a rectangular section, $h_{v_c} = \frac{d_c}{2}$.

Then

$$d_o = d_c + 0.5\, d_c + 0.5\, (0.5\, d_c)$$

or

$$d_o = 1.75\, d_c$$

If the required depth, d_o, in the inlet channel is not attainable without excessive ponding, or without use of excessively high banks or dikes, the drain inlet width, b, should be increased. The height of the flume walls should provide a freeboard of at least 1 foot above the inlet channel water surface. The invert at the downstream end of the flume should be at least 6 inches above the normal water surface in the canal.

(c) *Chute.*—The chute descends to the pool on an invert slope equal to the side slope of the canal, with the top of the chute walls coinciding with the canal bank: The wall height, measured normal to the chute slope, should be:

$$h_b = d_c + 1 \text{ foot minimum}$$

(d) *Outlet Pool.*—The pool dimensions are determined for the condition of full flow in the drain inlet when the canal is empty. For the more common inlet conditions, these dimensions are tabulated in figure 4-43. Pool dimensions for other inlet conditions should be determined from the basic equations for pools as discussed in chapter II.

The dimensions tabulated in figure 4-43 are based upon the assumption that critical depth, d_c, occurs at the beginning of the chute, and that F/d_c is equal to H/d_c in figure 2-37, chapter II, where the pool depth, d_1 associated with the hydraulic jump, is tabulated in terms of critical depth, and d_2 in terms of d_1. Friction loss in the chute is neglected.

The pool wall height, K, should be equal to or greater than d_2, and the pool length should be equal to 3K. The vertical wall at the end of the pool provides a positive tailwater to insure the hydraulic jump; therefore, pool and chute blocks, which would interfere with sediment removal, are not considered necessary. If the pool length, L, exceeds the bottom width of the canal, the drain inlet should be set back into the canal bank as required.

(e) *Percolation.*—Percolation of water from the inlet to an empty canal should be checked by Lane's weighted-creep analysis discussed in chapter VIII. Additional cutoff walls should be used if necessary.

(f) *Protection.*—Inlet protection should consist of 6 inches of coarse gravel, extending upstream a distance equal to 2.5 d_o (5 feet minimum), and to an elevation 1 foot above the water surface in the inlet channel (see fig. 4-43).

Outlet protection in the canal should consist of a minimum of 12 inches of coarse gravel, extending 5 feet upstream and downstream from the structure, and up the sloping canal banks to an elevation 1 foot above the normal water surface.

4-44. *Other Types of Drain Inlets.*—(a) *Rectangular Concrete Drain Inlets to a Concrete-lined Canal.*—The rectangular concrete drain inlet to a concrete-lined canal section should be similar to the concrete drain inlet to an earth section, except that the chute section is terminated at the top of the concrete lining (see fig. 4-39). A thicker lining with reinforcement should be considered for the section adjacent to the drain inlet.

(b) *Gravel Blanket Drain Inlet.*—The gravel blanket drain inlet, as shown in figure 4-42, channels the flow from the drainage channel into an earth section canal, utilizing a wide earth transition with a layer of gravel protection. Dimensions are as follows:

(1) *Inlet transition.*—The inlet transition should converge to a throat width equal to at least 1 foot for each cubic foot per second of design flow. Thus, a design flow of 3 cfs would require a throat width of 3 feet. The transition length should be sufficient to divert the flow, but at least twice the throat width, and not less than the normal canal bank width. The invert of the transition should slope toward the canal on a slope of 0.01 minimum, and 0.10 maximum.

The berms should provide a freeboard of 1 foot above the drain inlet water surface, assuming critical depth at the beginning of the canal bank slope. The angle of convergence should not exceed 30^o on each side.

(2) *Gravel blanket.*—The gravel blanket should consist of 12 inches of coarse gravel. The inlet protection should extend the full length and width of the inlet transition, to an elevation 1 foot above the inlet water surface for the design flow from the inlet channel.

Canal protection should extend down the upper canal bank and across the invert. It should extend longitudinally along the canal for a length equal to at least twice the inlet throat width.

4-45. *Design Example.*—A canal intercepts a drainage channel that has an invert above the normal canal water surface. The flow from the drainage area is expected to be about 6 cfs of clean water (relatively free of weeds or debris). The canal, having a normal flow of 75 cfs and a wasteway overflow not far downstream, can probably accommodate the addition of the 6 cfs from the drainage channel without excessive infringement upon canal freeboard. If this proves to be true, a drain inlet shall be designed to handle this flow, as shown in figure 4-44.

(a) *Assumptions.*—

(1) Properties of the earth section canal are as follows:

$Q = 75$ cfs	$n = 0.025$	$s = 0.0005$
$b = 8$ ft	$w_1 = 6$ ft.	$ss = 1\text{-}1/2{:}1$
$d_n = 2.9$ ft	$h_B = 4.7$ ft.	

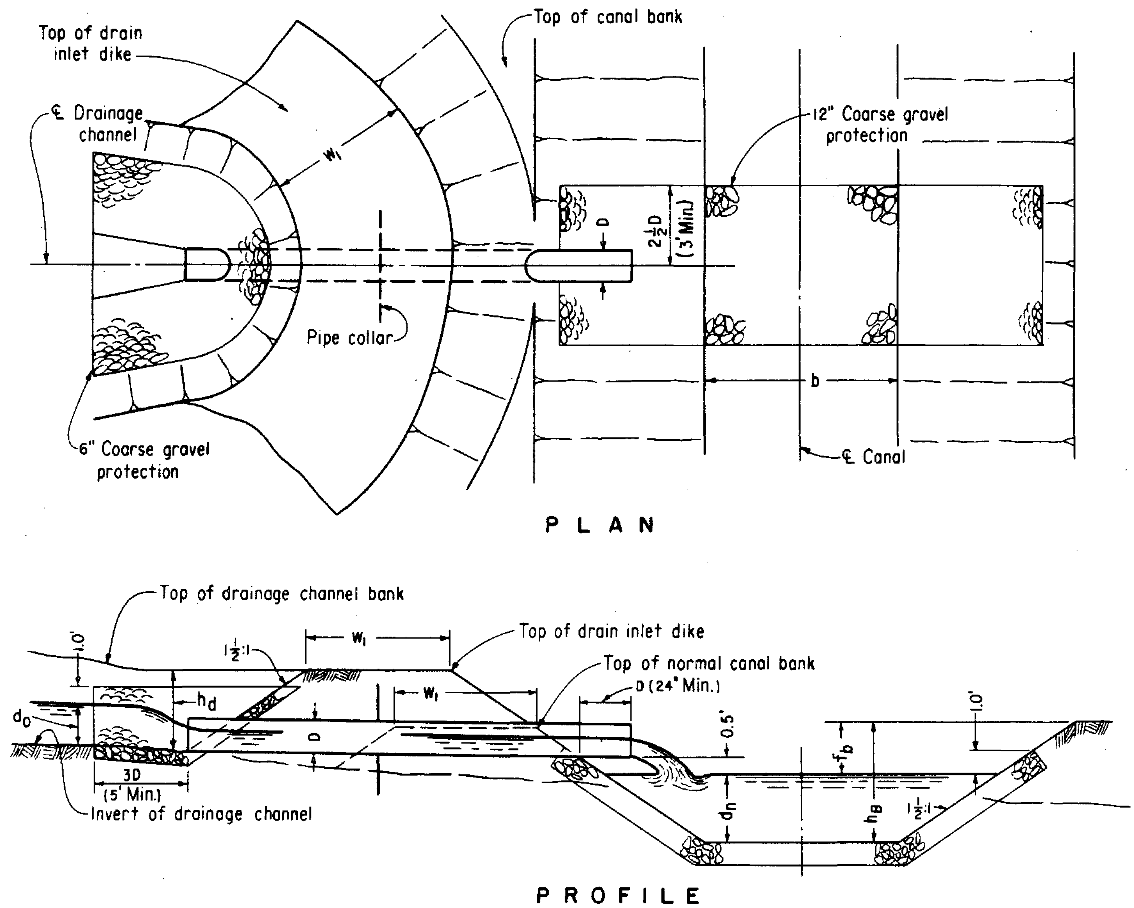


Figure 4-44. **Pipe drain inlet–plan, profile, and nomenclature. 103-D-1315**

(2) The drainage channel is well defined and has an invert elevation 1.5 feet above the normal water surface of the canal.

(3) Properties of the natural channel are approximately as follows:

$Q = 6$ cfs $\qquad d_n = 1.5$ ft.
$b = 6$ ft. $\qquad h_B = 3$ ft.

(b) *Solution (see fig. 4-44).*–

(1) *Canal freeboard.*–Determine the reduced freeboard in the canal resulting from the inflow of 6 cfs, which increases the canal flow from 75 to 81 cfs.

$$\text{Freeboard for 75 cfs} = h_B - d_n = 4.7 - 2.9 = 1.8 \text{ ft.}$$

Find normal depth in canal for 81 cfs:

Try d = 3.05 ft. and determine the resultant hydraulic properties,

$$A = bd + 1.5\,d^2 = 8 \times 3.05 + 1.5(3.05)^2 = 24.40 + 13.94 = 38.34 \text{ sq. ft.}$$

$wp = b + 3.606\,d$ (see bibliography ref. [4])

$$= 8 + 3.606 \times 3.05 = 8 + 11 = 19.0 \text{ ft.}$$

$$R = \frac{A}{wp} = \frac{38.34}{19.0} = 2.02$$

$R^{2/3} = 1.598$ (see table 63 [4])

Then

$$Q = \frac{1.486\,AR^{2/3}\,s^{1/2}}{n} = \frac{1.486 \times 38.34 \times 1.598 \times 0.02236}{0.025}$$

$= 81.4 > 81$ cfs (close enough)

Therefore, $d = 3.05$ ft. for $Q = 81$ cfs

Remaining freeboard $= h_B - d_{(81\ cfs)}$

$$= 4.7 - 3.1 = 1.6 \text{ ft.}$$

Infringement on normal freeboard $= 1.8 - 1.6$
$= 0.2$ ft.

Permissible infringement, $\frac{f_{b_n}}{4} = \frac{1.8}{4} = 0.45$ ft.
> 0.2 ft.

Therefore, the canal can accommodate the additional flow of 6 cfs without excessive infringement upon canal freeboard.

(2) *Drain inlet type.*—The well-defined inlet channel and the relatively small discharge indicate that a pipe drain inlet would be suitable.

(3) *Pipe diameter.*—It is determined that an earth inlet transition will be satisfactory, and the pipe diameter should be based upon a pipe velocity of 5 feet per second flowing full.

$$D = 1.128 \sqrt{\frac{Q}{V}}$$
$$= 1.128 \sqrt{\frac{6}{5}}$$
$$= 1.128\,(1.095)$$
$$= 1.24 \text{ ft.}$$

Use D = 15 inches. (Note that the minimum pipe diameter permitted for relatively clean water is 12 inches.)

Then, $A = 1.227$, and for a full pipe, $V = 4.89$, and $h_v = 0.37$ (from table 8-1, chapter VIII).

(4) *Invert slope, s_o.*—The invert of the pipe inlet will be set at the invert of the drainage channel, 1.5 feet above the normal water surface (NWS) of the canal. The outlet end of the pipe will be set 6 inches above the NWS, leaving a drop of 1.0 foot in the pipe. By scaling, the pipe length is found to be about 18 feet. The pipe slope is then $\frac{1.0}{18} = 0.056$, which is greater than the minimum required slope of 0.005.

(5) *Critical slope, s_c.*—Determine the critical slope for $Q = 6$ cfs, $D = 1.25$ feet, and $n = 0.024$, using tables 21, 22, 58, and 60 [4].

Find $D^{5/2} = 1.747$ from table 58.

Then, $\frac{Q}{D^{5/2}} = \frac{6}{1.747} = 3.435$,

and

$$\frac{d_c}{D} = 0.79 \text{ from table 22.}$$

Also, $\frac{Qn}{D^{8/3}\ s^{1/2}} = 0.447$ from table 21.

Then,

$$s_c^{1/2} = \frac{Qn}{0.447\ D^{8/3}} \text{ (where } D^{8/3} = 1.813 \text{ from table 60)}$$
$$= \frac{6 \times 0.024}{0.447 \times 1.813}$$
$$= 0.178$$

and

$$s_c = (0.178)^2 = 0.032 < 0.056$$

(6) *Location of hydraulic control.*—As the actual invert slope, s_o, is steeper than the critical slope, s_c, the hydraulic control is located at the inlet, just inside the pipe. Critical flow conditions at the inlet end of the pipe are then:

$$d_c = 0.79\ D$$
$$= 0.79 \times 1.25$$
$$= 0.99 \text{ ft.}$$

$$A_c = 0.666\ D^2 \text{ (from table 21)}$$
$$= 0.666 \times (1.25)^2$$
$$= 1.04 \text{ sq. ft.}$$

$$V_c = \frac{Q}{A_c} = \frac{6}{1.04} = 5.77 \text{ f.p.s.}$$

and $h_{v_c} = \frac{V^2}{2g} = 0.52$ ft.

Had the actual slope of the pipe been flatter than the critical slope, the hydraulic control would be located at the outlet. Then the hydraulics in the inlet end would be determined by the Bernoulli method, balancing energies from the outlet end to the inlet end, as shown in subsection 4-8(c).

(7) *Inlet water surface.*—The water surface in the inlet channel must rise to a depth sufficient to provide a specific energy equal to the specific energy in the pipe, plus inlet losses, h_i. Thus,

$$E_{s_o} = E_{s_c} + h_i$$
$$d_o + h_{v_o} = d_c + h_{v_c} + h_i$$

Assume that the velocity head in the inlet channel is negligible and that the inlet loss, $h_i = 0.5\ \Delta h_v$, where Δh_v is the difference in velocity heads, h_{v_c} for critical flow in the pipe and h_{v_o} for flow in the inlet channel.

Then

$$\begin{aligned} d_o + 0 &= d_c + h_{v_c} + 0.5\ h_{v_c} \\ d_o &= d_c + 1.5\ h_{v_c} \\ &= 0.99 + 1.5\ (0.52) \\ &= 1.77 \text{ ft.} \end{aligned}$$

Thus, the water surface in the inlet channel will be 1.77 feet above the invert of the pipe inlet, to produce a discharge of 6 cfs in the 15-inch pipe. The normal depth of 1.5 feet will be increased by 0.27 feet at the inlet.

(8) *Inlet dike height and pipe cover.*—The inlet dike must provide the required freeboard above the inlet water surface. Thus, the height of the dike, as shown in figure 4-44, is:

$$\begin{aligned} h_d &= f_b + d_o \\ &= 2.0 + 1.77 \\ &= 3.77 \text{ ft.} \end{aligned}$$

Check to see if this dike height will provide the required minimum cover, C, over the 15-inch-diameter pipe:

$$C = \frac{2}{3} D \text{ (18 in. min.)} + 6 \text{ in. min.}$$

As $\frac{2}{3}D = \left(\frac{2}{3}\right) \times 15 = 10$ inches, and $10 + 6 = 16$ inches, the 18-inch minimum controls, and

$$\begin{aligned} C &= 18 + 6 \\ &= 24 \text{ in. minimum} \\ &\quad (2.0 \text{ ft. minimum}) \end{aligned}$$

Pipe cover, C, resulting from the required freeboard:

$$\begin{aligned} C &= h_d - D \\ &= 3.77 - 1.25 \\ &= 2.52 > 2.0 \text{ ft.} \end{aligned}$$

Therefore, the height of the dike is controlled by the freeboard requirement.

(9) *Percolation.*—Percolation from the inlet pool to the canal should be checked by the Lane method as discussed in subsection VIII C.

(10) *Protection.*—Inlet protection should consist of 6 inches of coarse gravel, extending at least 12 inches above the maximum inlet water surface, for a length of 3D (5 feet minimum). Use 5-foot length (since 3 x 1.25 is only 3.75 feet).

Outlet protection should consist of 12 inches of coarse gravel, extending 12 inches above the normal water surface of the canal, as shown in figure 4-44, and 2–1/2 D, or 3.13 feet (use 3.5 feet) upstream and downstream from the pipe centerline.

D. BIBLIOGRAPHY

4-46. *Bibliography.*

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Water Measurement Structures

H. J. WARREN[1]

A. GENERAL

5-1. *Types of Measuring Structures.*—There are many different types of water measurement structures used in irrigation systems. The types most commonly used in Reclamation systems are Parshall flumes, figures 5-1 and 5-2; weirs, figure 5-3; open-flow meters, figures 5-4 and 3-22; and constant-head orifices (CHO). Constant-head orifices are mainly used in turnouts and have been previously discussed in chapter III. Acoustic velocity meters and magnetic flow meters are expected to offer a reliable method of flow measurement in irrigation systems but they are not included herein. Parshall flumes, weirs, weir boxes and open-flow meters are discussed in this chapter. A general description of each type of structure is given, along with its advantages and disadvantages. Design examples show how a particular size of water measurement structure is determined. Standard design drawings of Reclamation measuring structures are also included.

In a typical irrigation system, water is usually measured at the storage reservoir outlet, the canal headworks, and at lateral and farm turnouts. The type of measuring structure selected for these locations depends on availability of head, adaptability to site, economy of installation, and ease of operation.

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Figure 5-1. **Flow through a 9-inch Parshall flume.**
P 20-D-29299-520

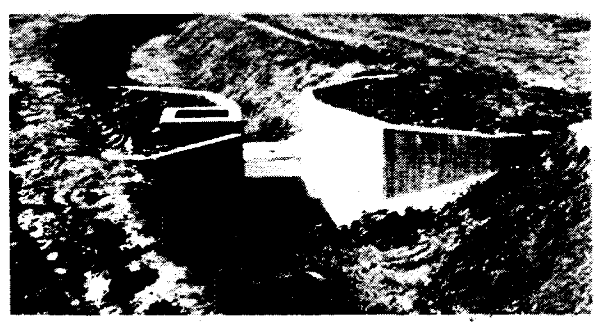

Figure 5-2. **Recently constructed Parshall flume.**
PX-D-72610

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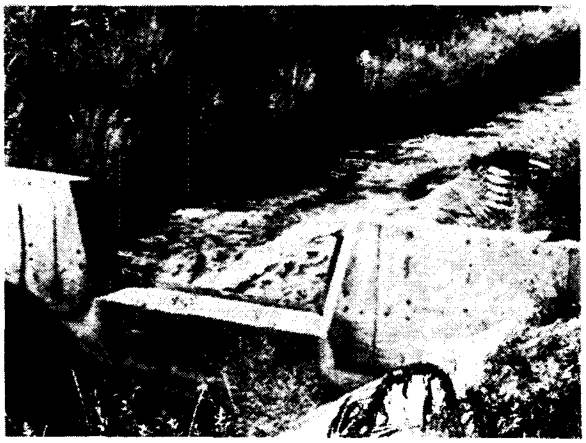

Figure 5-3. **Concrete weir structure with standard contracted Cipolletti weir. PX-D-72611**

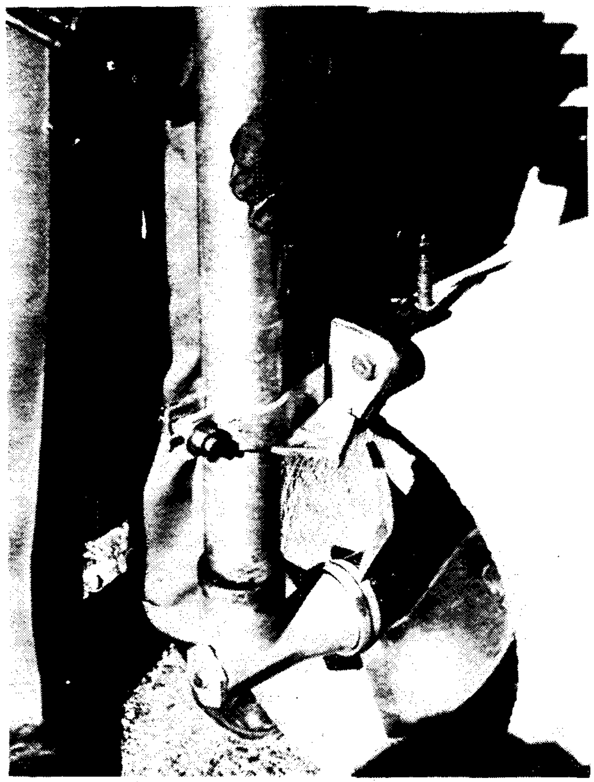

Figure 5-4. **Open-flow meter attached to a concrete pipe. 222-D-33586**

B. PARSHALL FLUMES

5-2. *Description.*—Parshall flumes are specially designed inline open channel measuring structures in which canal water flows over a broad, flat-converging section through a narrow downward sloping throat section and then diverges on an upward sloping floor (fig. 5-5).

The floor of the converging section is the crest of the flume. It is level both longitudinally and laterally and is usually set above the upstream canal invert.

The flume geometry forces the water at free-flow conditions to pass through critical depth on the crest, thereby providing a means of determining the rate of flow from a single water depth measurement. Free-flow conditions occur when the downstream canal water surface (tailwater) is low enough to have no effect on the depth of water on the crest. These conditions prevail over a wide range of tailwater depths. The tailwater elevation may be appreciably higher than the flume crest without affecting the free-flow discharge through the flume. However, if the

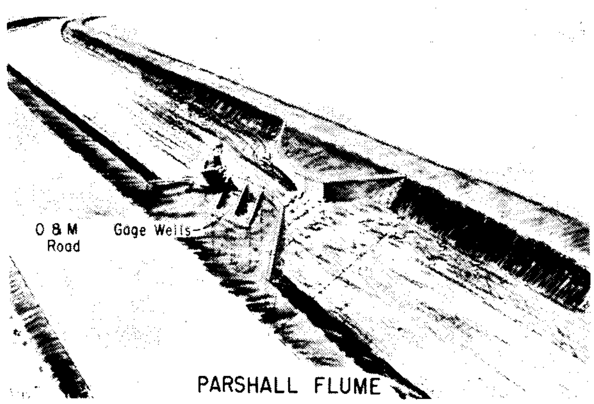

Figure 5-5. **Artist sketch showing Parshall flume in a canal. CTPX-D-31408.**

downstream water surface exceeds specified limits, submerged-flow conditions occur and two water depth measurements are required to determine the rate of flow.

Parshall flumes can be designed to measure flow from 0.01 cubic foot per second (cfs) to 3,000 cfs. This discussion will be limited to small Parshall flumes having free-flow capacities of 0.05 to 139.5 cfs. These discharges correspond to throat widths of 6 inches through 8 feet. For 6- and 9-inch flumes, the maximum degree of submergence ($\frac{H_b}{H_a}$ ratio, see fig. 5-6) for free-flow discharge is 60 percent, whereas, for flumes 1 through 8 feet in width, the maximum degree of submergence for free flow is 70 percent [1].[2] Gage zeros for both H_a and H_b are set at the flume crest elevation so that water depth measurements are depths above the crest. Correct zeroing and reading of the gages is necessary for accurate results which is usually within about 2 percent for free flows and about 5 percent for submerged flows [2].

Parshall flumes can be constructed of concrete, wood, galvanized metal or any other construction material that can be built to the given dimensions in the field or prefabricated in a shop. Care should be taken in construction so that the structure is built as closely as possible to the standard dimensions given in figure 5-7 with one exception; the wingwalls may be lengthened as required. Also, it is important to have the floor of the converging section level so that the same amount of water is passing over each increment of throat width. An angle iron is usually embedded flush with the floor and perpendicular to the flow at the downstream end of the crest to prevent crest erosion and to provide a smooth surface for setting gage zeros. Sidewalls of the throat must be parallel and vertical.

Parshall flumes should be located only in straight sections of channels where the flow is relatively smooth and uniform. They should never be located on a curve or at right angles to the canal flow, as in a turnout, unless the flow has been straightened and uniformly redistributed. Parshall flumes should be located as close as possible to canal discharge regulating gates for convenience of operation, but far enough away from the gates so that the flow is uniform and free from eddies, turbulence, and waves.

5-3. ***Advantages.***—Parshall flumes are recognized as accurate and reliable flow measuring structures and have the following advantages: (1) capable of measuring rate of flow with relatively small head loss, (2) capable of measuring a wide range of free-flow discharges with relatively high tailwater depths using a single water depth measurement, (3) capable of measuring submerged flow using two water depth measurements when the degree of submergence impedes the free-flow discharge, (4) virtually a self-cleaning structure because of flume geometry and the throat velocity, (5) can not be easily altered to obtain unallocated water, and (6) unaffected by velocity of approach, which is automatically controlled, when the flume is built to the given standard dimensions and used where the incoming flow is uniform, evenly distributed, and free of turbulence.

5-4. ***Disadvantages.***—Parshall flumes: (1) are usually more expensive to construct than weirs, (2) cannot be used in close combination with turnouts because the inflow must be uniform and the water surface relatively smooth, and (3) must be constructed carefully and accurately for satisfactory performance.

5-5. ***Size Selection.***—Parshall flume sizes are designated by their throat widths. Before a Parshall flume can be selected, the channel cross section, the range of discharges to be measured, allowable head loss through the flume, and the normal depth of the flow in the channel must be known. Minimizing construction costs is an important factor.

A particular discharge or discharge range can be measured by any one of several different size flumes. Final selection is based on the flume width which best fits the canal dimensions and hydraulic properties. As a general rule, the width of the Parshall flume should be about one-third to one-half the width of the upstream canal water surface at design discharge and normal depth. With these

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[2] Numbers in brackets refer to items in the bibliography, see 5-29.

conditions known, the selection and setting of a Parshall flume for free flow and submerged flow can be described best by design examples.

5-6. ***Design Example with Free-flow Discharge.***—In this example, assume that a Parshall flume is to be selected for use in a canal to measure flows from 4 to 40 cfs with free-flow discharge. The design capacity of the canal is 40 cfs, bottom width 6.0 feet, side slopes 1-1/2 to 1, normal depth of water in the canal 2.0 feet, width of water surface 12.0 feet and the canal bank freeboard 1.5 feet. Further assume that 0.65 foot of head is available if required for losses through the flume. From figure 5-7 "Standard Parshall Flume Dimensions," the Parshall flumes that have free-flow measuring capabilities within the desired discharge range are those with throat widths from 3.0 to 8.0 feet.

Using the general rule to determine the throat width, the size range would be 4.0 to 6.0 feet. Since the crest of the Parshall flume is set above the upstream canal invert a distance of $\frac{M}{4}$ a flume that is too narrow would cause the headwater to rise and encroach on the upstream canal freeboard. Consider for now only the size range where the depth at H_a for free-flow discharge is less than or equal to the upstream water depth.

Table 5-3 gives the following values of H_a for a discharge of 40 cfs for different flume widths that meet the criteria set forth in the preceding paragraph.

Flume size throat width, feet	Upper head H_a, feet
5	1.55
6	1.38
7	1.25
8	1.15

Allowable head loss through the Parshall flumes must also be considered. For flume sizes in this example, free-flow discharge exists until the degree of submergence exceeds 70 percent. The following head losses for a discharge of 40 cfs and a 70 percent degree of submergence are obtained from figure 5-8 (see subsection 5-8(c) for explanation on reading this chart).

Flume size throat width, feet	Head loss H_L, feet
5	0.58
6	0.49
7	0.42
8	0.37

The head loss, H_L, is the difference in water surface elevation of the headwater and tailwater as shown in figure 5-6.

The 5.0- through 8.0-foot flumes meet the depth and loss requirements; however, further investigation is required and the final selection of the flume is made on a trial and error basis. First try a 5-foot flume as it is the smallest and least expensive to build. The head loss for a 5-foot flume is 0.58 foot, therefore the tailwater elevation is set 0.58 foot lower than the headwater elevation. As noted from the dimensions in figure 5-7 the crest elevation is set $\frac{M}{4}$ above the upstream canal invert or 0.38 foot. Assume the upstream canal invert elevation is 100.00, and as previously indicated the normal depth of water in the canal is 2.0 feet, then the headwater elevation is 102.00. The depth at H_a for a free-flow discharge of 40 cfs is 1.55 feet. From the continuity equation, Q = VA where A is the cross-sectional area at H_a, the velocity at H_a is 3.85 feet per second (fps) and the velocity head, $\frac{v^2}{2g}$, is 0.23 foot. The energy at H_a is the crest elevation + H_a + $\frac{v^2}{2g}$ or (100.0 + 0.38) + 1.55 + 0.23 = 102.16 feet. If this flume were used, the headwater would rise about 0.16 foot and encroach on the upstream canal freeboard which is undesirable. Next check to see if this is a free-flow flume. Since the water surface at H_b is the same elevation as the tailwater, the degree of submergence is the ratio of $\frac{H_b}{H_a}$. Then H_b is the difference in elevation of the tailwater and the crest of the flume, or 1.04 feet. The ratio of $\frac{H_b}{H_a} = \frac{1.04}{1.55} = 67$ percent, which meets the free-flow criteria. The 6-foot flume is the next smallest, so using the same procedure try a 6-foot Parshall flume. Since the head loss through the flume is 0.49 foot, set the downstream water surface 0.49 foot below the upstream water surface. The crest elevation is the same as for the 5-foot flume, as dimension M is the same for both the 5- and 6-foot flumes. The depth at H_a for a discharge of 40 cfs is 1.38 feet. From the continuity equation the velocity at H_a is 3.70 feet per second and velocity head, $\frac{v^2}{2g}$, 0.21 foot. The energy at H_a is 100.38 + 1.38 + 0.21 = 101.97 which is slightly less than the upstream water surface elevation. When checking this flume for free flow, the degree of submergence is $\frac{H_b}{H_a} = \frac{1.13 \text{ feet}}{1.38 \text{ feet}}$ = 82 percent (should be 70 percent or less as previously determined), which does not meet the criteria for free-flow discharge. Recalling that 0.65 foot of head was initially available, the downstream water surface should be set at elevation 101.35 and the new H_b = 0.97 foot. The ratio of $\frac{H_b}{H_a}$ now equals $\frac{0.97 \text{ foot}}{1.38 \text{ feet}}$ or 70 percent which meets the criteria for free-flow conditions. The 6-foot flume should be selected because it is the least expensive to construct, can be set for free-flow conditions, and does not cause an encroachment on the upstream freeboard.

If a Parshall flume is always going to be operated at 70 percent or less submergence, that portion of the flume downstream from the throat section is not needed. If, however, the downstream channel should become silted up or clogged with vegetation, the submergence would increase to greater than 70 percent and the downstream portion of the flume with an H_b measuring well would be needed to determine the discharge for submerged flow.

5-7. ***Design Example with Submerged-flow Discharge.***—If possible Parshall flumes should be designed for free-flow discharge, however they will measure submerged flows up to 95 percent submergence. If a flume is set for this high submergence and hydraulic conditions in the downstream channel change to cause greater submergence, the flume becomes useless.

In the preceding example of the 6-foot Parshall flume assume that the flume is set for 90 percent submergence. Recall that the headwater elevation is 102.00, crest elevation is 100.38 and H_a for free-flow discharge is 1.38 feet. Then the ratio $\frac{H_b}{H_a}$ = 0.90 and H_b = 0.9 (1.38) = 1.24 feet. The tailwater surface is then tentatively set at elevation 101.62 or 1.24 feet above the crest. From figure 5-8 determine the head loss for 90 percent submergence, 40-cfs discharge and a 6-foot flume. Read a head loss of 0.17 foot. Submergence in excess of 70 percent, however, will influence the water surface at the H_a measuring well and an erroneous discharge would be read or computed using only this reading. Hence a new H_a and H_b for 90 percent submerged flow must be determined. Using a trial and error procedure, select values of H_a from table 5-3 and read the free-flow discharge for a 6-foot flume. Then from figure 5-11 read the correction for 90 percent submergence and subtract it from the free-flow discharge until an H_a is found that will give a correct reading for measuring 40 cfs through a 6-foot flume at 90 percent submergence.

An H_a of 1.58 feet is then determined to be satisfactory. Then the new H_b reading is 0.90 (1.58) = 1.42 feet. The new tailwater surface is set 1.42 feet above the crest elevation or 100.38 + 1.42 = 101.80. The flume is now set for 90 percent submergence and will measure 40 cfs with an H_a reading of 1.58 feet and an H_b reading of 1.42 feet. The actual

H_a, feet	Throat width, feet	Free-flow discharge, cfs	Submerged-flow correction (correction* times multiplying factor M)	Submerged-flow discharge, cfs
1.62	6.0	51.81	2.24 x 4.3 = 9.63	42.18
1.58	6.0	49.78	2.23 x 4.3 = 9.58	40.20
1.57	6.0	49.28	2.22 x 4.3 = 9.54	39.74

*This value is only an approximation determined from figure 5-11.

calculated difference between the headwater and tailwater elevation (102.00 − 101.80) is 0.20 foot, which is slightly greater than the 0.17-foot head loss taken from figure 5-8.

Engineers at Utah State University have developed submerged-flow calibration curves for a number of Parshall flumes. The discharge, Q, in cubic feet per second is plotted as the ordinate; ($H_a - H_b$), in feet as the abscissa; and the submergence ratio $\frac{H_b}{H_a}$ in percent as the varying parameter [3]. These curves eliminate the need for trial and error procedure as given in the example problem for submerged flow. Knowing the design discharge and the desired degree of submergence, the value of ($H_a - H_b$) can be read directly. The desired submergence ratio gives the relationship of H_b to H_a. The two simultaneous equations may be solved for H_a and H_b for a specified flow and a specified submergence. The downstream canal water surface is set above the crest an amount equal to H_b.

5-8. ***Use of Tables and Figures for Discharge and Free-flow Determinations.***—(a) *Free-flow Discharge.*—Free-flow discharge for 6-inch, 9-inch, and 1- through 8-foot Parshall flumes can be read directly from tables 5-1, 5-2, and 5-3, respectively [1]. When the flow is submerged, a correction must be applied to the free-flow discharges.

(b) *Submerged-flow Discharge.*—Figures 5-9 and 5-10 give the submerged-flow discharge directly for 6- and 9-inch flumes respectively [1].

For example, assume flow through a 6-inch flume with an H_a gage reading = 1.25 feet and H_b = 1.05 feet. The submergence ratio, $\frac{H_b}{H_a} = \frac{1.05}{1.25}$ = 84 percent. Find 84 percent submergence on the left side of figure 5-9. Proceed horizontally along the 84 percent submergence line to half way between the 1.2 and 1.3 sloping H_a lines. From this point proceed vertically downward and read a discharge of 2.25 cfs on the bottom line of the figure.

The corrections for submerged-flow discharge in 1- through 8-foot flumes are determined from figure 5-11 [1]. These discharge corrections are then subtracted from the free-flow values in table 5-3. For instance, the flow through a 6-foot flume with an H_a gage reading 1.58 feet and H_b gage reading 1.42 feet would be computed as follows: (1) compute submergence ratio, $\frac{H_b}{H_a} = \frac{1.42}{1.58} = 90$ percent. Submerged flow conditions exist because the submergence ratio is greater than 70 percent, (2) from table 5-3 the free-flow discharge for H_a = 1.58 would be 49.78 cfs. On figure 5-11 project a line horizontally to the right from 1.58 feet to the 90 percent sloping submergence line. From this point drop vertically downward to the base of the figure and read a correction of about 2.23 cfs. This correction should then be multiplied by the size or multiplying factor, M, for a 6-foot flume which is in the insert table on figure 5-11. In this case M = 4.3. The corrected discharge for the 90 percent submerged 6-foot flume is the free-flow discharge minus the correction times the size factor, or 49.78 cfs minus 2.23 (4.3) = 40.2 cfs.

(c) *Head Losses.*—Head losses for 1- through 8-foot Parshall flumes can be obtained from figure 5-8 by the following procedure: (1) for the free-flow design example find 70 percent submergence at the bottom of the chart. Follow the 70 percent submergence line vertically upward until it intersects the sloping discharge line where Q = 40 cfs, (2) from this intersection project a horizontal line to the 6-foot throat width, W, and (3) from this point go vertically downward and read the head loss of 0.49 foot.

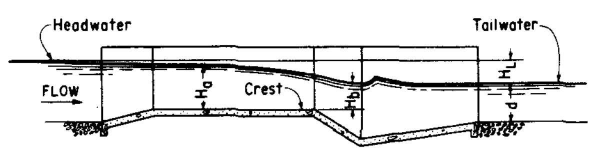
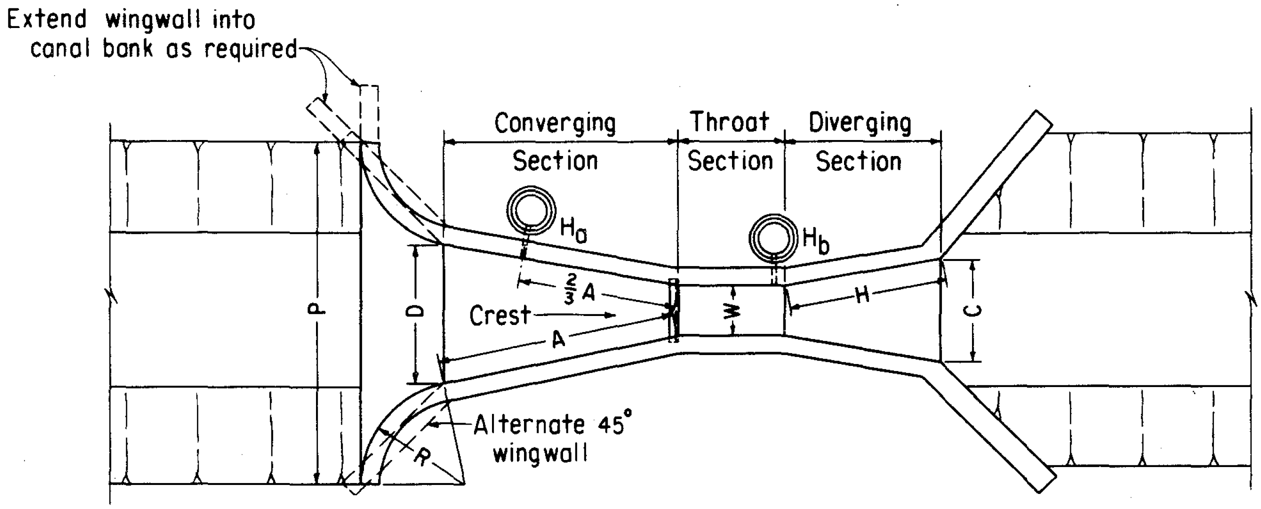
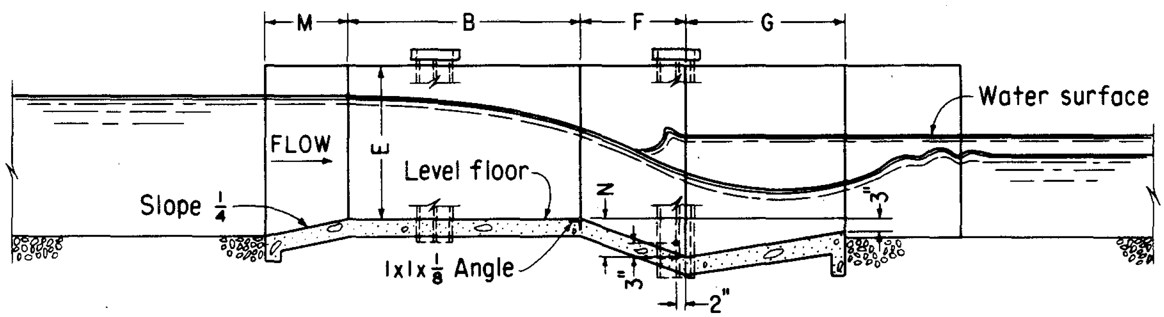


Figure 5-6. **Relationship of flow depths to the flume crest elevation. 103-D-1224**



PLAN



PROFILE

W		A		$\frac{2}{3}$A		B		C		D		E		F		G		M		N		P		R		FREE-FLOW CAPACITY MINIMUM	FREE-FLOW CAPACITY MAXIMUM
FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	CFS	CFS
0	6	2	$\frac{7}{16}$	1	$4\frac{5}{16}$	2	0	1	$3\frac{1}{2}$	1	$3\frac{5}{8}$	2	0	1	0	2	0	1	0	0	$4\frac{1}{2}$	2	$11\frac{1}{2}$	1	4	.05	3.9
	9	2	$10\frac{5}{8}$	1	$11\frac{1}{8}$	2	10	1	3	1	$10\frac{5}{8}$	2	6	1	0	1	6	1	0		$4\frac{1}{2}$	3	$6\frac{1}{2}$	1	4	.09	8.9
1	0	4	6	3	0	4	$4\frac{7}{8}$	2	0	2	$9\frac{1}{4}$	3	0	2	0	3	0	1	3		9	4	$10\frac{3}{4}$	1	8	.11	16.1
1	6	4	9	3	2	4	$7\frac{7}{8}$	2	6	3	$4\frac{3}{8}$	3	0	2	0	3	0	1	3		9	5	6	1	8	.15	24.6
2	0	5	0	3	4	4	$10\frac{7}{8}$	3	0	3	$11\frac{1}{2}$	3	0	2	0	3	0	1	3		9	6	1	1	8	.42	33.1
3	0	5	6	3	8	5	$4\frac{3}{4}$	4	0	5	$1\frac{7}{8}$	3	0	2	0	3	0	1	3		9	7	$3\frac{1}{2}$	1	8	.61	50.4
4	0	6	0	4	0	5	$10\frac{5}{8}$	5	0	6	$4\frac{1}{4}$	3	0	2	0	3	0	1	6		9	8	$10\frac{3}{4}$	2	0	1.3	67.9
5	0	6	6	4	4	6	$4\frac{1}{2}$	6	0	7	$6\frac{5}{8}$	3	0	2	0	3	0	1	6		9	10	$1\frac{1}{4}$	2	0	1.6	85.6
6	0	7	0	4	8	6	$10\frac{3}{8}$	7	0	8	9	3	0	2	0	3	0	1	6		9	11	$3\frac{1}{2}$	2	0	2.6	103.5
7	0	7	6	5	0	7	$4\frac{1}{4}$	8	0	9	$11\frac{3}{8}$	3	0	2	0	3	0	1	6		9	12	6	2	0	3.0	121.4
8	0	8	0	5	4	7	$10\frac{1}{8}$	9	0	11	$1\frac{3}{4}$	3	0	2	0	3	0	1	6		9	13	$8\frac{1}{4}$	2	0	3.5	139.5

Figure 5-7. **Standard Parshall flume dimensions. 103-D-1225**

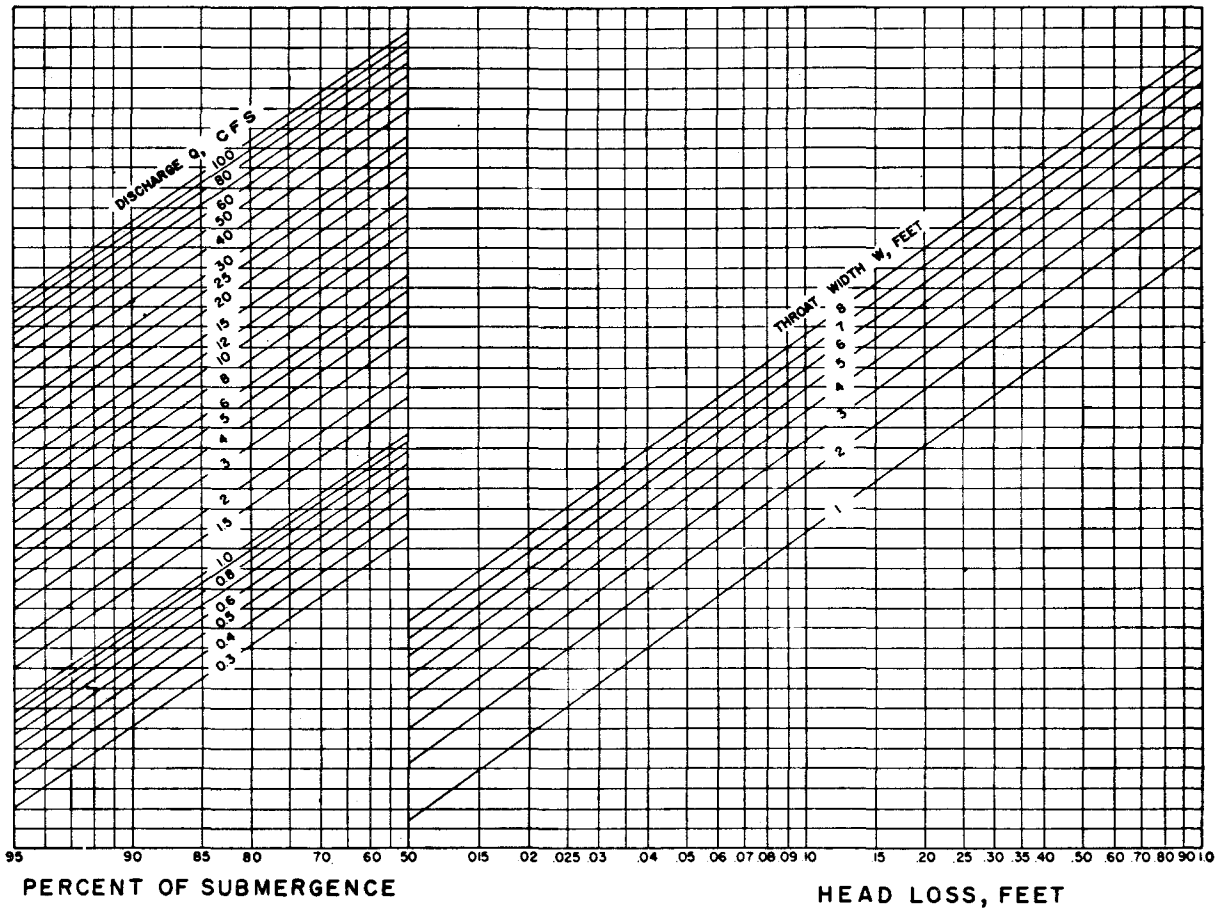


Figure 5-8. **Head loss through Parshall flumes, 1 through 8 feet wide. 103-D-1226**

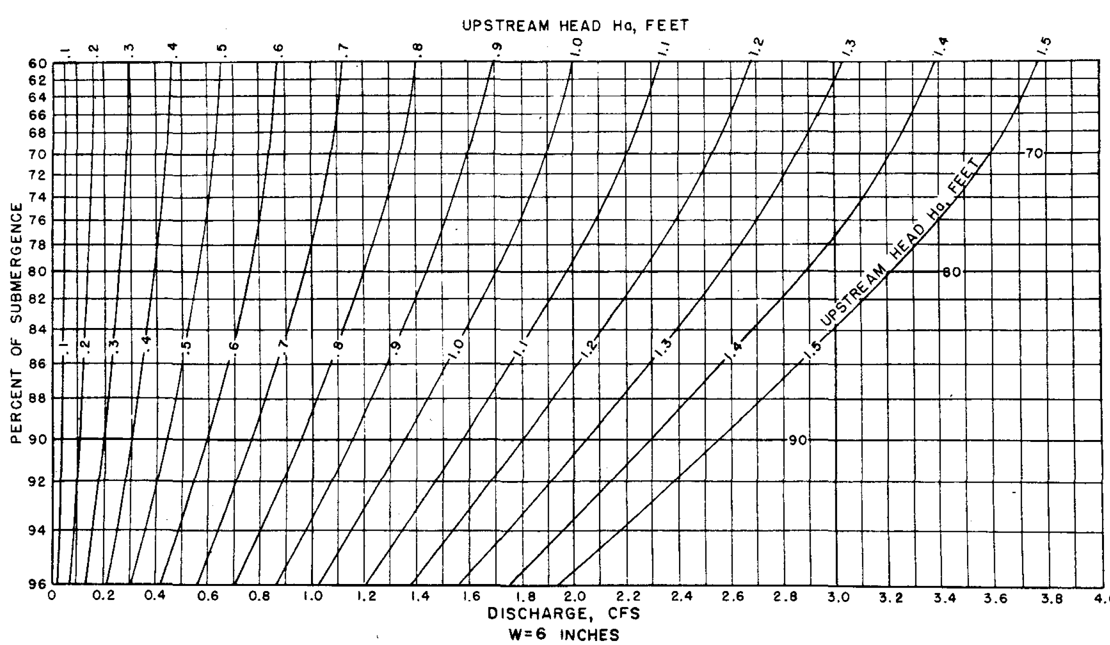


Figure 5-9. **Diagram for determining rate of submerged flow in a 6-inch Parshall flume. 103-D-1227**

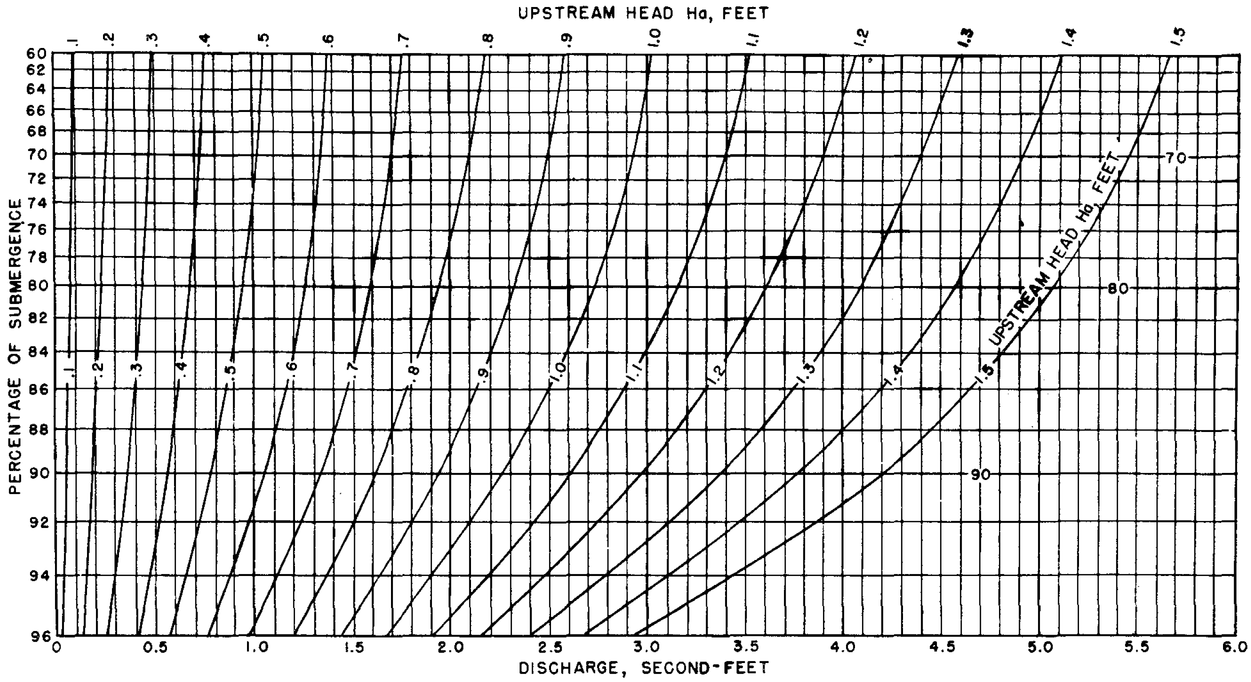
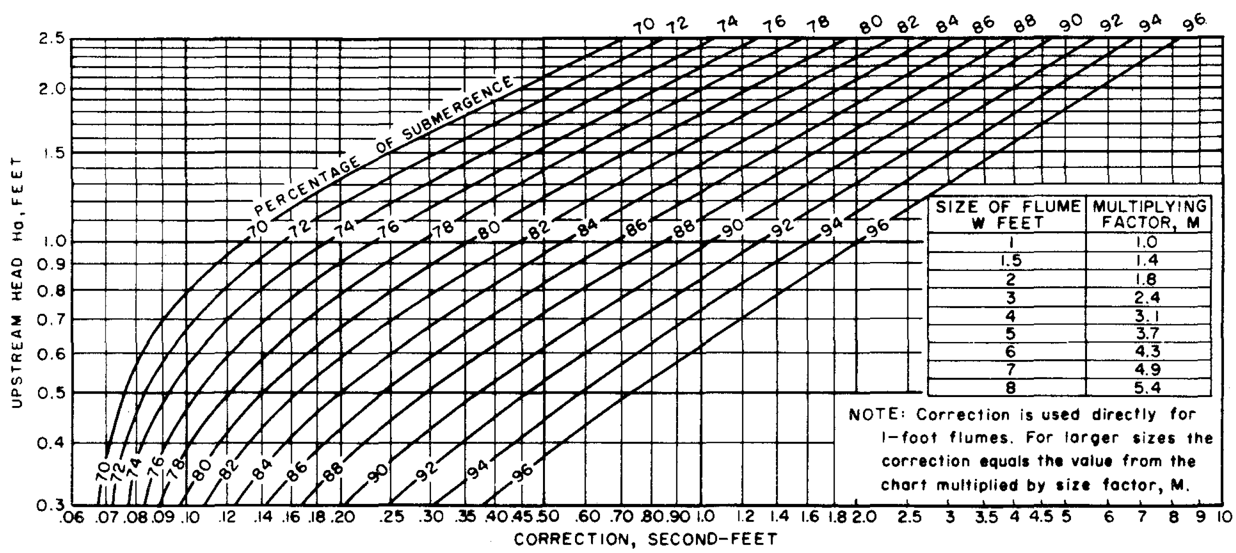


Figure 5-10. Diagram for determining rate of submerged flow in a 9-inch Parshall flume. 103-D-1228



SIZE OF FLUME W FEET	MULTIPLYING FACTOR, M
1	1.0
1.5	1.4
2	1.8
3	2.4
4	3.1
5	3.7
6	4.3
7	4.9
8	5.4

NOTE: Correction is used directly for 1-foot flumes. For larger sizes the correction equals the value from the chart multiplied by size factor, M.

Figure 5-11. Diagram for determining correction to be substracted from free-flow discharge to obtain rate of submerged-flow discharge through Parshall flumes, 1 through 8 feet wide. 103-D-1229

Table 5-1.–*Free-flow discharge through 6-inch Parshall measuring flume in cubic feet per second. Computed from the formula* $Q = 2.06\ H_a^{1.58}$. 103-D-1230

Head H_a, feet	Discharge, cfs	Head H_a, feet	Discharge, cfs	Head H_a, feet	Discharge, cfs
0.10	0.05	0.61	0.94	1.11	2.43
.11	.06	.62	.97	1.12	2.46
.12	.07	.63	.99	1.13	2.50
.13	.08	.64	1.02	1.14	2.53
.14	.09	.65	1.04	1.15	2.57
.15	.10	.66	1.07	1.16	2.60
.16	.11	.67	1.10	1.17	2.64
.17	.12	.68	1.12	1.18	2.68
.18	.14	.69	1.15	1.19	2.71
.19	.15	.70	1.17	1.20	2.75
.20	.16	.71	1.20	1.21	2.78
.21	.18	.72	1.23	1.22	2.82
.22	.19	.73	1.26	1.23	2.86
.23	.20	.74	1.28	1.24	2.89
.24	.22	.75	1.31	1.25	2.93
.25	.23	.76	1.34	1.26	2.97
.26	.25	.77	1.36	1.27	3.01
.27	.26	.78	1.39	1.28	3.04
.28	.28	.79	1.42	1.29	3.08
.29	.29	.80	1.45		
.30	.31	.81	1.48		
.31	.32	.82	1.50		
.32	.34	.83	1.53		
.33	.36	.84	1.56		
.34	.38	.85	1.59		
.35	.39	.86	1.62		
.36	.41	.87	1.65		
.37	.43	.88	1.68		
.38	.45	.89	1.71		
.39	.47	.90	1.74		
.40	.48	.91	1.77		
.41	.50	.92	1.81		
.42	.52	.93	1.84		
.43	.54	.94	1.87		
.44	.56	.95	1.90		
.45	.58	.96	1.93		
.46	.61	.97	1.97		
.47	.63	.98	2.00		
.48	.65	.99	2.03		
.49	.67	1.00	2.06		
.50	.69	1.01	2.09		
.51	.71	1.02	2.12		
.52	.73	1.03	2.16		
.53	.76	1.04	2.19		
.54	.78	1.05	2.22		
.55	.80	1.06	2.26		
.56	.82	1.07	2.29		
.57	.85	1.08	2.32		
.58	.87	1.09	2.36		
.59	.89	1.10	2.40		
.60	.92				

Table 5-2.–*Free-flow discharge through 9-inch Parshall measuring flume in cubic feet per second. Computed from the formula* $Q = 3.07\, H_a^{1.53}$. 103-D-1231

Head H_a, feet	Discharge, cfs	Head H_a, feet	Discharge, cfs	Head H_a, feet	Discharge, cfs
0.10	0.09	0.61	1.44	1.11	3.60
.11	.10	.62	1.48	1.12	3.65
.12	.12	.63	1.51	1.13	3.70
.13	.14	.64	1.55	1.14	3.75
.14	.15	.65	1.59	1.15	3.80
.15	.17	.66	1.63	1.16	3.85
.16	.19	.67	1.66	1.17	3.90
.17	.20	.68	1.70	1.18	3.95
.18	.22	.69	1.74	1.19	4.01
.19	.24	.70	1.78	1.20	4.06
.20	.26	.71	1.82	1.21	4.11
.21	.28	.72	1.86	1.22	4.16
.22	.30	.73	1.90	1.23	4.22
.23	.32	.74	1.94	1.24	4.27
.24	.35	.75	1.98	1.25	4.32
.25	.37	.76	2.02	1.26	4.37
.26	.39	.77	2.06	1.27	4.43
.27	.41	.78	2.10	1.28	4.48
.28	.44	.79	2.14	1.29	4.53
.29	.46	.80	2.18	1.30	4.59
.30	.49	.81	2.22	1.31	4.64
.31	.51	.82	2.27	1.32	4.69
.32	.54	.83	2.31	1.33	4.75
.33	.56	.84	2.35	1.34	4.80
.34	.59	.85	2.39	1.35	4.86
.35	.62	.86	2.44	1.36	4.92
.36	.64	.87	2.48	1.37	4.97
.37	.67	.88	2.52	1.38	5.03
.38	.70	.89	2.57	1.39	5.08
.39	.73	.90	2.61	1.40	5.14
.40	.76	.91	2.66	1.41	5.19
.41	.78	.92	2.70	1.42	5.25
.42	.81	.93	2.75	1.43	5.31
.43	.84	.94	2.79	1.44	5.37
.44	.87	.95	2.84	1.45	5.42
.45	.90	.96	2.88	1.46	5.48
.46	.94	.97	2.93	1.47	5.54
.47	.97	.98	2.98	1.48	5.59
.48	1.00	.99	3.02	1.49	5.65
.49	1.03	1.00	3.07	1.50	5.71
.50	1.06	1.01	3.12	1.51	5.77
.51	1.10	1.02	3.17	1.52	5.83
.52	1.13	1.03	3.21	1.53	5.89
.53	1.16	1.04	3.26	1.54	5.94
.54	1.20	1.05	3.31	1.55	6.00
.55	1.23	1.06	3.36	1.56	6.06
.56	1.26	1.07	3.40	1.57	6.12
.57	1.30	1.08	3.45	1.58	6.18
.58	1.33	1.09	3.50	1.59	6.24
.59	1.37	1.10	3.55		
.60	1.40				

Table 5-3.—*Free-flow discharge through Parshall measuring flumes, 1- through 8-foot size, in cubic feet per second. Computed from the formula* $Q = 4WH_a^{1.522W^{0.026}}$. 103-D-1232-1

Head H_a, feet	Width of throat, W, feet							
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
0.20	0.35	0.66	0.97	1.26				
.21	.37	.71	1.04	1.36				
.22	.40	.77	1.12	1.47				
.23	.43	.82	1.20	1.58				
.24	.46	.88	1.28	1.69				
.25	.49	.93	1.37	1.80	2.22	2.63		
.26	.51	.99	1.46	1.91	2.36	2.80		
.27	.54	1.05	1.55	2.03	2.50	2.97		
.28	.58	1.11	1.64	2.15	2.65	3.15		
.29	.61	1.18	1.73	2.27	2.80	3.33		
.30	.64	1.24	1.82	2.39	2.96	3.52	4.08	4.62
.31	.68	1.30	1.92	2.52	3.12	3.71	4.30	4.88
.32	.71	1.37	2.02	2.65	3.28	3.90	4.52	5.13
.33	.74	1.44	2.12	2.78	3.44	4.10	4.75	5.39
.34	.77	1.50	2.22	2.92	3.61	4.30	4.98	5.66
.35	.80	1.57	2.32	3.06	3.78	4.50	5.22	5.93
.36	.84	1.64	2.42	3.20	3.95	4.71	5.46	6.20
.37	.88	1.72	2.53	3.34	4.13	4.92	5.70	6.48
.38	.92	1.79	2.64	3.48	4.31	5.13	5.95	6.74
.39	.95	1.86	2.75	3.62	4.49	5.35	6.20	7.05
.40	.99	1.93	2.86	3.77	4.68	5.57	6.46	7.34
.41	1.03	2.01	2.97	3.92	4.86	5.80	6.72	7.64
.42	1.07	2.09	3.08	4.07	5.05	6.02	6.98	7.94
.43	1.11	2.16	3.20	4.22	5.24	6.25	7.25	8.24
.44	1.15	2.24	3.32	4.38	5.43	6.48	7.52	8.55
.45	1.19	2.32	3.44	4.54	5.63	6.72	7.80	8.87
.46	1.23	2.40	3.56	4.70	5.83	6.96	8.08	9.19
.47	1.27	2.48	3.68	4.86	6.03	7.20	8.36	9.51
.48	1.31	2.57	3.80	5.03	6.24	7.44	8.65	9.84
.49	1.35	2.65	3.92	5.20	6.45	7.69	8.94	10.17
.50	1.39	2.73	4.05	5.36	6.66	7.94	9.23	10.51
.51	1.44	2.82	4.18	5.53	6.87	8.20	9.53	10.85
.52	1.48	2.90	4.31	5.70	7.09	8.46	9.83	11.19
.53	1.52	2.99	4.44	5.88	7.30	8.72	10.14	11.54
.54	1.57	3.08	4.57	6.05	7.52	8.98	10.45	11.89
.55	1.62	3.17	4.70	6.23	7.74	9.25	10.76	12.24
.56	1.66	3.26	4.84	6.41	7.97	9.52	11.07	12.60
.57	1.70	3.35	4.98	6.59	8.20	9.79	11.39	12.96
.58	1.75	3.44	5.11	6.77	8.43	10.07	11.71	13.33
.59	1.80	3.53	5.25	6.96	8.66	10.35	12.03	13.70
.60	1.84	3.62	5.39	7.15	8.89	10.63	12.36	14.08
.61	1.88	3.72	5.53	7.34	9.13	10.92	12.69	14.46
.62	1.93	3.81	5.68	7.53	9.37	11.20	13.02	14.84
.63	1.98	3.91	5.82	7.72	9.61	11.49	13.36	15.23
.64	2.03	4.01	5.97	7.91	9.85	11.78	13.70	15.62
.65	2.08	4.11	6.12	8.11	10.10	12.08	14.05	16.01
.66	2.13	4.20	6.26	8.31	10.34	12.38	14.40	16.41
.67	2.18	4.30	6.41	8.51	10.59	12.68	14.75	16.81
.68	2.23	4.40	6.56	8.71	10.85	12.98	15.10	17.22
.69	2.28	4.50	6.71	8.91	11.10	13.28	15.46	17.63
.70	2.33	4.60	6.86	9.11	11.36	13.59	15.82	18.04

Table 5-3.–*Free-flow discharge through Parshall measuring flumes, 1- through 8-foot size, in cubic feet per second. Computed from the formula* $Q = 4WH_a^{1.522W^{0.026}}$ –Continued. 103-D-1232-2

Head H_a, feet	Width of throat, W, feet							
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
0.71	2.38	4.70	7.02	9.32	11.62	13.90	16.18	18.45
.72	2.43	4.81	7.17	9.53	11.88	14.22	16.55	18.87
.73	2.48	4.91	7.33	9.74	12.14	14.53	16.92	19.29
.74	2.53	5.02	7.49	9.95	12.40	14.85	17.29	19.71
.75	2.58	5.12	7.65	10.16	12.67	15.17	17.66	20.14
.76	2.63	5.23	7.81	10.38	12.94	15.49	18.04	20.57
.77	2.68	5.34	7.97	10.60	13.21	15.82	18.42	21.01
.78	2.74	5.44	8.13	10.81	13.48	16.15	18.81	21.46
.79	2.80	5.55	8.30	11.03	13.76	16.48	19.20	21.91
.80	2.85	5.66	8.46	11.25	14.04	16.81	19.59	22.36
.81	2.90	5.77	8.63	11.48	14.32	17.15	19.99	22.81
.82	2.96	5.88	8.79	11.70	14.60	17.49	20.39	23.26
.83	3.02	6.00	8.96	11.92	14.88	17.83	20.79	23.72
.84	3.07	6.11	9.13	12.15	15.17	18.17	21.18	24.18
.85	3.12	6.22	9.30	12.38	15.46	18.52	21.58	24.64
.86	3.18	6.33	9.48	12.61	15.75	18.87	21.99	25.11
.87	3.24	6.44	9.65	12.84	16.04	19.22	22.40	25.58
.88	3.29	6.56	9.82	13.07	16.33	19.57	22.82	26.06
.89	3.35	6.68	10.00	13.31	16.62	19.93	23.24	26.54
.90	3.41	6.80	10.17	13.55	16.92	20.29	23.66	27.02
.91	3.46	6.92	10.35	13.79	17.22	20.65	24.08	27.50
.92	3.52	7.03	10.53	14.03	17.52	21.01	24.50	27.99
.93	3.58	7.15	10.71	14.27	17.82	21.38	24.93	28.48
.94	3.64	7.27	10.89	14.51	18.13	21.75	25.36	28.97
.95	3.70	7.39	11.07	14.76	18.44	22.12	25.79	29.47
.96	3.76	7.51	11.26	15.00	18.75	22.49	26.22	29.97
.97	3.82	7.63	11.44	15.25	19.06	22.86	26.66	30.48
.98	3.88	7.75	11.63	15.50	19.37	23.24	27.10	30.98
.99	3.94	7.88	11.82	15.75	19.68	23.62	27.55	31.49
1.00	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00
1.01	4.06	8.12	12.19	16.25	20.32	24.38	28.45	32.52
1.02	4.12	8.25	12.38	16.51	20.64	24.77	28.90	33.04
1.03	4.18	8.38	12.57	16.76	20.96	25.16	29.36	33.56
1.04	4.25	8.50	12.76	17.02	21.28	25.55	29.82	34.08
1.05	4.31	8.63	12.96	17.28	21.61	25.94	30.28	34.61
1.06	4.37	8.76	13.15	17.54	21.94	26.34	30.74	35.14
1.07	4.43	8.88	13.34	17.80	22.27	26.74	31.20	35.68
1.08	4.50	9.01	13.54	18.07	22.60	27.13	31.67	36.22
1.09	4.56	9.14	13.74	18.34	22.93	27.53	32.14	36.76
1.10	4.62	9.27	13.93	18.60	23.26	27.94	32.62	37.30
1.11	4.68	9.40	14.13	18.86	23.60	28.35	33.10	37.84
1.12	4.75	9.54	14.33	19.13	23.94	28.76	33.58	38.39
1.13	4.82	9.67	14.53	19.40	24.28	29.17	34.06	38.94
1.14	4.88	9.80	14.73	19.67	24.62	29.58	34.54	39.50
1.15	4.94	9.94	14.94	19.94	24.96	30.00	35.02	40.06
1.16	5.01	10.07	15.14	20.22	25.31	30.41	35.51	40.62
1.17	5.08	10.20	15.34	20.50	25.66	30.83	36.00	41.18
1.18	5.15	10.34	15.55	20.78	26.01	31.25	36.50	41.75
1.19	5.21	10.48	15.76	21.05	26.36	31.68	37.00	42.32
1.20	5.28	10.61	15.96	21.33	26.71	32.10	37.50	42.89
1.21	5.34	10.75	16.17	21.61	27.06	32.53	38.00	43.47

Table 5-3.–*Free-flow discharge through Parshall measuring flumes, 1- through 8-foot size, in cubic feet per second. Computed from the formula* $Q = 4WH_a^{1.522W^{0.026}}$ –Continued. 103-D-1232-3

Head H_a, feet	Width of throat, W, feet							
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
1.22	5.41	10.89	16.38	21.90	27.42	32.96	38.50	44.05
1.23	5.48	11.03	16.60	22.18	27.78	33.39	39.00	44.64
1.24	5.55	11.17	16.81	22.47	28.14	33.82	39.51	45.22
1.25	5.62	11.31	17.02	22.75	28.50	34.26	40.02	45.80
1.26	5.69	11.45	17.23	23.04	28.86	34.70	40.54	46.38
1.27	5.76	11.59	17.44	23.33	29.22	35.14	41.05	46.97
1.28	5.82	11.73	17.66	23.62	29.59	35.58	41.57	47.57
1.29	5.89	11.87	17.88	23.92	29.96	36.02	42.09	48.17
1.30	5.96	12.01	18.10	24.21	30.33	36.47	42.62	48.78
1.31	6.03	12.16	18.32	24.50	30.70	36.92	43.14	49.38
1.32	6.10	12.30	18.54	24.80	31.07	37.37	43.67	49.99
1.33	6.18	12.44	18.76	25.10	31.44	37.82	44.20	50.60
1.34	6.25	12.59	18.98	25.39	31.82	38.28	44.73	51.22
1.35	6.32	12.74	19.20	25.69	32.20	38.74	45.26	51.84
1.36	6.39	12.89	19.42	25.99	32.58	39.20	45.80	52.46
1.37	6.46	13.03	19.64	26.30	32.96	39.66	46.35	53.08
1.38	6.53	13.18	19.87	26.60	33.34	40.12	46.89	53.70
1.39	6.60	13.33	20.10	26.90	33.72	40.58	47.44	54.33
1.40	6.68	13.48	20.32	27.21	34.11	41.05	47.99	54.95
1.41	6.75	13.63	20.55	27.52	34.50	41.52	48.54	55.58
1.42	6.82	13.78	20.78	27.82	34.89	41.99	49.09	56.22
1.43	6.89	13.93	21.01	28.14	35.28	42.46	49.64	56.86
1.44	6.97	14.08	21.24	28.45	35.67	42.94	50.20	57.50
1.45	7.04	14.23	21.47	28.76	36.06	43.42	50.76	58.14
1.46	7.12	14.38	21.70	29.07	36.46	43.89	51.32	58.78
1.47	7.19	14.54	21.94	29.38	36.86	44.37	51.88	59.43
1.48	7.26	14.69	22.17	29.70	37.26	44.85	52.45	60.08
1.49	7.34	14.85	22.41	30.02	37.66	45.34	53.02	60.74
1.50	7.41	15.00	22.64	30.34	38.06	45.82	53.59	61.40
1.51	7.49	15.16	22.88	30.66	38.46	46.31	54.16	62.06
1.52	7.57	15.31	23.12	30.98	38.87	46.80	54.74	62.72
1.53	7.64	15.47	23.36	31.30	39.28	47.30	55.32	63.38
1.54	7.72	15.62	23.60	31.63	39.68	47.79	55.90	64.04
1.55	7.80	15.78	23.84	31.95	40.09	48.28	56.48	64.71
1.56	7.87	15.94	24.08	32.27	40.51	48.78	57.06	65.38
1.57	7.95	16.10	24.32	32.60	40.92	49.28	57.65	66.06
1.58	8.02	16.26	24.56	32.93	41.33	49.78	58.24	66.74
1.59	8.10	16.42	24.80	33.26	41.75	50.28	58.83	67.42
1.60	8.18	16.58	25.05	33.59	42.17	50.79	59.42	68.10
1.61	8.26	16.74	25.30	33.92	42.59	51.30	60.02	68.79
1.62	8.34	16.90	25.54	34.26	43.01	51.81	60.62	69.48
1.63	8.42	17.06	25.79	34.60	43.43	52.32	61.22	70.17
1.64	8.49	17.22	26.04	34.93	43.86	52.83	61.82	70.86
1.65	8.57	17.38	26.29	35.26	44.28	53.34	62.42	71.56
1.66	8.65	17.55	26.54	35.60	44.70	53.86	63.03	72.26
1.67	8.73	17.72	26.79	35.94	45.13	54.38	63.64	72.96
1.68	8.81	17.88	27.04	36.28	45.56	54.90	64.25	73.66
1.69	8.89	18.04	27.30	36.62	46.00	55.42	64.86	74.37
1.70	8.97	18.21	27.55	36.96	46.43	55.95	65.48	75.08
1.71	9.05	18.38	27.80	37.30	46.86	56.48	66.10	75.79
1.72	9.13	18.54	28.06	37.65	47.30	57.00	66.72	76.50

Table 5-3.–*Free-flow discharge through Parshall measuring flumes, 1- through 8-foot size, in cubic feet per second. Computed from the formula* $Q = 4WH_a^{1.522W^{0.026}}$ –Continued. 103-D-1232-4

Head H_a, feet	Width of throat, W, feet							
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
1.73	9.21	18.71	28.32	38.00	47.74	57.53	67.34	77.22
1.74	9.29	18.88	28.57	38.34	48.17	58.06	67.96	77.94
1.75	9.38	19.04	28.82	38.69	48.61	58.60	68.59	78.66
1.76	9.46	19.21	29.08	39.04	49.05	59.13	69.22	79.38
1.77	9.54	19.38	29.34	39.39	49.50	59.67	69.85	80.10
1.78	9.62	19.55	29.60	39.74	49.94	60.20	70.48	80.83
1.79	9.70	19.72	29.87	40.10	50.38	60.74	71.11	81.56
1.80	9.79	19.90	30.13	40.45	50.83	61.29	71.75	82.29
1.81	9.87	20.07	30.39	40.80	51.28	61.83	72.39	83.03
1.82	9.95	20.24	30.65	41.16	51.73	62.38	73.03	83.77
1.83	10.04	20.42	30.92	41.52	52.18	62.92	73.68	84.51
1.84	10.12	20.59	31.18	41.88	52.64	63.46	74.33	85.25
1.85	10.20	20.76	31.45	42.24	53.09	64.01	74.98	86.00
1.86	10.29	20.93	31.71	42.60	53.55	64.57	75.63	86.75
1.87	10.38	21.10	31.98	42.96	54.00	65.13	76.28	87.50
1.88	10.46	21.28	32.25	43.32	54.46	65.69	76.93	88.25
1.89	10.54	21.46	32.52	43.69	54.92	66.25	77.58	89.00
1.90	10.62	21.63	32.79	44.05	55.39	66.81	78.24	89.76
1.91	10.71	21.81	33.06	44.42	55.85	67.37	78.90	90.52
1.92	10.80	21.99	33.33	44.79	56.32	67.93	79.56	91.29
1.93	10.88	22.17	33.60	45.16	56.78	68.50	80.23	92.05
1.94	10.97	22.35	33.87	45.53	57.25	69.06	80.90	92.82
1.95	11.06	22.53	34.14	45.90	57.72	69.63	81.57	93.59
1.96	11.14	22.70	34.42	46.27	58.19	70.20	82.24	94.36
1.97	11.23	22.88	34.70	46.64	58.67	70.78	82.91	95.14
1.98	11.31	23.06	34.97	47.02	59.14	71.35	83.58	95.92
1.99	11.40	23.24	35.25	47.40	59.61	71.92	84.26	96.70
2.00	11.49	23.43	35.53	47.77	60.08	72.50	84.94	97.48
2.01	11.58	23.61	35.81	48.14	60.56	73.08	85.62	98.26
2.02	11.66	23.79	36.09	48.52	61.04	73.66	86.30	99.05
2.03	11.75	23.98	36.37	48.90	61.52	74.24	86.99	99.84
2.04	11.84	24.16	36.65	49.29	62.00	74.83	87.68	100.6
2.05	11.93	24.34	36.94	49.67	62.48	75.42	88.37	101.4
2.06	12.02	24.52	37.22	50.05	62.97	76.00	89.06	102.2
2.07	12.10	24.70	37.50	50.44	63.46	76.59	89.75	103.0
2.08	12.19	24.89	37.78	50.82	63.94	77.19	90.44	103.8
2.09	12.28	25.08	38.06	51.21	64.43	77.78	91.14	104.6
2.10	12.37	25.27	38.35	51.59	64.92	78.37	91.84	105.4
2.11	12.46	25.46	38.64	51.98	65.41	78.97	92.54	106.2
2.12	12.55	25.64	38.93	52.37	65.91	79.56	93.25	107.0
2.13	12.64	25.83	39.22	52.76	66.40	80.15	93.95	107.9
2.14	12.73	26.01	39.50	53.15	66.89	80.75	94.66	108.7
2.15	12.82	26.20	39.79	53.54	67.39	81.36	95.37	109.5
2.16	12.92	26.39	40.08	53.94	67.89	81.97	96.08	110.3
2.17	13.01	26.58	40.37	54.34	68.39	82.58	96.79	111.1
2.18	13.10	26.77	40.66	54.73	68.89	83.19	97.51	111.9
2.19	13.19	26.96	40.96	55.12	69.39	83.80	98.23	112.8
2.20	13.28	27.15	41.25	55.52	69.90	84.41	98.94	113.6
2.21	13.37	27.34	41.54	55.92	70.40	85.02	99.66	114.4
2.22	13.46	27.54	41.84	56.32	70.90	85.63	100.4	115.3
2.23	13.56	27.73	42.13	56.72	71.41	86.25	101.1	116.1

Table 5-3.–*Free-flow discharge through Parshall measuring flumes, 1- through 8-foot size, in cubic feet per second. Computed from the formula* $Q = 4WH_a^{1.522W^{0.026}}$ –Continued. 103-D-1232-5

Head H_a, feet	Width of throat, W, feet							
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
2.24	13.65	27.92	42.43	57.12	71.92	86.87	101.8	116.9
2.25	13.74	28.12	42.73	57.52	72.43	87.49	102.6	117.8
2.26	13.84	28.31	43.02	57.93	72.94	88.11	103.3	118.6
2.27	13.93	28.50	43.32	58.34	73.46	88.73	104.0	119.5
2.28	14.02	28.70	43.62	58.74	73.97	89.35	104.8	120.3
2.29	14.12	28.90	43.92	59.15	74.49	89.98	105.5	121.2
2.30	14.21	29.09	44.22	59.56	75.01	90.61	106.2	122.0
2.31	14.30	29.29	44.52	59.96	75.53	91.24	107.0	122.9
2.32	14.40	29.49	44.83	60.37	76.05	91.87	107.7	123.7
2.33	14.49	29.69	45.13	60.79	76.57	92.50	108.5	124.6
2.34	14.59	29.89	45.43	61.20	77.09	93.14	109.2	125.4
2.35	14.68	30.08	45.74	61.61	77.61	93.77	110.0	126.3
2.36	14.78	30.28	46.04	62.03	78.13	94.41	110.7	127.2
2.37	14.87	30.48	46.35	62.44	78.66	95.05	111.5	128.0
2.38	14.97	30.69	46.66	62.86	79.19	95.69	112.2	128.9
2.39	15.07	30.89	46.96	63.27	79.72	96.33	113.0	129.8
2.40	15.16	31.09	47.27	63.69	80.25	96.97	113.7	130.7
2.41	15.26	31.29	47.58	64.11	80.78	97.62	114.5	131.5
2.42	15.35	31.49	47.89	64.53	81.31	98.27	115.3	132.4
2.43	15.45	31.69	48.20	64.95	81.84	98.91	116.0	133.3
2.44	15.55	31.89	48.51	65.38	82.38	99.56	116.8	134.2
2.45	15.64	32.10	48.82	65.80	82.92	100.2	117.6	135.1
2.46	15.74	32.30	49.13	66.23	83.45	100.9	118.3	135.9
2.47	15.89	32.50	49.45	66.65	83.99	101.5	119.1	136.8
2.48	15.94	32.70	49.76	67.07	84.53	102.2	119.9	137.7
2.49	16.03	32.90	50.08	67.50	85.07	102.8	120.6	138.6
2.50	16.13	33.11	50.39	67.93	85.62	103.5	121.4	139.5

C. MODIFIED PARSHALL FLUMES

5-9. *Description.*—Some of the Parshall flume dimensions shown in figure 5-7 are modified so that the flume fits the canal profile.

Generally, only the flumes operating at free-flow conditions are modified, and the major modifications are made downstream of the throat section. All critical factors which can significantly affect accurate measurement such as dimensions for the crest, throat, and location of the H_a pressure tap are constructed to the standard dimensions given in figure 5-7 [1]. For instance, a Parshall flume, chute, and stilling basin may be combined into one structure as shown in figure 5-12. As previously stated, for free-flow measurement, that portion of the flume downstream of the throat is not required. Therefore, the downward sloping floor of the throat section can be extended as a rectangular chute into a stilling pool without affecting the flow measurement accuracy and the standard free-flow discharge tables may be used.

If modifications are made for submerged flow (greater than 60 percent submergence for 6- and 9-inch flumes, and 70 percent for 1- through 8-foot flumes), then the flow should be calibrated with a current meter or some other suitable device to determine the effects of the modifications.

5-10. ***Determination of Upstream Canal Water Surface.***—The upstream canal water surface required for a modified, free-flow Parshall flume should be determined as follows: (1) from the given discharge and selected width of flume, read the H_a value from the appropriate Parshall flume tables 5-1, 5-2 or 5-3, (2) determine the width of the converging section at the H_a measuring point. Using this width and the depth, H_a, compute the velocity and velocity head in the flume. The energy is then the depth, H_a, plus the velocity head, h_{v_a}, (3) neglecting losses, the energy elevation of the upstream canal water will be the same elevation as the energy at H_a. The canal water surface is obtained by subtracting the velocity head in the canal from the energy elevation.

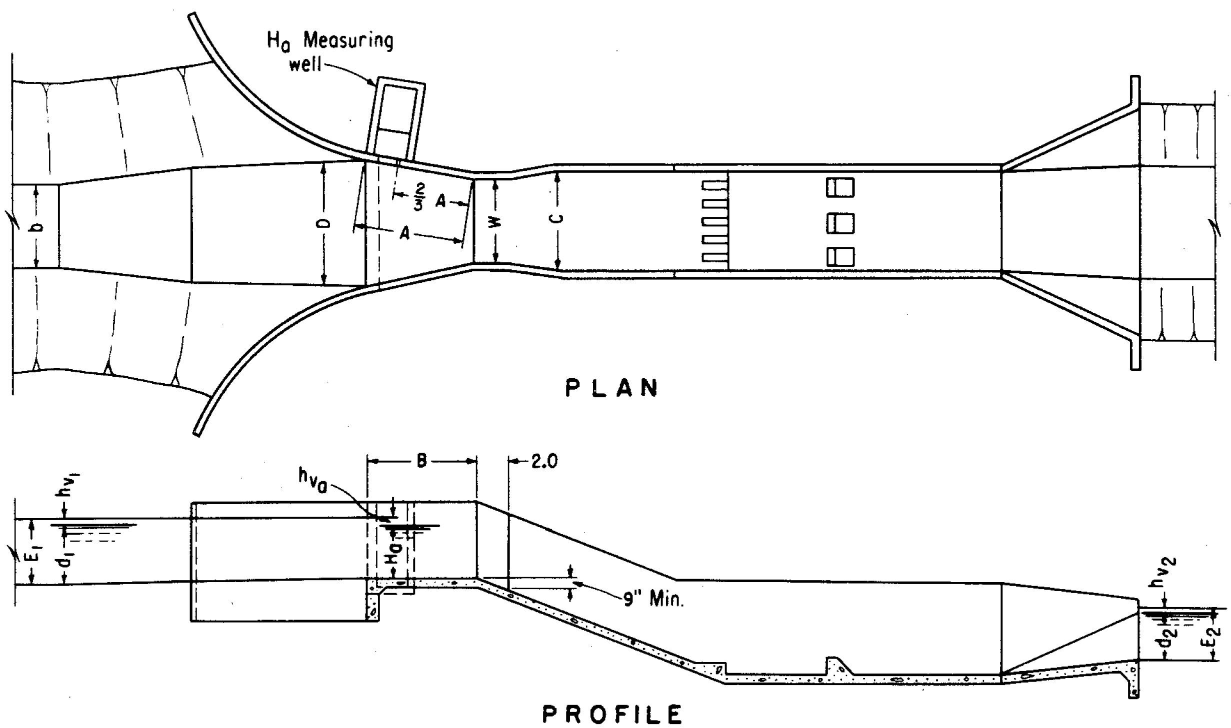


Figure 5-12. **Modified Parshall flume. 103-D-1233**

D. STILLING WELLS

5-11. *Description.*—Stilling wells are used in combination with Parshall flumes, figure 5-2; weirs, figure 5-3; and constant head orifices, figure 3-24, to permit more accurate reading of gages. They provide a water surface essentially free from surface fluctuations.

Stilling wells are connected to the measuring structures by small pipes and provide a place for the installation of staff gages, hook gages, float gage recorders or any other type of device suitable for measuring water surface levels. The water surface in the stilling well is essentially the same elevation as in the measuring structure at the pressure taps.

5-12. *Design Criteria.*—Gages for Parshall flumes and weirs should be set so they show the depth of the water above the crest and not above the pressure openings.

By restricting the area of inlet port to approximately $\frac{1}{1000}$ [1] of the inside horizontal cross-sectional area of the well, water surface in the measuring structure is effectively dampened. The inlet area of the conduit should be increased if the stilling well is offset from the channel by more than 20 to 30 feet.

Some dimensions [1] for stilling wells and their inlet pipe diameters are given in the following tabulation:

Float well dimensions	Diameter of inlet port, inches	Diameter of inlet conduit, 20 to 30 feet long, inches
12-inch diameter	1/2	1/2
16-inch diameter	1/2	3/4
20-inch diameter	5/8	3/4
24-inch diameter	3/4	1
30-inch diameter	1	1-1/2
36-inch diameter	1-1/4	2
3 feet by 3 feet square	1-1/4	2
3 feet by 4 feet rectangular	1-1/2	3
4 feet by 5 feet rectangular	2	4

Stilling wells can be constructed monolithically with a measuring structure or placed adjacent to it. In either case, since the primary purpose of the stilling well is to provide a smooth water surface, the structure must be well anchored to prevent movements that would otherwise cause undesirable water surface fluctuations.

Intake pipes to stilling wells should be cleaned occasionally either by hand or by some type of flushing system. Sediment or other foreign material lodged in the pipe could cause improper water levels to be transmitted to the stilling well. However, permanent flushing systems usually are not warranted except where the water normally carries a significant suspended sediment load.

E. WEIRS

5-13. *Purpose and Description.*—Weirs are overflow structures built across open channels to measure the rate of flow of water. They have been used for many years and offer a simple, reliable method for water measurement if they are built correctly and maintained properly. Weirs discussed herein are those that will measure flows from about 1 to 100 cfs.

5-14. *Types of Weirs.*—Weirs are identified by the shape of their openings. These openings can be either sharp-crested or broad-crested. Those most frequently used for water measurement and discussed herein are sharp-crested rectangular, trapezoidal or Cipolletti, and triangular or 90° V-notch weirs shown in figure 5-13.

Weirs also may be designated as suppressed or contracted. depending on whether or not the sides of the weir are coincident with the sides of the approach channel. If the sides of a rectangular weir are coincident with the sides of the approach channel and extend downstream from the weir, the sheet of water (nappe) leaving the weir crest does not contract laterally, thus the end contractions are suppressed and weir is called a suppressed weir (fig. 5-14). However, if the sides and crest of a rectangular, trapezoidal, or V-notch weir are far enough away from the sides and bottom of the approach channel, the nappe will fully

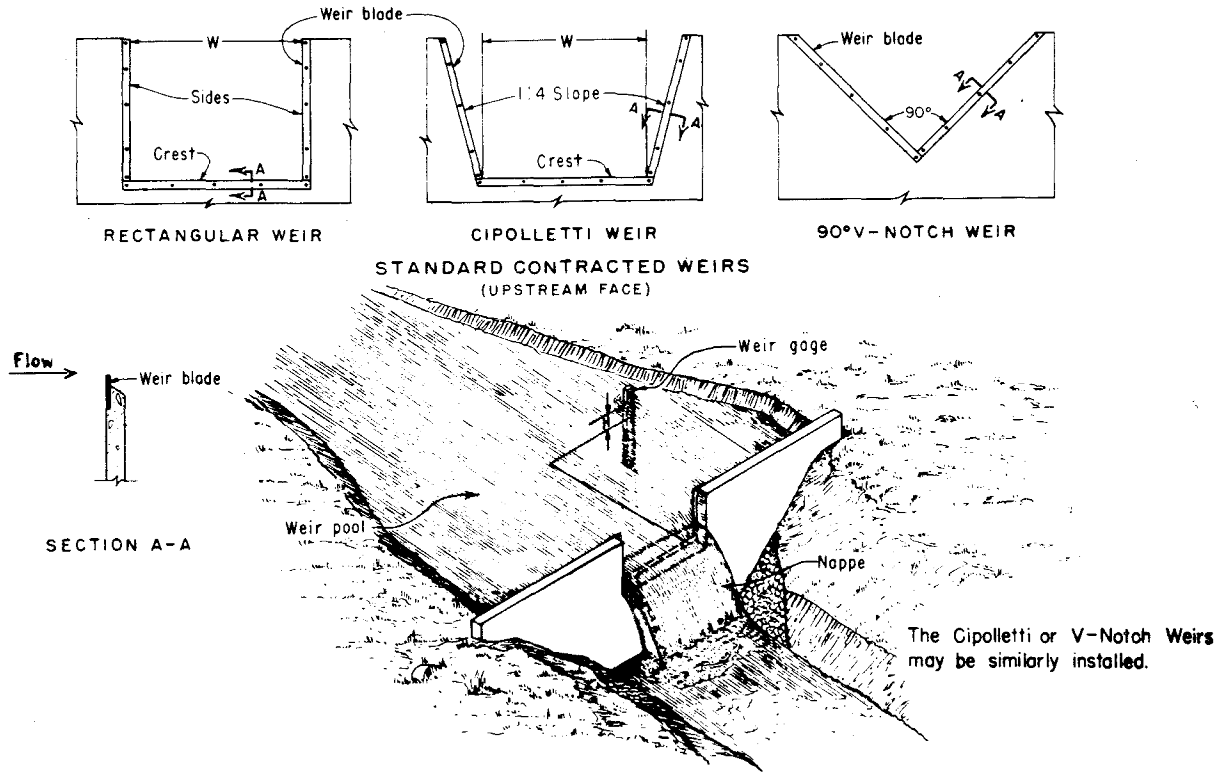


Figure 5-13. **Standard contracted weir discharging at free flow. 103-D-1234**

contract laterally at the ends and vertically at the crest of the weir. When these conditions occur, the weir is called a contracted weir. In this case water flows slowly and uniformly in its lateral approach to the weir ends. As the water nears the weir it accelerates and turns to pass through the opening. When the water turns, the streamlines contract and the water springs free (fig. 5-15) from the weir ends to form a jet narrower than the weir opening [1]. Similarly, streamlines from the bottom spring free from the crest. A contracted Cipolletti weir is shown in figure 5-16.

If the weirs and weir pools are built to specified shapes and definite dimensions as shown in figures 5-17 and 5-18, standard conditions exist and the weirs are classified as standard weirs. Standard type weirs used by Reclamation are standard contracted and standard suppressed rectangular weirs, standard Cipolletti weirs, and 90° V-notch weirs.

5-15. *Types of Flow.*—All weirs may be classified as free flow or submerged flow. Free-flow conditions exist when the water surface downstream of the weir is low enough to allow air to circulate between the weir and the underside of the nappe. In a suppressed weir the sides of the structure prevent the air from circulating under the nappe so the underside of the nappe has to be vented.

Submerged-flow conditions exist when the water surface downstream of the weir rises above the weir crest. Weirs should be designed to discharge freely rather than submerged because of greater measurement accuracy, although a slight submergence does not appreciably affect the discharge as much as the lack of ventilation under the nappe. The underside of the nappe on a free-flow weir should be properly vented so the nappe springs free from the weir crest and full contraction occurs.

5-16. *Application.*—Weirs can be used, where adequate head is available, as inline measuring structures as shown in figures 5-17 and 5-18, or in combination with division structures and turnouts as shown in figures 5-19 and 5-26, respectively.

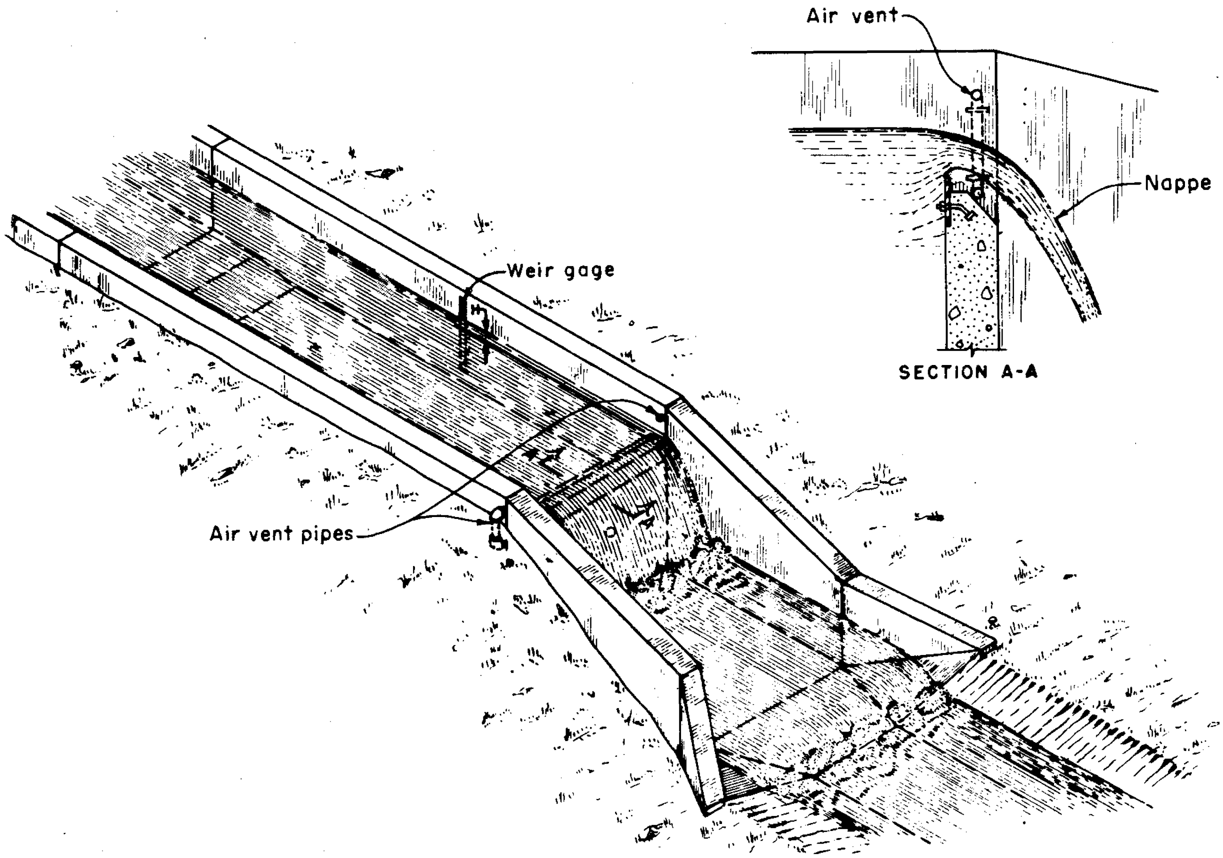


Figure 5-14. Typical suppressed weir in a flume drop. 103-D-1235

5-17. ***Adjustable Weirs.***—Weirs used in division structures are usually adjustable (fig. 5-20). Adjustable weirs can be rectangular, trapezoidal, or V-shaped. They are mounted in a frame and can be raised or lowered by a threaded stem and handwheel (fig. 5-21). Some adjustable weirs are attached permanently to the division structures while others are constructed so the weir and frame may be removed from one structure and used on another (figs. 5-22 and 5-23). Adjustable weirs installed in division structures serve as measuring devices but their primary purpose is to control or divide the flow. When two or more adjustable weirs are used in one division box (fig. 3-31), the proximity of the weirs and the small pool could have a detrimental effect on the head-discharge relationship of each weir. Consequently, the accuracy of the flow measurement is less than that expected from a weir operating under standard conditions. The division structures shown in figures 3-35 and 3-36 in chapter III do not include staff gages for measuring heads on the weirs. Instead a portable staff or weir gage (fig. 5-25) is placed on the crest of the weir to measure the head as shown in figure 5-19. Some of these gages or weir sticks contain both a piezometer and manometer to show the total head (depth over the weir and velocity head) causing the flow. After measuring the head, the flow can be determined from the standard weir tables. If a permanent-type weir gage installation is preferred over the portable gage, the weir gage may be installed as shown in figure 5-24. If greater accuracy is required for either the portable or permanent gage installation the weir should be calibrated to determine the head-discharge relationship.

5-18. ***Advantages and Disadvantages.***—Weirs have the following advantages:

(1) Capable of accurately measuring a wide range of flow.

(2) Easy to construct.

Figure 5-15. **Water flowing over a standard Cipolletti weir. P20-D-34861**

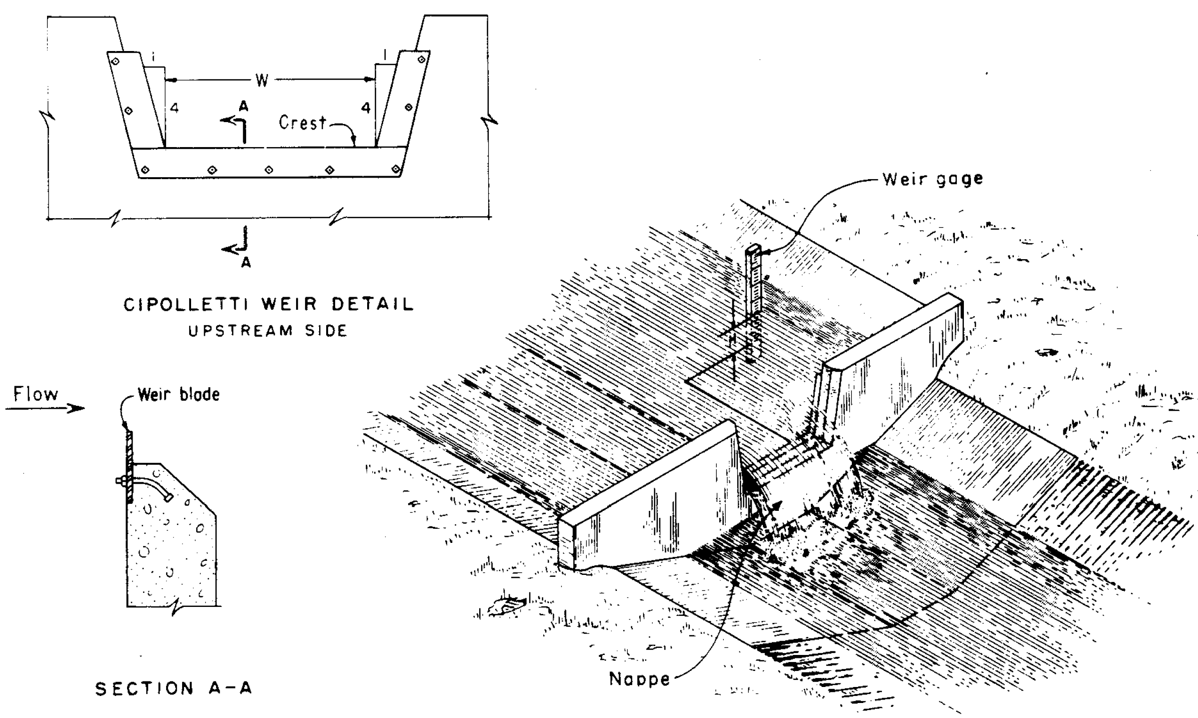
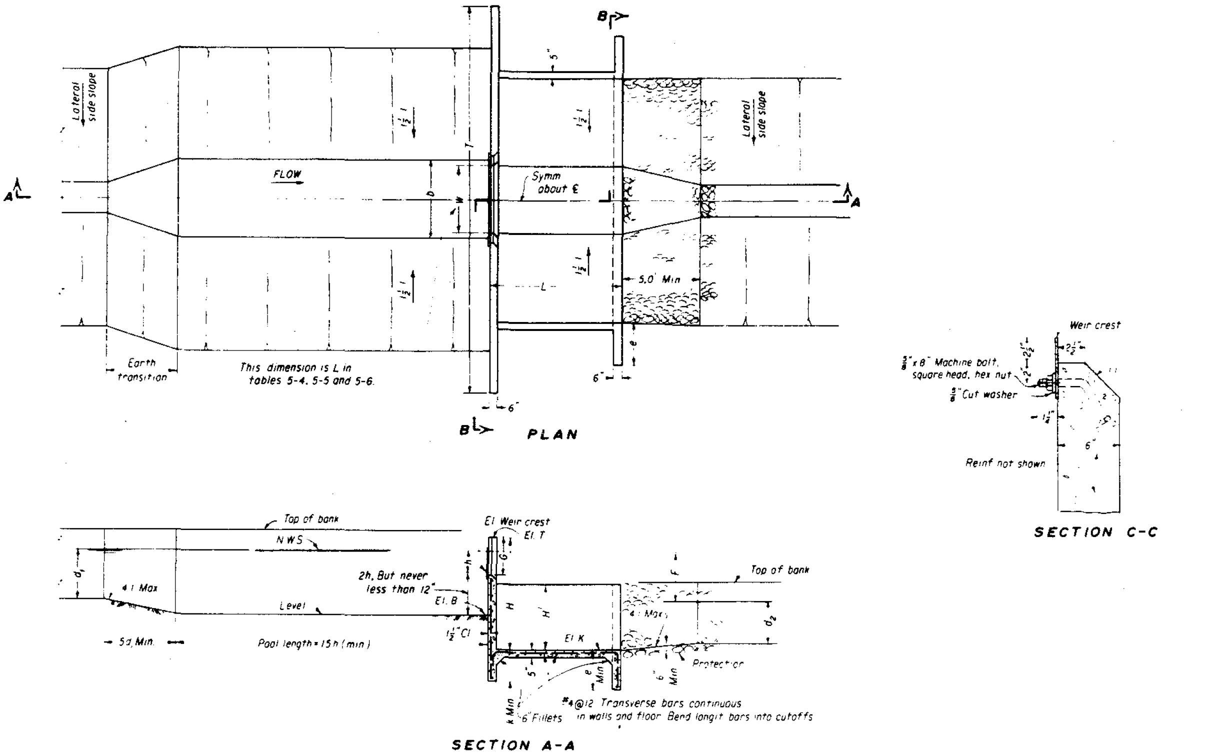
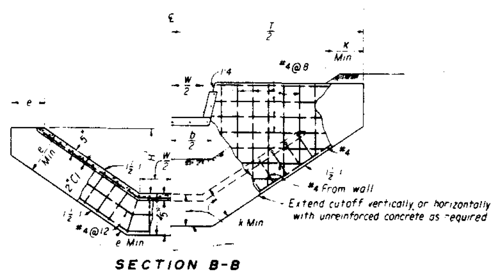
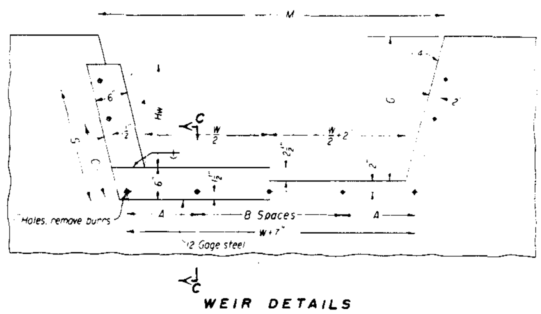


Figure 5-16. **Cipolletti weir in a permanent bulkhead–free-flow conditions. 103-D-1236**







ANCHOR BOLT AND WEIR BLADE DATA

W	H_w	G	M	A	B	S	C
12"	8"	18½"	2'-1¼"	9½"		10 9/16"	
18"	10"	20½"	2'-8¼"	12½"		12 3/8"	
24"	12"	22½"	3'-3¼"	15½"		14 7/15"	
3'-0"	16"	2'-2½"	4'-5¼"	14½"	1@14"	18 9/16"	10¼"
4'-0"	20"	2'-6½"	5'-7¼"	13½"	2@14"	22 11/16"	10¼"

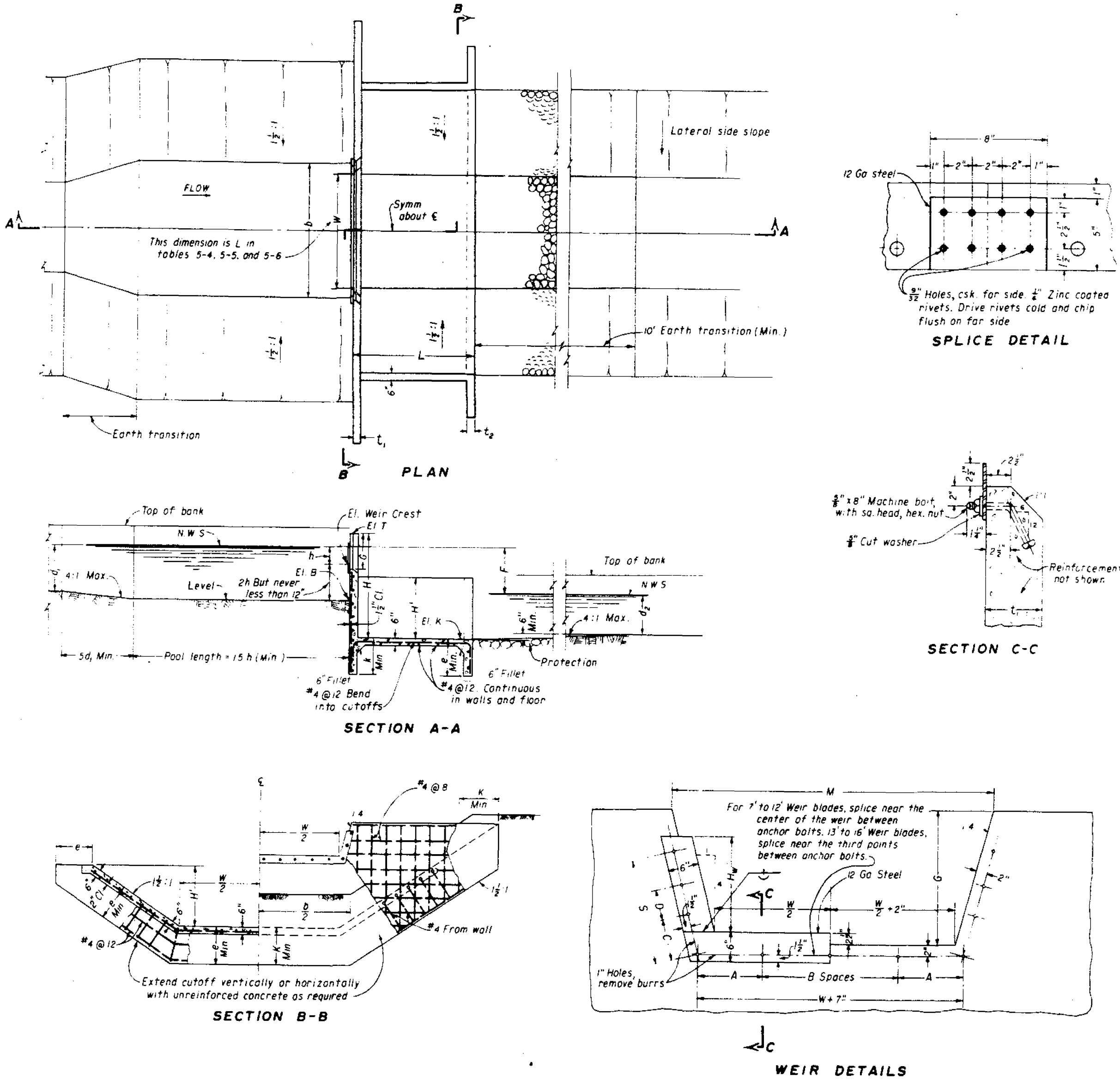
STANDARD DIMENSIONS

STR. NO.	W	Q MAX.	h MAX.	F FOR Q MAX. MIN.	F FOR Q MAX. MAX.	L	H	H'	b	K	e	T
1	12"	0.64	.33'	.33'		4'-0"	2'-6"	24"	18"	18"	18"	10'-6"
1A	12"	0.64	.33		3.0'	7'-0"	5'-0"	24"	18"	18"	18"	10'-6"
1.5	18"	1.78	.50'	.50'		4'-0"	2'-9"	2'-3"	2'-3"	18"	18"	12'-9"
1.5A	18"	1.78	.50'		3.0'	7'-0"	5'-3"	2'-3"	2'-3"	18"	18"	12'-9"
2	24"	3.09	.67'	.67'		4'-0"	3'-3"	2'-6"	3'-0"	18"	18"	15'-0"
2A	24"	3.09	.67'		3.0'	8'-0"	5'-6"	2'-6"	3'-0"	18"	24"	15'-0"
3	3'-0"	[illegible]	1.00'	1.00'		5'-0"	4'-0"	3'-0"	4'-6"	18"	24"	19'-6"
3A	3'-0"	10.1	1.00'		3.0'	9'-0"	6'-0"	3'-0"	4'-6"	18"	24"	19'-6"
4	4'-0"	20.7	1.33	1.33'		7'-0"	5'-0"	3'-6"	6'-0"	24"	24"	25'-0"
4A	4'-0"	20.7	1.33'		3.0'	10'-0"	6'-6"	3'-6"	6'-0"	24"	24"	25'-0"

NOTES

Maximum allowable drop in water surface, F, is 3.0 feet.
Minimum drop is equal to head, h, on the weir for design discharge.
Dimension H' provides 12" freeboard for the following assumed d_2 values for Str Nos 1 & 1A, .50'; 1.5 & 1.5A, .75'; 2 & 2A, 1.0'; 3 & 3A, 1.5'; 4 & 4A, 2.0'.
Dimension H provides for 12" freeboard for Q max, 4 inches on the weir blade and 8 inches on the structure.

Figure 5-17. **Cipolletti weir structures–12 inches to 4 feet. 103-D-1237**



WEIR STRUCTURES

STR. NO.	W	Q MAX cfs	h MAX	F FOR Q MAX. MIN.	MAX.
5	5'-0"	36	1.66'	1.66'	
5A	5'-0"	36	1.66'		3.00'
6	6'-0"	57	2.00'	2.00'	
6A	6'-0"	57	2.00'		3.00'
7	7'-0"	70	2.07'	2.07'	
7A	7'-0"	70	2.07'		3.00'
8	8'-0"	70	1.89'	1.89'	
8A	8'-0"	70	1.89'		3.00'
9	9'-0"	70	1.75'	1.75'	
9A	9'-0"	70	1.75'		3.00'
10	10'-0"	70	1.63'	1.63'	
10A	10'-0"	70	1.63'		3.00'
11	11'-0"	70	1.53'	1.53'	
11A	11'-0"	70	1.53'		3.00'
12	12'-0"	74	1.50'	1.50'	1.50'
13	13'-0"	80	1.50'	1.50'	1.50'
14	14'-0"	87	1.50'	1.50'	1.50'
15	15'-0"	93	1.50'	1.50'	1.50'
16	16'-0"	99	1.50'	1.50'	1.50'

ANCHOR BOLT AND WEIR BLADE DATA

W	H_W	G	M	A	B	S	C	D
5'-0"	24"	2'-10 1/2"	6'-9 1/4"	12 1/2"	3@14"	2'-2 13/16"	10 1/4"	8"
6'-0"	2'-4"	3'-2 1/2"	7'-11 1/4"	11 1/2"	4@14"	2'-6 13/16"	10 1/4"	10"
7'-0"	2'-4"	3'-2 1/2"	8'-11 1/4"	10 1/2"	5@14"	2'-6 13/16"	10 1/4"	10"
8'-0"	2'-4"	3'-2 1/2"	9'-11 1/4"	14"	5@15"	2'-6 13/16"	10 1/4"	10"
9'-0"	2'-4"	3'-2 1/2"	10'-11 1/4"	12 1/2"	6@15"	2'-6 13/16"	10 1/4"	10"
10'-0"	24"	2'-10 1/2"	11'-9 1/4"	14 1/2"	7@14"	2'-2 13/16"	10 1/4"	8"
11'-0"	24"	2'-10 1/2"	12'-9 1/4"	13 1/2"	8@14"	2'-2 13/16"	10 1/4"	8"
12'-0"	24"	2'-10 1/2"	13'-9 1/4"	12 1/2"	9@14"	2'-2 13/16"	10 1/4"	8"
13'-0"	24"	2'-10 1/2"	14'-9 1/4"	11 1/2"	10@14"	2'-2 13/16"	10 1/4"	8"
14'-0"	24"	2'-10 1/2"	15'-9 1/4"	10 1/2"	11@14"	2'-2 13/16"	10 1/4"	8"
15'-0"	24"	2'-10 1/2"	16'-9 1/4"	11"	11@15"	2'-2 13/16"	10 1/4"	8"
16'-0"	24"	2'-10 1/2"	17'-9 1/4"	15"	13@13"	2'-2 13/16"	10 1/4"	8"

NOTES

Maximum allowable drop in water surface, F, is 3.0 feet for discharges up to 70 cfs, and 1.5 feet for discharges greater than 70 cfs.

Minimum drop in water surface, F, is equal to the head, h, on the weir for design discharge

Weir blade to be galvanized by the hot dip process after fabrication

Figure 5-18. **Cipolletti weir structures–5 feet to 16 feet. 103-D-1238**

(3) Can be used in combination with turnouts.
(4) Can be used in division structures.
(5) Can be both movable (portable) and adjustable.

Some disadvantages of weirs are:
(1) Impose a greater head loss in the canal system than other water measurement structures.
(2) Weir pools must be cleaned of sediment periodically and kept free of weeds and trash.
(3) Can be easily altered to obtain unallocated water.
(4) Some leakage occurs around movable weirs.

5-19. ***Requirements for Accurate Measurement.***—To obtain the highest degree of measurement accuracy, standard weir structures must be accurately constructed and properly maintained. The necessary requirements for weir blade, nappe, approach channel-weir relationship and gage location are:

(a) *Weir Blade.*—The weir blade should be designed and installed as follows:

(1) The upstream face of the bulkhead and weir blade must be vertical.

(2) The crest of the weir blade should be level and have a sharp right angle corner on the upstream side.

(3) The sides of a rectangular contracted weir blade should be vertical, have a sharp right angle corner on the upstream side and form a right angle at the junction with the crest.

(4) The sides of a Cipolletti weir blade should be on 1 to 4 slope, and have sharp right angle corners on the upstream side.

(5) The sides of a 90° V-notch weir blade should have a sharp right angle corner on the upstream side.

(6) The downstream edge of the crest and sides of a movable weir blade should be chamfered at a 45° angle or more (crest should be about one-eighth inch thick); however, knife edges on all weirs should be avoided because they are difficult to maintain.

(7) Weir blades must be kept free of rust and nicks. Any form of roughness will cause the weir to discharge more water than indicated in the standard tables.

Figure 5-19. **Division structure with adjustable weirs. Operator is measuring depth of water on weir crest. P222-116-52034**

(b) *Nappe.*—The nappe requirements are:

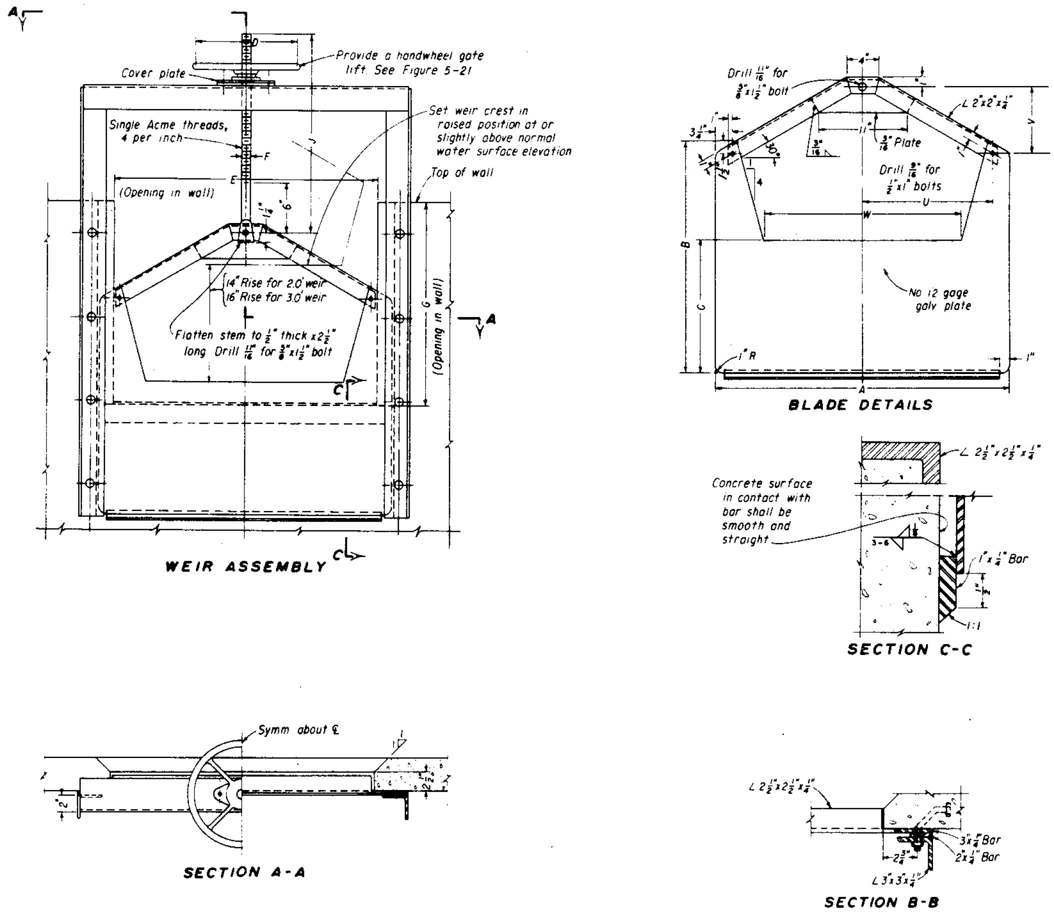
(1) Air should circulate freely under the nappe. In suppressed weirs the underside of the nappe should be vented from both sides. An increase in discharge as much as 25 percent may occur if the nappe is not properly vented [2].

(2) The nappe should only touch the upstream side of the crest and sides of the weir blade.

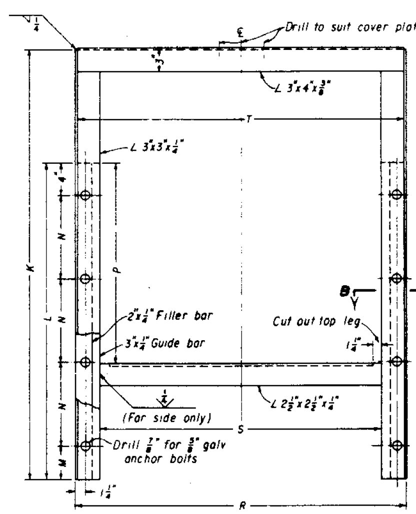
(c) *Approach Channel-to-weir Relationship.*—The following design criteria have been developed from extensive tests and prolonged use of standard weirs:

(1) The approach channel or weir pool should be designed so that the velocity of approach in the weir pool is about 0.5 foot per second. It should extend upstream from the weir a distance of 15 to 20 times the head on the weir. If for some reason the weir pool cannot be designed to meet this velocity requirement, then a discharge correction coefficient must be applied. The second edition of the Bureau of Reclamation "Water Measurement Manual" [1] contains a table for determining the discharge correction for various heads and approach velocities.

(2) The distance between the weir crest and the invert of the approach channel or



DIM.	2' WEIR	3' WEIR
A	3'-1 1/2"	4'-3 1/2"
B	2'-5"	3'-1"
C	1'-5"	1'-9"
D	1'-0"	1'-2"
E	2'-10"	4'-0"
F	0'-1"	0'-1 1/8"
G	2'-0 1/2"	2-4 1/2"
H	0'-10"	1'-2"
J	2'-0"	2'-4"
K	4'-4"	5'-9"
L	3'-2"	3'-10"
M	0'-4"	0'-3"
N	0'-10"	1'-1"
P	2'-0 1/4"	2'-4 1/4"
R	3'-6"	4'-8"
S	3'-0"	4'-2"
T	3'-5 1/2"	4'-7 1/2"
U	1'-3 1/2"	1'-10 1/2"
V	0'-8"	1'-0"
W	2'-0"	3'-0"



FRAME DETAILS
Tack weld guide bars, filler bars and 3"x3"x1/4" angles together

Figure 5-20. **Adjustable weirs–2 and 3 feet wide. 103-D-1239**

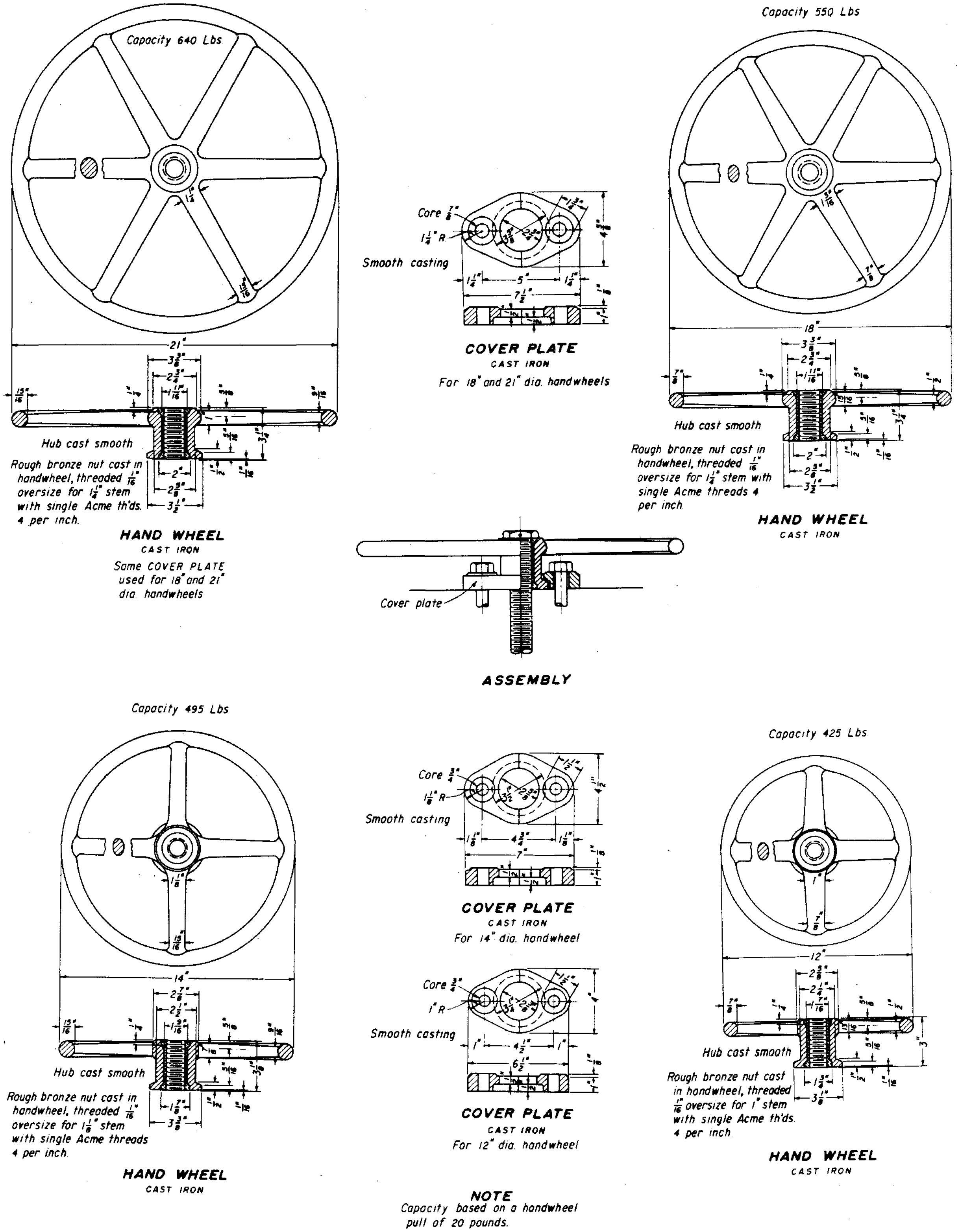


Figure 5-21. **Handwheels–12, 14, 18, and 21 inches. 103-D-1240**

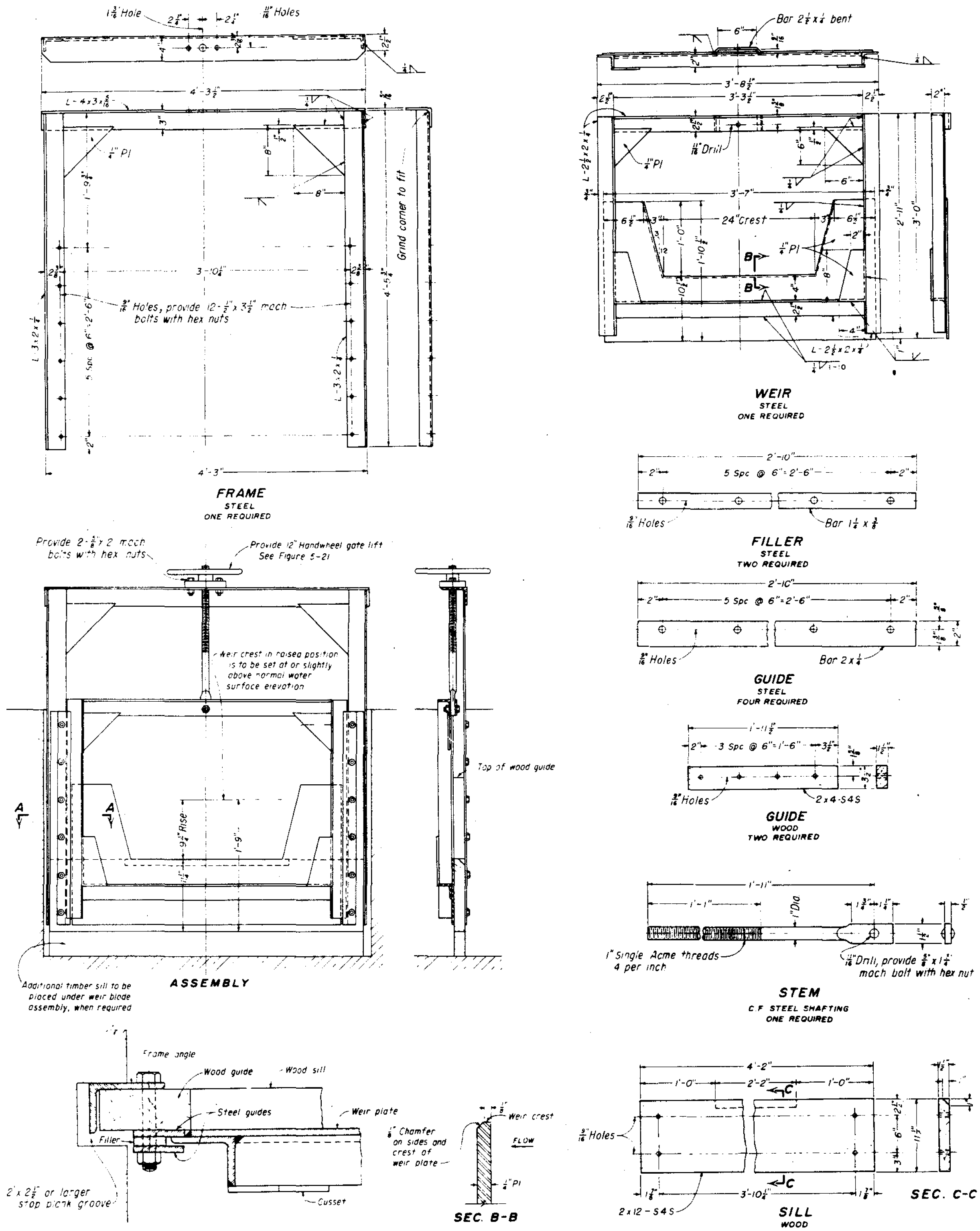


Figure 5-22. **Two-foot movable weir. 103-D-1241**

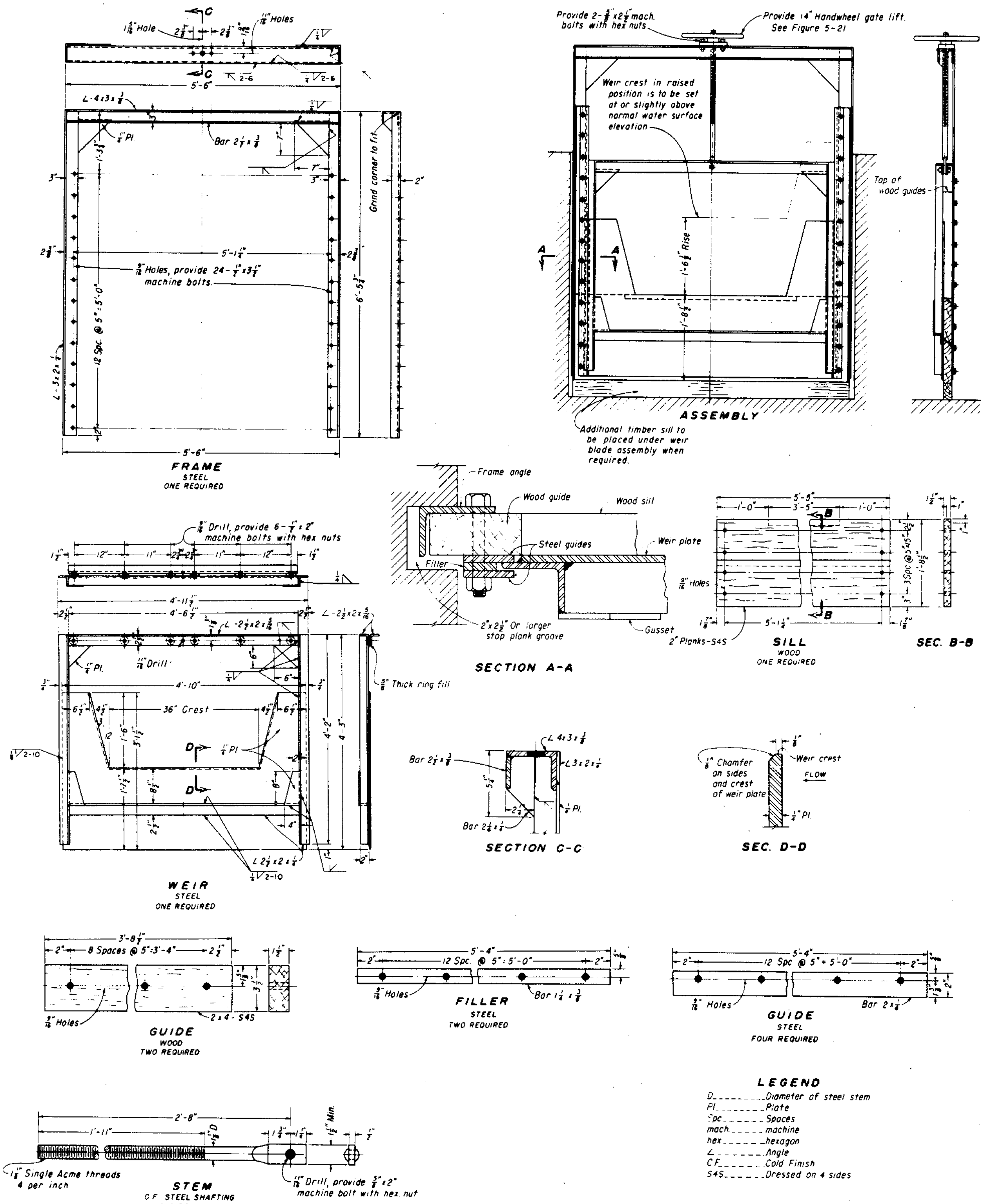
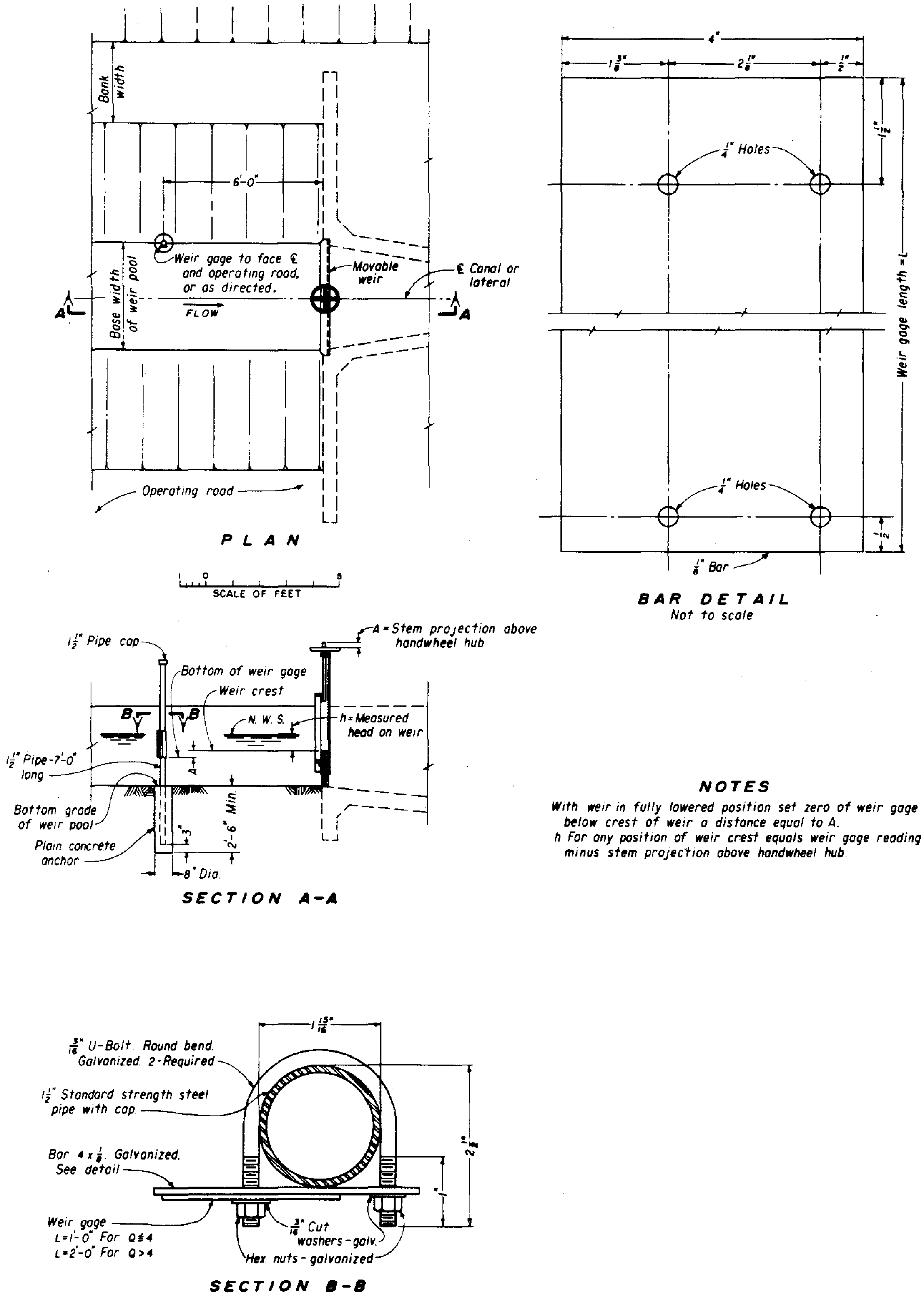


Figure 5-23. **Three-foot movable weir. 103-D-1242**

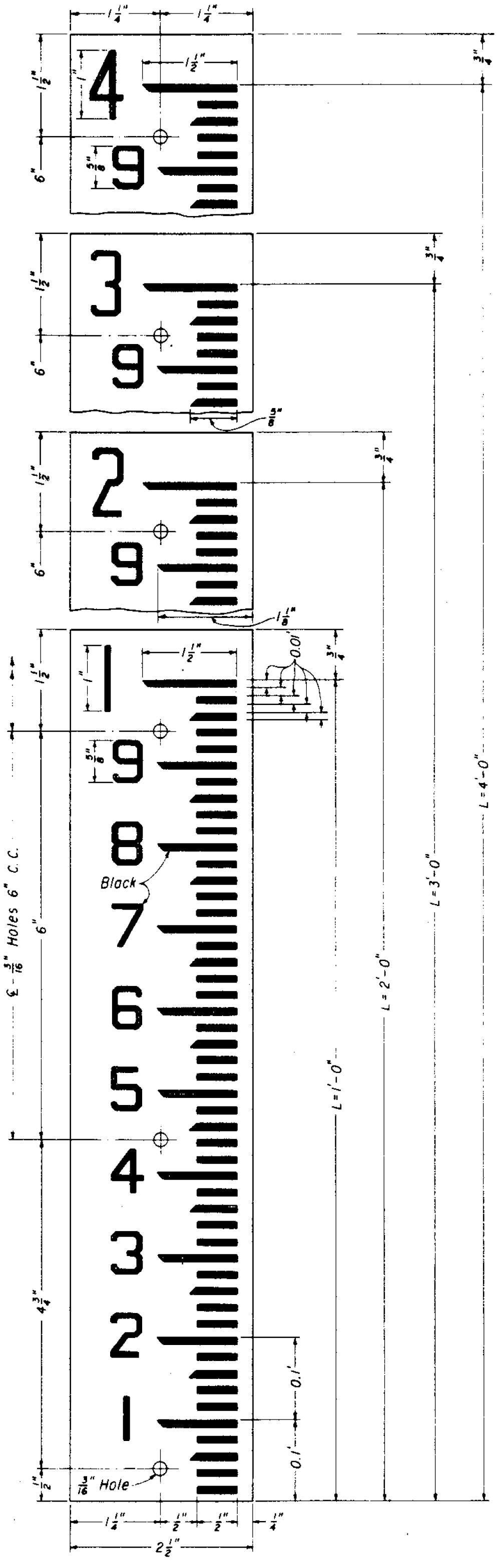


NOTES

With weir in fully lowered position set zero of weir gage below crest of weir a distance equal to A.

h For any position of weir crest equals weir gage reading minus stem projection above handwheel hub.

Figure 5-24. **Staff or weir gage installation. 103-D-1243**



NOTES

Gages to be made of No. 18 gage (U.S standard) mild steel plate and to be covered with vitreous enamel with a minimum thickness of 12 mils on numeral side and 3 mils on the reverse side and on edges where plate has been cut, punched or drilled.

All cutting, drilling and punching of the plates shall be completed before the vitreous enamel is applied

The face of the gage shall be white and all numerals and graduations shall be black.

Graduations shall be sharp and accurate to the dimensions shown.

In case a greater length than 4'-0" is required the details shall be similar to details shown for shorter lengths.

Figure 5-25. **Staff or weir gage. 103-D-1244** weir pool should be at least twice the head on the weir and never less than 1 foot for all weirs. The distance between the corners of the weir crest and the sides of the approach channel should also be at least twice the head on the weir and never less than 1 foot for all standard contracted weirs.

(d) *Head on Weir.*—The head on the weir is the difference in elevation between the weir crest and the water surface upstream. The head is measured by:

(1) Either a staff gage in the weir pool or a gage in a measuring well located upstream of the weir a distance of four times the maximum head on the weir.

5-20. ***Head-discharge Relationship.***—(a) *Rectangular and Cipolletti Weirs.*—The discharge in cubic feet per second of these standard weirs depends on the crest length and the head on the weir. Knowing the weir length and head on the weir, the free-flow discharge can be read directly from tables 5-4, 5-5, and 5-6.

The minimum head on standard rectangular and Cipolletti weirs is 0.2 foot. At heads less than 0.2 foot the nappe does not spring free of the crest and measurement error results.

The maximum head on standard rectangular and Cipolletti weirs is one-third of the crest length. For the weir structure shown in figure 5-18, the maximum head (h max.) on the weir should not exceed the values given in the table because in unlined canals excessive channel erosion may occur immediately downstream of the structure. If this structure is used in a hard surface lined canal where erosion is not a problem, the head on the weir can be one-third the crest length and the discharge computed from the appropriate weir formula.

(b) *V-notch Weirs.*—The discharge in cubic feet per second of a standard 90° V-notch weir is determined only by the head on the bottom of the V-notch. Table 5-7 gives the discharges for various heads. The V-notch weirs are usually limited to flows of 10 cfs or less. More head is required in V-notch weirs to pass a given discharge than in rectangular and Cipolletti weirs but for flows up to 1 cfs they are generally more accurate. The reason for this is that the nappe springs free from the V-shaped section even for small heads, whereas the nappe clings to the crest of other weirs. The minimum head on a V-notch weir should be 0.2 foot.

5-21. ***Design Example of Selecting and Setting a Weir.***—(a) *Requirements.*—Select and set a Cipolletti weir structure to measure a maximum flow of 40 cfs. The weir structure is to be constructed in a trapezoidal earth canal which has the following hydraulic properties: Q = 40 cfs; bottom width = 6.0 feet; side slopes 1-1/2 to 1; normal water depth upstream (d_1) and downstream (d_2) of weir = 2.0 feet; velocity = 2.22 feet per second; and canal bank freeboard = 1.5 feet. Assume a 3-foot drop (F) between upstream and downstream normal water surfaces has been provided in the canal profile.

(b) *Weir Selection.*—The table of weir structures in figure 5-18 gives some standard weir lengths with their corresponding maximum discharges, Q max., and minimum and maximum drops, F, between upstream and downstream water surfaces. Select the structure having the least weir length for a design discharge of 40 cfs and F = 3.0 feet. Structure No. 6A (from table on fig. 5-18), has a crest length, W, of 6 feet which meets this criteria. The maximum design capacity of this structure is 57 cfs and a maximum allowable F = 3 feet. Table 5-5 shows that for a 6-foot weir and a discharge of 40.1 cfs, the head, h, equals 1.58 feet.

(c) *Weir Setting.*—Set the weir pool invert so that elevation B shown in figure 5-18 is 3h below the upstream normal water surface (NWS) elevation. Assume the upstream canal invert elevation is 100.00 feet. Then

$$\text{El. B} = \text{Canal invert elevation} + d_1 - 3h$$
$$\text{El. B} = 100.00 + 2.00 - 3(1.58)$$
$$= 97.26 \text{ feet}$$

Since 2h = 3.16 feet is greater than the 12-inch minimum, El. B is satisfactory. The elevation of the top of weir wall (El. T) is given by the following equation:

$$\text{El. T} = \text{El. B} + 2h - \frac{2.50}{12} + G$$

$$\text{El. T} = 97.26 + 3.16 - \frac{2.50}{12} + 3.21$$

$$\text{El. T} = 103.42 \text{ feet}$$

In the above calculation, $\frac{2.50}{12}$ is the weir blade projection in feet above the concrete sill and G is the concrete depth of the weir notch given in the Anchor Bolt and Weir Blade Data table shown in figure 5-18.

Next calculate the elevation of the weir floor (El. K).

$$\text{El. K} = \text{NWS El. (upstm.)} - F - d_2 - 0.5\,(\text{min.})$$
$$\text{El. K} = 102.00 - 3.00 - 2.00 - 0.50$$
$$\text{El. K} = 96.50 \text{ feet}$$

In all cases El. K should be at or below El. B. The height of the weir wall, H, is:

$$H = \text{El. T} - \text{El. K}$$
$$H = 103.42 - 96.50$$
$$H = 6.92 \text{ feet}$$

The height of the sloping sidewall, H′, is:

$$H' = 0.5\,(\text{minimum}) + d_2 + \text{structure freeboard}$$

As the weir structure freeboard is 1 foot, then

$$H' = 0.50 + 2.00 + 1.00 = 3.50 \text{ feet}$$

The length, L, of the weir structure should be long enough to contain the plunging nappe and to provide a pool for stilling the water before it passes downstream into the canal. The length is computed by the following empirical formula:

$$L = 3h + 2F$$
$$L = 3(1.58) + 2(3.0) = 10.74 \text{ feet}$$

Therefore, the structure can be made 11 feet long.

Cutoff walls are added to provide additional percolation path to prevent piping of foundation materials from under the structure, to protect the structure from being undermined if channel erosion occurs, and to provide structure stability to resist overturning and sliding. As a general rule cutoff walls should extend below the canal invert as shown for the following water depths at the cutoff:

Water depth, feet	Cutoff depth, feet	Cutoff thickness, inches
0-3	2.0	6
3-6	2.5	8
>6	3.0	8

The upstream water depth (3h) adjacent to the upstream face of the structure for this example is 4.74 feet. Using the general rule, the cutoff should extend 2.5 feet below the invert. Then $K_{min.}$ shown in figure 5-18 is:

$$K_{min.} = 2.5 - (\text{El.B} - \text{El.K})$$
$$K_{min.} = 2.5 - (96.26 - 96.50)$$
$$K_{min.} = 2.5 - 0.76$$
$$K_{min.} = 1.74 \text{ feet}$$

Water depth at the downstream cutoff is 2.5 feet and using the general rule, $e_{min.}$ is 24 inches. Use 24 inches for both $K_{min.}$ and $e_{min.}$. Additional cutoff lengths may be required depending on the percolation factor as discussed in detail in section 8-10. Lane's weighted creep length in this example is:

Weighted creep length

$$= (\text{El. B} - \text{El. K}) + K_{min.} + \frac{L}{3} + (K_{min.} - \frac{6}{12}) + (e_{min.} - \frac{6}{12}) + (e_{min.} - \text{protection thickness})$$
$$= (97.26 - 96.50) + 2.00 + \frac{11}{3} + (2 - \frac{6}{12}) + (2 - \frac{6}{12}) + (2 - 1)$$
$$= 10.43 \text{ feet}$$

Next determine the maximum difference in water surface (ΔWS) between the upstream and downstream ends of the structure:

(1) For design discharge, $\Delta WS = 3.0$ feet

(2) For upstream water ponded to weir crest and downstream water at canal invert

$$\Delta WS = \text{Crest elevation} - \text{Downstream canal invert elevation}$$
$$= (\text{El. B}+2h) - (\text{El. K}+0.50)$$
$$= 100.42 - 97.00 = 3.42 \text{ feet}$$

Compute weighted creep ratio (percolation factor) for $\Delta WS = 3.42$ feet,

$$\text{Percolation factor} = \frac{10.43}{3.42} = 3.1$$

Assume the required percolation factor for the foundation material for this structure is 2.5. The percolation factor calculated is greater than 2.5 and therefore the cutoffs are adequate in length.

Table 5-4.–*Discharge of standard contracted rectangular weirs in cubic feet per second. Values below and to the left of heavy line determined experimentally; others computed from the formula* $Q = 3.33 (L-0.2h)h^{3/2}$. 103-D-1245-1

Head h, feet	Length of weir, L, feet						
	1.0	2.0	3.0	4.0	5.0	6.0	7.0
0.20	0.29	0.58	0.88	1.18	1.48	1.78	2.07
.21	.31	.63	.95	1.27	1.59	1.91	2.23
.22	.33	.67	1.02	1.36	1.70	2.05	2.39
.23	.35	.72	1.08	1.45	1.82	2.19	2.55
.24	.37	.76	1.16	1.55	1.94	2.33	2.72
.25	.40	.81	1.23	1.64	2.06	2.48	2.89
.26	.42	.86	1.30	1.74	2.18	2.63	3.07
.27	.44	.91	1.38	1.84	2.31	2.78	3.24
.28	.47	.96	1.45	1.95	2.44	2.93	3.43
.29	.49	1.01	1.53	2.05	2.57	3.09	3.61
.30	.51	1.06	1.61	2.16	2.70	3.25	3.80
.31	.54	1.11	1.69	2.26	2.84	3.41	3.99
.32	.56	1.17	1.77	2.37	2.98	3.58	4.18
.33	.59	1.22	1.85	2.48	3.12	3.75	4.38
.34	.62	1.28	1.94	2.60	3.26	3.92	4.58
.35	.66	1.33	2.02	2.71	3.40	4.09	4.78
.36	.69	1.39	2.11	2.82	3.54	4.26	4.98
.37	.71	1.44	2.19	2.94	3.69	4.44	5.19
.38	.74	1.50	2.28	3.06	3.84	4.62	5.40
.39	.77	1.56	2.37	3.18	3.99	4.80	5.61
.40	.80	1.62	2.46	3.30	4.14	4.99	5.83
.41	.83	1.68	2.55	3.42	4.30	5.17	6.05
.42	.86	1.74	2.64	3.55	4.46	5.36	6.27
.43	.89	1.80	2.74	3.68	4.61	5.55	6.49
.44	.92	1.86	2.83	3.80	4.77	5.75	6.72
.45	.95	1.92	2.92	3.93	4.94	5.94	6.95
.46	.98	1.98	3.02	4.06	5.10	6.14	7.18
.47	1.01	2.04	3.12	4.19	5.26	6.34	7.41
.48	1.04	2.11	3.22	4.32	5.43	6.54	7.64
.49	1.08	2.17	3.32	4.46	5.60	6.74	7.88
.50	1.11	2.24	3.41	4.59	5.77	6.95	8.12
.51		2.30	3.52	4.73	5.94	7.15	8.37
.52		2.37	3.62	4.86	6.11	7.36	8.61
.53		2.43	3.72	5.00	6.29	7.57	8.86
.54		2.50	3.82	5.14	6.46	7.79	9.11
.55		2.57	3.92	5.28	6.64	8.00	9.36
.56		2.64	4.03	5.43	6.82	8.22	9.61
.57		2.70	4.14	5.57	7.00	8.44	9.87
.58		2.77	4.24	5.71	7.18	8.66	10.1
.59		2.84	4.35	5.86	7.37	8.88	10.4
.60		2.91	4.46	6.00	7.55	9.10	10.6
.61		2.98	4.57	6.15	7.74	9.32	10.9
.62		3.05	4.68	6.30	7.93	9.55	11.2
.63		3.12	4.79	6.45	8.12	9.78	11.4
.64		3.19	4.90	6.60	8.31	10.0	11.7
.65		3.26	5.01	6.75	8.50	10.2	12.0
.66		3.34	5.12	6.91	8.69	10.5	12.3
.67		3.41	5.23	7.06	8.89	10.7	12.5
.68		3.58	5.35	7.22	9.08	11.0	12.8
.69		3.66	5.46	7.37	9.28	11.2	13.1
.70		3.74	5.58	7.53	9.48	11.4	13.4

Table 5-4.—*Discharge of standard contracted rectangular weirs in cubic feet per second. Values below and to the left of heavy line determined experimentally; others computed from the formula* $Q = 3.33\ (L-0.2h)h^{3/2}$.—Continued. 103-D-1245-2

Head h, feet	Length of weir, L, feet					
	2.0	3.0	4.0	5.0	6.0	7.0
0.71	3.82	5.69	7.69	9.68	11.7	13.7
.72	3.90	5.81	7.84	9.88	11.9	13.9
.73	3.98	5.93	8.00	10.1	12.2	14.2
.74	4.06	6.05	8.16	10.3	12.4	14.5
.75	4.14	6.16	8.33	10.5	12.7	14.8
.76	4.22	6.28	8.49	10.7	12.9	15.1
.77	4.30	6.40	8.65	10.9	13.2	15.4
.78	4.38	6.52	8.82	11.1	13.4	15.7
.79	4.46	6.64	8.98	11.3	13.7	16.0
.80	4.54	6.77	9.15	11.5	13.9	16.3
.81	4.62	6.89	9.32	11.7	14.2	16.6
.82	4.70	7.01	9.48	12.0	14.4	16.9
.83	4.78	7.14	9.65	12.2	14.7	17.2
.84	4.87	7.26	9.82	12.4	15.0	17.5
.85	4.96	7.38	10.0	12.6	15.2	17.8
.86	5.05	7.51	10.2	12.8	15.5	18.1
.87	5.14	7.64	10.3	13.0	15.7	18.4
.88	5.23	7.76	10.5	13.3	16.0	18.8
.89	5.32	7.89	10.7	13.5	16.3	19.1
.90	5.41	8.02	10.9	13.7	16.5	19.4
.91	5.50	8.15	11.0	13.9	16.8	19.7
.92	5.59	8.28	11.2	14.2	17.1	20.0
.93	5.68	8.40	11.4	14.4	17.4	20.4
.94	5.77	8.53	11.6	14.6	17.6	20.7
.95	5.86	8.66	11.7	14.8	17.9	21.0
.96	5.95	8.80	11.9	15.1	18.2	21.3
.97	6.04	8.93	12.1	15.3	18.5	21.7
.98	6.13	9.06	12.3	15.5	18.8	22.0
.99	6.22	9.19	12.5	15.8	19.0	22.3
1.00	6.31	9.32	12.7	16.0	19.3	22.6
1.01			12.8	16.2	19.6	23.0
1.02			13.0	16.5	19.9	23.3
1.03			13.2	16.7	20.2	23.6
1.04			13.4	16.9	20.5	24.0
1.05			13.6	17.2	20.7	24.3
1.06			13.8	17.4	21.0	24.7
1.07			14.0	17.6	21.3	25.0
1.08			14.1	17.9	21.6	25.4
1.09			14.3	18.1	21.9	25.7
1.10			14.5	18.4	22.2	26.0
1.11			14.7	18.6	22.5	26.4
1.12			14.9	18.9	22.8	26.7
1.13			15.1	19.1	23.1	27.1
1.14			15.3	19.3	23.4	27.4
1.15			15.5	19.6	23.7	27.8
1.16			15.7	19.8	24.0	28.2
1.17			15.9	20.1	24.3	28.5
1.18			16.1	20.3	24.6	28.9
1.19			16.3	20.6	24.9	29.2
1.20			16.5	20.8	25.2	29.6

Table 5-4.–*Discharge of standard contracted rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\,(L\text{-}0.2h)h^{3/2}$.–Continued. 103-D-1245-3

Head h, feet	Length of weir, *L*, feet				Head h, feet	Length of weir, *L*, feet	
	4.0	5.0	6.0	7.0		6.0	7.0
1.21	16.7	21.1	25.5	30.0	1.71	42.1	49.6
1.22	16.9	21.3	25.8	30.3	1.72	42.5	50.0
1.23	17.1	21.6	26.1	30.7	1.73	42.8	50.4
1.24	17.3	21.8	26.4	31.0	1.74	43.2	50.8
1.25	17.5	22.1	26.8	31.4	1.75	43.6	51.3
1.26	17.7	22.4	27.1	31.8	1.76	43.9	51.7
1.27	17.9	22.6	27.4	32.2	1.77	44.3	52.1
1.28	18.1	22.9	27.7	32.5	1.78	44.6	52.5
1.29	18.3	23.1	28.0	32.9	1.79	45.0	53.0
1.30	18.5	23.4	28.3	33.3	1.80	45.4	53.4
1.31	18.7	23.7	28.6	33.6	1.81	45.7	53.8
1.32	18.9	23.9	29.0	34.0	1.82	46.1	54.3
1.33	19.1	24.2	29.3	34.4	1.83	46.4	54.7
1.34		24.4	29.6	34.8	1.84	46.8	55.1
1.35		24.7	29.9	35.2	1.85	47.2	55.6
1.36		25.0	30.3	35.5	1.86	47.5	56.0
1.37		25.2	30.6	35.9	1.87	47.9	56.4
1.38		25.5	30.9	36.3	1.88	48.3	56.9
1.39		25.8	31.2	36.7	1.89	48.6	57.3
1.40		26.0	31.6	37.1	1.90	49.0	57.7
1.41		26.3	31.9	37.5	1.91	49.4	58.2
1.42		26.6	32.2	37.8	1.92	49.8	58.6
1.43		26.8	32.5	38.2	1.93	50.1	59.0
1.44		27.1	32.9	38.6	1.94	50.5	59.5
1.45		27.4	33.2	39.0	1.95	50.9	59.9
1.46		27.7	33.5	39.4	1.96	51.2	60.4
1.47		27.9	33.9	39.8	1.97	51.6	60.8
1.48		28.2	34.2	40.2	1.98	52.0	61.3
1.49		28.5	34.5	40.6	1.99	52.4	61.7
1.50		28.8	34.9	41.0	2.00	52.7	62.2
1.51		29.0	35.2	41.4	2.01		62.6
1.52		29.3	35.5	41.8	2.02		63.1
1.53		29.6	35.9	42.2	2.03		63.5
1.54		29.9	36.2	42.6	2.04		64.0
1.55		30.1	36.6	43.0	2.05		64.4
1.56		30.4	36.9	43.4	2.06		64.9
1.57		30.7	37.2	43.8	2.07		65.3
1.58		31.0	37.6	44.2	2.08		65.8
1.59		31.3	37.9	44.6	2.09		66.2
1.60		31.5	38.3	45.0	2.10		66.7
1.61		31.8	38.6	45.4	2.11		67.1
1.62		32.1	39.0	45.8	2.12		67.6
1.63		32.4	39.3	46.2	2.13		68.1
1.64		32.7	39.7	46.7	2.14		68.5
1.65		33.0	40.0	47.1	2.15		69.0
1.66		33.2	40.4	47.5	2.16		69.4
1.67			40.7	47.9	2.17		69.9
1.68			41.1	48.3			
1.69			41.4	48.7			
1.70			41.8	49.2			

Table 5-4.–*Discharge of standard contracted rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\,(L\text{-}0.2h)h^{3/2}$.–Continued. 103-D-1245-4

Head h, feet	Length of weir, *L*, feet						
	8.0	9.0	10.0	12.0	15.0	18.0	20.0
0.20	2.37	2.67	2.97	3.56	4.45	5.35	5.94
.21	2.55	2.87	3.19	3.83	4.79	5.75	6.39
.22	2.73	3.08	3.42	4.11	5.14	6.17	6.83
.23	2.92	3.29	3.66	4.39	5.49	6.59	7.33
.24	3.11	3.50	3.90	4.68	5.86	7.03	7.81
.25	3.31	3.72	4.14	4.97	6.22	7.47	8.30
.26	3.51	3.95	4.39	5.28	6.60	7.93	8.81
.27	3.71	4.18	4.65	5.58	6.98	8.38	9.32
.28	3.92	4.41	4.91	5.89	7.37	8.86	9.84
.29	4.13	4.65	5.17	6.21	7.77	9.33	10.4
.30	4.34	4.89	5.44	6.53	8.17	9.82	10.9
.31	4.56	5.14	5.71	6.86	8.59	10.3	11.5
.32	4.78	5.39	5.99	7.19	9.00	10.8	12.0
.33	5.01	5.64	6.27	7.53	9.43	11.3	12.6
.34	5.24	5.90	6.56	7.88	9.86	11.8	13.2
.35	5.47	6.16	6.85	8.23	10.3	12.4	13.7
.36	5.70	6.42	7.14	8.58	10.7	12.9	14.3
.37	5.94	6.69	7.44	8.94	11.2	13.4	14.9
.38	6.18	6.96	7.74	9.30	11.6	14.0	15.5
.39	6.42	7.24	8.05	9.67	12.1	14.5	16.2
.40	6.67	7.51	8.36	10.0	12.6	15.1	16.8
.41	6.92	7.80	8.67	10.4	13.0	15.7	17.4
.42	7.18	8.08	8.99	10.8	13.5	16.2	18.1
.43	7.43	8.37	9.31	11.2	14.0	16.8	18.7
.44	7.69	8.66	9.63	11.6	14.5	17.4	19.4
.45	7.95	8.96	9.96	12.0	15.0	18.0	20.0
.46	8.22	9.26	10.3	12.4	15.5	18.6	20.7
.47	8.48	9.56	10.6	12.8	16.0	19.2	21.4
.48	8.75	9.86	11.0	13.2	16.5	19.8	22.0
.49	9.03	10.2	11.3	13.6	17.0	20.4	22.7
.50	9.30	10.5	11.7	14.0	17.5	21.1	23.4
.51	9.58	10.8	12.0	14.4	18.1	21.7	24.1
.52	9.86	11.1	12.4	14.9	18.6	22.3	24.8
.53	10.1	11.4	12.7	15.3	19.1	23.0	25.6
.54	10.4	11.8	13.1	15.7	19.7	23.6	26.3
.55	10.7	12.1	13.4	16.2	20.2	24.3	27.0
.56	11.0	12.4	13.8	16.6	20.8	25.0	27.8
.57	11.3	12.7	14.2	17.0	21.3	25.6	28.5
.58	11.6	13.1	14.5	17.5	21.9	26.3	29.2
.59	11.9	13.4	14.9	17.9	22.5	27.0	30.0
.60	12.2	13.7	15.3	18.4	23.0	27.7	30.8
.61	12.5	14.1	15.7	18.8	23.6	28.4	31.5
.62	12.8	14.4	16.1	19.3	24.2	29.1	32.3
.63	13.1	14.8	16.4	19.8	24.8	29.8	33.1
.64	13.4	15.1	16.8	20.2	25.4	30.5	33.9
.65	13.7	15.5	17.2	20.7	25.9	31.2	34.7
.66	14.0	15.8	17.6	21.2	26.5	31.9	35.5
.67	14.4	16.2	18.0	21.7	27.1	32.6	36.3
.68	14.7	16.6	18.4	22.2	27.8	33.4	37.1
.69	15.0	16.9	18.8	22.6	28.4	34.1	37.9
.70	15.3	17.3	19.2	23.1	29.0	34.8	38.7

Table 5-4.–*Discharge of standard contracted rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\,(L\text{-}0.2h)h^{3/2}$.–Continued. 103-D-1245-5

Head h, feet	Length of weir, L, feet						
	8.0	9.0	10.0	12.0	15.0	18.0	20.0
0.71	15.7	17.6	19.6	23.6	29.6	35.6	39.6
.72	16.0	18.0	20.1	24.1	30.2	36.3	40.4
.73	16.3	18.4	20.5	24.6	30.9	37.1	41.2
.74	16.6	18.8	20.9	25.1	31.5	37.8	42.1
.75	17.0	19.1	21.3	25.6	32.1	38.6	42.9
.76	17.3	19.5	21.7	26.1	32.8	39.4	43.8
.77	17.7	19.9	22.2	26.7	33.4	40.2	44.7
.78	18.0	20.3	22.6	27.2	34.1	40.9	45.5
.79	18.3	20.7	23.0	27.7	34.7	41.7	46.4
.80	18.7	21.1	23.4	28.2	35.4	42.5	47.3
.81	19.0	21.5	23.9	28.7	36.0	43.3	48.2
.82	19.4	21.8	24.3	29.3	36.7	44.1	49.0
.83	19.7	22.2	24.8	29.8	37.4	44.9	49.9
.84	20.1	22.6	25.2	30.3	38.0	45.7	50.8
.85	20.4	23.0	25.7	30.9	38.7	46.5	51.8
.86	20.8	23.4	26.1	31.4	39.4	47.3	52.7
.87	21.1	23.8	26.6	32.0	40.1	48.2	53.6
.88	21.5	24.3	27.0	32.5	40.7	49.0	54.5
.89	21.9	24.7	27.5	33.1	41.4	49.8	55.4
.90	22.2	25.1	27.9	33.6	42.1	50.7	56.4
.91	22.6	25.5	28.4	34.2	42.8	51.5	57.3
.92	23.0	25.9	28.8	34.7	43.5	52.4	58.2
.93	23.3	26.3	29.3	35.3	44.2	53.2	59.2
.94	23.7	26.7	29.9	36.0	45.1	54.2	60.3
.95	24.1	27.2	30.2	36.4	45.7	54.9	61.1
.96	24.5	27.6	30.7	37.0	46.4	55.8	62.0
.97	24.8	28.0	31.2	37.6	47.1	56.6	63.0
.98	25.2	28.4	31.7	38.1	47.8	57.5	64.0
.99	25.6	28.9	32.2	38.7	48.6	58.4	65.0
1.00	26.0	29.3	32.6	39.3	49.3	59.3	65.9
1.01	26.4	29.8	33.1	39.9	50.0	60.2	66.9
1.02	26.7	30.2	33.6	40.5	50.7	61.0	67.9
1.03	27.1	30.6	34.1	41.0	51.5	61.9	68.9
1.04	27.5	31.1	34.6	41.7	52.3	62.9	69.9
1.05	27.9	31.5	35.1	42.2	53.0	63.7	70.9
1.06	28.3	31.9	35.6	42.8	53.7	64.6	71.9
1.07	28.7	32.4	36.1	43.4	54.5	65.6	72.9
1.08	29.1	32.8	36.6	44.0	55.2	66.4	74.3
1.09	29.5	33.3	37.1	44.6	56.0	67.4	75.0
1.10	29.9	33.7	37.6	45.3	56.8	68.3	76.0
1.11	30.3	34.2	38.1	45.9	57.6	69.3	77.1
1.12	30.7	34.6	38.6	46.5	58.3	70.1	78.0
1.13	31.1	35.1	39.1	47.1	59.1	71.1	79.1
1.14	31.5	35.6	39.6	47.7	59.9	72.0	80.1
1.15	31.9	36.0	40.1	48.3	60.6	73.0	81.2
1.16	32.3	36.5	40.6	48.9	61.4	73.9	82.2
1.17	32.7	36.9	41.2	49.6	62.3	74.9	83.3
1.18	33.1	37.4	41.7	50.2	63.0	75.8	84.4
1.19	33.6	37.9	42.2	50.8	63.8	76.8	85.4
1.20	34.0	38.3	42.7	51.5	64.6	77.7	86.5

Table 5-4.—*Discharge of standard contracted rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\,(L\text{-}0.2h)h^{3/2}$. —Continued. 103-D-1245-6

Head h, feet	Length of wier, *L*, feet						
	8.0	9.0	10.0	12.0	15.0	18.0	20.0
1.21	34.4	38.8	43.2	52.1	65.4	78.7	87.6
1.22	34.8	39.3	43.8	52.8	66.2	79.7	88.7
1.23	35.2	39.8	44.3	53.4	67.0	80.6	89.7
1.24	35.6	40.2	44.8	54.0	67.8	81.6	90.8
1.25	36.1	40.7	45.4	54.7	68.7	82.6	91.9
1.26	36.5	41.2	45.9	55.3	69.4	83.6	93.0
1.27	36.9	41.7	46.4	56.0	70.3	84.6	94.1
1.28	37.3	42.2	47.0	56.6	71.1	85.6	95.2
1.29	37.8	42.7	47.5	57.3	71.9	86.6	96.3
1.30	38.2	43.1	48.1	57.9	72.7	87.5	97.4
1.31	38.6	43.6	48.6	58.6	73.6	88.5	98.5
1.32	39.1	44.1	49.2	59.3	74.4	89.6	99.7
1.33	39.5	44.6	49.7	59.9	75.3	90.6	100.8
1.34	39.9	45.1	50.3	60.6	76.1	91.6	
1.35	40.4	45.6	50.8	61.3	77.0	92.6	
1.36	40.8	46.1	51.4	61.9	77.8	93.6	
1.37	41.3	46.6	51.9	62.6	78.7	94.7	
1.38	41.7	47.1	52.5	63.3	79.5	95.7	
1.39	42.1	47.6	53.1	64.0	80.4	96.7	
1.40	42.6	48.1	53.6	64.7	81.2	97.8	
1.41	43.0	48.6	54.2	65.3	82.0	98.8	
1.42	43.5	49.1	54.7	66.0	82.9	99.8	
1.43	43.9	49.6	55.3	66.7	83.8	100.9	
1.44	44.4	50.1	55.9	67.4	84.7		
1.45	44.8	50.6	56.5	68.1	85.5		
1.46	45.3	51.2	57.0	68.8	86.4		
1.47	45.7	51.7	57.6	69.5	87.3		
1.48	46.2	52.2	58.2	70.2	88.2		
1.49	46.6	52.7	58.8	70.9	89.1		
1.50	47.1	53.2	59.3	71.6	89.9		
1.51	47.6	53.7	59.9				
1.52	48.0	54.3	60.5				
1.53	48.5	54.8	61.1				
1.54	49.0	55.3	61.7				
1.55	49.4	55.8	62.3				
1.56	49.9	56.4	62.8				
1.57	50.3	56.9	63.4				
1.58	50.8	57.4	64.0				
1.59	51.3	58.0	64.6				
1.60	51.8	58.5	65.2				
1.61	52.2	59.0	65.8				
1.62	52.7	59.6	66.4				
1.63	53.2	60.1	67.0				
1.64	53.7	60.6	67.6				
1.65	54.1	61.2	68.3				
1.66	54.6	61.7	68.9				
1.67	55.1	62.3	69.5				
1.68	55.6	62.8					
1.69	56.1	63.4					
1.70	56.5	63.9					

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values below and to the left of heavy lines determined experimentally; others computed from the formula $Q = 3.367\ Lh^{3/2}$.* 103-D-1246-1

Head h, feet	Length of weir, *L*, feet						
	1.0	2.0	3.0	4.0	5.0	6.0	7.0
0.20	0.30	0.60	0.90	1.20	1.51	1.81	2.11
.21	.32	.65	.97	1.30	1.62	1.94	2.27
.22	.35	.70	1.04	1.39	1.74	2.08	2.43
.23	.37	.74	1.11	1.48	1.86	2.23	2.60
.24	.40	.79	1.19	1.58	1.98	2.38	2.77
.25	.42	.84	1.26	1.68	2.10	2.53	2.95
.26	.45	.89	1.34	1.78	2.23	2.68	3.12
.27	.47	.94	1.42	1.89	2.36	2.83	3.31
.28	.50	1.00	1.50	2.00	2.49	2.99	3.49
.29	.53	1.05	1.58	2.10	2.63	3.15	3.68
.30	.55	1.11	1.66	2.21	2.77	3.32	3.87
.31	.58	1.16	1.74	2.32	2.90	3.49	4.07
.32	.61	1.22	1.83	2.44	3.05	3.66	4.27
.33	.64	1.28	1.92	2.55	3.19	3.83	4.47
.34	.67	1.34	2.00	2.67	3.34	4.00	4.67
.35	.70	1.39	2.09	2.79	3.49	4.18	4.88
.36	.73	1.45	2.18	2.91	3.64	4.36	5.09
.37	.76	1.52	2.27	3.03	3.79	4.55	5.30
.38	.79	1.58	2.37	3.16	3.94	4.73	5.52
.39	.82	1.64	2.46	3.28	4.10	4.92	5.74
.40	.85	1.70	2.56	3.41	4.26	5.11	5.96
.41	.88	1.77	2.65	3.54	4.42	5.30	6.19
.42	.92	1.83	2.75	3.66	4.58	5.50	6.41
.43	.95	1.90	2.85	3.80	4.75	5.70	6.65
.44	.98	1.96	2.95	3.93	4.91	5.90	6.88
.45	1.02	2.03	3.05	4.06	5.08	6.10	7.11
.46	1.05	2.10	3.15	4.20	5.25	6.30	7.35
.47	1.08	2.17	3.25	4.34	5.42	6.51	7.59
.48	1.12	2.24	3.36	4.48	5.60	6.72	7.84
.49	1.16	2.31	3.46	4.62	5.77	6.93	8.08
.50	1.20	2.38	3.57	4.76	5.95	7.14	8.33
.51		2.45	3.68	4.90	6.13	7.36	8.58
.52		2.52	3.79	5.05	6.31	7.57	8.84
.53		2.60	3.90	5.20	6.50	7.79	9.09
.54		2.67	4.01	5.34	6.68	8.02	9.35
.55		2.75	4.12	5.49	6.87	8.24	9.61
.56		2.82	4.23	5.64	7.05	8.47	9.88
.57		2.90	4.35	5.80	7.24	8.69	10.1
.58		2.97	4.46	5.95	7.44	8.92	10.4
.59		3.05	4.58	6.10	7.63	9.15	10.7
.60		3.13	4.69	6.26	7.82	9.39	11.0
.61		3.21	4.81	6.42	8.02	9.62	11.2
.62		3.29	4.93	6.57	8.22	9.86	11.5
.63		3.37	5.05	6.73	8.42	10.1	11.8
.64		3.45	5.17	6.90	8.62	10.3	12.1
.65		3.53	5.29	7.06	8.82	10.6	12.4
.66		3.61	5.42	7.22	9.03	10.8	12.6
.67		3.69	5.54	7.38	9.23	11.1	12.9
.68		3.81	5.66	7.55	9.44	11.3	13.2
.69		3.90	5.79	7.72	9.65	11.6	13.5
.70		3.98	5.92	7.89	9.86	11.8	13.8

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values below and to the left of heavy lines determined experimentally; others computed from the formula $Q = 3.367\ Lh^{3/2}$.*–Continued. 103-D-1246-2

Head h, feet	Length of weir, L, feet					
	2.0	3.0	4.0	5.0	6.0	7.0
0.71	4.06	6.04	8.06	10.1	12.1	14.1
.72	4.15	6.17	8.23	10.3	12.3	14.4
.73	4.24	6.30	8.40	10.5	12.6	14.7
.74	4.33	6.43	8.57	10.7	12.9	15.0
.75	4.42	6.56	8.75	10.9	13.1	15.3
.76	4.51	6.69	8.92	11.2	13.4	15.6
.77	4.60	6.82	9.10	11.4	13.7	15.9
.78	4.69	6.96	9.28	11.6	13.9	16.2
.79	4.78	7.09	9.46	11.8	14.2	16.6
.80	4.87	7.23	9.64	12.0	14.5	16.9
.81	4.96	7.36	9.82	12.3	14.7	17.2
.82	5.05	7.50	10.0	12.5	15.0	17.5
.83	5.14	7.64	10.2	12.7	15.3	17.8
.84	5.24	7.78	10.4	13.0	15.6	18.1
.85	5.34	7.92	10.6	13.2	15.8	18.5
.86	5.44	8.06	10.7	13.4	16.1	18.8
.87	5.54	8.20	10.9	13.7	16.4	19.1
.88	5.64	8.34	11.1	13.9	16.7	19.5
.89	5.74	8.48	11.3	14.1	17.0	19.8
.90	5.84	8.62	11.5	14.4	17.3	20.1
.91	5.94	8.77	11.7	14.6	17.5	20.5
.92	6.04	8.91	11.9	14.9	17.8	20.8
.93	6.14	9.06	12.1	15.1	18.1	21.1
.94	6.25	9.20	12.3	15.3	18.4	21.5
.95	6.36	9.35	12.5	15.6	18.7	21.8
.96	6.47	9.50	12.7	15.8	19.0	22.2
.97	6.58	9.65	12.9	16.1	19.3	22.5
.98	6.69	9.80	13.1	16.3	19.6	22.9
.99	6.80	9.95	13.3	16.6	19.9	23.2
1.00	6.91	10.1	13.5	16.8	20.2	23.6
1.01		10.5	13.7	17.1	20.5	23.9
1.02		10.6	13.9	17.3	20.8	24.3
1.03		10.8	14.1	17.6	21.1	24.6
1.04		10.9	14.3	17.9	21.4	25.0
1.05		11.1	14.5	18.1	21.7	25.4
1.06		11.3	14.7	18.4	22.1	25.7
1.07		11.4	14.9	18.6	22.4	26.1
1.08		11.6	15.1	18.9	22.7	26.5
1.09		11.7	15.3	19.2	23.0	26.8
1.10		11.9	15.5	19.4	23.3	27.2
1.11		12.1	15.8	19.7	23.6	27.6
1.12		12.2	16.0	20.0	23.9	27.9
1.13		12.4	16.2	20.2	24.3	28.3
1.14		12.5	16.4	20.5	24.6	28.7
1.15		12.7	16.6	20.8	24.9	29.1
1.16		12.9	16.8	21.0	25.2	29.4
1.17		13.0	17.0	21.3	25.6	29.8
1.18		13.2	17.3	21.6	25.9	30.2
1.19		13.4	17.5	21.9	26.2	30.6
1.20		13.6	17.7	22.1	26.6	31.0

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values computed from the formula* $Q = 3.367\ Lh^{3/2}$.–Continued. 103-D-1246-3

Head h, feet	Length of weir, *L*, feet				Head h, feet	Length of weir, *L*, feet	
	4.0	5.0	6.0	7.0		6.0	7.0
1.21	17.9	22.4	26.9	31.4	1.71	45.2	52.7
1.22	18.1	22.7	27.2	31.8	1.72	45.6	53.2
1.23	18.4	23.0	27.6	32.2	1.73	46.0	53.6
1.24	18.6	23.2	27.9	32.5	1.74	46.4	54.1
1.25	18.8	23.5	28.2	32.9	1.75	46.8	54.6
1.26	19.0	23.8	28.6	33.3	1.76	47.2	55.0
1.27	19.3	24.1	28.9	33.7	1.77	47.6	55.5
1.28	19.5	24.4	29.3	34.1	1.78	48.0	56.0
1.29	19.7	24.7	29.6	34.5	1.79	48.4	56.4
1.30	20.0	25.0	29.9	34.9	1.80	48.8	56.9
1.31	20.2	25.2	30.3	35.3	1.81	49.2	57.4
1.32	20.4	25.5	30.6	35.7	1.82	49.6	57.9
1.33	20.7	25.8	31.0	36.2	1.83	50.0	58.3
1.34		26.1	31.3	36.6	1.84	50.4	58.8
1.35		26.4	31.7	37.0	1.85	50.8	59.3
1.36		26.7	32.0	37.4	1.86	51.2	59.8
1.37		27.0	32.4	37.8	1.87	51.7	60.3
1.38		27.3	32.8	38.2	1.88	52.1	60.8
1.39		27.6	33.1	38.6	1.89	52.5	61.2
1.40		27.9	33.5	39.0	1.90	52.9	61.7
1.41		28.2	33.8	39.5	1.91	53.3	62.2
1.42		28.5	34.2	39.9	1.92	53.7	62.7
1.43		28.8	34.5	40.3	1.93	54.2	63.2
1.44		29.1	34.9	40.7	1.94	54.6	63.7
1.45		29.4	35.3	41.2	1.95	55.0	64.2
1.46		29.7	35.6	41.6	1.96	55.4	64.7
1.47		30.0	36.0	42.0	1.97	55.9	65.2
1.48		30.3	36.4	42.4	1.98	56.3	65.7
1.49		30.6	36.7	42.9	1.99	56.7	66.2
1.50		30.9	37.1	43.3	2.00	57.1	66.7
1.51		31.2	37.5	43.7	2.01		67.2
1.52		31.5	37.9	44.2	2.02		67.7
1.53		31.8	38.2	44.6	2.03		68.2
1.54		32.2	38.6	45.0	2.04		68.7
1.55		32.5	39.0	45.5	2.05		69.2
1.56		32.8	39.4	45.9	2.06		69.7
1.57		33.1	39.7	46.4	2.07		70.2
1.58		33.4	40.1	46.8			
1.59		33.8	40.5	47.3			
1.60		34.1	40.9	47.7			
1.61		34.4	41.3	48.1			
1.62		34.7	41.7	48.6			
1.63		35.0	42.0	49.0			
1.64		35.4	42.4	49.5			
1.65		35.7	42.8	50.0			
1.66		36.0	43.2	50.4			
1.67			43.6	50.9			
1.68			44.0	51.3			
1.69			44.4	51.8			
1.70			44.8	52.2			

Table 5-5.—*Discharge of standard Cipolletti weirs in cubic feet per second. Values computed from the formula* $Q = 3.367\ Lh^{3/2}$.—Continued. 103-D-1246-4

Head h, feet	Length of weir, L, feet						
	8.0	9.0	10.0	11.0	12.0	13.0	14.0
0.20	2.41	2.71	3.01	3.31	3.61	3.91	4.22
.21	2.59	2.92	3.24	3.56	3.89	4.21	4.54
.22	2.78	3.13	3.47	3.82	4.17	4.52	4.86
.23	2.97	3.34	3.71	4.09	4.46	4.83	5.20
.24	3.17	3.56	3.96	4.35	4.75	5.15	5.54
.25	3.37	3.79	4.21	4.63	5.05	5.47	5.89
.26	3.57	4.02	4.46	4.91	5.36	5.80	6.25
.27	3.78	4.25	4.72	5.20	5.67	6.14	6.61
.28	3.99	4.49	4.99	5.49	5.99	6.49	6.98
.29	4.21	4.73	5.26	5.78	6.31	6.84	7.36
.30	4.43	4.98	5.53	6.09	6.64	7.19	7.75
.31	4.65	5.23	5.81	6.39	6.97	7.55	8.14
.32	4.88	5.48	6.09	6.70	7.31	7.92	8.53
.33	5.11	5.74	6.38	7.02	7.66	8.30	8.94
.34	5.34	6.01	6.68	7.34	8.01	8.68	9.35
.35	5.58	6.27	6.97	7.67	8.37	9.06	9.76
.36	5.82	6.54	7.27	8.00	8.73	9.45	10.2
.37	6.06	6.82	7.58	8.34	9.09	9.85	10.6
.38	6.31	7.10	7.89	8.68	9.46	10.3	11.0
.39	6.56	7.38	8.20	9.02	9.84	10.7	11.5
.40	6.81	7.67	8.52	9.37	10.2	11.1	11.9
.41	7.07	7.95	8.84	9.72	10.6	11.5	12.4
.42	7.33	8.25	9.16	10.1	11.0	11.9	12.8
.43	7.59	8.54	9.49	10.4	11.4	12.3	13.3
.44	7.86	8.84	9.83	10.8	11.8	12.8	13.8
.45	8.13	9.15	10.2	11.2	12.2	13.2	14.2
.46	8.40	9.45	10.5	11.6	12.6	13.7	14.7
.47	8.68	9.76	10.9	11.9	13.0	14.1	15.2
.48	8.96	10.1	11.2	12.3	13.4	14.6	15.7
.49	9.24	10.4	11.6	12.7	13.9	15.0	16.2
.50	9.52	10.7	11.9	13.1	14.3	15.6	16.7
.51	9.81	11.0	12.3	13.5	14.7	15.9	17.2
.52	10.1	11.4	12.6	13.9	15.2	16.4	17.7
.53	10.4	11.7	13.0	14.3	15.6	16.9	18.2
.54	10.7	12.0	13.4	14.7	16.0	17.4	18.7
.55	11.0	12.4	13.7	15.1	16.5	17.9	19.2
.56	11.3	12.7	14.1	15.5	16.9	18.3	19.8
.57	11.6	13.0	14.5	15.9	17.4	18.8	20.3
.58	11.9	13.4	14.9	16.4	17.9	19.3	20.8
.59	12.2	13.7	15.3	16.8	18.3	19.8	21.4
.60	12.5	14.1	15.7	17.2	18.8	20.3	21.9
.61	12.8	14.4	16.0	17.7	19.3	20.9	22.5
.62	13.2	14.8	16.4	18.1	19.7	21.4	23.0
.63	13.5	15.2	16.8	18.5	20.2	21.9	23.6
.64	13.8	15.5	17.2	19.0	20.7	22.4	24.1
.65	14.1	15.9	17.6	19.4	21.2	22.9	24.7
.66	14.4	16.3	18.1	19.9	21.7	23.5	25.3
.67	14.8	16.6	18.5	20.3	22.2	24.0	25.9
.68	15.1	17.0	18.9	20.8	22.7	24.5	26.4
.69	15.4	17.4	19.3	21.2	23.2	25.1	27.0
.70	15.8	17.8	19.7	21.7	23.7	25.6	27.6

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values computed from the formula $Q = 3.367\ Lh^{3/2}$.*–Continued. 103-D-1246-5

Head h, feet	Length of weir, *L*, feet						
	8.0	9.0	10.0	11.0	12.0	13.0	14.0
0.71	16.1	18.1	20.1	22.2	24.2	26.2	28.2
.72	16.5	18.5	20.6	22.6	24.7	26.7	28.8
.73	16.8	18.9	21.0	23.1	25.2	27.3	29.4
.74	17.2	19.3	21.4	23.6	25.7	27.9	30.0
.75	17.5	19.7	21.9	24.1	26.2	28.4	30.6
.76	17.8	20.1	22.3	24.5	26.8	29.0	31.2
.77	18.2	20.5	22.8	25.0	27.3	29.6	31.9
.78	18.6	20.9	23.2	25.5	27.8	30.2	32.5
.79	18.9	21.3	23.6	26.0	28.4	30.7	33.1
.80	19.3	21.7	24.1	26.5	28.9	31.3	33.7
.81	19.6	22.1	24.6	27.0	29.5	31.9	34.4
.82	20.0	22.5	25.0	27.5	30.0	32.5	35.0
.83	20.4	22.9	25.5	28.0	30.6	33.1	35.6
.84	20.7	23.3	25.9	28.5	31.1	33.7	36.3
.85	21.1	23.7	26.4	29.0	31.7	34.3	36.9
.86	21.5	24.2	26.9	29.5	32.2	34.9	37.6
.87	21.9	24.6	27.3	30.1	32.8	35.5	38.3
.88	22.2	25.0	27.8	30.6	33.4	36.1	38.9
.89	22.6	25.4	28.3	31.1	33.9	36.8	39.6
.90	23.0	25.9	28.8	31.6	34.5	37.4	40.3
.91	23.4	26.3	29.2	32.2	35.1	38.0	40.9
.92	23.8	26.7	29.7	32.7	35.7	38.6	41.6
.93	24.2	27.2	30.2	33.2	36.2	39.3	42.3
.94	24.6	27.6	30.7	33.8	36.8	39.9	43.0
.95	24.9	28.1	31.2	34.3	37.4	40.5	43.7
.96	25.3	28.5	31.7	34.8	38.0	41.2	44.3
.97	25.7	29.0	32.2	35.4	38.6	41.8	45.0
.98	26.1	29.4	32.7	35.9	39.2	42.5	45.7
.99	26.5	29.9	33.2	36.5	39.8	43.1	46.4
1.00	26.9	30.3	33.7	37.0	40.4	43.8	47.1
1.01	27.3	30.8	34.2	37.6	41.0	44.4	47.9
1.02	27.8	31.2	34.7	38.2	41.6	45.1	48.6
1.03	28.2	31.7	35.2	38.7	42.2	45.8	49.3
1.04	28.6	32.1	35.7	39.3	42.9	46.4	50.0
1.05	29.0	32.6	36.2	39.9	43.5	47.1	50.7
1.06	29.4	33.1	36.8	40.4	44.1	47.8	51.4
1.07	29.8	33.5	37.3	41.0	44.7	48.5	52.2
1.08	30.2	34.0	37.8	41.6	45.4	49.1	52.9
1.09	30.6	34.5	38.3	42.2	46.0	49.8	53.6
1.10	31.1	35.0	38.8	42.7	46.6	50.5	54.4
1.11	31.5	35.4	39.4	43.3	47.3	51.2	55.1
1.12	31.9	35.9	39.9	43.9	47.9	51.9	55.9
1.13	32.4	36.4	40.4	44.5	48.5	52.6	56.6
1.14	32.8	36.9	41.0	45.1	49.2	53.3	57.4
1.15	33.2	37.4	41.5	45.7	49.8	54.0	58.1
1.16	33.7	37.9	42.1	46.3	50.5	54.7	58.9
1.17	34.1	38.4	42.6	46.9	51.1	55.4	59.7
1.18	34.5	38.8	43.2	47.5	51.8	56.1	60.4
1.19	35.0	39.3	43.7	48.1	52.5	56.8	61.2
1.20	35.4	39.8	44.3	48.7	53.1	57.5	62.0

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values computed from the formula* $Q = 3.367\ Lh^{3/2}$.–Continued. 103-D-1246-6

Head h, feet	Length of weir, L, feet						
	8.0	9.0	10.0	11.0	12.0	13.0	14.0
1.21	35.9	40.3	44.8	49.3	53.8	58.3	62.7
1.22	36.3	40.8	45.4	49.9	54.5	59.0	63.5
1.23	36.7	41.3	45.9	50.5	55.1	59.7	64.3
1.24	37.2	41.8	46.5	51.1	55.8	60.4	65.1
1.25	37.6	42.4	47.1	51.8	56.5	61.2	65.9
1.26	38.1	42.9	47.6	52.4	57.2	61.9	66.7
1.27	38.6	43.4	48.2	53.0	57.8	62.7	67.5
1.28	39.0	43.9	48.8	53.6	58.5	63.4	68.3
1.29	39.5	44.4	49.3	54.3	59.2	64.1	69.1
1.30	39.9	44.9	49.9	54.9	59.9	64.9	69.9
1.31	40.4	45.4	50.5	55.5	60.6	65.6	70.7
1.32	40.9	46.0	51.1	56.2	61.3	66.4	71.5
1.33	41.3	46.5	51.6	56.8	62.0	67.1	72.3
1.34	41.8	47.0	52.2	57.5	62.7	67.9	73.1
1.35	42.2	47.5	52.8	58.1	63.4	68.7	73.9
1.36	42.7	48.1	53.4	58.7	64.1	69.4	74.8
1.37	43.2	48.6	54.0	59.4	64.8	70.2	75.6
1.38	43.7	49.1	54.6	60.0	65.5	71.0	76.4
1.39	44.1	49.7	55.2	60.7	66.2	71.7	77.3
1.40	44.6	50.2	55.8	61.4	66.9	72.5	78.1
1.41	45.1	50.7	56.4	62.0	67.7	73.3	78.9
1.42	45.6	51.3	57.0	62.7	68.4	74.1	79.8
1.43	46.1	51.8	57.6	63.3	69.1	74.9	80.6
1.44	46.5	52.4	58.2	64.0	69.8	75.6	81.5
1.45	47.0	52.9	58.8	64.7	70.6	76.4	82.3
1.46	47.5	53.5	59.4	65.3	71.3	77.2	83.2
1.47	48.0	54.0	60.0	66.0	72.0	78.0	84.0
1.48	48.5	54.6	60.6	66.7	72.8	78.8	84.9
1.49	49.0	55.1	61.2	67.4	73.5	79.6	85.7
1.50	49.5	55.7	61.9	68.0	74.2	80.4	86.6
1.51	50.0	56.2	62.5	68.7			
1.52	50.5	56.8	63.1	69.4			
1.53	51.0	57.3	63.7	70.1			
1.54	51.5	57.9	64.4				
1.55	52.0	58.5	65.0				
1.56	52.5	59.0	65.6				
1.57	53.0	59.6	66.2				
1.58	53.5	60.2	66.9				
1.59	54.0	60.8	67.5				
1.60	54.5	61.3	68.1				
1.61	55.0	61.9	68.8				
1.62	55.5	62.5	69.4				
1.63	56.1	63.1	70.1				
1.64	56.6	63.6					
1.65	57.1	64.2					
1.66	57.6	64.8					
1.67	58.1	65.4					
1.68	58.7	66.0					
1.69	59.2	66.6					
1.70	59.7	67.2					

Table 5-5.–*Discharge of standard Cipolletti weirs in cubic feet per second. Values computed from the formula* $Q = 3.367\ Lh^{3/2}$.–Continued. 103-D-1246-7

Head h, feet	Length of weir, *L*, feet		Head h, feet	Length of weir, *L*, feet		Head h, feet	Length of weir, *L*, feet	
	15.0	16.0		15.0	16.0		15.0	16.0
0.20	4.52	4.82	0.71	30.2	32.2	1.21	67.2	71.7
			.72	30.9	32.9	1.22	68.1	72.6
.21	4.86	5.18	.73	31.5	33.6	1.23	68.9	73.5
.22	5.21	5.56	.74	32.2	34.3	1.24	69.7	74.4
.23	5.57	5.94	.75	32.8	35.0	1.25	70.6	75.3
.24	5.94	6.33						
.25	6.31	6.73	.76	33.5	35.7	1.26	71.4	76.2
			.77	34.1	36.4	1.27	72.3	77.1
.26	6.70	7.14	.78	34.8	37.1	1.28	73.1	78.0
.27	7.09	7.56	.79	35.5	37.8	1.29	74.0	78.9
.28	7.48	7.98	.80	36.1	38.6	1.30	74.9	79.9
.29	7.89	8.41						
.30	8.30	8.85	.81	36.8	39.3	1.31	75.7	80.8
			.82	37.5	40.0	1.32	76.6	81.7
.31	8.72	9.30	.83	38.2	40.7	1.33	77.5	82.6
.32	9.14	9.75	.84	38.9	41.5	1.34	78.3	83.6
.33	9.57	10.2	.85	39.6	42.2	1.35	79.2	84.5
.34	10.0	10.7						
.35	10.5	11.2	.86	40.3	43.0	1.36	80.1	85.4
			.87	41.0	43.7	1.37	81.0	86.4
.36	10.9	11.6	.88	41.7	44.5	1.38	81.9	87.3
.37	11.4	12.1	.89	42.4	45.2	1.39	82.8	88.3
.38	11.8	12.6	.90	43.1	46.0	1.40	83.7	89.2
.39	12.3	13.1						
.40	12.8	13.6	.91	43.8	46.8	1.41	84.6	90.2
			.92	44.6	47.5	1.42	85.5	91.2
.41	13.3	14.1	.93	45.3	48.3	1.43	86.4	92.1
.42	13.8	14.7	.94	46.0	49.1	1.44	87.3	93.1
.43	14.2	15.2	.95	46.8	49.9	1.45	88.2	94.1
.44	14.7	15.7						
.45	15.3	16.3	.96	47.5	50.7	1.46	89.1	95.0
			.97	48.3	51.5	1.47	90.0	96.0
.46	15.8	16.8	.98	49.0	52.3	1.48	90.9	97.0
.47	16.3	17.4	.99	49.8	53.1	1.49	91.9	98.0
.48	16.8	17.9	1.00	50.5	53.9	1.50	92.8	99.0
.49	17.3	18.5						
.50	17.9	19.1	1.01	51.3	54.7			
			1.02	52.0	55.5			
.51	18.4	19.6	1.03	52.8	56.3			
.52	18.9	20.2	1.04	53.6	57.1			
.53	19.5	20.8	1.05	54.3	58.0			
.54	20.0	21.4						
.55	20.6	22.0	1.06	55.1	58.8			
			1.07	55.9	59.6			
.56	21.2	22.6	1.08	56.7	60.5			
.57	21.7	23.2	1.09	57.5	61.3			
.58	22.3	23.8	1.10	58.3	62.2			
.59	22.9	24.4						
.60	23.5	25.0	1.11	59.1	63.0			
			1.12	59.9	63.9			
.61	24.1	25.7	1.13	60.7	64.7			
.62	24.7	26.3	1.14	61.5	65.6			
.63	25.3	26.9	1.15	62.3	66.4			
.64	25.9	27.6						
.65	26.5	28.2	1.16	63.1	67.3			
			1.17	63.9	68.2			
.66	27.1	28.9	1.18	64.7	69.1			
.67	27.7	29.5	1.19	65.6	69.9			
.68	28.3	30.2	1.20	66.4	70.8			
.69	29.0	30.9						
.70	29.6	31.6						

Table 5-6.–*Discharge of standard suppressed rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\ Lh^{3/2}$. 103-D-1247-1

Head h, feet	Length of weir, *L*, in feet					
	1.0	1.5	2.0	3.0	4.0	5.0
0.20	0.30	0.45	0.60	0.89	1.19	1.49
.21	.32	.48	.64	.96	1.28	1.60
.22	.34	.52	.69	1.03	1.37	1.72
.23	.37	.55	.74	1.10	1.47	1.84
.24	.39	.59	.78	1.18	1.57	1.96
.25	.42	.62	.83	1.25	1.67	2.08
.26	.44	.66	.88	1.33	1.77	2.21
.27	.47	.70	.93	1.40	1.87	2.34
.28	.49	.74	.99	1.48	1.97	2.47
.29	.52	.78	1.04	1.56	2.08	2.60
.30	.55	.82	1.09	1.64	2.19	2.74
.31	.58	.86	1.15	1.72	2.30	2.87
.32	.60	.90	1.21	1.81	2.41	3.01
.33	.63	.95	1.26	1.89	2.53	3.16
.34	.66	.99	1.32	1.98	2.64	3.30
.35		1.03	1.38	2.07	2.76	3.45
.36		1.08	1.44	2.16	2.88	3.60
.37		1.12	1.50	2.25	3.00	3.75
.38		1.17	1.56	2.34	3.12	3.90
.39		1.22	1.62	2.43	3.24	4.06
.40		1.26	1.68	2.53	3.37	4.21
.41		1.31	1.75	2.62	3.50	4.37
.42		1.36	1.81	2.72	3.63	4.53
.43		1.41	1.88	2.82	3.76	4.70
.44		1.46	1.94	2.92	3.89	4.86
.45		1.51	2.01	3.02	4.02	5.03
.46		1.56	2.08	3.12	4.16	5.19
.47		1.61	2.15	3.22	4.29	5.37
.48		1.66	2.22	3.32	4.43	5.54
.49		1.71	2.28	3.43	4.57	5.71
.50		1.77	2.35	3.53	4.71	5.89
.51			2.43	3.64	4.85	6.06
.52			2.50	3.75	5.00	6.24
.53			2.57	3.85	5.14	6.43
.54			2.64	3.96	5.29	6.61
.55			2.72	4.08	5.43	6.79
.56			2.79	4.19	5.58	6.98
.57			2.87	4.30	5.73	7.17
.58			2.94	4.41	5.88	7.35
.59			3.02	4.53	6.04	7.55
.60			3.10	4.64	6.19	7.74
.61			3.17	4.76	6.35	7.93
.62			3.25	4.88	6.50	8.13
.63			3.33	5.00	6.66	8.33
.64			3.41	5.12	6.82	8.53
.65			3.49	5.24	6.98	8.73
.66			3.57	5.36	7.14	8.93
.67			3.65	5.48	7.31	9.13
.68				5.60	7.47	9.34
.69				5.73	7.63	9.54
.70				5.85	7.80	9.75

Table 5-6.–*Discharge of standard suppressed rectangular weirs in cubic feet per second. Computed from the formula* $Q = 3.33\ Lh^{3/2}$.–Continued. 103-D-1247-2

Head h, feet	Length of weir, L, in feet			Head h, feet	Length of weir, L, in feet	
	3.0	4.0	5.0		4.0	5.0
0.71	5.98	7.97	9.96	1.21	17.7	22.2
.72	6.10	8.14	10.2	1.22	17.9	22.4
.73	6.23	8.31	10.4	1.23	18.2	22.7
.74	6.36	8.48	10.6	1.24	18.4	23.0
.75	6.49	8.65	10.8	1.25	18.6	23.3
.76	6.62	8.83	11.0	1.26	18.8	23.6
.77	6.75	9.00	11.3	1.27	19.1	23.8
.78	6.88	9.18	11.5	1.28	19.3	24.1
.79	7.02	9.35	11.7	1.29	19.5	24.4
.80	7.15	9.53	11.9	1.30	19.7	24.7
.81	7.28	9.71	12.1	1.31	20.0	25.0
.82	7.42	9.89	12.4	1.32	20.2	25.3
.83	7.55	10.1	12.6	1.33	20.4	25.5
.84	7.69	10.3	12.8	1.34		25.8
.85	7.83	10.4	13.1	1.35		26.1
.86	7.97	10.6	13.3	1.36		26.4
.87	8.11	10.8	13.5	1.37		26.7
.88	8.25	11.0	13.8	1.38		27.0
.89	8.39	11.2	14.0	1.39		27.3
.90	8.53	11.4	14.2	1.40		27.6
.91	8.67	11.6	14.5	1.41		27.9
.92	8.82	11.8	14.7	1.42		28.2
.93	8.96	11.9	14.9	1.43		28.5
.94	9.10	12.1	15.2	1.44		28.8
.95	9.25	12.3	15.4	1.45		29.1
.96	9.40	12.5	15.7	1.46		29.4
.97	9.54	12.7	15.9	1.47		29.7
.98	9.69	12.9	16.2	1.48		30.0
.99	9.84	13.1	16.4	1.49		30.3
1.00	9.99	13.3	16.7	1.50		30.6
1.01		13.5	16.9	1.51		30.9
1.02		13.7	17.2	1.52		31.2
1.03		13.9	17.4	1.53		31.5
1.04		14.1	17.7	1.54		31.8
1.05		14.3	17.9	1.55		32.1
1.06		14.5	18.2	1.56		32.4
1.07		14.7	18.4	1.57		32.8
1.08		14.9	18.7	1.58		33.1
1.09		15.2	19.0	1.59		33.4
1.10		15.4	19.2	1.60		33.7
1.11		15.6	19.5	1.61		34.0
1.12		15.8	19.7	1.62		34.3
1.13		16.0	20.0	1.63		34.7
1.14		16.2	20.3	1.64		35.0
1.15		16.4	20.5	1.65		35.3
1.16		16.6	20.8	1.66		35.6
1.17		16.9	21.1			
1.18		17.1	21.3			
1.19		17.3	21.6			
1.20		17.5	21.9			

Table 5-7.–*Discharge of 90° V-notch weirs in cubic feet per second. Computed from the formula $Q = 2.49h^{2.48}$.* 103-D-1248

Head h, feet	Discharge, cfs	Head h, feet	Discharge, cfs	Head h, feet	Discharge, cfs
0.20	0.04	0.55	0.56	0.90	1.92
.21	.05	.56	.59	.91	1.97
.22	.06	.57	.62	.92	2.02
.23	.06	.58	.64	.93	2.08
.24	.07	.59	.67	.94	2.13
.25	.08	.60	.70	.95	2.19
.26	.09	.61	.73	.96	2.25
.27	.10	.62	.76	.97	2.31
.28	.11	.63	.79	.98	2.37
.29	.12	.64	.82	.99	2.43
.30	.13	.65	.85	1.00	2.49
.31	.14	.66	.89	1.01	2.55
.32	.15	.67	.92	1.02	2.61
.33	.16	.68	.96	1.03	2.68
.34	.17	.69	.99	1.04	2.74
.35	.18	.70	1.03	1.05	2.81
.36	.20	.71	1.06	1.06	2.87
.37	.21	.72	1.10	1.07	2.94
.38	.23	.73	1.14	1.08	3.01
.39	.24	.74	1.18	1.09	3.08
.40	.26	.75	1.22	1.10	3.15
.41	.27	.76	1.26	1.11	3.22
.42	.29	.77	1.30	1.12	3.30
.43	.31	.78	1.34	1.13	3.37
.44	.32	.79	1.39	1.14	3.44
.45	.34	.80	1.43	1.15	3.52
.46	.36	.81	1.48	1.16	3.59
.47	.38	.82	1.52	1.17	3.67
.48	.40	.83	1.57	1.18	3.75
.49	.42	.84	1.61	1.19	3.83
.50	.45	.85	1.66	1.20	3.91
.51	.47	.86	1.71	1.21	3.99
.52	.49	.87	1.76	1.22	4.07
.53	.52	.88	1.81	1.23	4.16
.54	.54	.89	1.86	1.24	4.24
				1.25	4.33

F. WEIR BOXES

5-22. ***Purpose and Description.***—Weir boxes are small structures used in combination with pipe turnouts (fig. 5-26) to dissipate excess energy and measure rate of flow of water to laterals or farm turnouts. Three-foot weir boxes developed by Reclamation have been successfully used for a number of years. They are equipped with 3-foot suppressed rectangular weirs and are capable of measuring flows up to 5 cfs with effective heads up to 6 feet on the control gate. These structures are economical to build, easy to operate and have proven to be an accurate and reliable method of measuring small flows. Recently a 4-foot weir box was developed and calibrated by Reclamation. The box is capable of measuring flows up to about 12 cfs with effective heads of 6 feet on the control gate. In both of these structures, the water passes from the canal through a gated turnout, into a short length of pipe, and through a system of baffles into the weir box. Although the baffle arrangement in the 3-foot weir box is different from that in the 4-foot weir box, both baffle assemblies dissipate excess energy and distribute the inflow from the pipe so that the water surface in the weir pool is smooth and free from turbulence as it passes over the weir.

5-23. ***Design Considerations.***—(a) *Head Discharge Relationship.*—Figures 5-27 and 5-28 are design drawings for 3- and 4-foot weir boxes respectively. Examination of these drawings show that the weir pool dimensions and the placement of the weir gage do not conform to the standard conditions given in subchapter E for standard weirs. Therefore, the standard discharge equation for rectangular weirs is not applicable to weir boxes. The discharges for various heads are given on the design drawings. These discharges were determined from full-scale model studies conducted in Reclamation's Hydraulic Laboratory in Denver, CO. [4] [5].

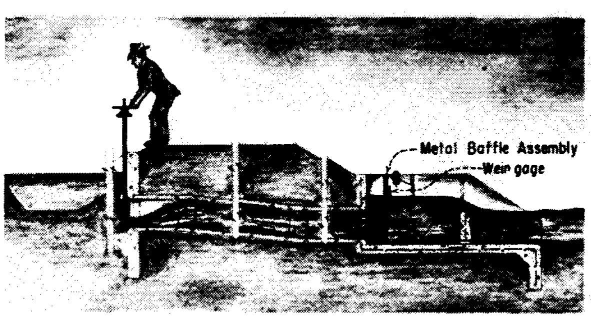


Figure 5-26. **Longitudinal section turnout and weir box. PX-D-31417**

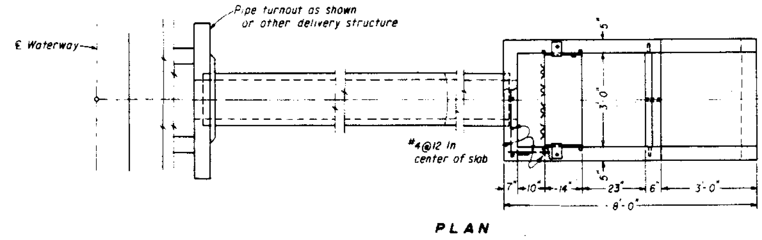
(b) *Head Losses.*—The inlet loss to the pipe is 0.5 of the velocity head in the pipe with the slide gate fully opened. The pipe loss is controlled by the pipe diameter, length of pipe, and discharge. Normally the pipe should be sized so that the water velocity in the full pipe at the entrance to the box does not exceed about 4 feet per second. The loss through the pipe outlet, baffle assembly, and into the stilling well vary with the discharge. In the 4-foot weir box, the loss varies uniformly as shown in figure 5-29 from 0.045 foot for a discharge of 3 cfs to 0.33 foot for a discharge of 12.4 cfs. No figures are available on the losses through the 3-foot weir box baffle assembly, however, experience indicates that for a flow of 5 cfs the head loss through the baffle assembly and stilling pool is about 0.15 foot.

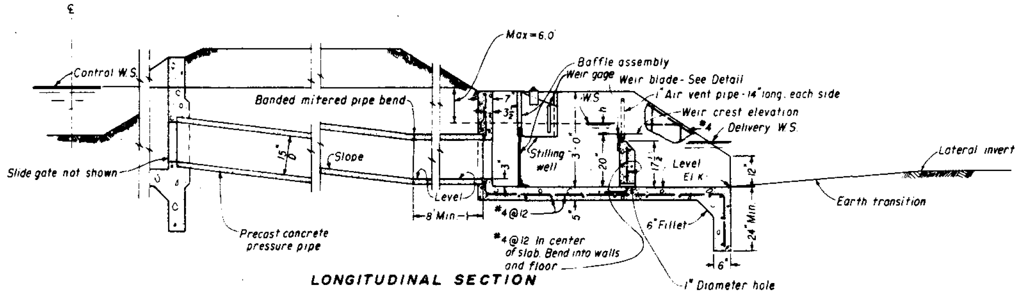
The following example problem gives a procedure for selecting and setting a weir box.

5-24. ***Design Example.***—For this example, assume that water is to be turned out from a canal to a lateral through a turnout with a weir box as shown in figure 5-28. The control water surface in the canal is assumed to be at elevation 103.00. The delivery water surface elevation required in the lateral is elevation 101.35. The normal water depth in the lateral is 1.0 foot. Assume the length of pipe between the turnout gate and the weir box is 50 feet. Further assume that there is one 5° mitered bend in the pipe. Then, select and set a weir box at the downstream end of the turnout to measure a flow of 9 cfs.

(a) *Solution.*—From the continuity equation, $Q = V_1 A_1 = V_2 A_2$, find the area of pipe to satisfy the allowable velocity of 4 feet per second in the pipe.

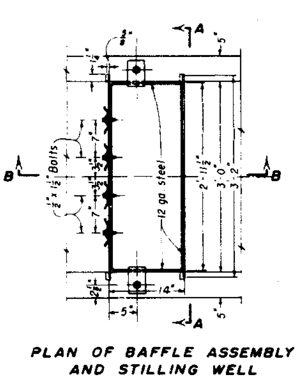
$$Q = V A$$
$$A = 9/4$$
$$A = 2.25 \text{ square feet}$$

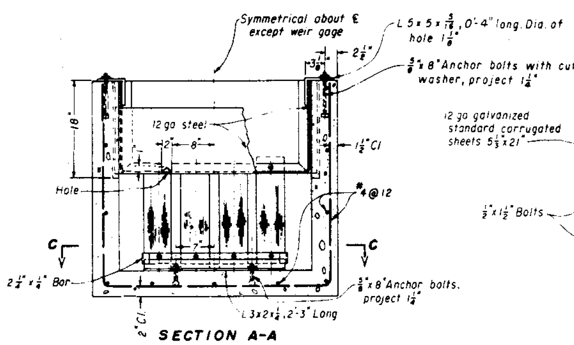


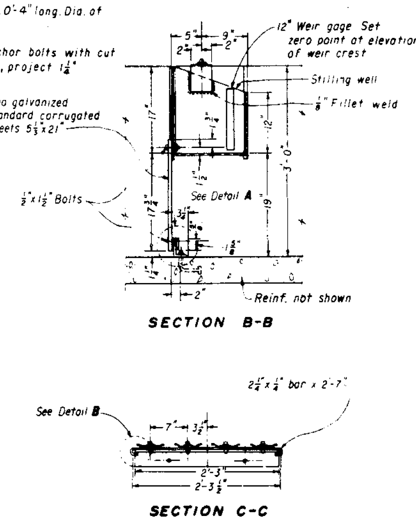


DISCHARGE TABLE

h (ft.)	Q (cfs)
.10	.39
.12	.48
.14	.58
.16	.68
.18	.78
.20	.90
.22	1.05
.24	1.20
.26	1.37
.28	1.55
.30	1.73
.32	1.91
.34	2.09
.36	2.27
.38	2.45
.40	2.64
.42	2.83
.44	3.02
.46	3.22
.48	3.43
.50	3.65
.52	3.87
.54	4.10
.56	4.34
.58	4.58
.60	4.83
.62	5.00

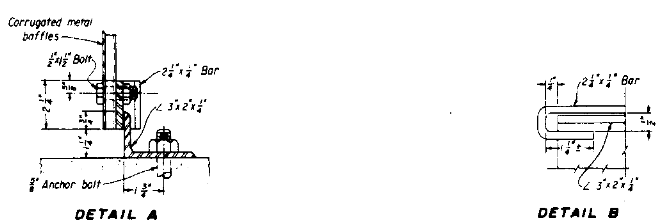


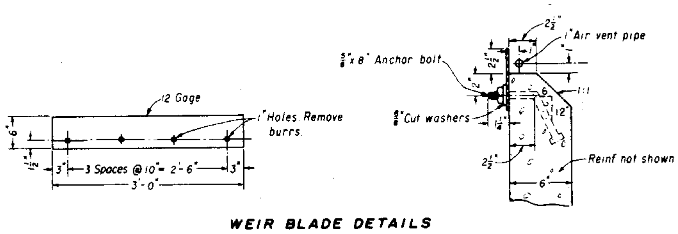




ESTIMATED QUANTITIES
(Weir box)

Concrete .... 1.5 Cu Yds
Reinforcement steel .... 150 Lbs
Miscellaneous metalwork .... 18 Lbs
Baffle assembly .... 1 Each

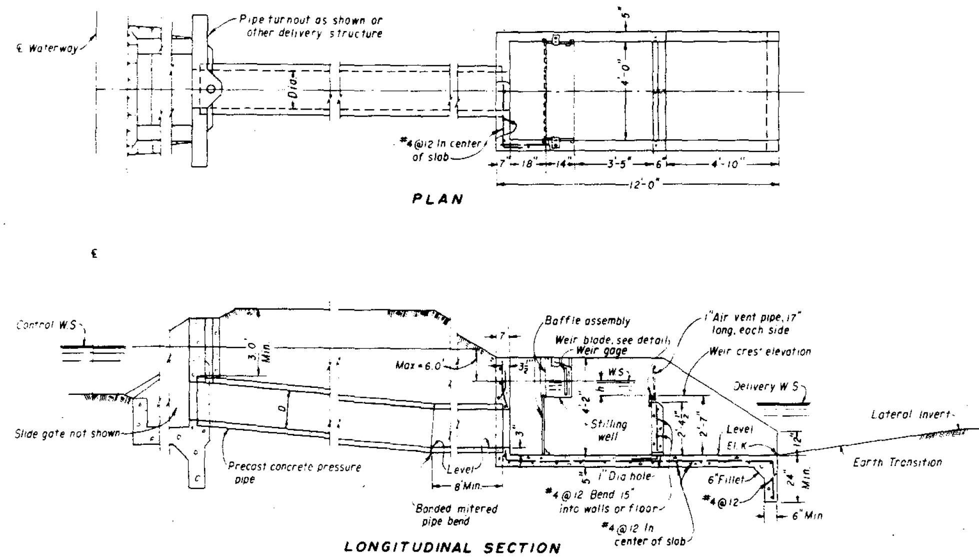




NOTES

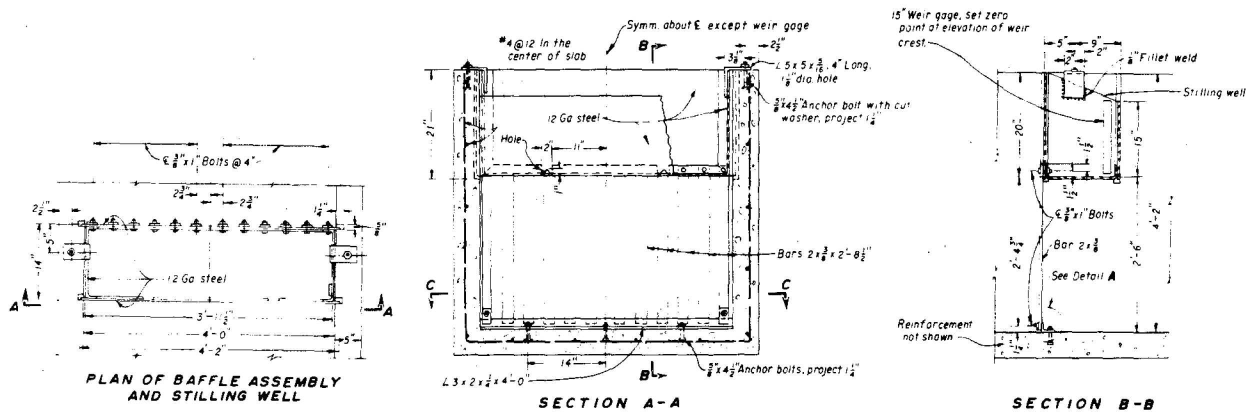
Anchor bolts to have square heads and hex nuts
For weir gage, see Figure 5-25
"h" is head on weir, for Q shown in table
Baffle assembly and all miscellaneous metalwork shall be galvanized by the hot dip process after fabrication
Baffle assembly includes all materials required for installation
For precast concrete pipe embedment, see Figure 8-14
For precast concrete pipe bends, see Figure 8-10
Bolted or welded fabrication of the stilling well may be used; if bolted use $\frac{1}{4}$"x$\frac{1}{2}$" bolts @ 6"±; if welded use continuous $\frac{1}{8}$" fillet welds

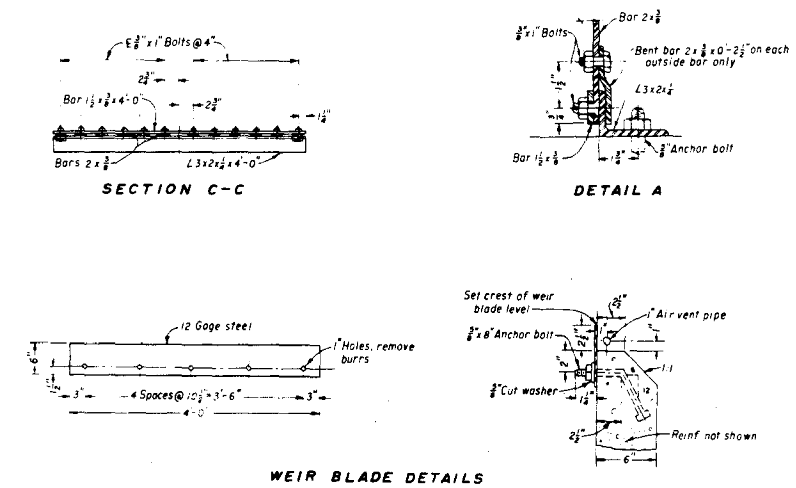
Figure 5-27. **Three-foot weir box. 103-D-1249**



DISCHARGE TABLE

h (ft)	Q (cfs)	h (ft)	Q (cfs)
.10	.4	.50	4.9
.12	.6	.52	5.2
.14	.7	.54	5.5
.16	.8	.56	5.8
.18	1.0	.58	6.1
.20	1.2	.60	6.4
.22	1.4	.62	6.8
.24	1.6	.64	7.1
.26	1.8	.66	7.5
.28	2.0	.68	7.9
.30	2.2	.70	8.3
.32	2.5	.72	8.7
.34	2.7	.74	9.1
.36	2.9	.76	9.5
.38	3.2	.78	9.9
.40	3.5	.80	10.4
.42	3.7	.82	10.9
.44	4.0	.84	11.4
.46	4.3	.86	11.9
.48	4.6	.88	12.4





ESTIMATED QUANTITIES
(Weir box)

Concrete .... 2.8 Cu. Yds
Reinforcement steel .... 280 Lbs
Miscellaneous metalwork .... 18 Lbs
Baffle assembly .... 1 Each

NOTES

Anchor bolts to have square heads and hex nuts
"h" is head on weir, for Q shown in table.
Baffle assembly and all miscellaneous metalwork shall be galvanized by the hot dip process after fabrication.
Baffle assembly includes all materials required for installation
Bolted or welded fabrication of the stilling well may be used; if bolted, use 1/4" x 1/2" bolts @ 6" ±; if welded, use continuous 1/8" fillet welds
For precast concrete pipe embedment, see Figure 8-14
For weir gage, see Figure 5-25
For precast concrete pipe bends, see Figure 8-10

Figure 5-28. **Four-foot weir box. 103-D-1250**

Then from table 8-1 select a 21-inch-diameter pipe which has more than adequate pipe area. The mean velocity of water in the pipe, V, velocity head, h_v, and the friction loss per 1,000 feet of pipe, H, can also be read from table 8-1 after the pipe diameter has been determined as shown above.

$$\text{Pipe inlet loss} = 0.5\ h_v = 0.5\ (0.22) = 0.11 \text{ foot}$$

$$\text{and the pipe loss} = \frac{H}{1000}(L) = \frac{3.22}{1000}(50) = 0.16 \text{ foot}$$

From figure 8-1, Loss Coefficients for Pipe Bends, the loss coefficient (ζ) for a single angle miter bend of 5^o is 0.013. The bend loss is then:

$$\text{Bend loss} = 0.013\ h_v = 0.013\ (0.22) = \text{about } 0.003 \text{ foot}$$

Since the bend loss is so small, it can be neglected for this problem.

The pipe outlet loss and the baffle assembly loss are determined from figure 5-29. With a discharge of 9 cfs, the pipe outlet loss and baffle assembly loss is 0.15 foot. From the discharge table on the weir box drawing, figure 5-28, a 4-foot weir box will measure a flow of 9 cfs with 0.74 foot of head on the weir. The total head loss from the canal to the weir is the sum of the pipe inlet loss, pipe loss, baffle assembly loss, bend loss if any, and the head on the weir. Therefore

$$H_L = 0.11 + 0.16 + 0.15 + 0.74$$

$$H_L = 1.16 \text{ feet}$$

The maximum elevation of the weir crest is:

$$\text{El. weir crest} = 103.00 - 1.16 = 101.84 \text{ feet}$$

then:

El. K (fig. 5-28)
= El. weir crest − weir height

El. K = 101.84 − 2.58 = 99.26 feet

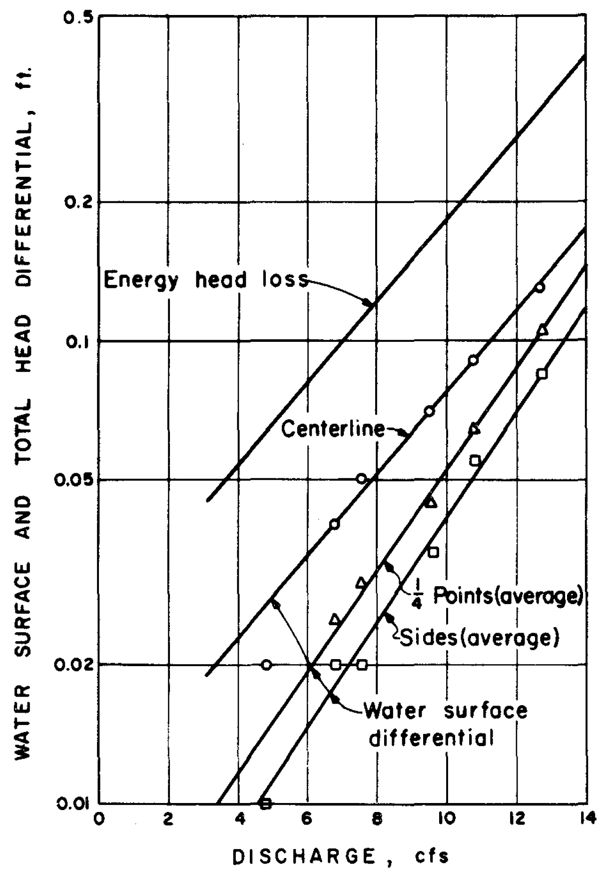


Figure 5-29. **Water surface drop and total head loss across baffles and weir gage stilling well of 4-foot weir box. 103-D-1251**

Since the delivery water surface (DWS) elevation required in the lateral was 101.35, and the normal depth in the lateral was 1.0 foot, set the invert of the lateral at elevation 100.35. In this problem the DWS could have been as high as 101.59 without submerging the weir.

Always set the crest of the weir at least 0.25 foot above the downstream water surface to prevent submergence and to assure air circulation under the nappe of the weir.

If the differential head between the control water surface in the canal and the water surface in the weir box exceeds the head loss through the structure, the turnout gate can be partially closed to obtain the appropriate water surface elevation in the weir box for the desired discharge.

G. OPEN-FLOW METERS

5-25. ***Purpose and Description.***—Open-flow meters are propeller-type meters which may be installed at the ends of gravity pipe turnouts as shown in figures 5-30 and 5-31, to measure and record the rate of flow of water. The multibladed, conical propeller is rotated by the energy of the moving water. The rotation of the propeller actuates a register which totals the flow and gives a direct volumetric reading in gallons, cubic feet, acre-feet, or in any other desired units common to water measurement. Some meters not only totalize the flow but give an instantaneous discharge reading in desired units on a dial, similar to a speedometer in an automobile.

Open-flow meters are commercially available for pipe diameters ranging from 4 to 72 inches. The meter size is designated according to the diameter of the conduit where it will be used. Thus, a 24-inch meter would be used in a 24-inch-diameter pipe.

Open-flow meters have a reasonably accurate measurement range of about 1 to 10, that is, the meter can measure and total flows up to 10 times the minimum flow. They are generally accurate to plus or minus 2 to 5 percent of the actual discharge. Some manufacturers guarantee accuracy to 2 percent of the true flow for all flows above the minimum specified rate.

5-26 ***Design Considerations.***—(a) *Velocity.*—Open-flow meters provide accurate measurement for flow velocities of 0.5 to 17 feet per second; however, measurement accuracy is likely to be affected when the velocity of the water is less than about 1.5 feet per second. For velocities exceeding 8 feet per second, standard meter bearings require excessive maintenance. Heavy-duty meters are usually used for velocities greater than 8 feet per second.

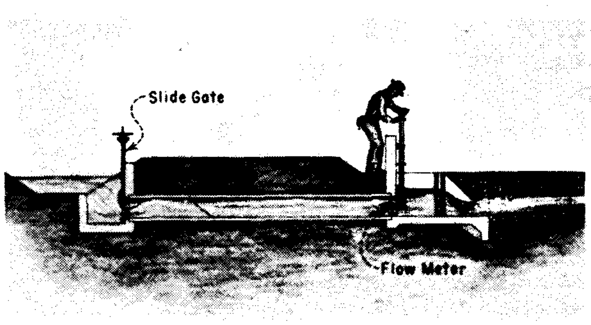


Figure 5-30. **Artist sketch of turnout with open-flow meter attached. PX-D-31428**

(b) *Requirements for Accurate Flow Measurement.*—When used properly, the propeller should always be submerged and the conduit should be flowing full. To provide for this requirement, stoplog guides are included in the outlet transition as shown on figure 5-31 to accommodate placing stoplogs when required to force the conduit to flow full.

The propeller should be suspended so that the propeller hub is in the center of the conduit which can be either rectangular or round. A slight eccentricity or misalinement of the propeller will affect measurement accuracy. Therefore, anchor bolts for the mounting bracket must be located very accurately (fig. 3-23).

Propeller meters are very sensitive to spiral flow, and large measurement errors result from this condition. Flow straightening vanes, several pipe diameters in length, installed a short distance upstream of the propeller will reduce the measurement errors caused by spiral flow. Straightening vanes are not required if a straight level section of pipe, as recommended by the meter manufacturer, is used immediately upstream from the meter location.

(c) *Size Determination.*—The selection of the size of the meter is a very important design consideration. Many meters have been taken out of service because they were too large to accurately measure the average day-to-day flows. If possible, a meter that measures flow in the midrange of its capability should be selected. Meter propeller diameters generally range from 0.5 to 0.8 of the pipe diameter. Laboratory tests show that the best accuracy is obtained when the propeller diameter is 75 percent or more of the pipe diameter.

(d) *Head Loss.*—The head loss across a propeller-type meter is usually considered to be negligible; however, some allowance for loss should be included in the design, particularly if head is a critical consideration. The meter

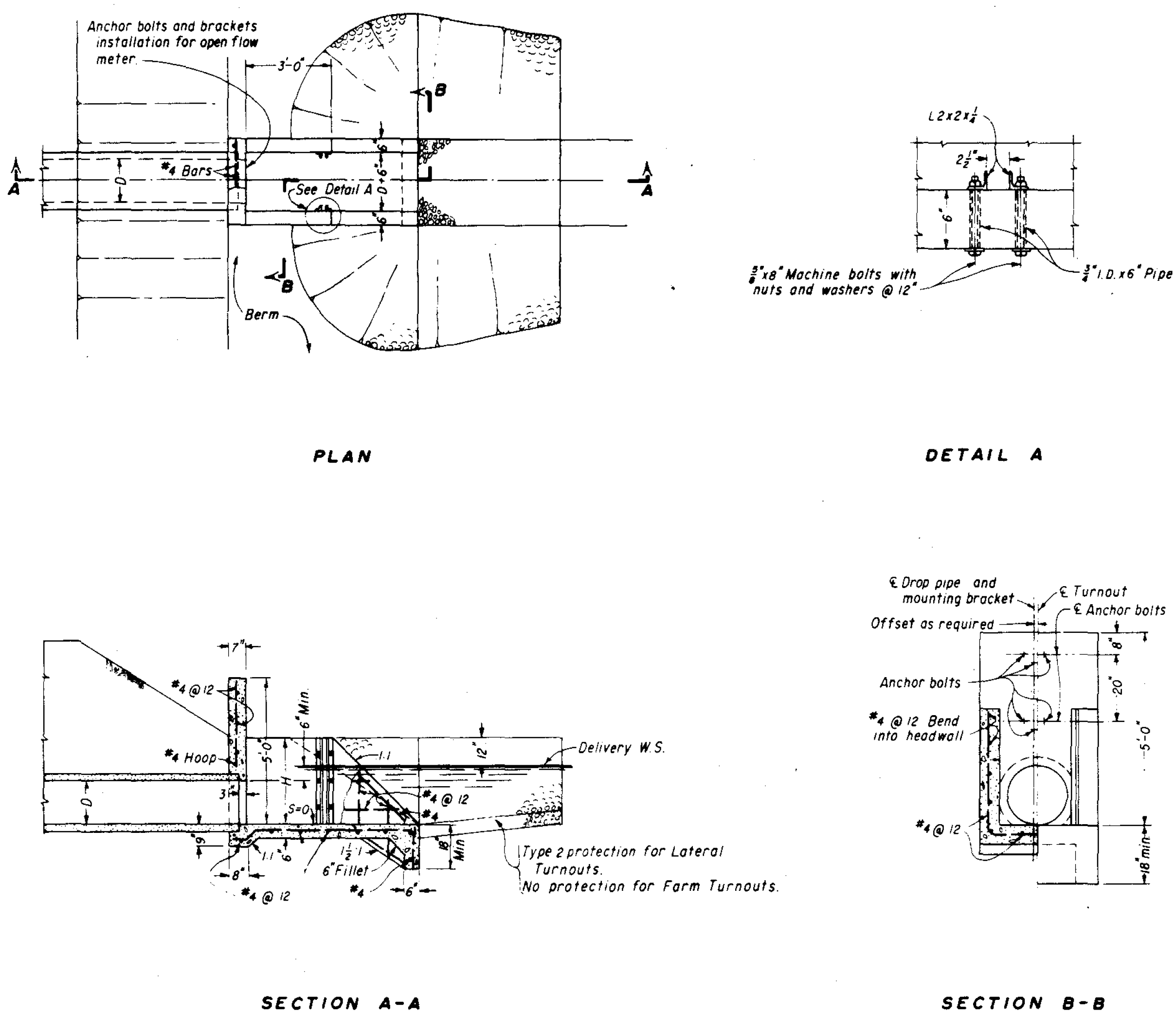


Figure 5-31. **Outlet transition for open-flow meter.**
103-D-1252

manufacturer can usually provide meter head loss information.

5-27. ***Advantages.***—Portability of a meter permits the user to measure flows at several turnouts which have the same pipe diameters or turnouts having different pipe diameters if the meter is specially equipped with a removable register and change gears. Flow rate and volume can be read directly thus eliminating computations. As previously mentioned, these meters have a flow range in excess of 10 to 1 above the minimum flow and the head loss is very small.

5-28. ***Disadvantages.***—Initial cost of an open-flow meter is usually relatively high. Maintenance costs may also be high, particularly if the meter is used in water that contains appreciable sediment. The sediment causes the bearings to wear and this becomes a problem unless a well-planned maintenance program is followed. Meters should be inspected and serviced at regular intervals as corrosion and wear may affect the calibration.

Although the propellers are designed to pass some weeds and debris, there is a limit to the amount that can pass without clogging or affecting the water measurement.

Spiral flow, air in the line, or any other factor that affects the rate of propeller rotation affects the measurement. Practically all conditions which affect the propeller rotation rate reduce the number of propeller revolutions and result in underregistration.

H. BIBLIOGRAPHY

5-29. ***Bibliography***.

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[4] Simmons, W. P., Case, W. C., "Hydraulic Model Studies of Small Weir Box Turnout Structures for General Irrigation Use" Hyd-396, Bureau of Reclamation, 1954.
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Energy Dissipators

A. GENERAL

6-1. *Energy Dissipators.*—Energy dissipators are used to dissipate excess kinetic energy possessed by flowing water. This energy or velocity head is acquired by the water where the velocity is high, such as in a chute or drop and energy dissipators are incorporated into the design of these structures. An effective energy dissipator must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipators direct the water into an obstruction that diverts the flow in all directions and in this manner dissipates the energy in the flow. In some structures the flow plunges into a pool of water where the energy is diffused. Check drops and vertical drops (chapter III C), baffled outlets, baffled aprons, and vertical stilling wells are all impact-type energy dissipators. The use of a vertical drop in combination with an overchute is shown in chapter IV C.

Other energy dissipators use the hydraulic jump to dissipate excess head. In this type of structure water flowing at a higher than critical velocity is forced into a hydraulic jump and energy is dissipated in the resulting turbulence. Stilling pools (chapter II F) contain the turbulent water until it can be discharged into the downstream channel without damage to the channel. The sumped pipe drop (chapter II E) is a closed conduit drop in which the hydraulic jump occurs within the pipe.

The impact-type of energy dissipator is considered to be more efficient than the hydraulic jump type. Generally, the use of an impact-type energy dissipator results in smaller and more economical structures.

The design of energy dissipators has been the subject of many model studies and the designs recommended for this type of structure are based primarily on empirical data resulting from these studies. The studies continue in an attempt to develop more effective and more economical energy dissipators.

B. BAFFLED APRON DROPS

R. B. HAYES[1]

6-2. *General.*—(a) *Usage.*—Baffled apron drops are used in canals or wasteway channels to provide dissipation of excess energy at drops in grade. Energy dissipation occurs as the water flows over the concrete baffle blocks, which are located along the floor of the chute. The ability of the baffled apron drop to accommodate a widely fluctuating tailwater elevation makes it especially suitable as an

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energy dissipator at the end of a canal or wasteway that discharges into a reservoir. The length of the baffled apron does not affect the efficacy of the structure. It is effective in dissipating excess energy for drops of any magnitude (see fig. 6-1), but it becomes uneconomical for large flows with great drops, due to the wide section and numerous blocks required. Where an excess of trash, trees, or tumbleweeds accompanies the flow, they may become lodged in the baffle blocks, restricting the flow. Removal of this material is sometimes difficult.

(b) *Inlet Control Features.*—Various types of inlet control features are utilized to maintain an upstream water surface as required for turnouts; or to provide a velocity of approach consistent with the scouring tolerance of the upstream section; or to avoid the excessive splashing that would result from supercritical flow at the inlet. The more common types of inlet control features are as follows:

(1) *Sill control.*—A sill may be provided at the inlet, as shown in figure 6-2A, to reduce the velocity of approach, and minimize scour in the upstream section. The sill also provides a controlled water surface for upstream turnouts. To permit complete drainage of the upstream pool, a slot is provided through the crest, as shown in figure 6-5. The inlet should be kept free of sediment deposits, as the extensive accumulation of sediment would allow the flow to sweep over the sill at a velocity too fast for effective dissipation of energy.

(2) *Control notch.*—A control notch may be provided at the inlet, as shown in figures 6-2B and 6-3, to control the upstream water depth. While the control notch is designed to maintain normal depth and velocity in the upstream section, it produces a fast velocity in the inlet itself, causing splashing as the fast velocity flow strikes the first row of baffle blocks. Excessive splashing may require frequent maintenance of the erosion protection. The control notch should be kept free of trash.

(3) *Inlet without control.*—The simplest type of inlet (shown in fig. 6-2C) is used where there is not a requirement to control the upstream water surface for turnout deliveries, and where the upstream channel is sufficiently stable to withstand (without erosion) the higher velocities associated with water surface drawdown. To minimize splashing as the flow strikes the first row of baffle blocks, an invert curve may be provided to allow the flow to strike the blocks in a direction normal to their upstream face. Where flows are infrequent, and some splashing is permissible, the curve is sometimes omitted, as shown in figure 6-2B.

(c) *Miscellaneous Features.*—

(1) *Bridge deck.*—Where a crossing is required, a bridge deck may be incorporated in the inlet design.

(2) *Cutoff walls and wingwalls.*—In addition to the inlet cutoff walls, wingwalls are provided at, or near the downstream end of the structure, to decrease percolation and to retain the backfill along the slope. Where the downstream channel is subject to degradation, a cutoff wall is extended down

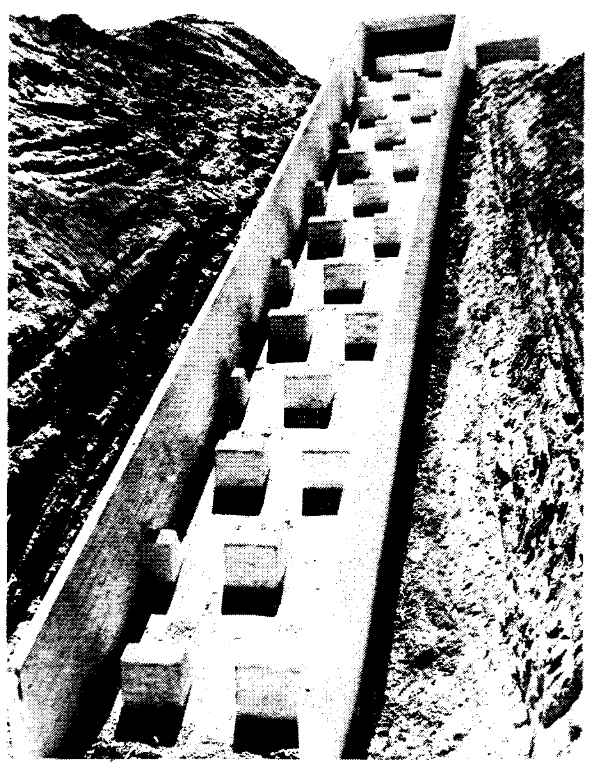

Figure 6-1. **Baffled apron drop before backfilling. P-328-701-9501**

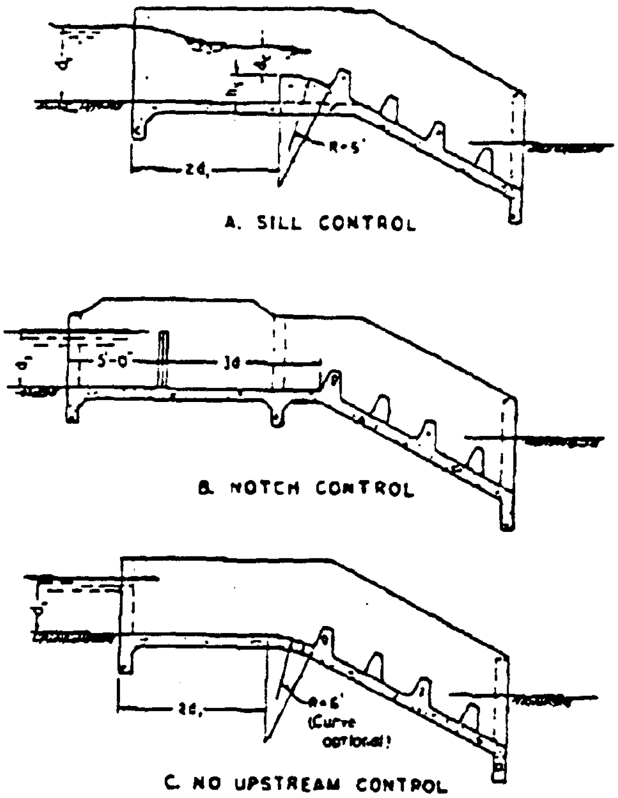

Figure 6-2. Typical inlet types. 103-D-1334

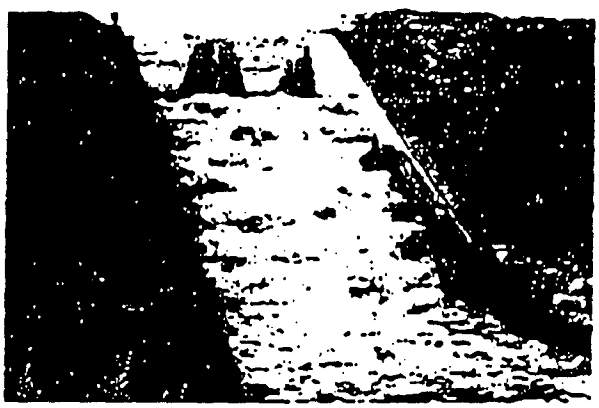

Figure 6-3. Baffled apron drop with inlet control notches. PX-D-72607

from the invert, as shown in figure 6-5. The wingwalls may be located at the end of the structure to coincide with the cutoff, but they are often located a few feet upstream from the end as shown in figure 6-4. This provides a better stilling action at the outlet, and raises the top elevation of the wingwall,

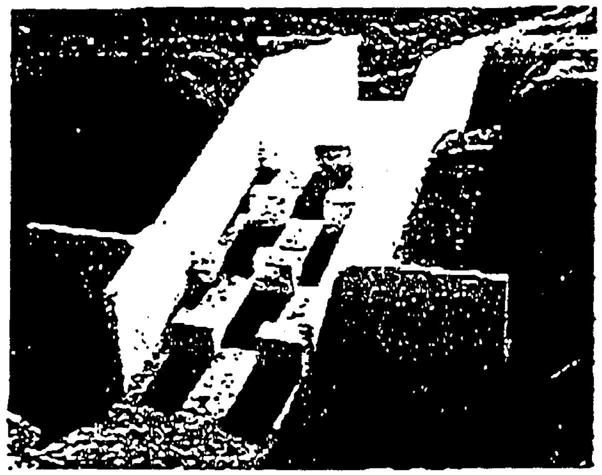

Figure 6-4. Baffled apron drop with wingwalls located upstream from the outlet end. P-492-701-1102A

which should be located above the tailwater elevation to minimize erosion.

(3) *Protective drains.*—Protective drains are sometimes provided under the invert of the baffled apron drop to relieve the uplift pressure following the termination of flow.

6-3. *Design Considerations.*—(a) *General.*—For consideration of the baffled apron drop to provide dissipation of the excess energy at a wasteway turnout, see subsection 4-2(b) Wasteway Outlets. For the application of baffled apron drops to cross-drainage structures, see subchapter IV C.

(b) *Capacity.*—The capacity of the baffled apron drop is a function of the allowable discharge, q, per foot of width, as shown in table 6-1. They have been operated for short periods at about twice the design capacity without excessive erosion.

Table 6-1.—*Recommended discharge*

Q capacity, cfs	q *Discharge per foot of chute width, cfs
0 to 39	5 to 10
40 to 99	10 to 15
100 to 189	15 to 20
190 to 460	20 to 30

*Discharge per foot of width, q should be interpolated within the range indicated.

(c) *Inlet.*—The inlet should be the same width as the baffled apron, and should provide a velocity of approach slower than the critical velocity, V_c. Where splashing must be minimized [1],[2] the entrance velocity should not exceed about $\frac{V_c}{2}$, where

$$V_c = \sqrt{gq}$$

in the rectangular inlet section. Other design considerations are as follows:

(1) *Sill control.*—The inlet length should be at least $2d_1$, as shown in figure 6-2A. The required height of the sill above the inlet floor may be determined from the energy balance between the inlet and the upstream channel.

Thus,

$$E_{s_1} = E_{s_c} + h_i + h_s$$

or

$$h_s = E_{s_1} - E_{s_c} - h_i$$

where

h_s is the sill height,

$E_{s_1} = d_1 + h_{v_1}$ in the upstream channel,

$E_{s_c} = d_c + h_{v_c}$ in the control section at the sill,

and

$$h_i = 0.5\Delta h_v$$

$$= 0.5\,(h_{v_c} - h_{v_1})$$

$$= 0.5\left[\frac{V_c^2}{2g} - \frac{V_1^2}{2g}\right]$$

The curvature of the sill crest should terminate at its point of tangency with the slope of the downstream apron (see subsec. 6-4(b)(7)). This point should not be more than 12 inches in elevation below the crest [1]. This is assured by limiting the radius of curvature to a maximum of 9 feet. A 6-foot radius is frequently used. The sill has a 6-inch-wide slot to provide drainage of the upstream pool.

(2) *Control notch.*—A control notch, where used, should conform to the design requirements of subchapter III G. The rectangular inlet section should begin 5 feet upstream from the control notch, and the length between the notch and the sill should be equal to three times the upstream water depth, as shown in figure 6-2B, to permit the flow to spread to the full width of the section.

(d) *Baffled Apron Dimensions (see fig. 6-5).*—The following steps are suggested as a guide to be used in setting the dimensions [2]:

(1) Set the longitudinal slope of the chute floor and sidewalls at 2 to 1 ($\tan \phi = 0.5$).

(2) Approximate width of structure should be set by the relation,

$$B = \frac{Q}{q}$$

where

B = width,
Q = maximum total discharge, and
q = allowable discharge per foot of width (see table 6-1).

For the permissible entrance velocity, see subsection 6-3(c).

(3) Set the first row of baffles so that the base of the upstream face is at the downstream end of the invert curve and no more than 12 inches in elevation below the crest.

(4) Baffle block height, h_b, should be about 0.9 times critical depth, d_c, to nearest inch.

(5) Baffle block widths and spaces should be equal, and not less than h_b, but not more than 1-1/2 h_b. Partial blocks, having a width not less than 1/3 h_b and not more than 2/3 h_b should be placed against the sidewalls in rows 1, 3, 5, 7, etc. Alternate rows of baffle blocks should be staggered so that each block is downstream from a space in the

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[2] Numbers in brackets refer to references in bibliography, see sec. 6-14.

adjacent row. The structure width, B, determined above, should be adjusted so convenient baffle block widths can be used.

(6) The slope distance, s, between rows of baffle blocks, as shown in figure 6-5, should be at least 2 h_b, but no greater than 6 feet. A spacing of 6 feet may be used for all blocks equal to or less than 3 feet in height.

(7) A minimum of four rows of baffle blocks should be used. The baffled apron should be extended so that the top of at least one row of baffle blocks will be below the bottom grade of the outlet channel as discussed in subsection 6-3(g). The apron should be extended beyond the last row of blocks a distance equal to the clear space between block rows.

(8) Baffle blocks are constructed with their upstream faces normal to the chute floor. The longitudinal thickness, T, of the baffle blocks at the top should be at least 8 inches, but not more than 10 inches. See block detail, figure 6-5.

(9) Suggested height of the walls to provide adequate freeboard is three times the baffle block height measured normal to the chute floor. It is generally not feasible to set the freeboard for these structures to contain all of the spray and splash.

(e) *Uplift Stability.*—The net force causing flotation of the structure should be considered, assuming a sudden cessation of flow in the channel. The net flotation force is equal to the weight of the empty structure minus the hydrostatic force remaining in the soil around the structure. The magnitude of the maximum hydrostatic force, or uplift pressure varies with the theoretical height of the percolation gradient as determined by Lane's weighted-creep method [3], assuming an upstream gradient at the normal water surface elevation.

(f) *Sliding Stability.*—

(1) *Long baffled apron drops.*—The slope stability or tendency of long baffled aprons to slide down the 2 to 1 slope should be checked. This is particularly important in channels where erosion may remove the earth material at the downstream toe of the apron. In model studies performed on baffled aprons, piezometer readings on the baffle blocks have indicated an average net water pressure on the blocks in the downstream direction of between 4 and 5 feet of water. This is equivalent to a force of between 250 and 310 pounds per square foot of block area.

(2) *Short baffled apron drops.*—Short baffled apron drops also should be checked for sliding. Complete removal by erosion of the earth material downstream from the downstream cutoff walls should be assumed, unless the stability of the downstream channel is assured. A horizontal sliding plane should be assumed rather than the sloping plane assumed for long baffled apron drops.

Major forces, F_s, tending to induce sliding along the horizontal plane, during maximum flow in the channel are as follows:

$$F_s = F_1 + F_2 + F_3 + F_4,$$

where

F_1 is the hydrostatic force on the upstream face of the upstream cutoff walls,

F_2 is the hydrostatic force on the vertical face of the sill,

F_3 is the horizontal component of the hydrostatic force on the upstream face of the baffle blocks, and

F_4 is the force of the saturated earth load on the upstream face of the downstream cutoff walls (see bibliography, reference [5], in chapter I).

Major forces F_R, resisting sliding during maximum flow in the channel are frictional resistance and passive earth pressure. Frictional resistance is considered for the inlet portion only (length $L_1 + L_2$), as the sloping portion can move horizontally with no frictional resistance. The frictional resistance is a function of the structure weight reduced by uplift, and the passive earth pressure is a function of the angle of internal friction of the earth material. The hydrostatic load on the downstream side of the cutoff walls is omitted as a safety factor, and a safety factor is also included in the coefficient of sliding.

Thus,

$F_R = \mu\ (W_c + W_w - U)$ + passive earth force,

where

μ is the coefficient of sliding friction, generally assumed to be equal to 0.35 (see chapter I),

W_c is the weight of concrete in the inlet portion of the structure,

W_w is the weight of water in the inlet portion of the structure,

U is the vertical uplift force as determined by Lane's weighted-creep method. An approximate method of determining the hydrostatic uplift pressure may be used by assuming a pressure gradient extending from the maximum upstream water surface to the downstream water surface.

Passive earth force is the total force resulting from passive earth pressure on the downstream side of the cutoff walls (see bibliography reference [5] in chapter I).

If the forces tending to induce sliding are greater than the forces resisting sliding (using suitable safety factors), additional cutoff walls should be included.

(g) *Miscellaneous Considerations.*—

(1) Gravel or riprap should be provided on each side of the structure from the top of the slope to the downstream wingwall, extending laterally a distance equal to the wall height. This protection above the maximum downstream water surface is to prevent erosion from splashing. Below this water surface the protection is required to prevent erosion by eddy currents. Wingwalls (see fig. 6-4) hold the slope protection in place. Channel protection downstream from the structure should be in accordance with subchapter VII B. Rockfill at the bottom of the apron may be unnecessary.

(2) The channel grade may be controlled by a downstream structure, by geologic formation, or by being on a stable slope for the design capacity. A slope of 0.0018 will usually be stable for storm waterflows, but for normal canal flows the assumed slope should be no steeper than that of a canal in the same material.

The bottom row of blocks should be set below the computed channel invert elevation. When scour has occurred, to provide for future storms, the apron and walls should be extended by exposing the reinforcement in the end of the structure and bonding a new extension to the original installation.

(3) Greater economy may be achieved by precasting the concrete baffle blocks.

6-4. *Design Example.*—A wasteway channel descends a slope that requires energy dissipation to prevent excessive erosion. As a dependable tailwater cannot be assured, it is decided that a baffled apron drop would best meet the need.

(a) *Assumptions.*—

(1) On the basis of soil type and operating conditions, it is decided that little splashing should be permitted, and a sill-type control is preferable (see fig. 6-2 A).

(2) Hydraulic properties of the wasteway channel are as follows:

$Q = 120$ cfs	$r = 2.55$
$b = 8$ ft.	$n = 0.025$
$d_1 = 4.10$ ft.	$s = 0.00035$
$A_1 = 58.02$ sq. ft.	ss = 1-1/2:1
$v_1 = 2.08$ f.p.s.	$f_b = 2.0$ ft.
$h_{v_1} = 0.07$ ft.	$h_B = 6.1$ ft.

(3) A drop of 6 feet in invert elevation is required.

(b) *Solution (see fig. 6-5 for dimension nomenclature).*—

(1) From table 6-1, find the recommended discharge per foot, q, for a total capacity of 120 cfs, and determine a preliminary chute width.

For

$Q = 120$ cfs,

find by interpolation

$q = 16$ cfs (approximately).

Then

$$B = \frac{Q}{q} = \frac{120}{16} = 7.5 \text{ ft.}$$

(2) Determine the limits of baffle block dimensions, based upon critical depth, d_c.

For d_c = 2.0 ft. (from table 17 in bibliography reference [4]),

Block height, $h_b = 0.9\ d_c = 1.8$.
Say 1 ft. 10 in., or 1.83 ft.
Block width and space, w:
Min. $w = h_b = 1.83$ ft.
Max. $w = 1.5\ h_b = 1.5 \times 1.83 = 2.75$ ft.

(3) Determine exact dimensions of baffle blocks and chute width. As the partial block width,

$$w_p = \frac{1}{3} h_b \text{ min.} = 0.61 \text{ min.},$$

$$\text{and } \frac{2}{3} h_b \text{ max.} = 1.22 \text{ max.},$$

Try $w_p = \frac{1}{2} h_b$, for simplicity.

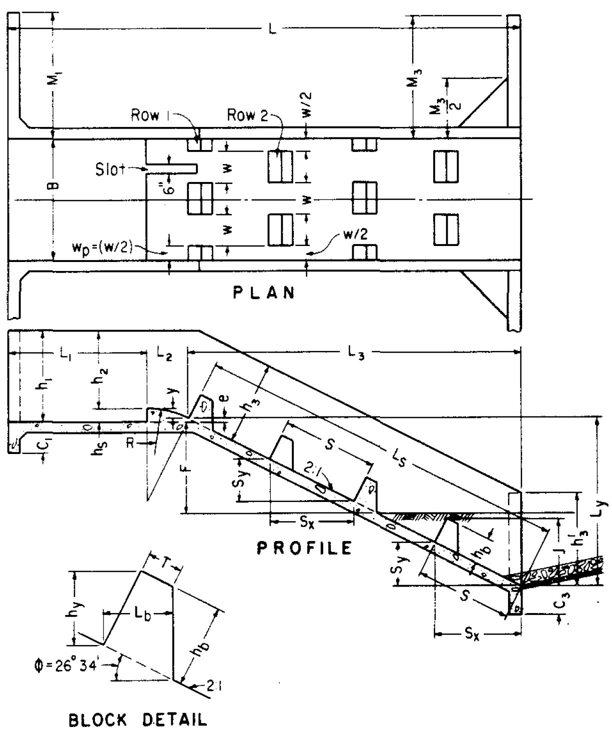

Figure 6-5. **Baffled apron drop design. 103-D-1335**

Then, use alternate rows as follows:

Rows 1 and 3:
1 full block = 1w
2 full spaces = 2w
2 half blocks = lw
B = 4w

Rows 2 and 4:
2 full blocks = 2w
1 full space = lw
2 half spaces = lw
B = 4w

Thus, the total width, B, of any row is

$$B = 4w$$

Using the minimum block width of 1.83 ft.

$$B = 4 \times 1.83 = 7.32 \text{ ft.}$$

To simplify dimensions,

Use
w = 2.0 ft. (>1.83<2.75),
and
w_p = 1.0 ft. (>0.61<1.22).

Then,

$$B = 4w = 4 \times 2 = 8 \text{ ft.},$$

$$q = \frac{120}{8} = 15 \text{ cfs},$$

d_c = 1.91 ft.,

h_b = 1.72 ft.

Use

h_b = 1 ft. 9 in.

Select T = 9 inches (see block detail, fig. 6-5)
> 8 inches
< 10 inches

(4) Determine inlet length, L_1:

$L_1 = 2d_1 = 2 \times 4.1 = 8.2$ ft.
Use L_1 = 8 ft. 3 in.

(5) Determine inlet sill height, h_s. (Although the baffle blocks just downstream from the crest may have some effect on the water depth over the sill, the assumption that critical depth occurs at the sill is adequate for determining the sill height.)

Using B = 8 ft., q = 15 cfs, d_c = 1.91, h_{v_c} = 0.96, and V_c = 7.86 f.p.s.

Then

$$h_s = E_{s_1} - E_{s_c} - h_i \text{ (where } h_i = \text{inlet loss)}$$

$$= (d_1 + h_{v_1}) - (d_c + h_{v_c} - 0.5(h_{v_c} - h_{v_1})$$

$$= (4.10 + 0.07) - (1.91 + 0.96) - 0.5(0.96 - 0.07)$$

$$= 0.85 \text{ ft.}$$

Use

$$h_s = 10 \text{ inches} = 0.83 \text{ foot}$$

(6) Check inlet velocity to minimize splashing:

Determine depth, d_1, at inlet cutoff:

$$d_1 = h_s + d_c + h_{v_c}$$

$$= h_s + d_c + \frac{d_c}{2}$$

$$= 0.83 + 1.91 + 0.96$$

$$= 3.70 \text{ feet}$$

The entrance velocity is then

$$V_1 = \frac{Q}{A_1} = \frac{Q}{d_1 B}$$

$$= \frac{120}{3.7 \times 8} = 4.0 \text{ f.p.s.}$$

Determine critical velocity, V_c over crest:

$$V_c = \frac{Q}{A} = \frac{120}{d_c B}$$

$$= \frac{120}{1.91 \times 8} = 7.85 \text{ f.p.s.}$$

Thus, the inlet velocity is approximately equal to half the critical velocity, and splashing will be minimized.

(7) Determine sill length, L_2, and dimension e, as shown in figure 6-6.

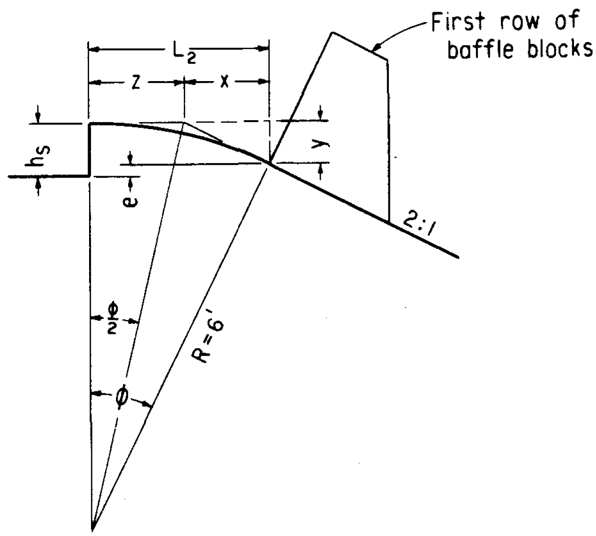

Figure 6-6. **Sill curve dimensions. 103-D-1336**

Using a radius, R = 6 ft., and an invert slope of 2 to 1,

$$\phi = 26^\circ 34'$$
$$\sin\phi = 0.4472 = y/z$$
$$\tan\phi = 0.5 = y/x$$
$$\phi/2 = 13^\circ 17'$$
$$\tan\phi/2 = 0.2361 = z/R$$

Substituting,

$$z = 0.2361R = 1.42 \text{ ft.}$$
$$y = 0.4472z = 0.63 \text{ ft.}$$
$$x = \frac{y}{0.5} = 1.26 \text{ ft.}$$

Then,

$$L_2 = x + z = 1.26+1.42 = 2.68 \text{ feet,}$$

and

$$e = h_s - y = 0.83 - 0.63 = 0.20 \text{ foot.}$$

(8) Determine the slope distance, S, between rows of baffle blocks, as shown in figure 6-5.

$$S = 2\,h_b \text{ min.}$$
$$= 2\,(1.75) = 3.5 \text{ ft.}$$

Use

S = 6 feet (see subsec. 6-3(d)(6)).

(9) Determine minimum depth of cover, j, at outlet to insure that the last row of baffle blocks will be covered by the backfill, placed in the structure to the elevation of the downstream grade.

$$S_y = S \sin \phi = 6(0.4472) = 2.68 \text{ ft.}$$
$$h_y = h_b \cos \phi = 1.75(0.8944) = 1.57 \text{ ft.}$$
$$j = S_y + h_y = 4.25 \text{ ft.}$$

(10) Determine apron lengths, L_3 and L_s, for a drop, F = 6 feet.

Minimum distance,

$$L_y = e + F + j$$
$$= 0.20 + 6 + 4.25 = 10.45 \text{ ft.}$$

Minimum rows of blocks:

$$\text{Rows} = \frac{L_y}{S_y} = \frac{10.45}{2.68} = 3.9$$

Use 4 rows.

Where the ratio, $\frac{L_y}{S_y}$ indicates that fewer rows would be adequate, the minimum number of four rows should be used by extending two or more rows below the downstream grade.

Finally,

$$L_s = 4\,S$$
$$= 4\,(6) = 24 \text{ ft.}$$
$$L_y = 4\,S_y$$
$$= 4\,(2.68) = 10.72 \text{ ft.}$$
$$L_3 = 4\,S_x$$
$$= 4(6 \cos \phi)$$
$$= 24 \times 0.8944 = 21.47 \text{ ft.}$$

(11) Determine overall length of structure.

$$L = L_1 + L_2 + L_3$$
$$= 8.25 + 2.68 + 21.47$$
$$= 32.40 \text{ ft.}$$

(12) Determine the following wall heights (see fig. 6-5):

$$h_1 = d_1 + 1 \text{ ft.} = 4.10 + 1 = 5.1 \text{ ft.}$$

Use h_1 = 5 ft. 2 in.

With a level invert, and with the top of the walls level from h_1 to h_2,

$$h_2 = h_1 - h_s$$
= 5 ft. 2 in. minus 10 in.
= 4 ft. 4 in.

The height of the chute walls,

$$h_3 = 3h_b = 3(1.75) = 5.25 \text{ ft.}$$
= 5 ft. 3 in.

(13) Determine length, M_1, of the upstream wingwalls, as shown in figure 6-7.

$$M_1 = 1.5h_1 + C_1,$$

where the cutoff depth, C_1 = 2.5 feet for a water depth,

d_1 = 4.1 ft. (see fig. 7-2).

Then,

$$M_1 = 1.5\,(5.17) + 2.5$$
= 10.25 ft.
= 10 ft. 3 in.

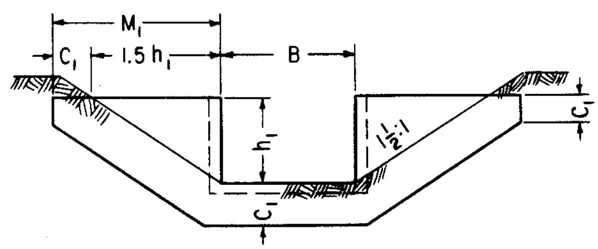

Figure 6-7. **Upstream wingwalls. 103-D-1337**

(14) Determine length, M_3, of downstream wingwalls, as shown in figure 6-8.

$$M_3 = 1.5\,h_3' + C_3,$$

where the cutoff depth, C_3 = 2.5 feet for an assumed channel water depth of 4.1 feet (see fig. 7-2), and

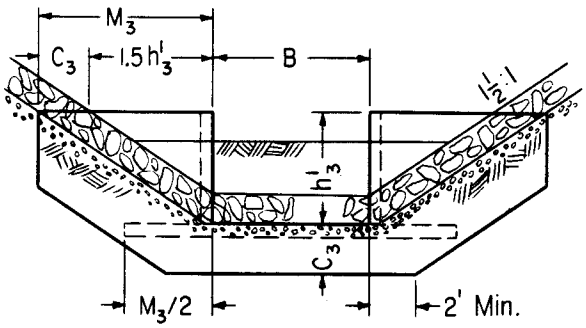

Figure 6-8. **Downstream wingwalls. 103-D-1338**

$$h_3' = \frac{h_3}{\cos\phi} \text{ (with } \phi = 26^{o}34'\text{)}$$
$$= \frac{5.25}{0.8944}$$
$$= 5.87 \text{ ft.}$$

Then

$$M_3 = 1.5(5.87) + 2.5$$
$$= 8.81 + 2.5$$
$$= 11.31$$

Use

$$M_3 = 11 \text{ ft. } 4 \text{ in.}$$

(15) Check flotation of the structure (due to uplift forces) according to the requirements of subsection 6-3(e).

(16) Check sliding stability of the structure according to the requirements of subsection 6-3(f).

(17) Determine protection requirements. Select type of protection from figure 7-8, chapter VII, for inclined drops with a water depth of 4.1 feet.

Inlet Protection: Type 1, extending a distance d_1 upstream, and up the sloping sides to an elevation 1 foot above the normal water surface.

Slope Protection: Type 1, as described in subsection 6-3(g).

Outlet Protection: Type 3, extending downstream a distance $4d_1$, and up the sloping sides to an elevation 1 foot above the assumed water surface, or to top of constructed bank, whichever is greater. Note that the backfill, placed over the last row of blocks, covers the riprap to the elevation of the natural grade (see fig. 6-5).

(18) Check percolation by the requirements of chapter VIII, and provide cutoff walls if needed.

C. BAFFLED OUTLETS

R. B. YOUNG[3]

6-5. ***Purpose and Description.***—Excess energy forces in flowing or falling water must be effectively dissipated to prevent erosion damage to downstream channels. Pipe drops, pipe chutes, and pipe cross-drainage culverts are examples of structures that require some form of energy dissipator. Usually the energy is dissipated in a sumped pipe, a stilling pool, or a baffled outlet. A hydraulic jump is involved with the energy dissipation in a sumped pipe and a stilling pool whereas the baffled outlet is an impact-type energy dissipator.

The baffled outlet is a boxlike structure having a vertical hanging baffle and an end sill (figs. 6-9, 6-10, 2-23, 2-25, and 4-24). Excess energy of the incoming water jet is dissipated primarily by striking the baffle and to a lesser degree by eddies that are formed after the jet strikes the baffle. A tailwater depth is not required for satisfactory hydraulic performance as is the case for a hydraulic jump basin; although a smoother outlet water surface will sometimes result if there is tailwater. For the best operation, the tailwater should be about $(\frac{b}{2} + f)$ above the invert of the baffled outlet. The height of tailwater above the baffled outlet invert should never exceed b + f because then some of the flow will not strike the baffle. If the tailwater depth is uncontrolled, the baffled outlet invert is usually a distance f below the downstream channel invert. Because the baffled outlet does not require tailwater, this

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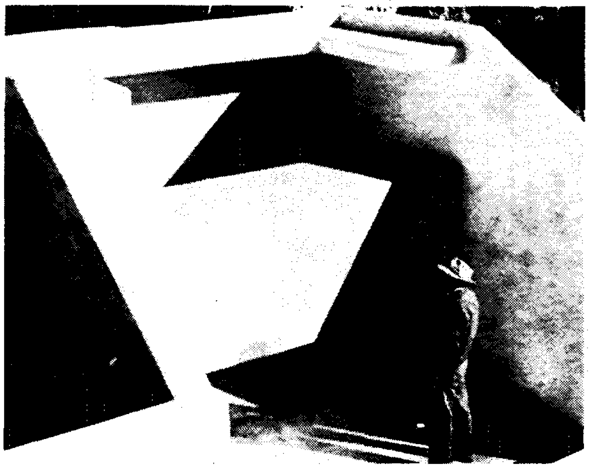


Figure 6-9. **Large baffled outlet. P455-520-408**

outlet is therefore particularly useful where tailwater depth is uncontrolled or where the rate of discharge increase is sudden and the tailwater buildup is slow. The baffled outlet, if properly designed, is a more effective energy dissipator than the hydraulic jump [5].

6-6. *Hydraulic Considerations.*—The baffled outlet was developed using hydraulic model studies [5] from which detailed dimensions for the baffled outlet were determined for various Froude numbers. To standardize the method of computing Froude numbers, the shape of the jet is assumed to be square; thus, the depth of the incoming flow, d, is considered to be the square root of its cross-sectional area using the equation $A = \frac{Q}{V}$. In this equation V is the theoretical velocity and is equal to $\sqrt{2gh}$ [6]. The head, h, is the head to be dissipated and it is usually sufficiently accurate to use the difference in the channel invert elevations at the inlet and outlet ends of the structure for this value. However, friction loss in long chutes may be significant and therefore it should be considered in the determination of h.

Water surface roughness and downstream channel erosion together with the ability of the basin to contain the flow were used as guidelines in evaluating the hydraulic performance of the test flows. Each of the test flows was judged to be satisfactory or unsatisfactory and plotted in dimensionless terms using Froude number, F, of the incoming flow and the ratio of basin width to the incoming depth of flow, $\frac{W}{d}$. From this data a recommended design curve for determining proper basin widths could be drawn and is shown on figure 6-10. Because the size of the jet was becoming very small in relationship to the width of the basin, the curve was not extended beyond a width-to-depth ratio of 10, which corresponded to flow having a Froude number of about 9. For a $\frac{W}{d}$ ratio of about 3, tests showed that the corresponding Froude number of the incoming flow was about 1. For this Froude number excess energy is minimal. Therefore the use of this basin for excess energy dissipation appeared to be impractical for $\frac{W}{d}$ ratios smaller than 3.

The curve on figure 6-10 indicates the minimum width of basin that should be used for a given Froude number. However, if the basin is too wide the energy will not be effectively dissipated because the incoming jet will spread and pass under the baffle rather than strike the baffle. Also the depth of the baffle should not be less than the diameter of the incoming pipe to prevent the jet from passing over the baffle. For partial discharge as well as design discharge, best overall dissipation is achieved only if the basin design width is equal to or slightly greater than the width determined from the curve for design discharge. Other basin dimensions are ratios of the width as shown on figure 6-10.

To prevent cavitation or impact damage to the basin, the theoretical pipe velocity ($\sqrt{2gh}$) should be limited to 50 feet per second.

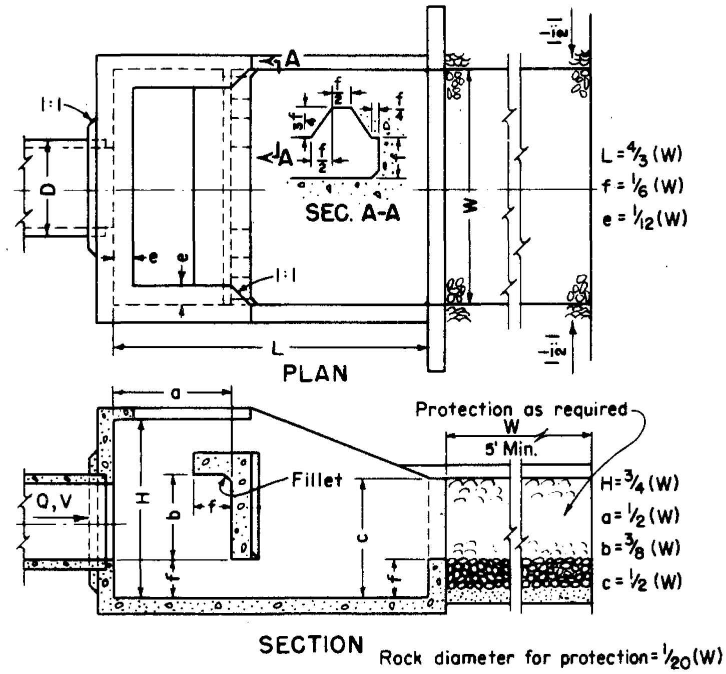
The diameter of the pipe outletting into the baffled outlet structure, should be determined using a velocity of 12 feet per second assuming the pipe is flowing full.

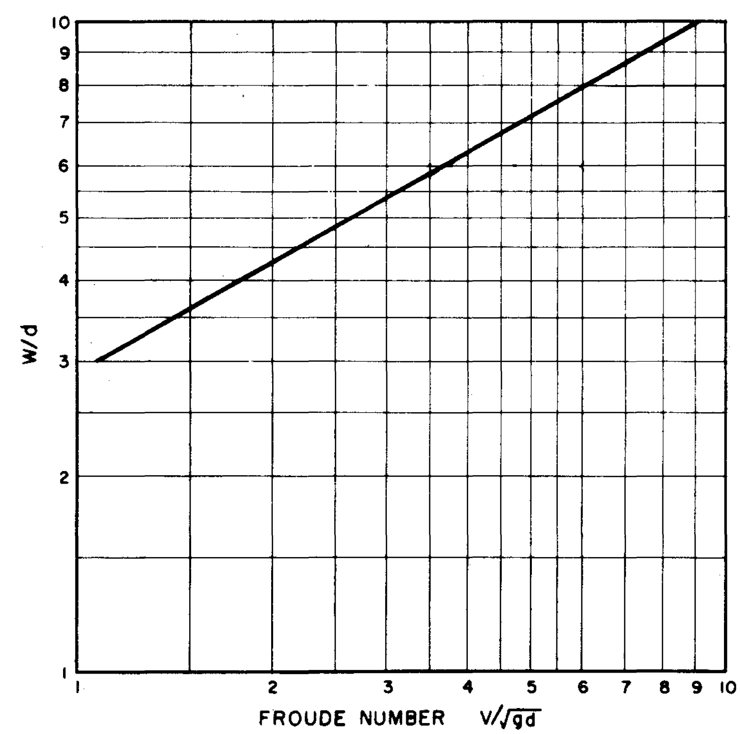
If the entrance pipe slopes downward, the outlet end of the pipe should be turned horizontal for a length of at least 3 pipe diameters so as to direct the jet into the baffle.

If there is a possibility of both the upstream and downstream ends of the pipe being sealed, an air vent near the upstream end may be necessary to prevent pressure fluctuations and associated surging of flow in the system.

Figures 6-11 through 6-20 show complete

(continued page 321)





W, ft., Is the inside width of the basin
V, fps, Is the theoretical velocity of the incoming flow and is $\sqrt{2gh}$
h, ft., Is the head to be dissipated
A, ft.^{2}, Is the area of flow entering the basin and is Q/V
d, ft., Represents the depth of flow entering the basin and is $\sqrt{A}$

Figure 6-10. **Design width of basin. 103-D-1339**

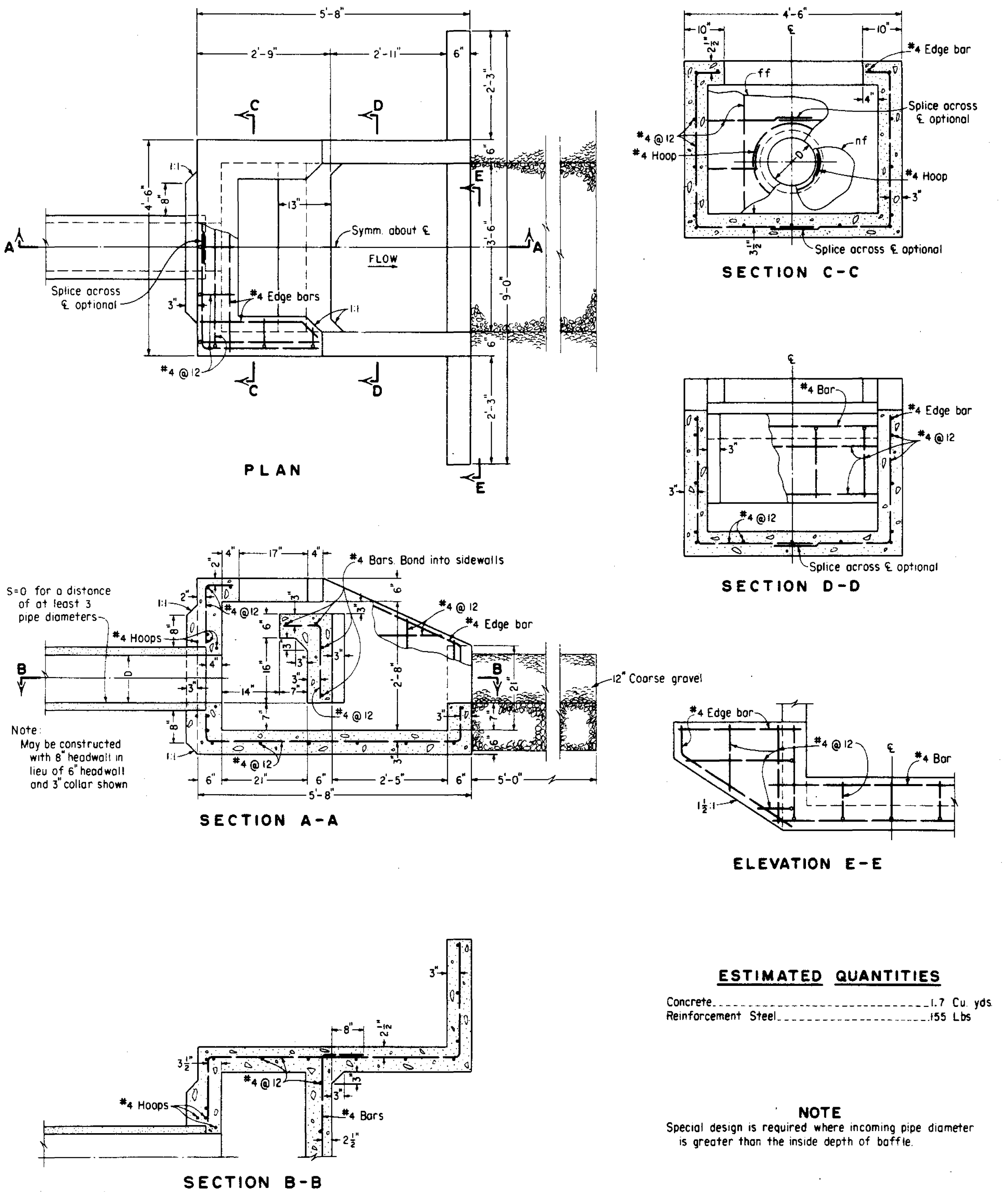


Figure 6-11. **Type 1 baffled outlet. 103-D-1340**

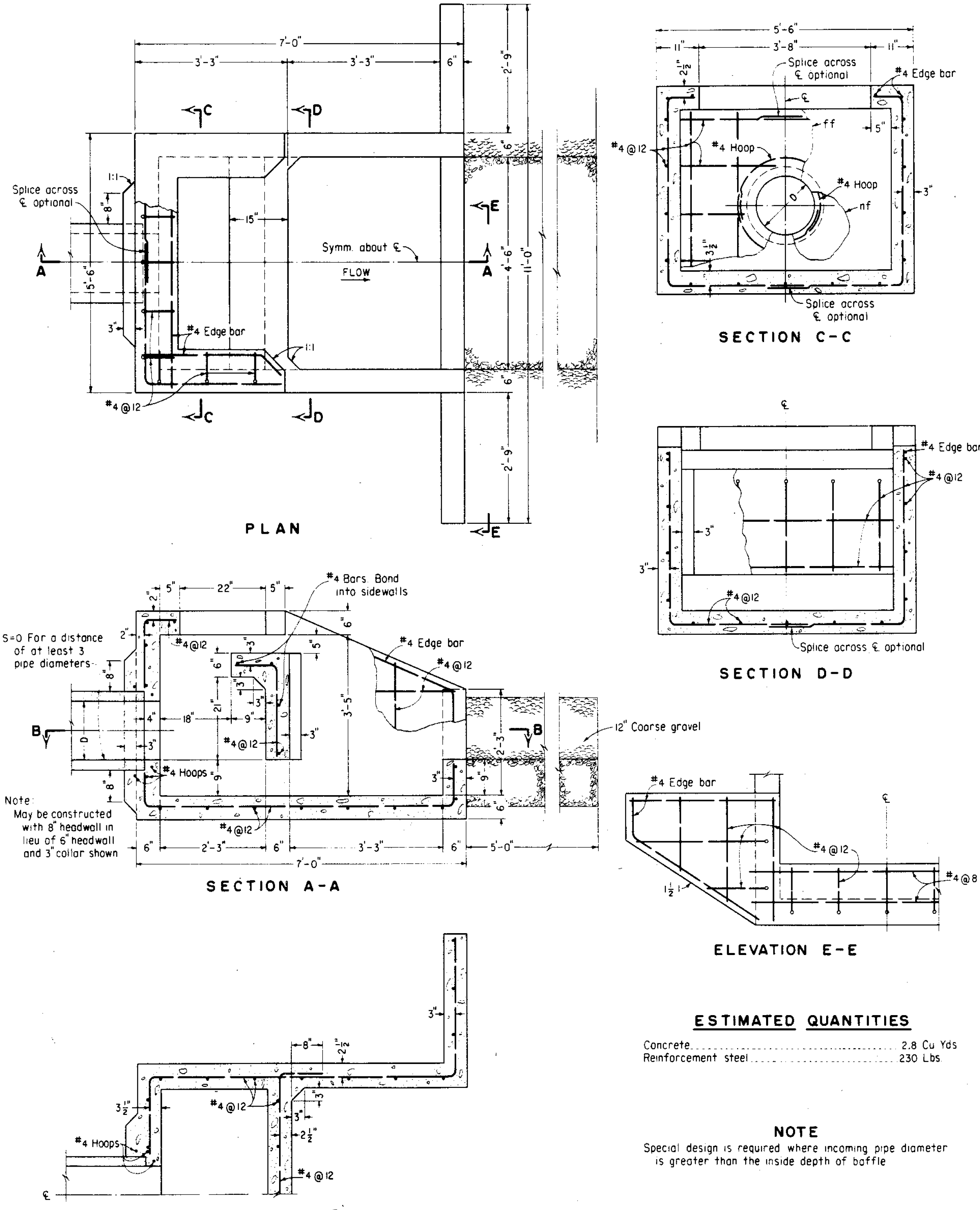


Figure 6-12. **Type 2 baffled outlet. 103-D-1341**

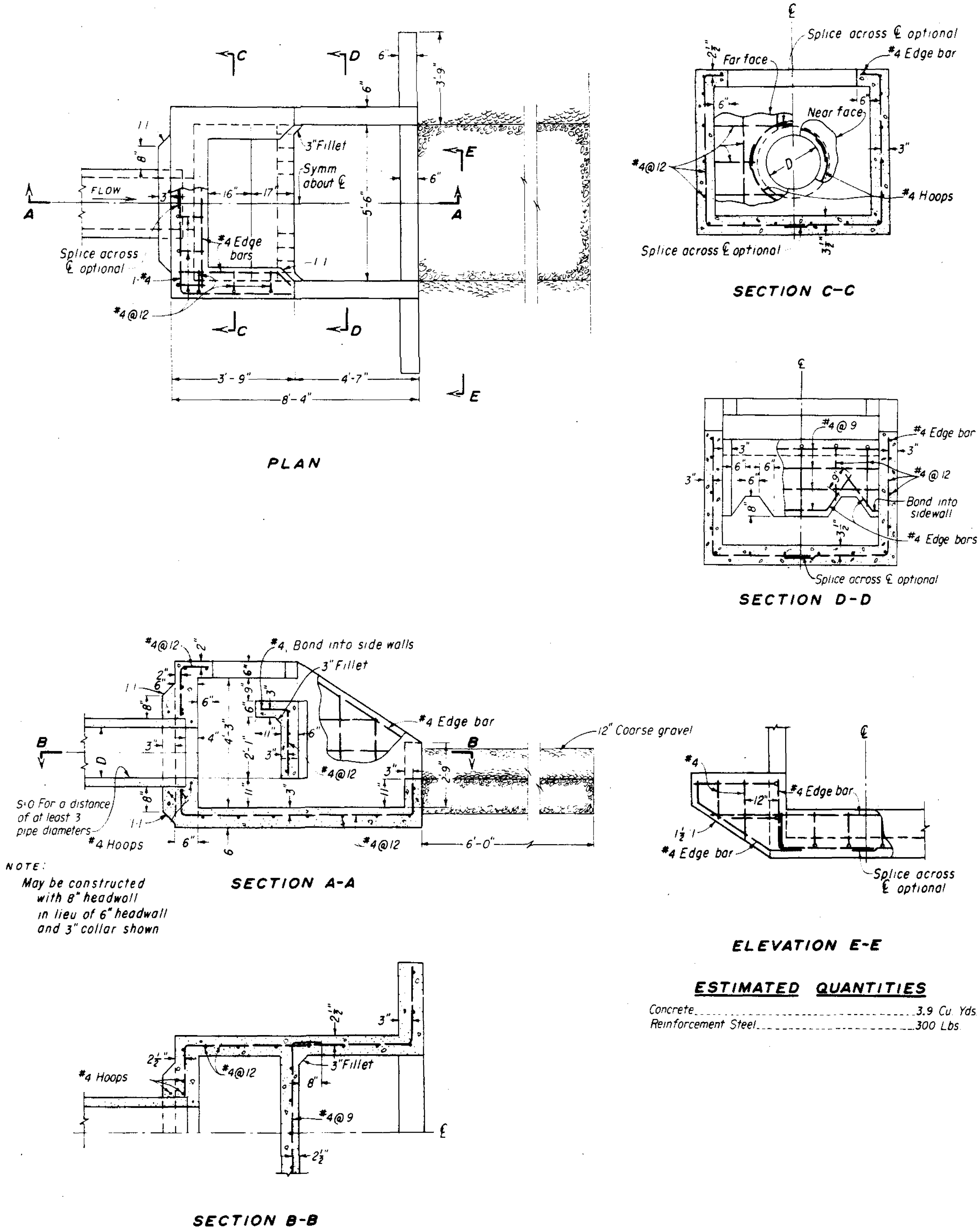


Figure 6-13. **Type 3 baffled outlet. 103-D-1342**

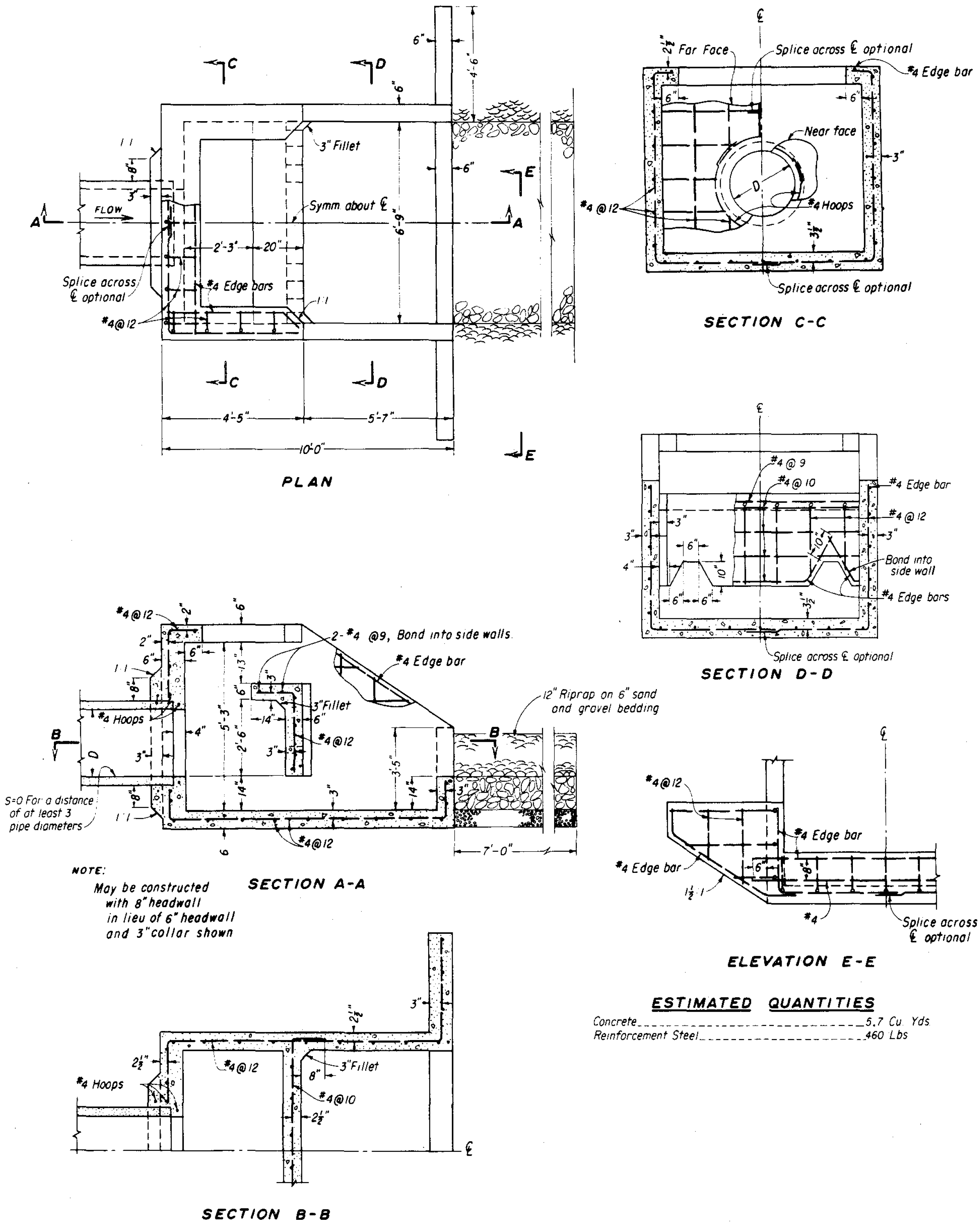


Figure 6-14. **Type 4 baffled outlet. 103-D-1343**

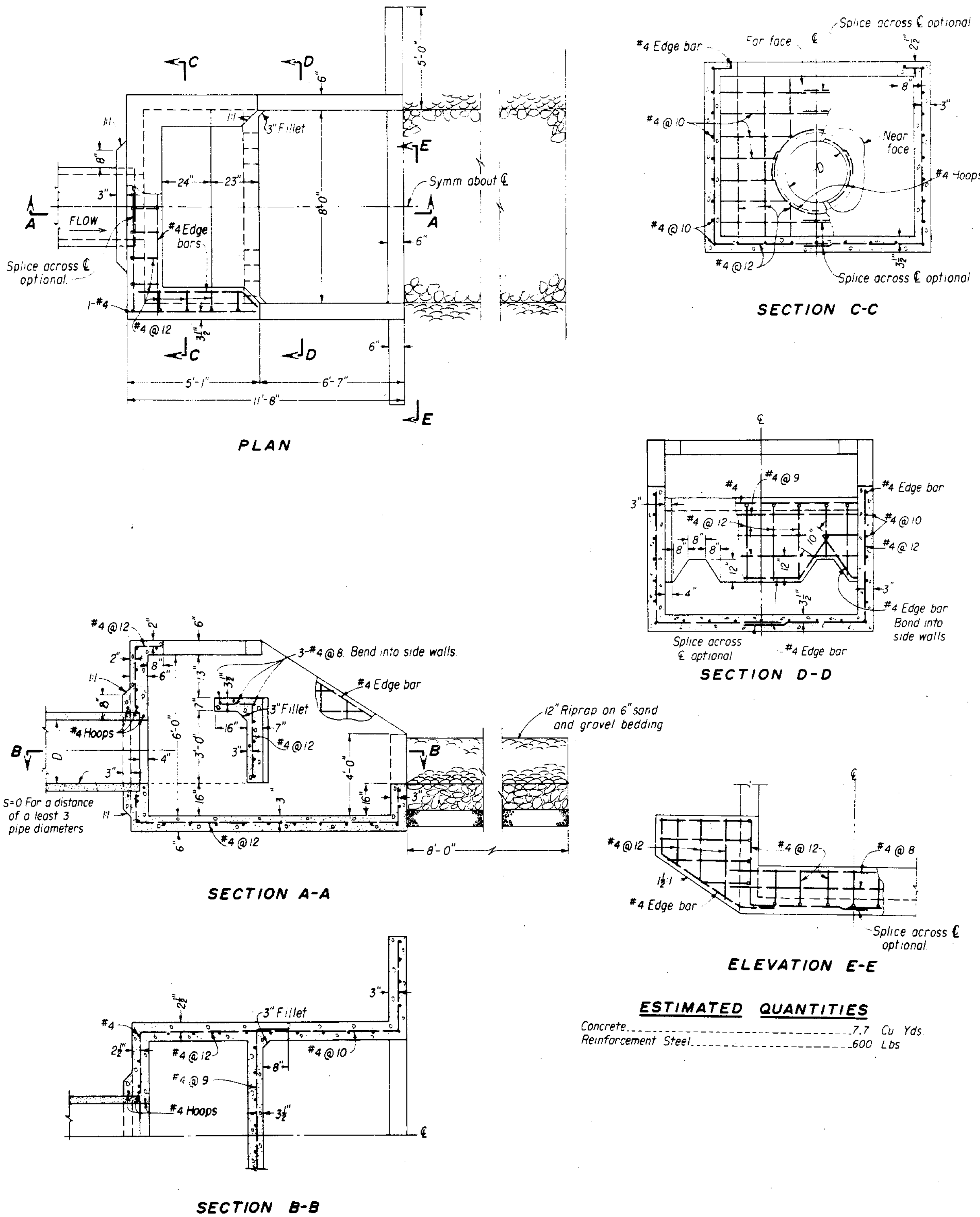


Figure 6-15. **Type 5 baffled outlet. 103-D-1344**

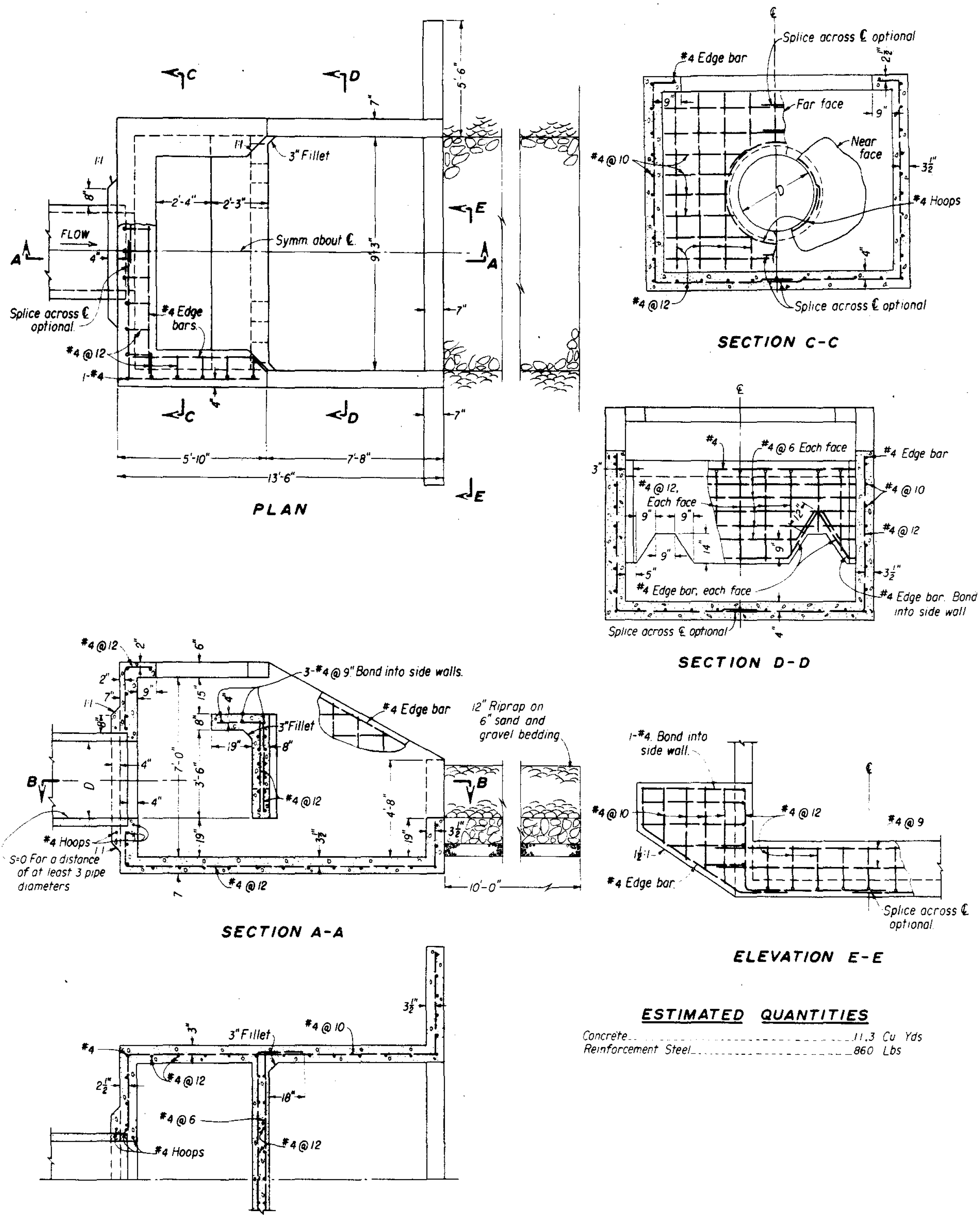


Figure 6-16. **Type 6 baffled outlet. 103-D-1345**

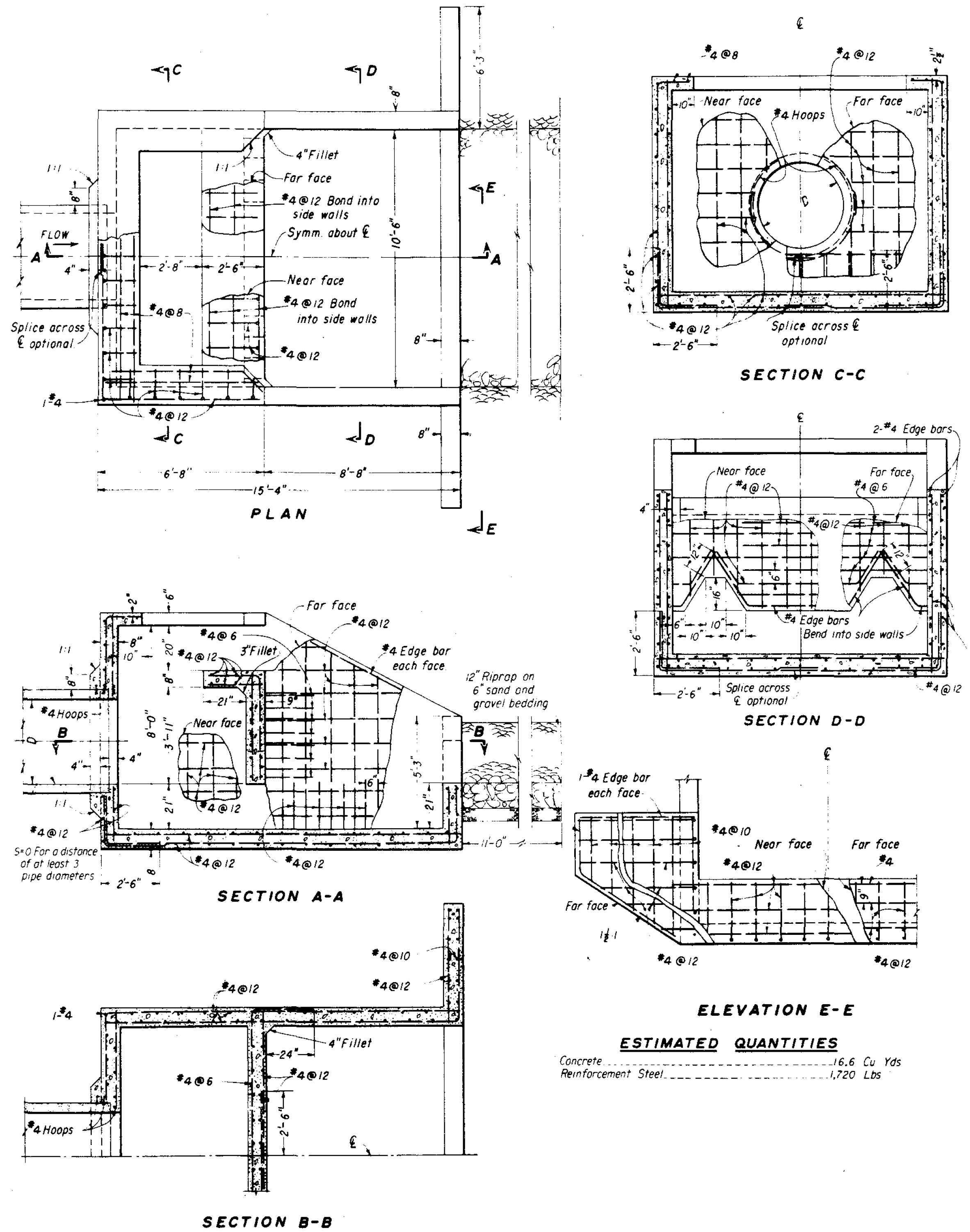


Figure 6-17. **Type 7 baffled outlet. 103-D-1346**

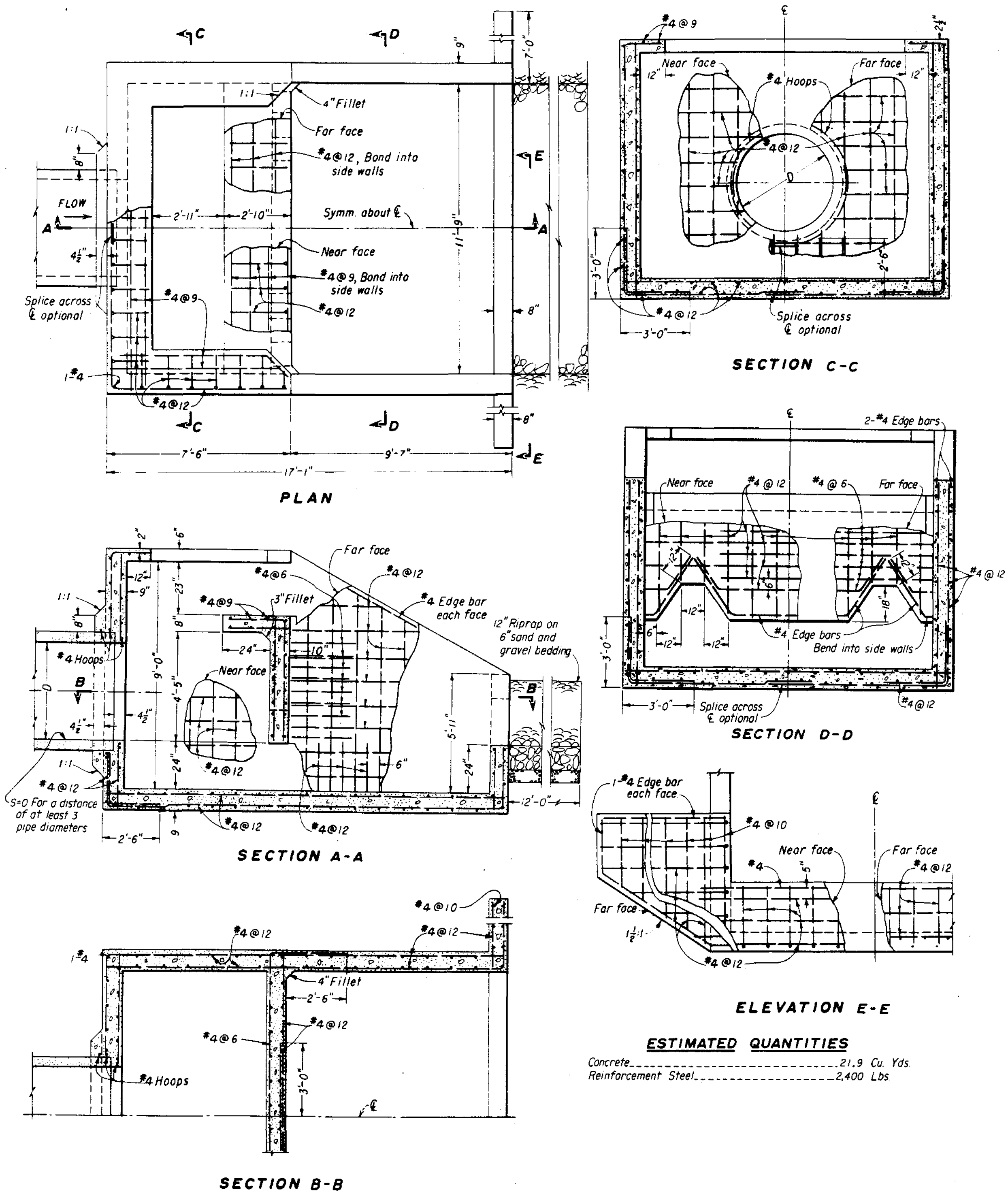


Figure 6-18. **Type 8 baffled outlet. 103-D-1347**

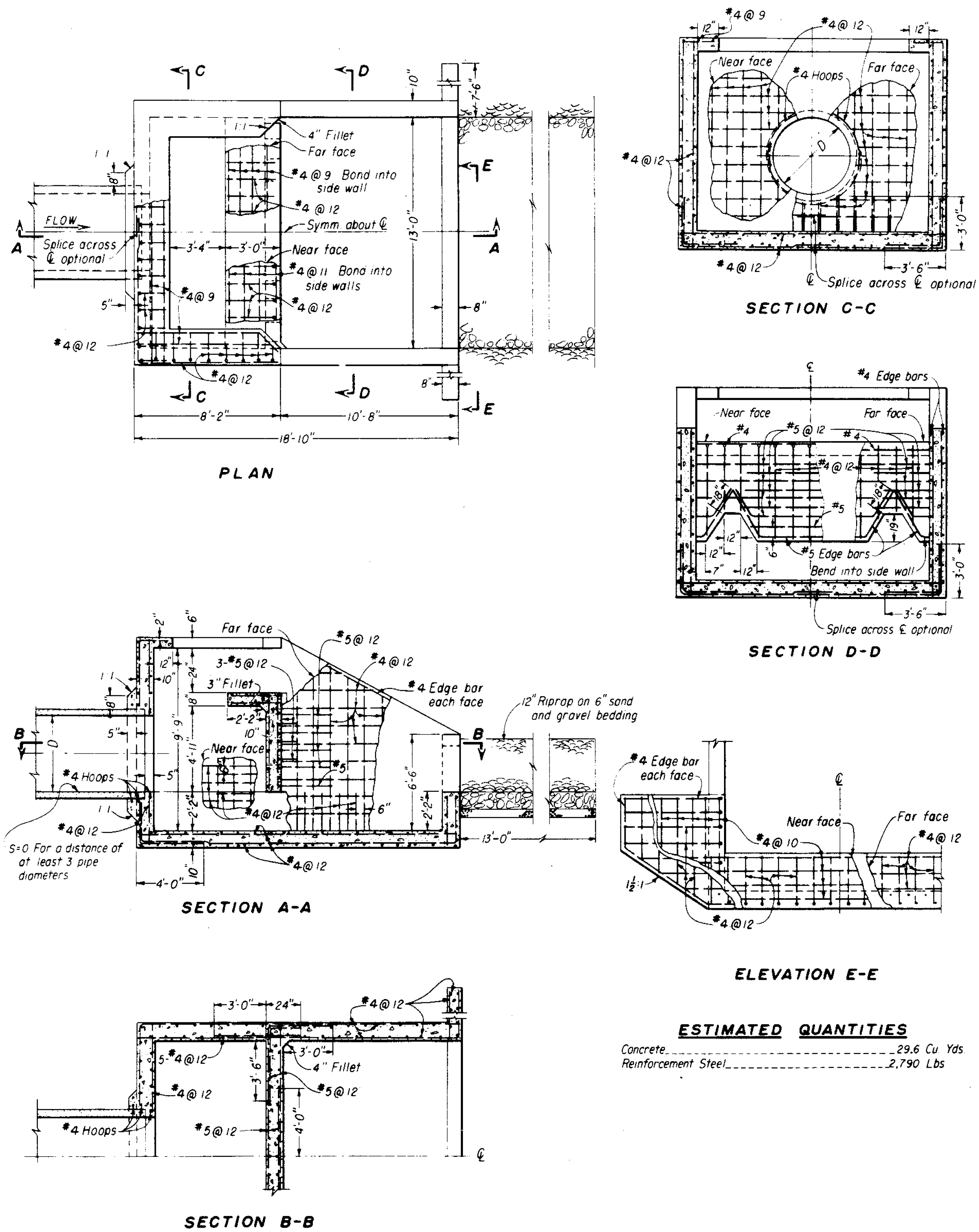


Figure 6-19. **Type 9 baffled outlet. 103-D-1348**

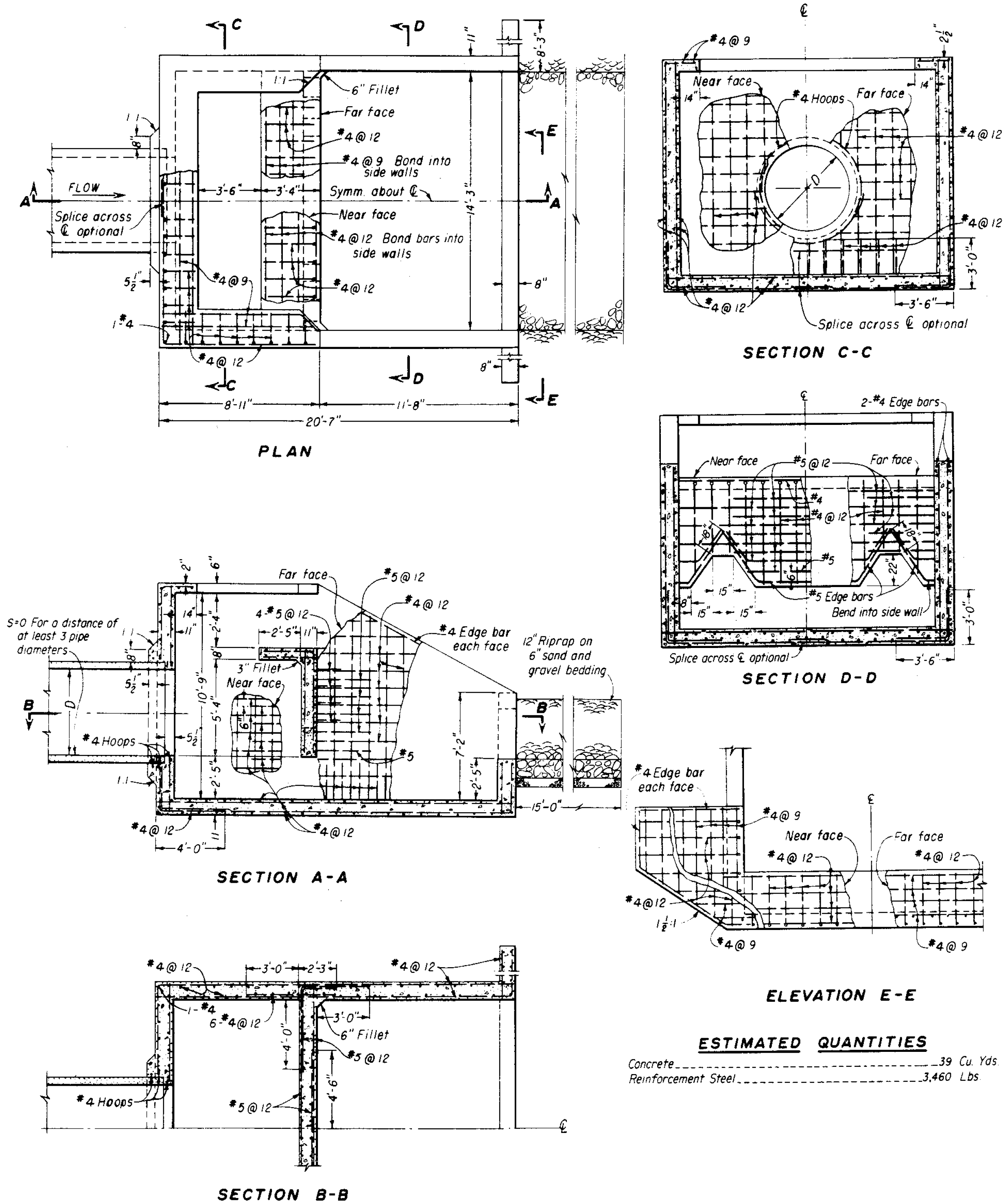


Figure 6-20. **Type 10 baffled outlet. 103-D-1349**

design dimensions and details for baffled outlets ranging in widths from 3 feet 6 inches to 14 feet 3 inches with flow entering the baffled outlet from pipes. Although uncommon, flow may also enter the basin from a rectangular open channel. In this case the channel walls should be as high as the basin walls and the invert should be horizontal for a minimum of three channel widths upstream from the basin.

6-7. ***Sediment and Debris.***—During periods of very low flow or nonoperation, sediment may accumulate in the basin. Notches in the baffle permit concentrated jets to form which will normally start erosion of the sediment and eventually wash the sediment from the basin with an increase in flow. However, as an added safety precaution, the basin is capable of satisfactorily discharging the entire design flow over the top of the baffle.

There is no practical method of making the basin self-cleaning of debris, such as Russian thistle. This is a serious deficiency of this structure in many locations. Where debris is a problem, other types of energy dissipators should be used; or if practical, screening devices are recommended at the entrance to the incoming pipe and at some locations it may be advantageous to screen the top of the baffled outlet structure itself. If thistles are allowed to enter the basin, they will not wash out.

6-8. ***Protection.***—Protection with a well-graded mixture of rocks, most having diameters equal to one-twentieth of the basin width, should be placed to a depth equal to the height of the end sill for a distance equal to one basin width downstream from the end sill. See figure 7-8. This protection will generally consist either of 12-inch coarse gravel or 12-inch riprap on a 6-inch sand and gravel bedding.

6-9. ***Basin Width Design Procedure.***—For a given design Q and head, h, the width of the baffled outlet basin is determined as follows:

(1) Compute the theoretical velocity in feet per second, $V = \sqrt{2gh}$

(2) Then compute the cross-sectional area of the incoming flow in square feet, $A = \frac{Q}{V}$

(3) Next compute the depth of flow, d in feet, $d = \sqrt{A}$ (This assumes the shape of the jet is square.)

(4) Compute the Froude number, $F = \frac{V}{\sqrt{gd}}$

(5) For this Froude number read $\frac{W}{d}$ ratio from the curve on figure 6-10.

(6) Then W in feet $= d(\frac{W}{d})$. This is the minimum width which should be used.

6-10. ***Design Example.***—(a) *Given*:

(1) *Hydraulic data.*—Cross-drainage pipe structure with a design Q of 150 cfs, and a head, h, of 30 feet.

(2) *Select pipe diameter.*—Selection of the pipe diameter is based on a maximum pipe velocity of 12 feet per second assuming the pipe is flowing full. Minimum pipe area required $= \frac{Q}{V} = \frac{150}{12} = 12.5$ ft.^{2}. Select 48-inch-diameter pipe (A = 12.57 ft.^{2}).

(3) *Air vent requirement.*—The water surface required at the inlet will submerge the top of the pipe and the outlet of the pipe is assumed to be sealed for this example. An air vent into the pipe near the inlet is therefore required. The vent diameter should be about $\frac{1}{6}$ of the pipe diameter or $\frac{48}{6} = 8$ inches.

(b) *Determination of the basin dimensions*:

(1) Theoretical velocity

$$V = \sqrt{2gh} = \sqrt{2 \times 32.2 \times 30} = 43.93 \text{ f.p.s.}$$

The maximum allowable theoretical velocity is 50 feet per second for a baffled outlet. Therefore a baffled outlet may be used.

(2) Area of flow

$$A = \frac{Q}{V} = \frac{150}{43.93} = 3.42 \text{ ft.}^2$$

(3) Depth of flow

$$d = \sqrt{A} = \sqrt{3.42} = 1.85 \text{ ft.}$$

(4) Froude number

$$F = \frac{V}{\sqrt{gd}} = \frac{43.93}{\sqrt{32.2 \times 1.85}} = 5.7$$

(5) $\frac{W}{d}$ Ratio

For

$F = 5.7$, read $\frac{W}{d} = 7.7$ from curve on figure 6-10.

(6) Minimum width of basin

$$W = d \times \frac{W}{d} = 1.85 \times 7.7 = 14.25 \text{ ft.}$$

(7) Baffled outlet design width

Use

$$W = 14 \text{ feet } 3 \text{ inches}$$

(8) Basin invert.—Set the basin invert a distance f below the natural ground surface.

(9) Other dimensions, including the notch dimensions are related to the design width and may be computed using relationships to W as shown on figure 6-10. For this particular example, figure 6-20 would be selected to construct the basin.

(c) *Determination of erosion protection requirements*:

(1) Required rock diameter $= \frac{W}{20} = \frac{14.25}{20} = 0.71$ foot. The volume of a rock 0.71 foot in diameter is $\frac{\pi \, dia^3}{6} = 0.19$ cubic foot. From subchapter VII B, Erosion Protection, it can be seen that coarse gravel protection has a maximum size of $\frac{1}{8}$ or 0.13 cubic foot which is less than required. Therefore, coarse gravel is too small and should not be used. The next size protection is 12-inch riprap on 6-inch sand and gravel bedding and is adequate, since the average volume of the largest and smallest permissible rock for 12-inch riprap = 0.5 cubic foot.

(2) Length of protection required W = 14.25 feet. Use 15 feet.

(3) Depth of protection required $= \frac{W}{6} = \frac{14.25}{6} = 2.38$ feet. Twelve-inch riprap on 6-inch sand and gravel bedding has a total thickness of only 18 inches (1.5 feet), but the size of rock is much larger than required so this thickness will be considered adequate.

For this particular example, if the baffled outlet in figure 6-20 had been selected the protection would agree with the above protection.

D. VERTICAL SLEEVE VALVE STILLING WELLS

R. B. HAYES[1]

6-11. ***General***.—(a) *Description*.—The vertical sleeve valve stilling well is capable of dissipating an excess head as great as 800 feet, discharging a tranquil flow to the downstream canal. The sleeve valve can also be used to regulate the flow. A gate valve is usually required upstream from the sleeve valve to permit closing down the installation for maintenance or for the end of the seasonal operation.

Field usage and laboratory studies have contributed to the development of design criteria for optimum dissipation of energy in sleeve valve stilling wells. The general arrangement is of two types:

(1) *Basic arrangement (descending flow)*.—This is the usual arrangement, with the inflow entering near the top of the well, where an elbow directs it vertically downward through the sleeve valve. Dissipation of energy occurs as the vertical jet strikes the floor and floor cone and as the subsequent radial flow strikes the sidewalls of the well (see fig. 6-21A).

Other losses occur through pipeline friction, bend losses, and valve losses. This arrangement represents the configuration for which design parameters are presented in detail in subsequent paragraphs.

(2) *Alternate arrangement (ascending flow)*.—To conform to the ground surface and pipeline profile another configuration is

[1] Op. cit., p. 299.

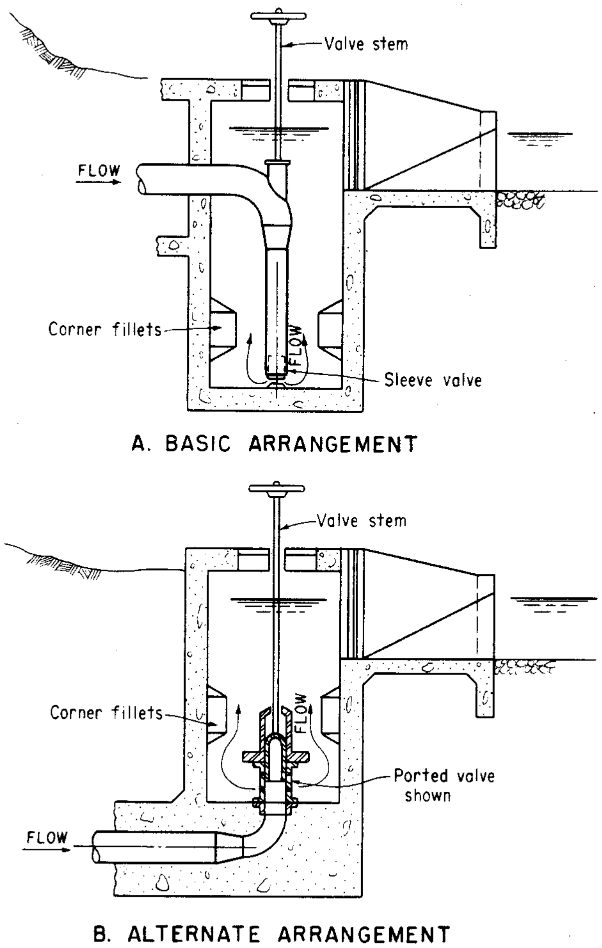


Figure 6-21. **Sleeve valve stilling well arrangements. 103-D-1350.**

sometimes used, with the pipe entering the well near the bottom and discharging upward through the sleeve valve (see fig. 6-21B). Dissipation of energy occurs in a manner similar to that described for the basic arrangement, except that the initial jet strikes a baffle instead of the floor. This arrangement has the advantage that the sleeve valve control stem is not subjected to the high velocity flow encountered in the basic arrangement, giving it the capability of withstanding higher velocities without excessive vibration.

(3) *Horizontal arrangement.*—A third arrangement, currently being developed in the Bureau of Reclamation Hydraulics Laboratory, consists of a horizontal sleeve valve discharging into a horizontal chamber.

(4) *Other features.*—To assist in obtaining optimum dissipation of energy two innovations have been used in the corners of the stilling well:

The *corner fillet* design [7] has resulted in a very smooth water surface at the outlet. The *corner angle* design, as tested in the USBR Hydraulics Laboratory, also yields a very smooth outlet water surface and is more economical. As the corner angle design has not yet been proven in the field, the corner fillet design is illustrated in the following paragraphs.

To improve the energy dissipation characteristics of the well for the basic arrangement, the earlier designs used a pedestal on the floor of the well. The pedestal also provided a convenient means of seating and anchoring the valve. Recent experience has indicated that the pedestal may be causing concrete erosion in the well and that a more effective arrangement is offered by omitting the pedestal and lining the floor and sides with steel, as indicated in subsection 6-12(e).

The standard sleeve valve, as described in the following paragraph, has in the past been supported by legs extending from the bottom of the valve to the floor or pedestal, as shown in figure 6-22. As this location is in the zone of high velocity and concrete erosion has been experienced on the sidewalls opposite these legs, it is suspected that the legs may be creating the erosion problem on the sidewalls [7]. Subsequent designs may locate the supports higher on the vertical pipe, extending to the sidewalls, provided mounting and alinement difficulties can be overcome.

(b) *Types of Vertical Sleeve Valves.*—Two basic types of vertical sleeve valves are used:

(1) The *standard sleeve valve* includes two basic designs, each using a plain sleeve (without ports). The sleeve may be inside or outside the valve housing. Most of the earlier sleeve valves incorporated an *internal sleeve* fastened to an internal valve stem which extended through the vertical pipe and 90° elbow to the handwheel.

Turbulence, vibration, noise, and cavitation resulted from the location of the

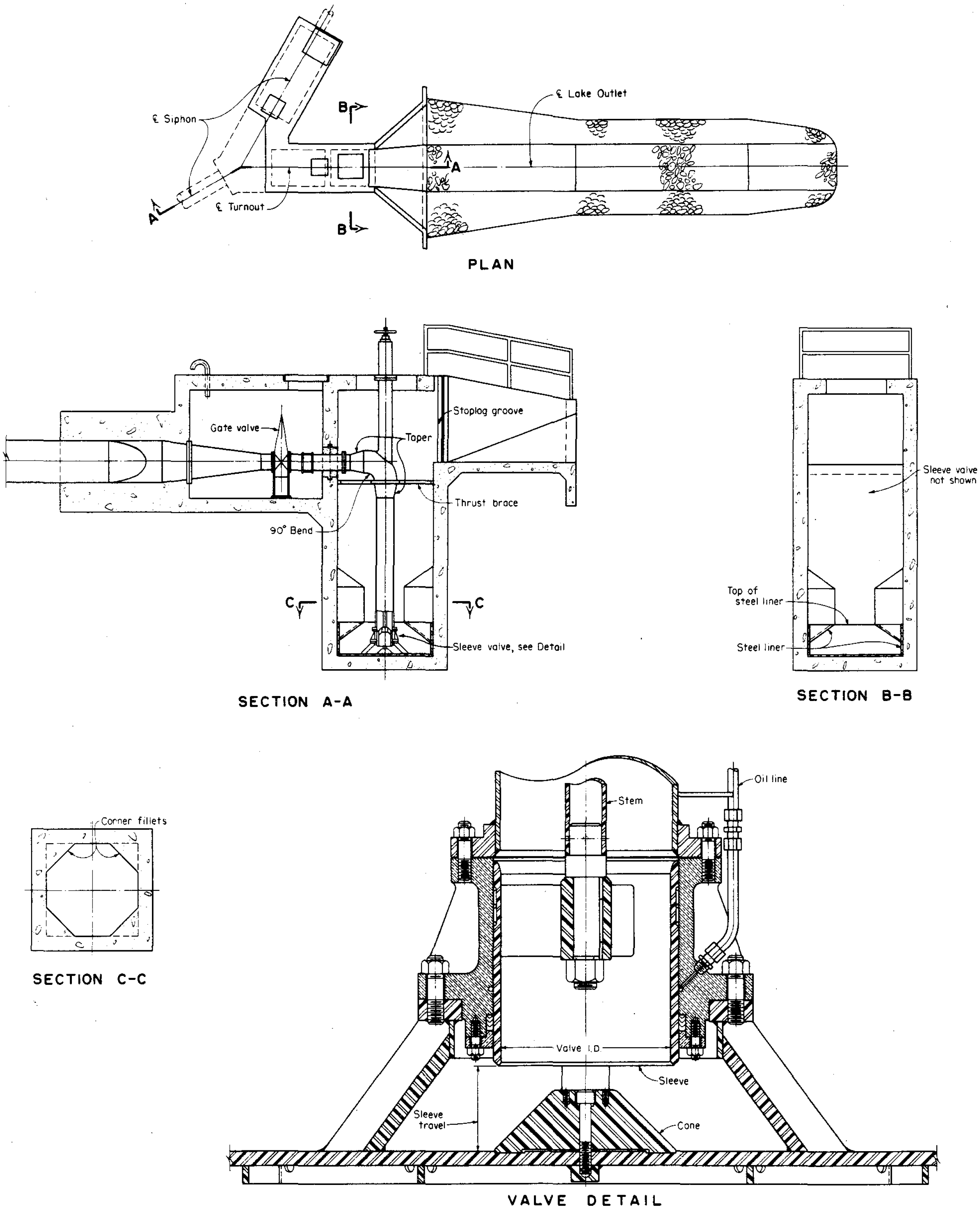


Figure 6-22. **Typical sleeve valve installation for pipe turnout. 103-D-1351.**

stem in the high velocity flow. A modification [8, 10] to the internal sleeve design alleviated this problem by increasing the diameter of the pipe used for the 90° bend, providing a gross cross-sectional area of about twice the area of the vertical pipe. Further, the stem connections to the internal sleeve were streamlined to reduce turbulence.

Subsequent designs have incorporated an *external sleeve*, with stem connections outside the valve and therefore outside the high velocity flow (see fig. 6-23A). This design greatly reduces the possibility of cavitation and vibration in the valve and obviates the need for an enlarged bend.

(2) The *ported sleeve valve* (multijet), as shown in figure 6-23B, is being used to dissipate heads considerably greater than those permitted for the standard sleeve valve. The ported sleeve valve incorporates a perforated housing with many tapered holes or slots of carefully determined sizes and configurations. Flow from the small jets mixes with the water in the well, dissipating the energy with very little vibration, noise, or cavitation.

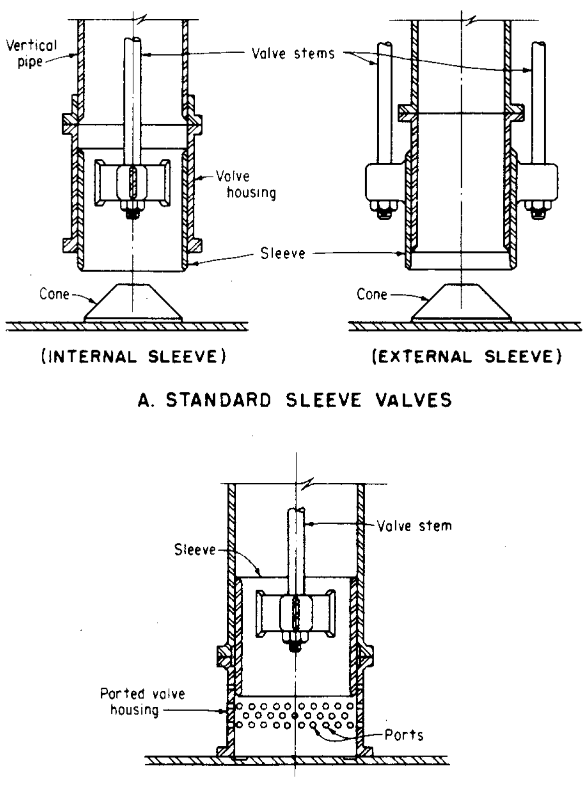


Figure 6-23. **Typical vertical sleeve valves. 103-D-1352.**

(c) *Usage of Vertical Sleeve Valve Stilling Wells.*—The ability of the vertical sleeve valve stilling well to dissipate high heads is utilized in the following design conditions:

(1) *Canal headworks.*—Excess energy may exist at the outlet of a pipe extending from a reservoir to a canal.

(2) *Pipe drop.*—A canal may have a need for a drop on a slope that is too deep for other types of energy dissipators.

(3) *Turnout.*—A turnout for water delivery may be required from a siphon, pump discharge line, or pipeline at a location where a considerable excess of head exists (see fig. 6-22).

(4) *Powerplant bypass.*—A powerplant bypass, carrying the discharge that would otherwise flow through the penstocks and the powerplant, must dissipate a head approximately equal to the operating head on the plant.

(d) *Limitations.*—In the design of the vertical sleeve valve stilling well, a few major aspects warrant careful consideration:

(1) *Head and velocity.*—Standard sleeve valves have been used by the Bureau of Reclamation for heads in excess of 400 feet. However, experience indicates that damage from severe vibration of the internal valve control stem may result where they are used with velocities greater than 30 feet per second in the vertical pipe.

With proper design of the valve, head in itself need not be a limiting factor. However, to allow for widely varying conditions of operation, the failure to suggest some head limitation may constitute an irresponsible omission. Therefore the following recommendations are offered as flexible limitations on the maximum permissible pressure head and velocity at the valve while discharging the design flow:

150 feet – Standard sleeve valve with internal sleeve and enlarged pipe for bend (30 fps),

400 feet – Standard sleeve valve with external sleeve (38 fps),
600 feet – Ported sleeve valve with enlarged pipe for bend (38 fps).

Current investigations arouse confidence that improvement to the ported sleeve valve will soon permit satisfactory performance for heads to 800 feet.

(2) *Water quality.*–The standard sleeve valve can tolerate a small amount of the weed or trash load which usually accompanies the flow in an open irrigation system. The ported sleeve valve, however, should be used only where clean water is assured by screening. Inlet trashracks should be submerged to facilitate the intake of clean water. Where a pipe turnout branches off from a pipeline, the centerline of the turnout pipe should be set at the centerline of the pipeline.

(3) *Operational limitations.*–To avoid air problems in the pipe upstream from the stilling well, the sleeve valve should be operated in such a way that the pipe will flow full. As the canal discharge varies, the sleeve valve opening should be adjusted to conform to the change in discharge. This adjustment can be simplified by the use of automatic or remote control to synchronize the pipe flow with the upstream discharge.

Rapid closure of the sleeve valve should be avoided, as the resulting water-hammer pressures [9] could destroy the installation.

6-12. *Design Considerations (Standard Sleeve Valve with Corner Fillets in Well).*–(a) *Inlet Conditions.*–To minimize noise, vibration, and cavitation, the standard sleeve valve with an internal sleeve and stem should have an inlet bend which provides a gross cross-sectional area equal to about twice the area of the vertical pipe, thus reducing the velocity in the bend [10]. A reducer is required to connect the bend to the vertical pipe. It should have a convergence no greater than 1 in 2 on each side [8]. The minimum length of the reducer is then equal to ΔD, the difference in diameters.

A thrust force on the pipe bend results from the change in direction of flow and should be balanced by an opposing force supplied by suitable anchorage as shown in figure 6-22. To provide adequate anchorage of the sleeve valve, the total thrust force, F_t, should be resolved into two forces, F_x and F_y (see fig. 6-24), representing the horizontal and vertical components of the thrust force.

The thrust force is a function of two forces as follows:

(1) *Hydrostatic force.*–The horizontal and vertical components of the hydrostatic force on the 90° bend are:

$$F_{h_x} = F_{h_y} = wAH = 62.4 \frac{\pi D^2}{4} H = 50\,D^2 H$$

where

w is the weight of water per cubic foot,
A is the cross-sectional area of the pipe,
D is the inside diameter of the pipe, and
H is the hydrostatic head at the centerline of the pipe.

(2) *Acceleration force.*–The horizontal and vertical components of the acceleration force on the 90° bend are as follows:

$$F_{a_x} = -\,Q\rho\,(V_2 \cos\alpha - V_1)$$
$$\text{and } F_{a_y} = Q\rho\,(V_2 \sin\alpha - 0)$$

where

Q is the discharge in cubic feet per second,
$\rho = w/g = 62.4/32.2 = 1.94$,
V is the velocity in feet per second, and
α is the change in direction of flow in degrees.

When

$$V_1 = V_2,\ \text{and}\ \alpha = 90^\circ,$$
$$F_{a_x} = F_{a_y} = Q\rho V$$

The horizontal and vertical components of the total thrust force are shown to be equal where a 90° pipe bend has a uniform diameter of pipe. Thus,

$$F_{T_x} = F_{T_y} = 50D^2 H + Q\rho V$$

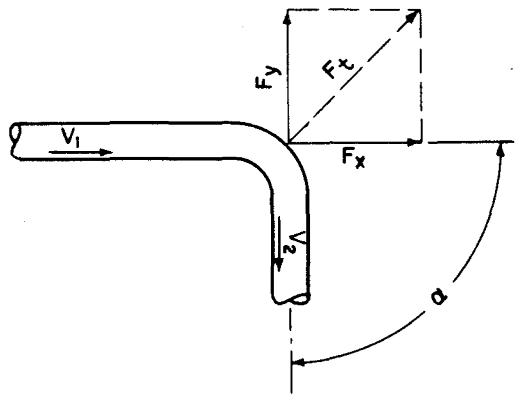


Figure 6-24. **Thrust forces on bend. 103-D-1353.**

The horizontal thrust force, F_{T_x} should be resisted by adequate horizontal bracing, as shown in figure 6-22, and the vertical thrust force, F_{T_y} should be resisted by the legs placed at or near the base of the vertical pipe.

(b) *Sleeve Valve Design.*—The sleeve valve is often designed for manual operation with adaptability to power operation. A handwheel attached to the valve stem is operated to adjust the valve opening as follows:

(1) *Standard sleeve valve.*—The standard sleeve valve usually has a sleeve travel equal to one-half the sleeve diameter. The sleeve diameter should be large enough to permit discharge of the design flow when fully open. Complete closure of the standard sleeve valve would shut off all flow. A steel truncated cone is located under and concentric to the valve. It provides a seat for the beveled edge of the sleeve and seals off the flow when the valve is fully closed. The cone also improves the sensitivity of the valve to small adjustments.

The sleeve diameter, travel, and cone angle can be varied to meet specific conditions. For control of flow velocities at higher heads, it may be desirable to oversize the sleeve and increase the cone angle.

(2) *Ported sleeve valve.*—The ported sleeve valve has a sleeve similar to the internal sleeve of the standard sleeve valve. The sleeve is raised by the handwheel, exposing the ported (or perforated) housing.

Valves have been tested with various port sizes and shapes. The smaller the individual port, the more rapid is the diffusion of its jet and the greater the energy dissipation. However, the smaller the port, the smaller the discharge and the greater its susceptibility to plugging. The bottom row of ports should be about 24 port diameters above the base to eliminate cavitation damage [7]. Further testing is necessary before optimum port size, shape, and distance from the well surfaces can be established.

(c) *Hydraulic Requirements.*—

(1) *Head.*—The head, ΔH, to be dissipated by the sleeve valve is the differential pressure head across the valve equal to $H_1 - H_2$ as shown in figure 6-28. The head, H_1, is measured from the centerline of the pipe to the hydraulic gradient immediately upstream from the valve elbow. The head, H_2, is measured from the same pipe centerline elevation to the water surface in the well. As the head, H_1, varies with the upstream water surface and also with the discharge, the worst condition is not always apparent without a hydraulic analysis (see comments in (3) below, and in section 6-11(d)(3).

(2) *Velocity.*—To reduce the threat of cavitation in the valve, a conservative velocity limitation should not exceed 30 feet per second when using an internal sleeve and 38 feet per second when using an external sleeve.

(3) *Sleeve valve diameter.*—The diameter of the vertical pipe and sleeve valve should be determined for maximum design discharge when minimum head exists. This diameter may not be the same as the diameter of the pipeline or of the pipe bend.

(4) *Orifice area (standard sleeve valve).*—The standard sleeve valve usually has a maximum sleeve travel equal to $\frac{D_v}{2}$. When fully open, the orifice area of the sleeve valve is greater than the cross-sectional area of the valve. The orifice of a valve with a 45° cone, as shown in figure 6-25, has a flow area, A_o, equal to the cross-sectional area of the valve, A_v, when the vertical opening, $a = 0.46\ D_v$, and the diagonal opening, $a_o = 0.707a$. Therefore, the valve

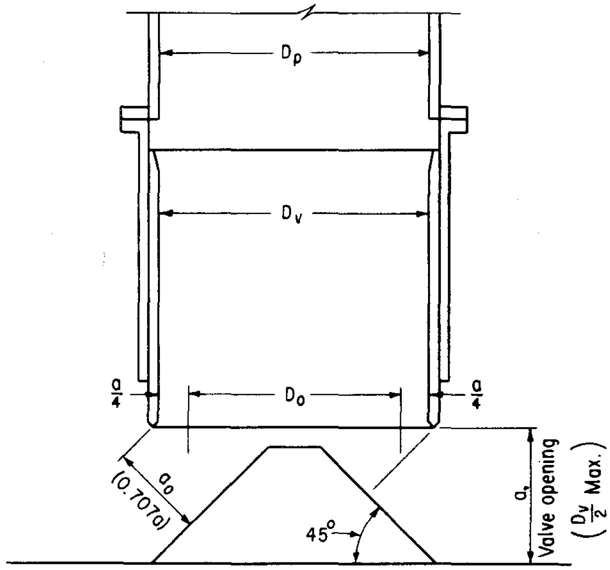


Figure 6-25. **Orifice of the standard sleeve valve. 103-D-1354.**

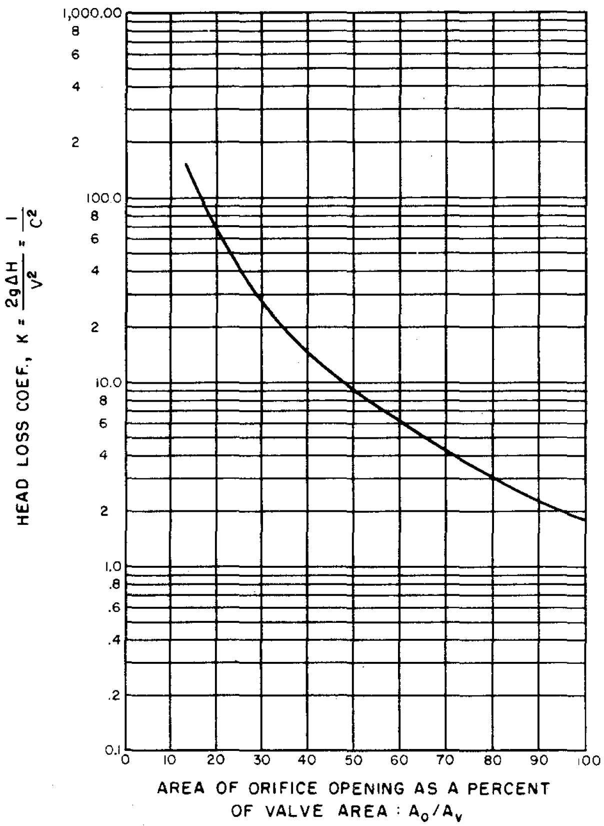


Figure 6-26. **Head loss coefficient for the standard sleeve valve. 103-D-1355.**

can be considered 100 percent open when

$$a = 0.46\ D_v$$

The orifice area,

$$A_o = 0.707a\ (\pi D_o).$$

Substituting

$$D_o = D_v - \frac{a}{2}\ (\text{see fig. 6-25}),$$

$$A_o = 0.707a\pi\ (D_v - \frac{a}{2}).$$

(5) *Discharge coefficient for standard sleeve valve.*—Laboratory studies [7] indicate a head loss coefficient,

$$K = \frac{2g(\Delta H)}{V^2} = 1.84\ (\text{see fig. 6-26}),$$

when the valve is fully open ($a = 0.46D_v$),
where ΔH is the differential head across the valve, and V is the velocity in the vertical pipe.

The valve discharge coefficient, C, is related to the head loss coefficient, K, as follows:

$$K = \frac{1}{C^2}$$

or

$$C = \frac{1}{\sqrt{K}}$$

When the valve is fully open,

$$C = \frac{1}{\sqrt{1.84}} = 0.735$$

Use C = 0.73 for the fully open valve.

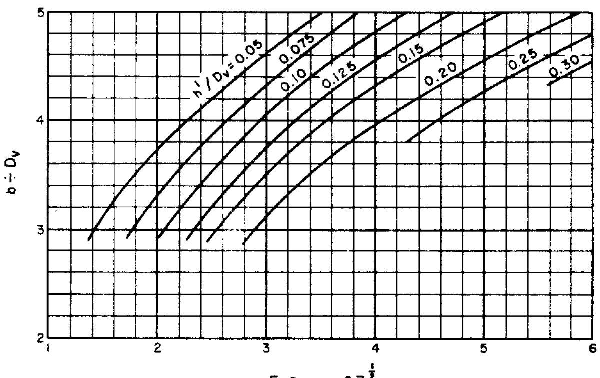
(d) *Well Dimensions (see fig. 6-28 for nomenclature).*—Well dimensions are determined from figure 6-27 as follows:

Enter along the abscissa with $\left[\frac{Q^2}{g\ D_v^{\ 5}}\right]^{1/2}$

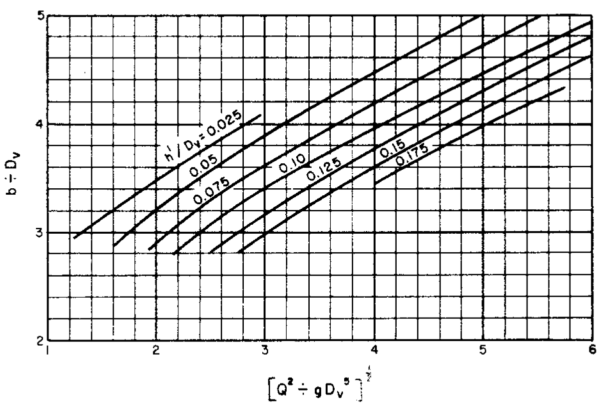
Follow vertically to the curve representing $\frac{h'}{D_v}$ and then horizontally to the ordinate, and read the ratio $\frac{b}{D_v}$

where

Q is the design discharge in cubic feet per second.



A. Based on a ratio, $d_w = 1.5b$



B. Based on a ratio, $d_w = 2b$

Figure 6-27. **Vertical sleeve valve stilling well dimensions. 103-D-1356.**

g is the acceleration of gravity (32.2 feet per second per second),

D_v is the valve diameter in feet,

$h' = \dfrac{h}{\sin\phi}$ (see fig. 6-28)

or $h' = 1.8h$ for 1-1/2 to 1 side slope and $h' = 2.23h$ for 2 to 1 side slope,

b is the width of the well in feet (3 ft minimum required for access), and

d_w is the depth of the well in feet as shown in figure 6-28.

From the Bureau of Reclamation Hydraulics Laboratory Studies [7], it has been established that satisfactory performance is achieved using well depth to width ratios,

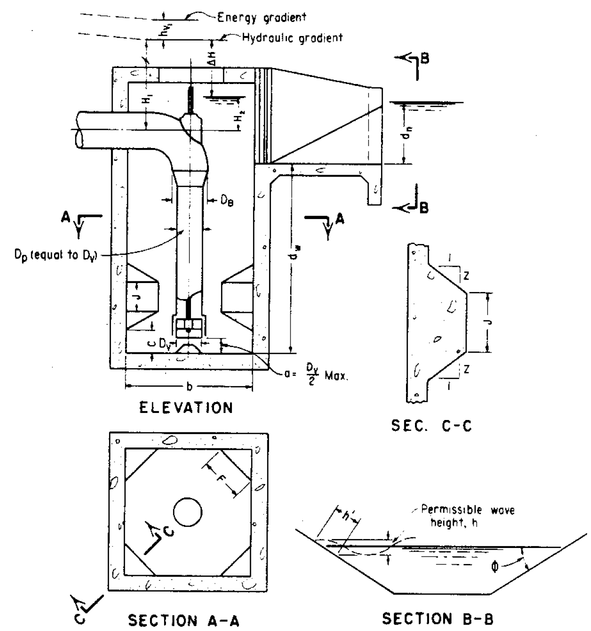


Figure 6-28. **Typical sleeve valve and stilling well. 103-D-1357.**

$$\frac{d_w}{b} = 1.5$$

and

$$\frac{d_w}{b} = 2.0$$

Figures 6-27A and 6-27B are provided to utilize both ratios.

The foregoing well dimensions are valid for a tailwater depth to well width ratio,

$$\frac{d_n}{b} = 0.5$$

or

$$d_n = \frac{b}{2}$$

After determining the well width and depth from figure 6-27,

$$\text{where } d_n > \frac{b}{2},$$

the dimension d_w could be reduced by the amount,

$$d_n - \frac{b}{2}$$

and

$$\text{where } d_n < \frac{b}{2},$$

the dimension d_w should be increased by the amount

$$\frac{b}{2} - d_n$$

Corner fillet dimensions are determined from the following relationships:

$$C = 0.100b$$
$$J = 0.210b$$
$$F = 0.417b$$
$$\text{and } Z = 0.715$$

See figure 6-28 for nomenclature.

Establishing a permissible tailwater wave height is a judgment factor based upon the depth, freeboard, and erosion resistance of the tailwater channel or canal. For the range of discharges treated in this text, a permissible wave height, h, of about 0.02 to 0.05 times the normal depth is recommended (see fig. 6-28).

(e) *Armor Plating.*—To prevent cavitation or abrasion damage to the floor and walls of the stilling well, they should be covered with a steel cover as follows:

1/2-inch-thick stainless steel on the floor and on the sidewalls to a height of 1.5 D_v.

The steel plating should be welded at junctions and should be anchored to the concrete with stainless steel anchors to prevent loosening. To avoid damage from galvanic action, care should be taken to prevent contact between the stainless steel and the reinforcement steel.

(f) *Outlet Transition.*—The flow from the well is discharged through a rectangular opening, through a transition to the outlet channel or canal. The transition is usually the broken-back type, which is suitable for a well-defined channel or canal section.

The water surface in the well must be higher than the tailwater elevation by an amount equal to the velocity head, h_v, in the downstream section, plus transition losses. Allowing a loss of $0.3\Delta h_v$ for the broken-back transition, the total water surface differential is equal to $1.3\Delta h_v$, where Δh_v is the difference in velocity heads in the well and in the downstream section. The velocity head in the well is considered to be zero (in the direction of the downstream flow). Therefore, the total water surface differential is equal to 1.3 times the downstream velocity head.

The design of the outlet transition should be similar to that discussed for bench flumes (subchapter II D). Where the tailwater elevation may vary considerably, as between the maximum and minimum water surfaces in a reservoir, a baffled apron drop is sometimes used in conjunction with, or downstream from, the sleeve valve stilling well.

(g) *Outlet Protection.*—To prevent erosion damage in the outlet channel, protection should be provided in accordance with figure 7-8, as indicated for a closed conduit drop with baffled pipe outlet.

6-13. *Design Example.*—A canal profile requires a drop of 150 feet on a steep slope. It is decided to accomplish the drop with a pipe and vertical sleeve valve stilling well.

(a) *Assumptions.*—

(1) Canal properties are as follows:

Maximum Q = 70 c.f.s.	d_n = 3.2 ft.
b = 8 ft.	v = 1.69 f.p.s.
ss = 1-1/2:1	h_v = 0.04 ft.
n = 0.025	s = 0.0003

(2) The maximum differential head, ΔH, across the closed valve is 150 feet, measured from the upstream canal water surface to the downstream canal water surface.

(3) At the design flow of 70 cfs, friction and bend losses plus pipe velocity head are equal to a total of 50 feet, leaving a differential head:

$$\Delta H = 150 - 50 = 100 \text{ feet}$$

(4) The basic arrangement (downward flow) best fits the profile.

(5) Trashracks at the inlet to the pipe drop assure fairly clean water through the valve, removing all but small debris.

(6) A tailwater wave height, h, equal to 0.10 foot (0.031 d_n), measured vertically from trough to crest (see fig. 6-28), is considered to be permissible.

(b) *Solution.*—

(1) *Sleeve valve type.*—A standard sleeve valve with an internal sleeve is adequate for dissipating the maximum head of 150 feet, or the minimum head of 100 feet during maximum discharge, and it is better able than the ported valve to pass the small debris that may pass through the trashracks.

(2) *Diameter.*—To minimize cavitation, the diameter of the vertical pipe and sleeve valve is determined on the basis of a maximum velocity of 30 feet per second.

Thus

$$A = \frac{Q}{V} = \frac{70}{30} = 2.33 \text{ sq. ft.}$$

Then, the diameter of the valve (and vertical pipe),

$$D_v = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 2.33}{\pi}} = 1.725 \text{ feet.}$$

Use

$$D_v = 1.75 \text{ feet,}$$
$$A_v = 2.41 \text{ sq. ft.}$$

and

$$V_v = 29.1 \text{ f.p.s.}$$

(3) *90^o elbow and taper.*—The long-radius bend should have a pipe diameter providing a gross area equal to twice the area of the vertical pipe,

or

$$2 \times 2.41 = 4.82 \text{ sq. ft.}$$

Then, the pipe diameter for the bend,

$$D_B = 2.48 \text{ ft. minimum}$$

Use

$$D_B = 2.50 \text{ ft.}$$

The length of the taper should be at least:

$$\Delta D = (2.5 - 1.75) = 0.75 \text{ ft.}$$

Use 1 foot.

Anchorage of the bend should be designed to resist the thrust forces discussed in subsection 6-12(a).

(4) *Discharge.*—Check the adequacy of the 21-inch-diameter sleeve valve, when fully open ($A_o = A_v$), to discharge the design flow, using a coefficient of discharge,

$$C = 0.73,$$
$$A_o = A_v = 2.41 \text{ sq. ft.,}$$
$$\text{and } \Delta H = 100 \text{ ft.}$$

From the orifice equation,

$$\begin{aligned} Q &= CA_o\sqrt{2g(\Delta H)} \\ &= 0.73\,(2.41)\sqrt{64.4 \times 100} \\ &= 141 > 70 \text{ cfs} \end{aligned}$$

Therefore, the diameter of 21 inches (determined to prevent cavitation) is more than adequate to discharge the design flow.

(5) *Required valve opening.*—Using a trial-and-error method, determine the required valve opening, a, for a discharge of 70 cfs, with a differential head, $\Delta H = 100$ ft. Try an area ratio, A_o/A_v, equal to 72 percent.

Thus,

$$\begin{aligned} A_o &= 0.72A_v \\ &= 0.72 \times 2.41 = 1.74 \text{ sq. ft.} \end{aligned}$$

From figure 6-26, with A_o/A_v equal to 72 percent,

$$K = 4.0$$

giving

$$C = \frac{1}{\sqrt{4.0}} = 0.50$$

From the orifice equation, the required orifice area,

$$\begin{aligned} A_o &= \frac{Q}{C\sqrt{2g\Delta H}} \\ &= \frac{70}{0.50\sqrt{64.4 \times 100}} \\ &= 1.75 > 1.74 \text{ sq. ft. (close enough)} \end{aligned}$$

Therefore, an orifice area equal to 72 percent of the valve area is sufficient for the design discharge at the design head.

From the equation for area of the sleeve valve orifice (see subsection 6-12(c)(4)),

$$A_o = 0.707\, a\pi(D_v - \frac{a}{2})$$

$$1.75 = 2.22\, a\,(1.75 - \frac{a}{2})$$

$$1.75 = 3.88\, a - 1.11a^2$$

Then, by trial, or from the quadratic equation, the required sleeve travel,

$$a = 0.55 \text{ ft.}$$

(6) Determine well dimensions, b and d_w, using figure 6-27A, with a ratio $\frac{d_w}{b} = 1.5$

On the abscissa,

$$\left[\frac{Q^2}{g(D_v)^5}\right]^{1/2} = \left[\frac{(70)^2}{g(16.41)}\right]^{1/2}$$

$$= (9.27)^{1/2} = 3.05$$

Using the curve,

$$\frac{h'}{D_v} = \frac{1.8h}{D_v} = \frac{1.8 \times 0.10}{1.75} = 0.10,$$

find

$$\frac{b}{D_v} = 4.1.$$

Then,

$$b = 4.1 \times 1.75 = 7.18 \text{ ft.}$$
$$(\text{say } b = 7 \text{ ft. } 2 \text{ in.}),$$
$$\text{and } d_w = 1.5\, b$$
$$= 1.5 \times 7.17 = 10.76 \text{ feet}$$

These values for b and d_w are valid, as indicated in subsection 6-12(d), only if:

$$d_n = \frac{b}{2}.$$

Actually

$$d_n = 3.2 \text{ feet}$$

and

$$\frac{b}{2} = \frac{7.17}{2} = 3.58 \text{ feet}$$

Therefore, the well depth, d_w, should be increased to compensate for the relative deficiency, δ, in tailwater depth,

where

$$\delta = \frac{b}{2} - d_n$$

$$= 3.58 - 3.2 = 0.38 \text{ feet}$$

Thus, d_w should be increased, resulting in a depth

$$d_w = 10.76 + 0.38 = 11.14 \text{ feet}$$

$$\text{Say } d_w = 11 \text{ ft. } 2 \text{ in.}$$

(7) For an economic comparison, the dimensions, b and d_w, should be determined again, using figure 6-27B, with a ratio,

$$\frac{d_w}{b} = 2.0$$

On the abscissa,

$$\left[\frac{Q^2}{g(D_v)^5}\right]^{1/2} = 3.05$$

as before, and using the same curve,

where

$$\frac{h'}{D_v} = 0.10,$$

find

$$\frac{b}{D_v} = 3.45$$

Then

$$b = 3.45 \times 1.75 = 6.04 \text{ feet}$$
$$(\text{use } b = 6 \text{ ft.})$$

and

$$d_w = 2b = 12 \text{ ft.}$$

These dimensions are valid only if

$$d_n = \frac{b}{2}$$

Actually

$$d_n = 3.2 \text{ feet}$$

and $$\frac{b}{2} = \frac{6}{2} = 3.0 \text{ feet}$$

Therefore, the well depth, d_w, could be reduced to compensate for the relative excess of tailwater depth,

$$\delta = d_n - \frac{b}{2}$$
$$= 3.2 - 3.0 = 0.2 \text{ ft.}$$

Thus, d_w could be reduced, resulting in a depth of

$$12.0 - 0.2 = 11.8 \text{ ft.}$$

Say 11 ft. 10 in.

Assuming the more economical well is obtained by using the dimensions resulting in the smaller concrete area, the structure designed with a ratio,

$$\frac{d_w}{b} = 1.5,$$

is slightly more economical.

(8) Determine the following dimensions from the established ratios:

$$\begin{aligned} C &= 0.100\ b \\ &= 0.100 \times 7.17 \\ &= 0.72 \text{ ft. (use 9 in.)} \end{aligned}$$

$$\begin{aligned} J &= 0.210\ b \\ &= 0.210 \times 7.17 \\ &= 1.51 \text{ ft. (use 18 in.)} \end{aligned}$$

$$\begin{aligned} F &= 0.417\ b \\ &= 0.417 \times 7.17 \\ &= 2.99 \text{ ft. (use 3 ft.)} \end{aligned}$$

(9) Steel plating should be provided in accordance with subsection 6-12(e), extending up the sidewalls to a minimum height:

$$1.5D_v = 1.5 \times 1.75 = 2.63 \text{ ft.}$$

Use 2 ft. 8 in.

(10) *Outlet transition and losses.*—As the canal section is well defined, a broken-back transition should be used, and the water surface differential from the well to the canal is equal to:

$$1.3\Delta h_v = 1.3 \times 0.04 = 0.05 \text{ ft.}$$

(11) *Outlet protection.*—The earth section of the canal downstream from the vertical sleeve valve stilling well should be protected from erosion by providing type 2 protection, extending 14 feet beyond the concrete transition, in accordance with figure 7-8.

E. BIBLIOGRAPHY

6-14. ***Bibliography***.

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[10] Miller, E., "Some Recent Developments in Flow Regulating Valves," Water Power, September 1969.

Transitions and Erosion Protection

R. B. YOUNG[1]

A. TRANSITIONS

1. General

7-1. ***Purpose and Description.***—Transitions usually produce gradual changes in water prism cross sections and are used at structure inlets and outlets and at changes in canal sections to: (1) provide smoother water flow, (2) reduce energy loss, (3) minimize canal erosion, (4) reduce ponded water surface elevations at cross-drainage structures, (5) provide additional stability to adjacent structures because of the added resistance to percolation, and (6) to retain earthfill at the ends of structures.

Transitions usually produce gradually accelerating velocities in inlet transitions and gradually decelerating velocities in outlet transitions. Because of the improved flow conditions at the ends of a pipe structure, the allowable pipe velocity may be increased and the size of pipe may be decreased if sufficient head is available.

Transitions are either open (no top) or closed. Closed transitions are used to further reduce energy losses for pipe structures by providing an additional gradual change of water prism cross section from rectangular to round. The refinement of rectangular to round transitions is usually not justified for capacities discussed in this publication. Open transitions may be either concrete or earth. Earth transitions are used to transition base width, invert elevation, and side slopes from a canal structure or concrete transition to that of the waterway section.

7-2. ***Types.***—(a) *Inline Canal Structure Transitions.*—The most common concrete transitions for inline canal structures are: (1) streamlined warp, (2) straight warp, and (3) broken-back. Broken-back refers to the intersection of the vertical and sloping plane surfaces on the sides of the transition as shown on figures 1-12, 2-6, 7-1, and 7-2 and is sometimes also referred to as dogleg. Broken-back transitions used with structures other than pipe structures (fig. 2-19) are discussed in other chapters. However, criteria in this chapter for water surface angles, cutoff dimensions, and freeboard at the cutoff are applicable. The streamlined and straight warp transitions will not be discussed because their refinement is usually not justified for the capacity range in this publication.

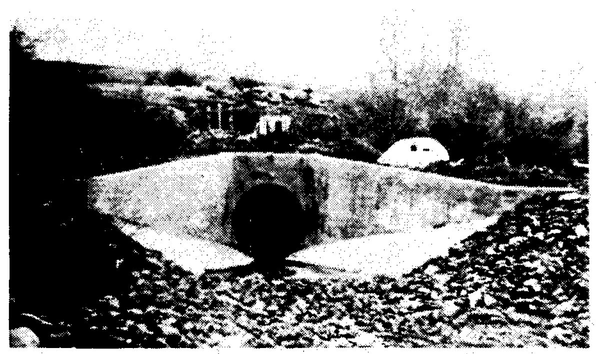


Figure 7-1. **Type 1 concrete transition. P33-D-25693**

[1]Civil Engineer, Hydraulic Structures Branch, Bureau of Reclamation.

A type 5 transition (fig. 7-3) is sometimes used to transition from a concrete-lined canal to a pipe structure. The transition energy head loss will be greater with this transition than with a more streamlined transition.

(b) *Cross-drainage Pipe Structure Transitions.*—The most common concrete transitions used with cross-drainage structures are type 1 (broken-back), figure 7-2; type 2, figures 7-4 and 4-19; type 3, figures 7-5 and 4-22; and type 4, figures 7-6 and 7-7. Addition of an inlet transition to a cross-drainage pipe structure permits the pipe inlet to be lowered which results in lowering of the required upstream water surface (provided the control is upstream). Lower upstream water surface minimizes inundation of farmland and increases the canal embankment freeboard.

A type 1 transition is used if the natural drainage channel has a well-defined cross section with dimensions that can reasonably be transitioned to the broken-back cross section. Where the uphill canal bank obstructs storm runoff from a relatively wide or lesser defined drainage channel, a type 2, 3, or 4 transition is usually more suitable. See chapter IV for further discussion of cross-drainage structures.

2. *Design Considerations for Pipe Structure Transitions*

7-3. ***Hydraulic Design.***—(a) *Pipe Submergence.*—Inlet transitions to pipe structures where the hydraulic control is at the downstream end of the structure should have a seal of 1.5 times the difference of velocity heads in the pipe and canal (1.5 Δh_v) or 3 inches minimum. The seal is measured between the upstream water surface of the inlet transition and the top of the opening in the transition headwall. This inlet submergence allows for a pipe entrance loss and a conversion of static head in the canal to full-pipe velocity head. For minimum head loss, the top of the opening at the outlet transition headwall should have little or no submergence. If the submergence exceeds one-sixth of the depth of the opening at the outlet, the head loss should be computed on the basis of a sudden enlargement rather than as an outlet transition.

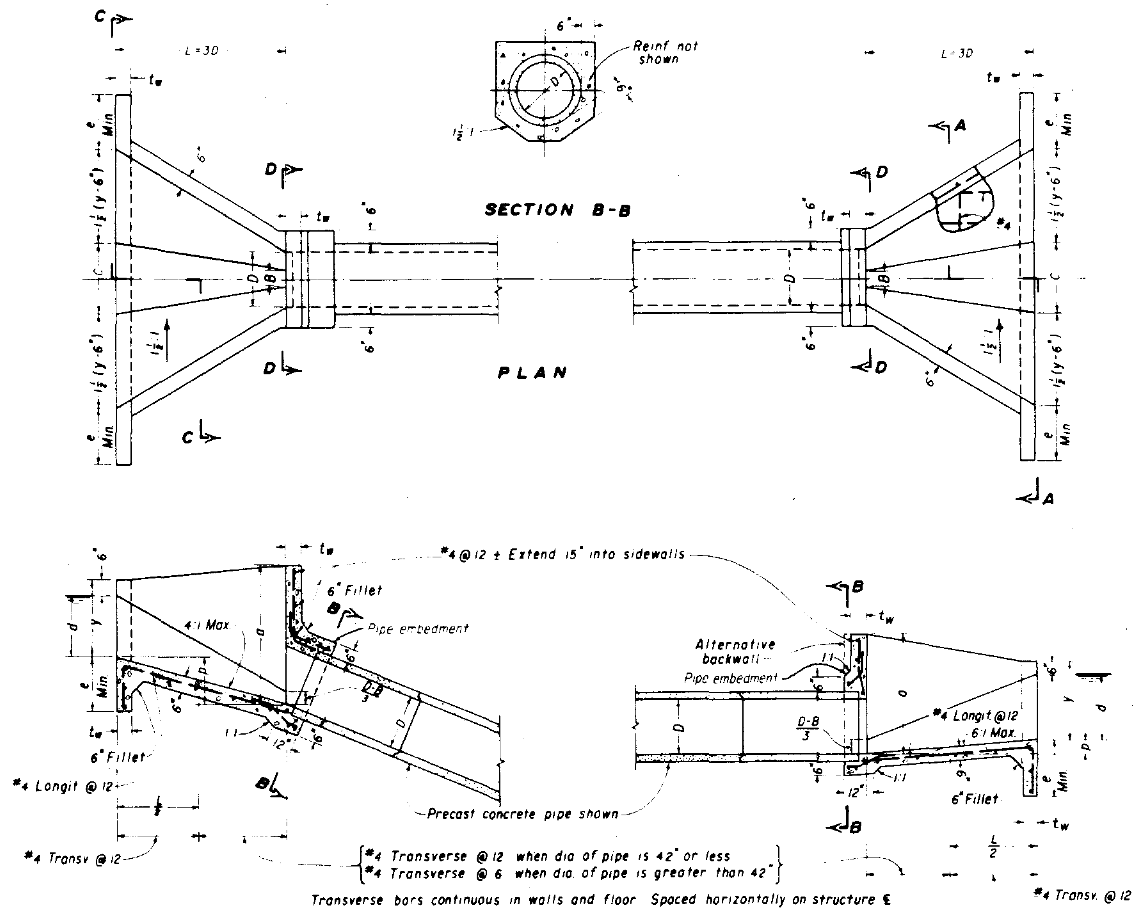
Theoretical differences in water surface in the canal and immediately inside the conduit at the headwalls are: $\Delta WS = (1+K_1)\,\Delta h_v$ at the inlet, and $\Delta WS = (1-K_2)\,\Delta h_v$ at the outlet. These values omit the small transition friction losses, and K_1 and K_2 are transition head loss coefficients described in the following paragraphs.

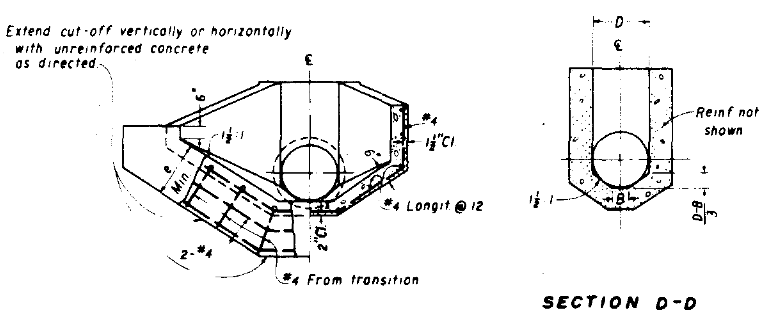
Where an inlet transition connects to a free-flow closed conduit in such a way that the conduit inlet is sealed, the head required to discharge the design flow can be determined by the orifice equation [1],[2] $Q = CA\sqrt{2gh}$ = discharge coefficient x pipe area x $\sqrt{2 \times g \times \text{head}}$. The head is measured from the center of the headwall opening to the inlet water surface, and a discharge coefficient of $C = 0.6$ should be used.

(b) *Head Losses.*—The energy head loss in a concrete transition will depend primarily on the difference between the velocity heads (Δh_v) at the cutoff end of the transition (usually taken to be the canal velocity head) and at the normal to centerline section of the closed conduit at the headwall. Friction losses for short transitions associated with capacities up to 100 cfs will be small and are usually omitted. Coefficients used with Δh_v which are considered adequate for determining energy losses in broken-back transitions are $K_1 = 0.4$ for the inlet and $K_2 = 0.7$ for the outlet or inlet loss = 0.4 Δh_v, and outlet loss = 0.7 Δh_v. Dimensions for broken-back transitions are usually such that additional transitioning to the canal section must be made with an earth transition where the canal is earth and a lined transition where the canal is lined. However, energy losses attributed to these transitions are small, and it is usually considered adequate in the hydraulic design to use only the concrete transition losses with the assumption that the velocity at the transition cutoff is the same as the velocity in the canal.

Coefficients of Δh_v considered adequate for determining energy losses for earth transitions connecting a canal section to a pipe are $K_1 = 0.5$ for the inlet and $K_2 = 1.0$ for the outlet.

[2] Numbers in brackets refer to items in the bibliography, see sec 7-16.





DIMENSIONS e AND t_w

DEPTH (d) OF WATER IN CANAL	e	t_w
0' to 3'	24"	6"
3' to 6'	2'-6"	8"
Greater than 6'	3'-0"	8"

NOTES

The minimum freeboard at the transition cutoff of structures in earth lined or unlined canals should be 6 inches for depths of water at the cutoff of 0 to 1.25 feet, 9 inches for depths 1.26 to 2.00 feet, and 12 inches for depths 2.00 to 5.00 feet

The freeboard at the transition cutoff of structures in hard surface lined canals should be the same as the freeboard of the lining

The freeboard at transition headwalls should be equal to or greater than the freeboard at the cutoff for 24 inch pipe and smaller and the freeboard should increase as the size of the structure increases.

B = .303D

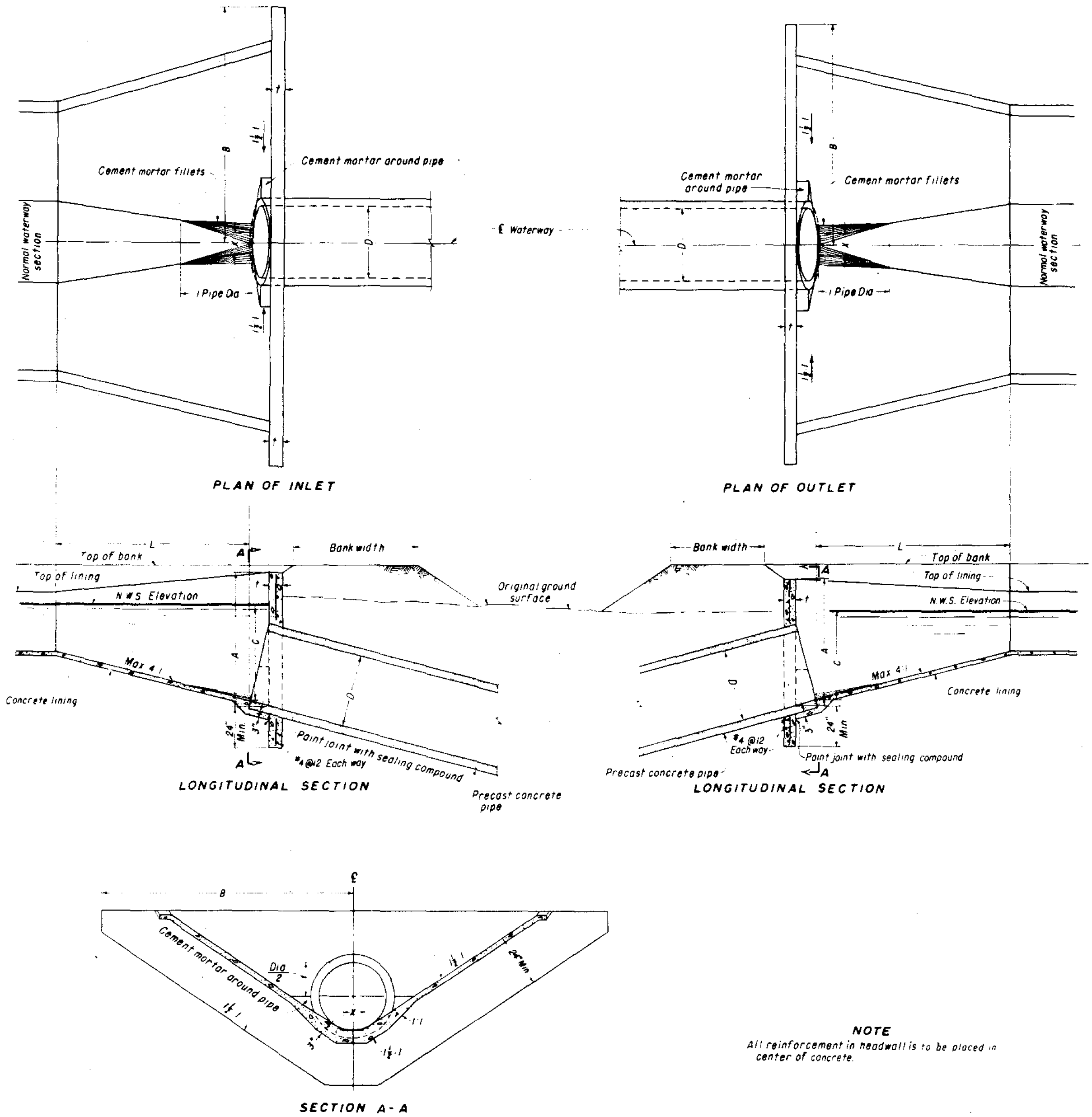
y = Depth (d) at cutoff plus freeboard at cutoff

The elevation of the invert at the headwall is determined from the required submergence at the headwall opening.

DIMENSION C

RELATIONSHIP OF PIPE DIAMETER (D) TO DEPTH (d) IN CANAL	C WITH WATER SURFACE ANGLE OF 22 ½°	C WITH WATER SURFACE ANGLE OF 25°	C WITH WATER SURFACE ANGLE OF 27 ½°
D = d	.5D	.8D	1.1D
D = 1.25d	1.1D	1.4D	1.7D
D = 1.5d	1.5D	1.8D	2.1D
D = 2d	2.0D	2.3D	2.6D

Figure 7-2. **Concrete transitions–type 1. 103-D-1288**



NOTE
All reinforcement in headwall is to be placed in center of concrete.

				INLET HEADWALL					OUTLET HEADWALL				
DIA.	L	t	X	C	A	B	CONCRETE (CU. YD.)	REINF. STEEL (LB.)	C	A	B	CONCRETE (CU. YD.)	REINF. STEEL (LB.)
12"	5'-0"	6"	4"	2.00'	3'-7"	7'-6"	1.0	70	1.30'	2'-9"	6'-4"	.7	50
15"	5'-0"	5"	5"	2.25'	3'-10"	8'-0"	1.1	75	1.60'	3'-2"	6'-11"	.9	60
18"	5'-0"	6"	6"	2.50'	4'-1"	8'-5"	1.2	85	2.00'	3'-6"	7'-6"	1.0	70
21"	5'-3"	6"	7"	2.75'	4'-4"	8'-10"	1.2	90	2.30'	3'-10"	8'-0"	1.1	80
24"	6'-0"	6"	7"	3.00'	4'-8"	9'-3"	1.3	95	2.70'	4'-3"	8'-7"	1.2	85
27"	6'-9"	7"	8"	3.25'	4'-11"	9'-9"	1.7	105	3.00'	4'-7"	9'-2"	1.5	95
30"	7'-6"	7"	9"	3.50'	5'-2"	10'-1"	1.8	110	3.30'	4'-11"	9'-9"	1.7	105
36"	9'-0"	7"	11"	4.00'	5'-8"	10'-11"	2.0	125	4.00'	5'-8"	10'-11"	2.0	125

The tabulated dimensions are for a full pipe velocity of 5 fps

Figure 7-3. **Concrete transitions–type 5. 103-D-1289**

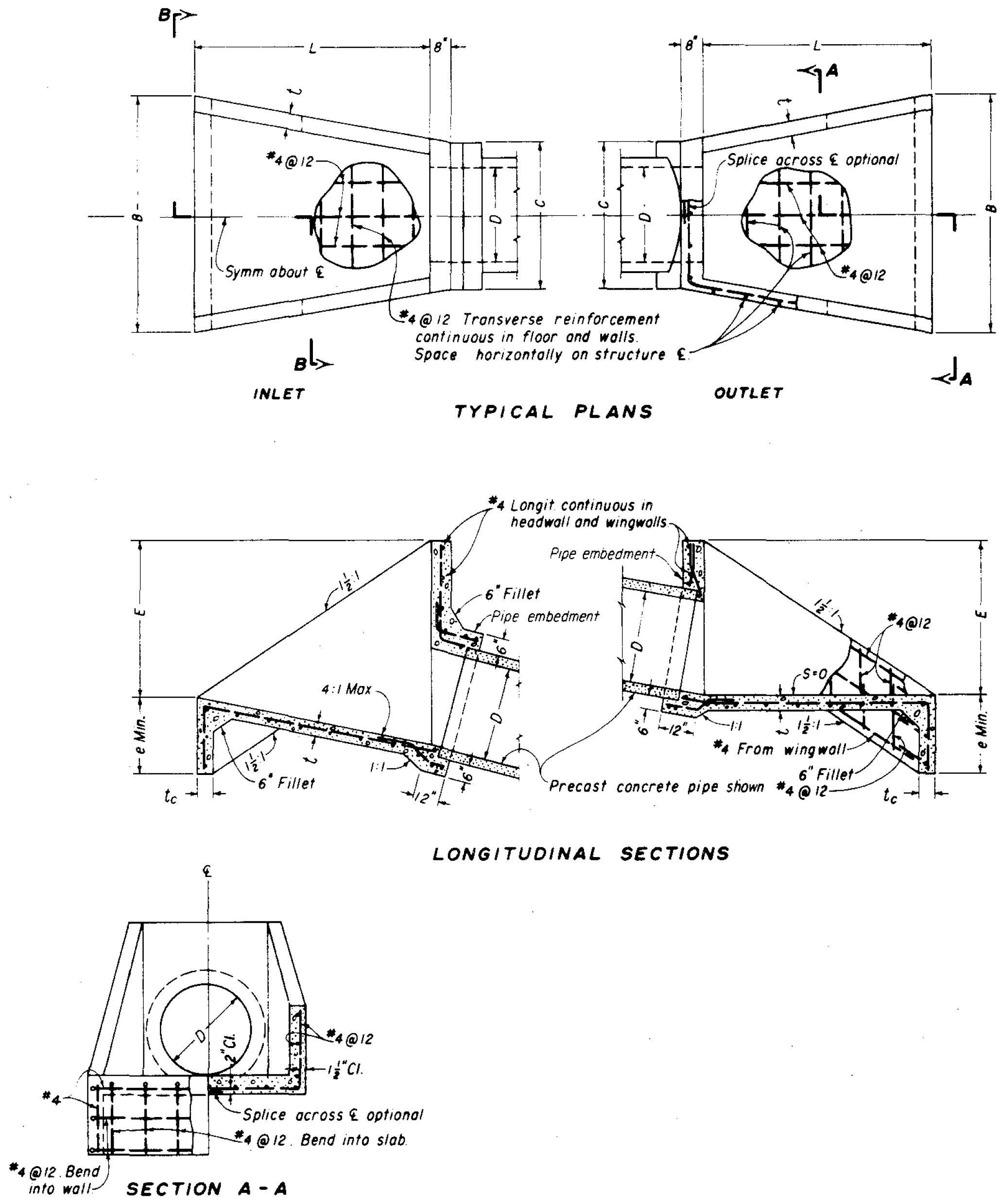
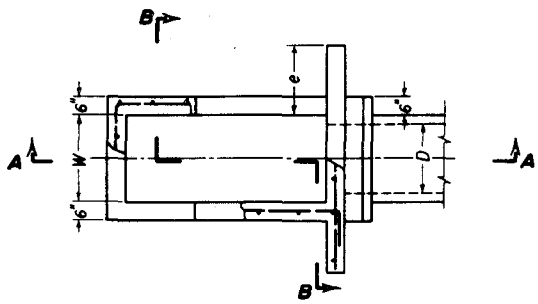


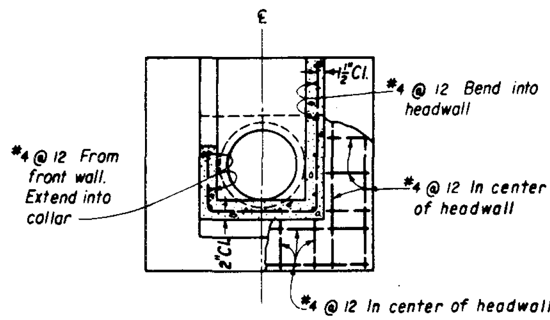
TABLE OF DIMENSIONS AND EST. QUANTITIES

D	E	e	L	B	C	t	t_c	CONC. (CU. YDS.)	REINF. (LBS.)
24"	4'-0"	24"	6'-0"	5'-6"	3'-6"	5"	6"	1.7	140
27"	4'-6"	24"	6'-9"	5'-6"	3'-9"	5"	6"	2.0	160
30"	4'-6"	24"	6'-9"	6'-6"	4'-2"	5"	6"	2.2	180
33"	5'-0"	2'-6"	7'-6"	7'-6"	4'-4"	5"	8"	2.4	200
36"	5'-0"	2'-6"	7'-6"	7'-6"	4'-8"	5"	8"	2.7	220
39"	5'-6"	2'-6"	8'-3"	9'-0"	5'-0"	6"	8"	3.5	280
42"	5'-6"	2'-6"	8'-3"	9'-0"	5'-3"	6"	8"	3.6	290
45"	6'-0"	2'-6"	9'-0"	10'-6"	5'-6"	7"	8"	4.7	370
48"	6'-0"	2'-6"	9'-0"	10'-6"	6'-0"	7"	8"	4.8	380

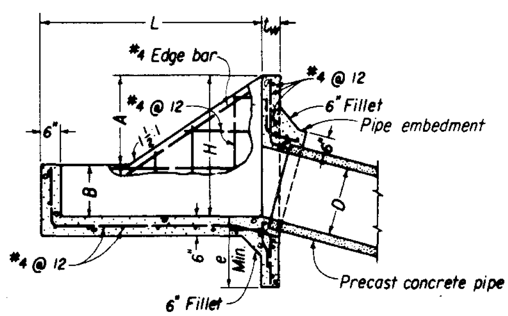
Figure 7-4. **Concrete transitions–type 2. 103-D-1290**



PLAN



SECTION B-B

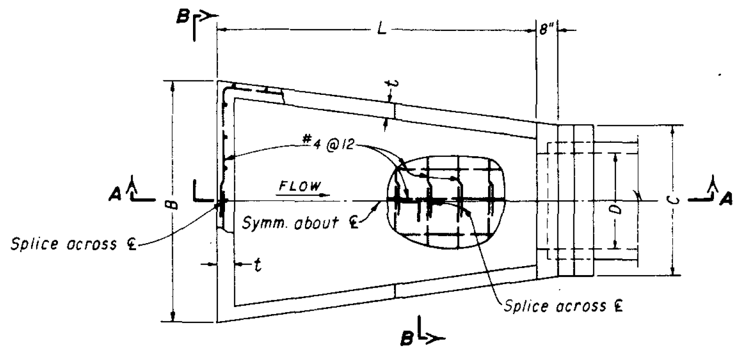


SECTION A-A

STR. No.	MAX. Q	DIMENSIONS								EST. QUANTITIES	
		D	L	W	H	A	B	t_w	e	CONCRETE CUBIC YDS.	REINF. STEEL LBS.
24-1	16	24"	6'-0"	2'-6"	4'-0"	2'-6"	18"	6"	24"	1.8	140
24-2	21	24"	6'-0"	2'-6"	4'-6"	2'-6"	24"	6"	24"	2.0	160
24-3	26	24"	6'-0"	2'-6"	5'-0"	2'-6"	2'-6"	6"	24"	2.2	180
24-4	31	24"	6'-0"	2'-6"	5'-8"	2'-6"	3'-2"	6"	24"	2.4	200
27-1	35	27"	6'-9"	2'-9"	5'-6"	2'-6"	3'-0"	6"	24"	2.6	210
27-2	40	27"	6'-9"	2'-9"	6'-0"	2'-6"	3'-6"	6"	24"	2.8	220
30-1	45	30"	7'-6"	3'-3"	6'-0"	3'-0"	3'-0"	6"	24"	3.0	240
30-2	50	30"	7-6"	3'-3"	6'-6"	3'-0"	3'-6"	6"	24"	3.2	260
33-1	55	33"	9-0"	3'-9"	6'-0"	3'-0"	3'-0"	8"	2'-6"	4.3	290
33-2	60	33"	9'-0"	3'-9"	6'-6"	3'-0"	3'-6"	8"	2'-6"	4.6	320
36-1	70	36"	9'-0"	3'-9"	7'-0"	3'-0"	4'-0"	8"	2'-6"	5.0	340

Tabulated dimensions and maximum Q's, provide freeboard at the headwall, with full-pipe velocities ranging up to 10 fps.

Figure 7-5. Concrete inlet transitions–type 3. 103-D-1291



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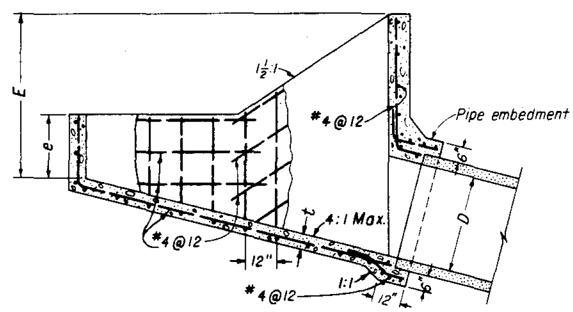
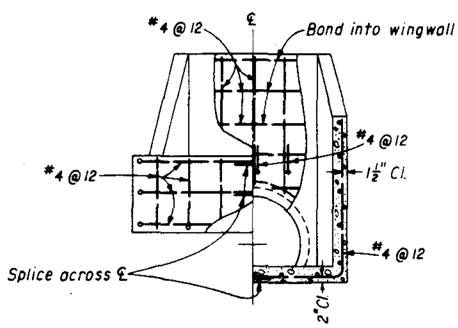


SECTION A-A



SECTION B-B

TABLE OF DIMENSIONS AND QUANTITIES

DIA.	E	e	L	B	C	t	CONC. (CU.YDS.)	REINF. (LBS.)
24"	4'-0"	24"	8'-0"	5'-0"	3'-6"	5"	1.7	120
30"	4'-6"	24"	9'-0"	6'-6"	4'-2"	5"	2.2	150
36"	5'-0"	24"	10'-0"	7'-6"	4'-8"	5"	2.7	190
42"	5'-6"	2'-6"	11'-0"	9'-0"	5'-3"	6"	4.0	280
48"	6'-0"	2'-6"	12'-0"	10'-6"	6'-0"	7"	4.8	340
54"	6'-6"	2'-6"	13'-0"	12'-0"	6'-7"	7"	5.6	390
60"	7'-0"	2'-6"	14'-0"	13'-6"	7'-2"	7"	6.5	460

Tabulated dimensions provide for control at the headwall with a full pipe velocity of 12 fps

Figure 7-6. **Concrete inlet transitions–type 4. 103-D-1292**

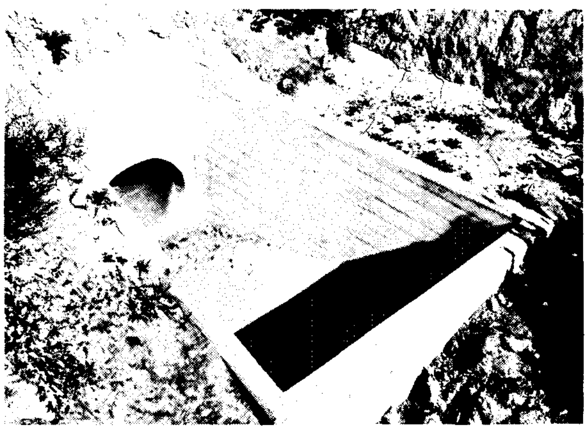


Figure 7-7. **Type 4 inlet concrete transition. P-328-701-9300**

(c) *Water Surface Angle.*—To obtain the most desirable hydraulic conditions, the angle between the water surface and the transition centerline should not exceed 27-1/2° for inlet transitions and 22-1/2° for outlet transitions. For some structure designs it may be economical to use an angle of 25° to allow the same concrete transition to be used for both inlet and outlet. For this angle the loss coefficients remain 0.5 for the inlet and 1.0 for the outlet.

(d) *Channel Erosion.*—To prevent undue channel erosion downstream from a structure outlet, the following criteria for pipe velocity should be observed. If the pipe velocity is equal to or less than 3.5 feet per second, an earth outlet transition is usually sufficient. If the pipe velocity is greater than 3.5 feet per second a concrete outlet transition or other outlet structure is required. If the pipe velocity is greater than 10 feet per second a baffled outlet or a stilling pool should be used.

7-4. ***Cutoffs.***—Cutoffs are provided to reduce percolation around transitions and to add stability and structural strength to transitions. Cutoffs are required at the ends of transitions in concrete-lined canals as well as in other lined or earth canals.

Cutoff walls should, in general, be a minimum of 24 inches deep for water depths up to 3 feet at the cutoff; 2 feet 6 inches deep for water depths of 3 to 6 feet; and 3 feet for water depths greater than 6 feet. For some small structures, 18-inch cutoffs may be satisfactory. The minimum concrete thickness should be 6 inches for 18- and 24-inch cutoffs and 8 inches for cutoffs deeper than 24 inches. Excavation for the structure may disclose soils that are unusually susceptible to piping, in which case the cutoff should be extended vertically or horizontally, or both, beyond these minimums to provide adequate protection against percolation. Nonreinforced concrete may be used for the extension.

7-5. ***Standardization.***—Concrete transitions may be standardized as a means of reducing cost by designing them to fit a range of conditions thereby rendering them applicable for a number of transition installations. If concrete transitions are standardized for inline canal structures it will probably be necessary to supplement the concrete transitions with earth or concrete lining transitions to complete the transitioning to the canal section. Transition losses for these supplemental transitions are usually neglected.

7-6. ***Type 1 Transitions (Broken-back).***—Figure 7-2 shows a typical type 1 transition. The type 1 transition is generally used with inline structures because of its applicability to a well-defined channel cross section.

A transition length L equal to three times the pipe diameter has given satisfactory performance in providing the necessary distance for smoothly changing the water velocity.

Dimension B is chosen so that the 1-1/2 to 1 sloping walls are approximately tangent to the opening at the headwall, and may be determined using the relationship, B = 0.303 times the pipe diameter. The computed value is rounded to the nearest greater inch.

The base width C at the cutoff walls is dependent on the design refinement of the water surface angle. If y − 6 inches is assumed to be approximately the same as the depth d in the canal at the cutoff, an acceptable C value can be determined by using the following water surface angles, equations, and relationships for pipe diameter D to depth d:

For a water surface angle of 22-1/2°:

$C = 0.5D$ when $D = d$,
$C = 1.1D$ when $D = 1.25d$,
$C = 1.5D$ when $D = 1.5d$, and
$C = 2D$ when $D = 2d$

For a water surface angle of 25°:

$C = 0.8D$ when $D = d$,
$C = 1.4D$ when $D = 1.25d$,
$C = 1.8D$ when $D = 1.5d$, and
$C = 2.3D$ when $D = 2d$

For a water surface angle of $27\text{-}1/2^{\circ}$:

$C = 1.1D$ when $D = d$,
$C = 1.7D$ when $D = 1.25d$,
$C = 2.1D$ when $D = 1.5d$, and
$C = 2.6D$ when $D = 2d$

Additional transitioning to the canal base width, if required, can be accomplished with an earth or concrete-lined transition.

Dimension y should not be less than the sum of the water depth at the cutoff and design freeboard at the cutoff. To avoid unnecessary erosion in an earth canal, it is desirable to set the invert of the transition cutoff at the canal invert. Freeboard at the transition cutoff adjacent to concrete canal lining or other hard surface or buried-membrane canal lining is usually the same as that of the lining. For capacities up to 50 cfs this freeboard will usually be 6 inches, and for capacities between 50 and 100 cfs the freeboard will generally range between 6 and 9 inches. In unlined and earth-lined canals the minimum freeboard at broken-back transition cutoffs should be as follows:

Water depth at cutoff, feet	Minimum freeboard, inches
0 to 1.25	6
1.26 to 2.00	9
2.01 to 5.00	12

The value for p is the difference in elevations of the inverts at the transition cutoff and at the headwall opening. The invert at the headwall opening is established by the required submergence of the top of the opening as previously discussed, and the invert at the transition cutoff is assumed to be the same as the canal invert. The p value should not exceed $\frac{3}{4}D$ for an inlet transition or $\frac{1}{2}D$ for an outlet transition. These dimensions provide maximum floor slopes of 4 to 1 for inlet transitions and 6 to 1 for outlet transitions. If additional transitioning to the canal invert is required, it should be accomplished in the adjacent earth transitions, or with concrete lining for a concrete-lined canal.

Dimension "a" is dependent on the design headwall freeboard and the invert of the opening established by required submergence as previously discussed. The freeboard at the broken-back transition headwall should be as great as or greater than the freeboard shown in the preceding tabulation for freeboard at the cutoff. Headwall freeboard for transitions connected to 24-inch-diameter pipe and smaller may be the same as the freeboard at the cutoff, therefore the tops of broken-back transition walls are level for this pipe diameter range. For larger diameters, the transition headwall freeboard should increase as the size of the structure increases; frequently, freeboard at the headwall will be twice that at the cutoff.

Cutoff dimensions have previously been discussed in section 7-4, and required pipe embedment at the headwall is discussed in chapter VIII.

7-7. ***Type 2 Transition.***—Figure 7-4 shows a typical type 2 transition. Dimensions in the table are for pipe diameters sized for a full-pipe flow velocity of 10 feet per second, and a free-flow pipe with hydraulic control at the inlet transition headwall. A free-flow pipe is common for cross-drainage culvert structures where the water surface at the outlet is usually considerably below the invert of the opening at the inlet headwall. A maximum full-pipe flow velocity of 10 feet per second is permitted for cross-drainage culvert structures having concrete outlet transitions.

To prevent degradation at the inlet, the invert at the transition cutoff is located at or near existing ground surface. Sloping the transition floor lowers the headwall opening, and because the hydraulic control for the design flow is at the inlet headwall, the water surface required to discharge the flow is also lowered.

Inlet sidewalls are flared for three reasons: (1) to produce a more hydraulically efficient entrance condition for the opening (orifice) at the headwall, (2) to provide a width at the cutoff sufficient to insure that the hydraulic control of the water surface is at the pipe entrance, and (3) to provide a greater width at the cutoff which reduces the likelihood of erosion by reducing the depth and velocity for flows less than design flow. Flaring the outlet sidewalls also allows the water to be released at the cutoff with less likelihood of erosion for partial flows.

The tabulated dimensions (fig. 7-4) provide for freeboard at the inlet headwall which increases as the size of the structure increases. If submergence of the top of the headwall for design flow is not objectionable, the listed transition dimensions may also be used for the design flow for a full-pipe velocity of 12 feet per second. This velocity is allowed if a baffled outlet is used.

To provide adequate freeboard for the canal, the inlet water surface for design flow should be at least 2 feet below the top of the canal bank. The orifice equation, [1] $Q = CA\sqrt{2gh}$ may be used to calculate the inlet water surface required to discharge the design flow. For a type 2 inlet transition, a discharge coefficient, $C = 0.6$ may be used. The head, h, measured from the centerline of the opening to the water surface for free flow may be conveniently determined by rearranging the orifice equation and making appropriate substitutions:

$$h = 0.0433V^2$$

where V is the design velocity for the pipe.

7-8. ***Type 3 Transitions.***—Figure 7-5 shows a typical type 3 transition. Dimensions provided in the table are for capacities from 16 to 70 cfs and pipe diameters of 24 through 36 inches. Full-pipe velocities range from about 5 feet per second for 24-inch-diameter pipe to about 10 feet per second for all pipe diameters listed. The dimensions provide control at the inlet headwall and also freeboard at the headwall for the design capacity and free-flow pipe. A maximum full-pipe flow velocity of 10 feet per second is permitted for cross-drainage culverts having concrete outlet transitions.

To prevent degradation at the inlet, the top of the inlet wall is placed at or near existing ground surface. Lowering the transition floor by an amount equal to B lowers the headwall opening and, because the hydraulic control is at the inlet headwall, the water surface required to discharge the design flow is also lowered.

Structures numbered 24-4, 27-2, 30-2, 33-2, and 36-1 may also be used for capacities greater than those tabulated with resulting full-pipe velocities up to 12 feet per second, provided there is a baffled outlet or a stilling pool for these higher capacities and provided freeboard at the headwall is not required.

To provide adequate canal bank freeboard, the inlet water surface for design flow should be at least 2 feet below the top of the canal bank. The orifice equation [1], $Q = CA\sqrt{2gh}$, may be used to calculate the inlet water surface required to discharge the design flow. For type 3 inlet transitions, a discharge coefficient, $C = 0.6$ may be used. The head, h, measured from the centerline of the opening to the water surface for free flow may be conveniently determined by rearranging the orifice equation and making appropriate substitutions:

$$h = 0.0433V^2$$

where V is the design velocity for the pipe.

7-9. ***Type 4 Transitions.***—Figure 7-6 shows a typical type 4 transition. Dimensions in the table are for pipe diameters sized for a full-pipe flow velocity of 12 feet per second with a free-flow pipe inlet. The dimensions provide control at the inlet headwalls for design capacity and free-flow pipe. A maximum full-pipe flow velocity of 12 feet per second is permitted for cross-drainage culvert structures having baffled outlets or stilling pools.

To prevent degradation at the inlet, the top of the inlet wall is placed at or near existing ground surface. Dropping the transition floor by an amount equal to e, and sloping the transition floor lowers the headwall opening. Because of this and as the hydraulic control is at the inlet, the water surface required to discharge the design flow is also lowered.

Inlet sidewalls are flared to provide a width at the cutoff sufficient to insure that the hydraulic control of the water surface is at the headwall and to provide a greater width at the cutoff which reduces the likelihood of erosion by reducing the depth and velocity for flows less than design flow.

To provide adequate canal bank freeboard, the inlet water surface for design flow should be at least 2 feet below the top of the canal bank. The orifice equation [1], $Q = CA\sqrt{2gh}$, may be used to calculate the inlet water surface required to discharge the design flow. For a type 4 inlet transition, a discharge coefficient, $C = 0.6$ may be used. The head, h, measured from the centerline of the opening to the water surface for free flow may be conveniently determined by rearranging the orifice equation and making appropriate substitutions:

$$h = 0.0433V^2$$

where V is the design velocity of the pipe.

7-10. ***Type 5 Transitions.***—Figure 7-3 shows a typical type 5 transition. These transitions are simply an extension of the concrete canal lining which matches the normal concrete-lined section at one end and has a headwall on the pipe end. These transitions may be used where minimum head loss is not a factor. Figure 7-3 has a table of dimensions for pipes up to 36 inches in diameter. Because of headwall stability considerations, the maximum pipe diameter used with type 5 transitions is 36 inches.

The table of dimensions provide for the following:

1. Full-pipe velocity of 5 feet per second.
2. Transition length equal to 3 pipe diameters or 5 feet minimum.
3. Maximum invert slope of 4 to 1.
4. Inlet pipe submergence of at least 1.5 pipe velocity heads when full-pipe velocity equals 5 feet per second.
5. Pipe submergence at outlet sufficient to cause pipe to flow full.
6. Inlet and outlet freeboard varying from the lining freeboard to about 1.5 feet at the headwall.

7-11. ***Earth Transitions.***—Earth transitions may be used for transitioning from a canal section to a canal structure where structure velocities do not exceed 3.5 feet per second. Lengths of earth transitions are usually related to the size of the structure. For pipe structures, inlet and outlet earth transition lengths are both usually equal to 3 pipe diameters or a minimum of 5 feet. For other structures, earth transition lengths are usually 5 feet for relatively small capacity structures and 10 feet for other structures. Invert slopes should not be steeper than 4 to 1 for both inlet and outlet transitions.

Lengths used for earth transitions in conjunction with concrete transitions should be 10 feet long or as otherwise required so that invert slopes are not steeper than the maximum allowable for the type 1 concrete transitions, 4 to 1 for inlets and 6 to 1 for outlets.

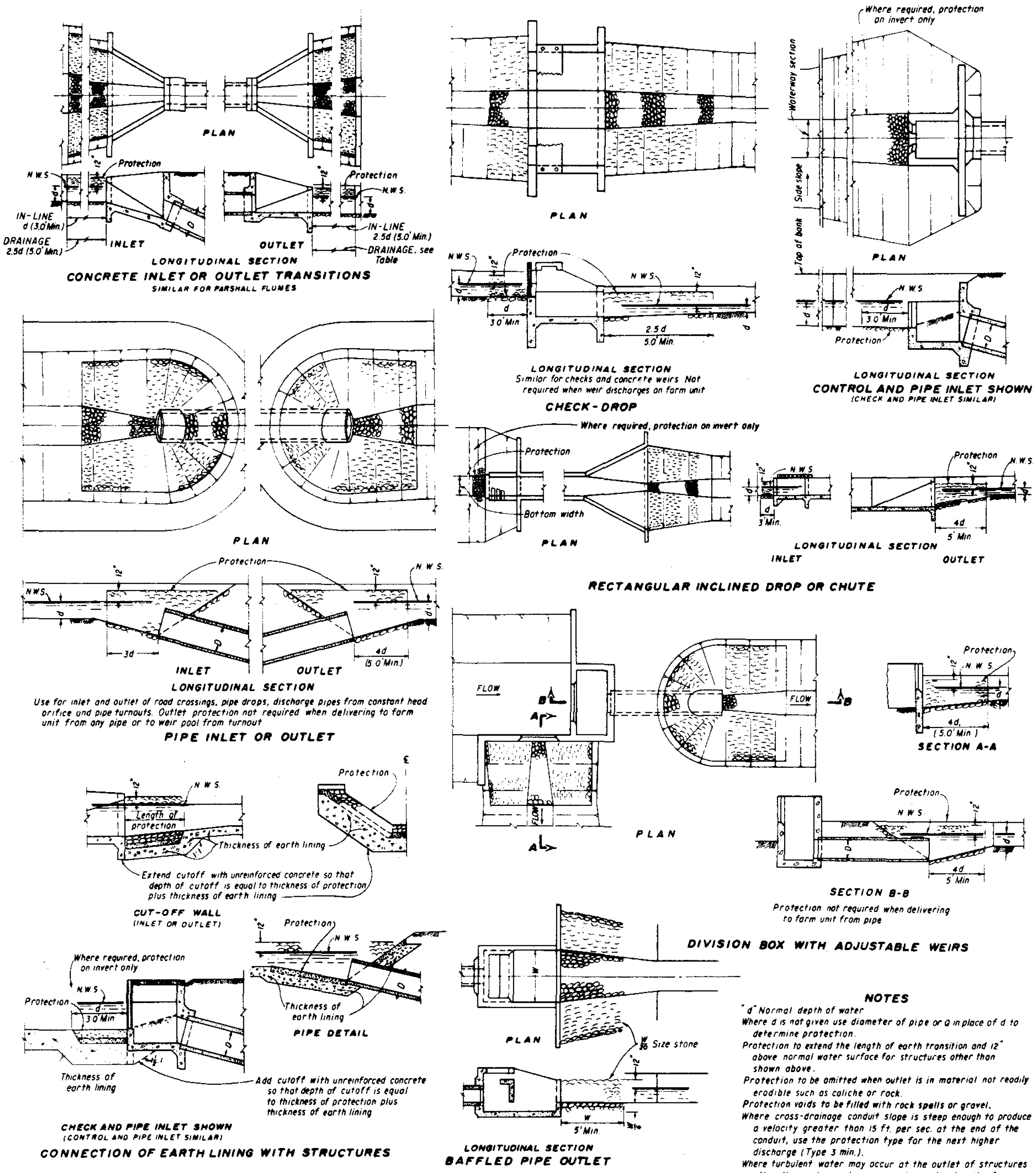
B. EROSION PROTECTION

7-12. ***Purpose and Description.***—Riprap and gravel protection (fig. 7-8) is often used adjacent to structures and at other locations in earth-surfaced canals where erosion may occur. Local conditions must be considered in determining the type and the amount of protection to be provided. These conditions include the cost of riprap; cost of gravel; danger to structures and crops or to human life should scour occur; rodent damage; type of soil; and velocity of water. The following protection requirements should be used as a guide only. The types shown represent minimum thicknesses and sizes of material to be used, and adjustments should be made to meet the local conditions mentioned above.

Type 1—6-inch coarse gravel
Type 2—12-inch coarse gravel
Type 3—12-inch riprap on 6-inch sand and gravel bedding
Type 4—18-inch riprap on 6-inch sand and gravel bedding

Except for cross-drainage structures, type 3 minimum protection should be used where velocities exceed 5 feet per second, regardless of water depth.

7-13. ***Inverted Siphons.***—The following protection is considered minimum for inverted siphons.



NOTES

"d" Normal depth of water
Where d is not given use diameter of pipe or Q in place of d to determine protection.
Protection to extend the length of earth transition and 12" above normal water surface for structures other than shown above.
Protection to be omitted when outlet is in material not readily erodible such as caliche or rock.
Protection voids to be filled with rock spalls or gravel.
Where cross-drainage conduit slope is steep enough to produce a velocity greater than 15 ft. per sec. at the end of the conduit, use the protection type for the next higher discharge (Type 3 min.).
Where turbulent water may occur at the outlet of structures other than siphon and cross-drainage, the length of protection should be increased to 4 depths.
Except for cross-drainage structures, type 3 minimum protection should be used where velocity is greater than 5 fps regardless of depth.

MINIMUM REQUIRED PROTECTION

d	SIPHONS		CHECKS, PARSHALL FLUMES, INCLINED DROPS, CHUTES, CHECK AND PIPE INLET, CONTROL AND PIPE INLET, ROAD CROSSINGS, PIPE DROPS, AND TURNOUTS, ETC.		CROSS DRAINAGE STRUCTURES				PROTECTION	
	INLET	OUTLET	INLET	OUTLET	Q (c.f.s.)	INLET	OUTLET	OUTLET LENGTH	Type 1	6" Coarse gravel
0 to 2.00'	None	None	None	Type 2	0 to 30	None	Type 2	8'	Type 2	12" Coarse gravel
2.01' to 3.50'	None	Type 1	None	Type 2	31 to 90	None	Type 2	12'	Type 3	12" Riprap on 6" sand and gravel bedding
3.51' to 7.00'	Type 1	Type 2	Type 1	Type 3	91 to 240	Type 1	Type 3	16'	Type 4	18" Riprap on 6" sand and gravel bedding

Figure 7-8. Erosion protection. 103-D-1293

Water depth, feet	Type of protection		Length of inlet protection	Length of outlet protection
	Inlet	Outlet		
0 to 2.00	None	None	--	--
2.01 to 3.50	None	Type 1	--	2.5 depths (5 ft. min.)
3.51 to 7.00	Type 1	Type 2	1 depth (3 ft. min.)	2.5 depths (5 ft. min.)

7-14. ***Cross-drainage Structures.***—The following protection is considered minimum for cross-drainage structures with concrete transitions.

Q, cfs	Type of protection		Outlet length, feet
	Inlet	Outlet	
0 to 30	None	Type 2	8
31 to 90	None	Type 2	12
91 to 240	Type 1	Type 3	16

Where the velocity in the conduit is greater than 15 feet per second at the outlet, use the protection type for the next higher discharge (type 3 minimum). Where baffled outlets are provided at the outlet of a structure the protection should be a thickness of $\frac{W}{6}$ with the minimum diameter of rock equal to $\frac{W}{20}$ and extending a distance W (5 feet minimum) beyond the baffled outlet. W is the inside width of the baffled outlet box.

7-15. ***Other structures.***—The following protection is considered minimum for Parshall flumes, checks, check-drops, inclined drops, chutes, turnouts, road crossings and pipe drops with the hydraulic control section on concrete, that is, where critical depth does not occur beyond the concrete structure. Where critical depth may occur beyond the concrete, the next higher type of protection should be used at the inlet.

Water depth, feet	Type of protection	
	Inlet	Outlet
0 to 2.00	None	Type 2
2.01 to 3.50	None	Type 2
3.51 to 7.00	Type 1	Type 3

Length of protection for outlets should normally be 2.5 depths (5.0 feet minimum), but where turbulent water may occur at the outlet, the length of protection should be increased to 4 depths. Gates or stoplogs near the outlet increase turbulence.

The rock for riprap and gravel protection should be hard, dense, durable, and should be reasonably well graded. The size range of rock used for 18-inch riprap should have a maximum size of 1/8 cubic yard and a minimum size of 1/10 cubic foot. The size range used for 12-inch riprap should have a maximum size of 1 cubic foot and a minimum size of 1-1/2 inches. The size range used in coarse gravel protection should have a maximum size of 1/8 cubic foot and a minimum size of 3/16 inches.

The 6-inch sand and gravel bedding for riprap should be a continuous layer of sand and gravel or sand and crushed rock, reasonably well graded to a maximum of 1-1/2 inches in size.

C. BIBLIOGRAPHY

7-16. ***Bibliography.***

[1] King, H. W. and Brater, E. F., "Handbook of Hydraulics," Fifth Edition, McGraw-Hill Book Co., New York, 1963.

Pipe and Pipe Appurtenances

R. B. YOUNG[1]

A. GENERAL

8-1. *Pipe and Pipe Appurtenances.*—In this publication, pipe is considered to be any circular conduit used for conveying water and may be manufactured from many different materials such as reinforced concrete, corrugated metal, asbestos cement, reinforced plastic mortar, and welded steel.

Appurtenances to a pipe are those structural elements necessary to provide efficient hydraulics, structural integrity, effective watertightness, adequate percolation path, easy access for inspection and maintenance, and effective safety.

A minimum pipe diameter of 12 inches should be used for structures in an open irrigation system to reduce the possibility of clogging by sediment or debris. It may be desirable to use a larger minimum diameter if clogging by indigenous weeds is a major consideration. A pipe diameter range of 12 through 72 inches for this publication was established by the above 12-inch minimum diameter and by what was considered to be a large enough pipe for cross-drainage structures that will be encountered in this publication.

8-2. *Hydraulic Head Losses.*—The main energy losses (hydraulic head losses) associated with water flowing through a pipe include friction losses, entrance and exit losses, and bend losses.

Friction losses can be determined by using Manning's formula [1][2]:

$$V = \frac{1.486\, r^{2/3}\, s^{1/2}}{n}$$

r = hydraulic radius (water area divided by wetted perimeter)

For a pipe flowing full

$$r = \frac{A_p}{wp}$$

$$= \frac{\pi R^2}{2\pi R} = \frac{R}{2} = \frac{D}{4}$$

then

$$V = \frac{0.59\, D^{2/3} s^{1/2}}{n} \quad \text{or} \quad s = \left(\frac{Vn}{0.59 D^{2/3}}\right)^2$$

where

- V = velocity of water in feet per second,
- r = hydraulic radius
- s = slope of energy gradient in feet per foot,
- A_p = cross-sectional area of full pipe in square feet,
- wp = wetted perimeter, in feet
- R = radius of pipe in feet,
- D = diameter of pipe in feet, and
- n = coefficient of roughness (see sec. 1-16). The assumed roughness

(continued on page 359)

[1]Civil Engineer, Hydraulic Structures Branch, Bureau of Reclamation.

[2]Numbers in brackets refer to items in the bibliography, see sec. 8-19.

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Table 8-1.–*Friction losses for circular conduits.* 103-D-1316-1

Q	12" A = 0.785			15" A = 1.227			18" A = 1.767			21" A = 2.405			Q
c.f.s.	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c.f.s.
1.50	1.91	.06	1.77	1.22	.02	.53							1.50
1.60	2.04	.06	2.02	1.30	.03	.60							1.60
1.70	2.16	.07	2.26	1.38	.03	.68							1.70
1.80	2.29	.08	2.54	1.47	.03	.77							1.80
1.90	2.42	.09	2.84	1.55	.04	.86							1.90
2.00	2.55	.10	3.15	1.63	.04	.95	1.13	.02	.36				2.00
2.10	2.67	.11	3.46	1.71	.05	1.05	1.19	.02	.40				2.10
2.20	2.80	.12	3.80	1.79	.05	1.15	1.25	.02	.44				2.20
2.30	2.93	.13	4.17	1.87	.05	1.26	1.30	.03	.47				2.30
2.40	3.06	.15	4.55	1.96	.06	1.38	1.36	.03	.52				2.40
2.50	3.18	.16	4.91	2.04	.06	1.50	1.41	.03	.56	1.04	.02	.24	2.50
2.60	3.31	.17	5.32	2.12	.07	1.62	1.47	.03	.61	1.08	.02	.26	2.60
2.70	3.44	.18	5.75	2.20	.08	1.74	1.53	.04	.66	1.12	.02	.28	2.70
2.80	3.56	.20	6.15	2.28	.08	1.87	1.58	.04	.70	1.16	.02	.31	2.80
2.90	3.69	.21	6.61	2.36	.09	2.01	1.64	.04	.76	1.21	.02	.33	2.90
3.00	3.82	.23	7.09	2.44	.09	2.14	1.70	.04	.81	1.25	.02	.36	3.00
3.10	3.95	.24	7.58	2.53	.10	2.31	1.75	.05	.86	1.29	.03	.38	3.10
3.20	4.07	.26	8.04	2.61	.11	2.45	1.81	.05	.92	1.33	.03	.40	3.20
3.30	4.20	.27	8.57	2.69	.11	2.61	1.87	.05	.98	1.37	.03	.43	3.30
3.40	4.33	.29	9.11	2.77	.12	2.76	1.92	.06	1.04	1.41	.03	.45	3.40
3.50	4.46	.31	9.66	2.85	.13	2.93	1.98	.06	1.10	1.46	.03	.49	3.50
3.60	4.58	.33	10.19	2.93	.13	3.09	2.04	.06	1.17	1.50	.03	.51	3.60
3.70	4.71	.35	10.78	3.01	.14	3.26	2.09	.07	1.23	1.54	.04	.54	3.70
3.80	4.84	.36	11.38	3.10	.15	3.46	2.15	.07	1.30	1.58	.04	.57	3.80
3.90	4.97	.38	12.00	3.18	.16	3.64	2.21	.08	1.38	1.62	.04	.60	3.90
4.00	5.10	.40	12.63	3.26	.17	3.83	2.26	.08	1.44	1.66	.04	.63	4.00
4.20	5.35	.44	13.90	3.42	.18	4.22	2.38	.09	1.60	1.75	.05	.70	4.20
4.40	5.60	.49	15.23	3.59	.20	4.65	2.49	.10	1.75	1.83	.05	.77	4.40
4.60	5.86	.53	16.68	3.75	.22	5.07	2.60	.11	1.91	1.91	.06	.84	4.60
4.80	6.11	.58	18.14	3.91	.24	5.51	2.72	.11	2.09	2.00	.06	.92	4.80
5.00	6.37	.63	19.71	4.07	.26	5.97	2.83	.12	2.26	2.08	.07	.99	5.00
5.20	6.62	.68	21.29	4.24	.28	6.48	2.94	.13	2.44	2.16	.07	1.07	5.20
5.40	6.88	.73	23.00	4.40	.30	6.98	3.06	.15	2.65	2.24	.08	1.15	5.40
5.60	7.13	.79	24.70	4.56	.32	7.50	3.17	.16	2.84	2.33	.08	1.25	5.60
5.80	7.39	.85	26.53	4.73	.35	8.07	3.28	.17	3.04	2.41	.09	1.33	5.80
6.00	7.64	.91	28.36	4.89	.37	8.62	3.40	.18	3.27	2.49	.10	1.42	6.00
6.20				5.05	.40	9.20	3.51	.19	3.48	2.58	.10	1.53	6.20
6.40				5.22	.42	9.83	3.62	.20	3.70	2.66	.11	1.63	6.40
6.60				5.38	.45	10.44	3.74	.22	3.95	2.74	.12	1.73	6.60
6.80				5.54	.48	11.07	3.85	.23	4.19	2.83	.12	1.84	6.80
7.00				5.70	.51	11.72	3.96	.24	4.43	2.91	.13	1.95	7.00
7.20				5.87	.54	12.43	4.07	.26	4.68	2.99	.14	2.06	7.20
7.40				6.03	.57	13.12	4.19	.27	4.96	3.08	.15	2.18	7.40
7.60				6.19	.60	13.82	4.30	.29	5.23	3.16	.16	2.30	7.60
7.80				6.36	.63	14.59	4.41	.30	5.50	3.24	.16	2.41	7.80
8.00				6.52	.66	15.34	4.53	.32	5.80	3.33	.17	2.55	8.00
8.20				6.68	.69	16.10	4.64	.33	6.09	3.41	.18	2.67	8.20
8.40				6.85	.73	16.93	4.75	.35	6.38	3.49	.19	2.80	8.40
8.60				7.01	.76	17.73	4.87	.37	6.71	3.58	.20	2.95	8.60
8.80				7.17	.80	18.55	4.98	.39	7.01	3.66	.21	3.08	8.80
9.00				7.34	.84	19.44	5.09	.40	7.33	3.74	.22	3.22	9.00
9.20							5.21	.42	7.68	3.82	.23	3.36	9.20
9.40							5.32	.44	8.00	3.91	.24	3.52	9.40
9.60							5.43	.46	8.34	3.99	.25	3.66	9.60
9.80							5.55	.48	8.71	4.07	.26	3.81	9.80
10.00							5.66	.50	9.06	4.16	.27	3.98	10.00
10.20							5.77	.52	9.42	4.24	.28	4.14	10.20
10.40							5.89	.54	9.81	4.32	.29	4.30	10.40
10.60							6.00	.56	10.18	4.41	.30	4.48	10.60
10.80							6.11	.58	10.56	4.49	.31	4.64	10.80
11.00							6.22	.60	10.94	4.57	.33	4.81	11.00
11.20							6.34	.62	11.37	4.66	.34	5.00	11.20
11.40							6.45	.65	11.77	4.74	.35	5.17	11.40
11.60							6.56	.67	12.17	4.82	.36	5.35	11.60
11.80							6.68	.69	12.62	4.91	.37	5.55	11.80
12.00							6.79	.72	13.04	4.99	.39	5.73	12.00
12.20							6.90	.74	13.47	5.07	.40	5.92	12.20
12.40							7.02	.77	13.94	5.16	.41	6.13	12.40
12.60							7.13	.79	14.38	5.24	.43	6.32	12.60
12.80							7.24	.82	14.83	5.32	.44	6.52	12.80
13.00							7.36	.84	15.33	5.40	.45	6.71	13.00
13.20										5.49	.47	6.94	13.20
13.40										5.57	.48	7.14	13.40
13.60										5.65	.50	7.35	13.60
13.80										5.74	.51	7.59	13.80
14.00										5.82	.53	7.80	14.00
14.20										5.90	.54	8.02	14.20
14.40										5.99	.56	8.26	14.40
14.60										6.07	.57	8.49	14.60
14.80										6.15	.59	8.71	14.80
15.00										6.24	.60	8.97	15.00
15.20										6.32	.62	9.20	15.20
15.40										6.40	.64	9.43	15.40
15.60										6.49	.65	9.70	15.60
15.80										6.57	.67	9.94	15.80
16.00										6.65	.69	10.19	16.00
16.20										6.74	.71	10.46	16.20
16.40										6.82	.72	10.71	16.40
16.60										6.90	.74	10.97	16.60
16.80										6.98	.76	11.22	16.80
17.00										7.07	.78	11.51	17.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$.

A = Area of pipe in square feet.

V = Velocity in feet per second.

h_v = Velocity head in feet.

n = Coefficient of roughness.

r = Hydraulic radius in feet.

s = Friction loss in feet per foot.

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS

MANNING'S FORMULA n = .013

12", 15", 18", AND 21", DIAMETERS

Sheet 1 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-2

Q	24" A = 3.142			27" A = 3.976			30" A = 4.909			33" A = 5.940			Q
c.f.s.	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c.f.s.
3.00	.95	.01	.17										3.00
3.20	1.02	.02	.20										3.20
3.40	1.08	.02	.22										3.40
3.60	1.15	.02	.25										3.60
3.80	1.21	.02	.28										3.80
4.00	1.27	.03	.31										4.00
4.20	1.34	.03	.34										4.20
4.40	1.40	.03	.37										4.40
4.60	1.46	.03	.41										4.60
4.80	1.53	.04	.45										4.80
5.00	1.59	.04	.48										5.00
5.20	1.65	.04	.52										5.20
5.40	1.72	.05	.57										5.40
5.60	1.78	.05	.61										5.60
5.80	1.84	.05	.65										5.80
6.00	1.91	.06	.70	1.51	.04	.37							6.00
6.20	1.97	.06	.74	1.56	.04	.40							6.20
6.40	2.03	.06	.79	1.61	.04	.42							6.40
6.60	2.10	.07	.85	1.66	.04	.45							6.60
6.80	2.16	.07	.89	1.71	.05	.48							6.80
7.00	2.22	.08	.95	1.76	.05	.51							7.00
7.20	2.29	.08	1.01	1.81	.05	.53							7.20
7.40	2.35	.09	1.06	1.86	.05	.57							7.40
7.60	2.42	.09	1.12	1.91	.06	.60							7.60
7.80	2.48	.10	1.18	1.96	.06	.63							7.80
8.00	2.54	.10	1.24	2.01	.06	.66	1.63	.04	.38	1.35	.03	.22	8.00
8.20	2.61	.11	1.31	2.06	.07	.69	1.67	.04	.39	1.38	.03	.24	8.20
8.40	2.67	.11	1.37	2.11	.07	.73	1.71	.05	.41	1.41	.03	.25	8.40
8.60	2.74	.12	1.44	2.16	.07	.76	1.75	.05	.43	1.45	.03	.26	8.60
8.80	2.80	.12	1.51	2.21	.08	.80	1.79	.05	.45	1.48	.03	.27	8.80
9.00	2.86	.13	1.57	2.26	.08	.84	1.83	.05	.47	1.52	.04	.29	9.00
9.20	2.92	.13	1.64	2.31	.08	.87	1.87	.05	.50	1.55	.04	.30	9.20
9.40	2.99	.14	1.72	2.36	.09	.91	1.91	.06	.52	1.58	.04	.31	9.40
9.60	3.05	.14	1.79	2.41	.09	.95	1.95	.06	.54	1.62	.04	.33	9.60
9.80	3.11	.15	1.86	2.46	.09	.99	1.99	.06	.56	1.65	.04	.34	9.80
10.00	3.18	.16	1.95	2.51	.10	1.03	2.04	.06	.59	1.68	.04	.35	10.00
11.00	3.50	.19	2.36	2.76	.12	1.25	2.24	.08	.71	1.85	.05	.43	11.00
12.00	3.82	.23	2.81	3.02	.14	1.50	2.44	.09	.85	2.02	.06	.51	12.00
13.00	4.13	.27	3.28	3.27	.17	1.76	2.65	.11	1.00	2.19	.07	.60	13.00
14.00	4.45	.31	3.81	3.52	.19	2.04	2.85	.13	1.16	2.35	.09	.69	14.00
15.00	4.77	.35	4.38	3.77	.22	2.34	3.05	.15	1.33	2.52	.10	.80	15.00
16.00	5.09	.40	4.99	4.02	.25	2.66	3.26	.17	1.52	2.69	.11	.91	16.00
17.00	5.41	.46	5.64	4.27	.28	3.00	3.46	.19	1.71	2.86	.13	1.03	17.00
18.00	5.73	.51	6.33	4.53	.32	3.38	3.66	.21	1.91	3.03	.14	1.15	18.00
19.00	6.04	.57	7.03	4.77	.35	3.75	3.87	.23	2.14	3.19	.16	1.28	19.00
20.00	6.36	.63	7.80	5.03	.39	4.17	4.07	.26	2.37	3.36	.18	1.42	20.00
21.00	6.68	.69	8.60	5.28	.43	4.59	4.28	.28	2.62	3.53	.19	1.57	21.00
22.00	7.00	.76	9.44	5.53	.48	5.04	4.48	.31	2.87	3.70	.21	1.72	22.00
23.00	7.32	.83	10.33	5.78	.52	5.50	4.68	.34	3.13	3.87	.23	1.88	23.00
24.00	7.64	.91	11.25	6.03	.57	5.99	4.89	.37	3.42	4.04	.25	2.05	24.00
25.00	7.95	.98	12.18	6.28	.61	6.50	5.09	.40	3.71	4.21	.28	2.23	25.00
26.00	8.27	1.06	13.18	6.54	.67	7.04	5.29	.44	4.00	4.37	.30	2.40	26.00
27.00	8.59	1.15	14.23	6.79	.72	7.59	5.50	.47	4.33	4.54	.32	2.59	27.00
28.00	8.91	1.23	15.31	7.04	.77	8.16	5.70	.51	4.65	4.71	.34	2.79	28.00
29.00	9.23	1.32	16.42	7.29	.83	8.75	5.90	.56	4.98	4.88	.37	3.00	29.00
30.00	9.55	1.42	17.58	7.54	.88	9.37	6.11	.58	5.34	5.05	.40	3.21	30.00
31.00	9.87	1.51	18.78	7.79	.94	10.00	6.31	.62	5.70	5.22	.42	3.43	31.00
32.00	10.18	1.61	19.98	8.04	1.01	10.65	6.51	.66	6.06	5.38	.45	3.65	32.00
33.00	10.50	1.71	21.26	8.30	1.07	11.35	6.72	.70	6.46	5.55	.48	3.88	33.00
34.00	10.82	1.82	22.57	8.55	1.14	12.04	6.92	.74	6.85	5.72	.51	4.12	34.00
35.00				8.80	1.20	12.76	7.13	.79	7.28	5.89	.54	4.37	35.00
36.00				9.05	1.27	13.49	7.33	.84	7.69	6.06	.57	4.63	36.00
37.00				9.31	1.35	14.28	7.53	.88	8.12	6.22	.60	4.87	37.00
38.00				9.56	1.42	15.06	7.74	.93	8.58	6.39	.63	5.15	38.00
39.00				9.81	1.49	15.86	7.94	.98	9.02	6.56	.67	5.42	39.00
40.00				10.06	1.57	16.68	8.14	1.03	9.48	6.73	.70	5.71	40.00
41.00				10.31	1.65	17.52	8.35	1.08	9.98	6.90	.74	6.00	41.00
42.00							8.55	1.13	10.46	7.07	.78	6.30	42.00
43.00							8.76	1.19	10.99	7.24	.81	6.61	43.00
44.00							8.96	1.24	11.49	7.40	.85	6.90	44.00
45.00							9.17	1.31	12.04	7.58	.89	7.24	45.00
46.00							9.37	1.36	12.57	7.74	.93	7.55	46.00
47.00							9.57	1.42	13.11	7.91	.97	7.89	47.00
48.00							9.78	1.49	13.69	8.08	1.01	8.23	48.00
49.00							9.98	1.55	14.26	8.25	1.06	8.58	49.00
50.00							10.19	1.61	14.87	8.41	1.10	8.92	50.00
51.00							10.39	1.68	15.46	8.59	1.15	9.30	51.00
52.00										8.75	1.19	9.65	52.00
53.00										8.92	1.24	10.03	53.00
54.00										9.09	1.28	10.42	54.00
55.00										9.26	1.33	10.81	55.00
56.00										9.42	1.38	11.19	56.00
57.00										9.60	1.43	11.62	57.00
58.00										9.76	1.48	12.01	58.00
59.00										9.93	1.53	12.43	59.00
60.00										10.10	1.58	12.86	60.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$.

A = Area of pipe in square feet.

V = Velocity in feet per second.

h_v = Velocity head in feet.

n = Coefficient of roughness.

r = Hydraulic radius in feet.

s = Friction loss in feet per foot.

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS

MANNING'S FORMULA n = .013

24", 27", 30", AND 33", DIAMETERS

Sheet 2 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-3

Q	36" A = 7.068			39" A = 8.296			42" A = 9.621			45" A = 11.045			Q
c.f.s.	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c.f.s.
8.00	1.13	.02	.14										8.00
8.20	1.16	.02	.15										8.20
8.40	1.19	.02	.15										8.40
8.60	1.22	.02	.16										8.60
8.80	1.25	.02	.17										8.80
9.00	1.27	.02	.18										9.00
9.20	1.30	.03	.18										9.20
9.40	1.33	.03	.19										9.40
9.60	1.36	.03	.20										9.60
9.80	1.39	.03	.21										9.80
10.00	1.41	.03	.22										10.00
11.00	1.56	.04	.27										11.00
12.00	1.70	.04	.32										12.00
13.00	1.84	.05	.38										13.00
14.00	1.98	.06	.44										14.00
15.00	2.12	.07	.50										15.00
16.00	2.26	.08	.57										16.00
17.00	2.40	.09	.64										17.00
18.00	2.54	.10	.72										18.00
19.00	2.68	.11	.80										19.00
20.00	2.82	.12	.89										20.00
21.00	2.97	.14	.99										21.00
22.00	3.11	.15	1.08										22.00
23.00	3.25	.16	1.18										23.00
24.00	3.39	.18	1.29										24.00
25.00	3.53	.19	1.39										25.00
26.00	3.67	.21	1.51	3.13	.15	.98	2.70	.11	.66	2.35	.09	.46	26.00
27.00	3.82	.23	1.63	3.25	.16	1.06	2.81	.12	.72	2.45	.09	.50	27.00
28.00	3.96	.24	1.76	3.37	.18	1.14	2.91	.13	.77	2.53	.10	.53	28.00
29.00	4.10	.26	1.88	3.49	.19	1.22	3.01	.14	.82	2.63	.11	.57	29.00
30.00	4.24	.28	2.01	3.61	.20	1.31	3.12	.15	.89	2.71	.11	.61	30.00
31.00	4.38	.30	2.15	3.74	.22	1.41	3.22	.16	.94	2.81	.12	.65	31.00
32.00	4.52	.32	2.29	3.86	.23	1.50	3.32	.17	1.00	2.89	.13	.69	32.00
33.00	4.66	.34	2.43	3.98	.25	1.59	3.43	.18	1.07	2.99	.14	.74	33.00
34.00	4.81	.36	2.59	4.10	.26	1.69	3.53	.19	1.13	3.07	.15	.78	34.00
35.00	4.95	.38	2.75	4.22	.28	1.79	3.64	.21	1.21	3.17	.16	.83	35.00
36.00	5.09	.40	2.90	4.34	.29	1.90	3.74	.22	1.27	3.26	.16	.88	36.00
37.00	5.23	.43	3.07	4.46	.31	2.00	3.85	.23	1.35	3.35	.17	.93	37.00
38.00	5.37	.45	3.23	4.58	.33	2.11	3.95	.24	1.42	3.44	.18	.98	38.00
39.00	5.51	.47	3.40	4.70	.34	2.22	4.05	.26	1.49	3.53	.19	1.03	39.00
40.00	5.65	.50	3.58	4.82	.36	2.34	4.15	.27	1.57	3.62	.20	1.09	40.00
41.00	5.80	.52	3.77	4.95	.38	2.47	4.26	.28	1.65	3.71	.21	1.14	41.00
42.00	5.94	.55	3.96	5.06	.40	2.58	4.36	.29	1.73	3.80	.22	1.20	42.00
43.00	6.08	.58	4.15	5.19	.42	2.71	4.47	.31	1.82	3.89	.24	1.26	43.00
44.00	6.22	.60	4.34	5.30	.44	2.83	4.57	.32	1.90	3.98	.25	1.32	44.00
45.00	6.37	.63	4.55	5.43	.46	2.97	4.68	.34	2.00	4.07	.26	1.38	45.00
46.00	6.50	.66	4.74	5.54	.48	3.09	4.78	.35	2.08	4.16	.27	1.44	46.00
47.00	6.65	.69	4.96	5.67	.50	3.24	4.89	.37	2.18	4.26	.28	1.51	47.00
48.00	6.79	.72	5.17	5.79	.52	3.38	4.99	.39	2.27	4.34	.29	1.57	48.00
49.00	6.93	.75	5.39	5.91	.54	3.52	5.09	.40	2.36	4.44	.31	1.64	49.00
50.00	7.07	.78	5.61	6.03	.56	3.67	5.19	.42	2.46	4.52	.32	1.70	50.00
51.00	7.22	.81	5.85	6.15	.59	3.81	5.30	.44	2.56	4.62	.33	1.78	51.00
52.00	7.35	.84	6.06	6.27	.61	3.96	5.40	.45	2.66	4.71	.34	1.85	52.00
53.00	7.50	.87	6.31	6.39	.64	4.12	5.51	.47	2.77	4.80	.36	1.92	53.00
54.00	7.63	.90	6.53	6.51	.66	4.27	5.61	.49	2.87	4.89	.37	1.99	54.00
55.00	7.78	.94	6.79	6.63	.68	4.43	5.72	.51	2.99	4.98	.39	2.06	55.00
56.00	7.92	.97	7.04	6.75	.71	4.59	5.82	.52	3.09	5.07	.40	2.14	56.00
57.00	8.06	1.01	7.29	6.88	.73	4.77	5.92	.55	3.20	5.16	.41	2.22	57.00
58.00	8.20	1.04	7.55	6.99	.76	4.93	6.03	.56	3.32	5.25	.43	2.29	58.00
59.00	8.35	1.08	7.83	7.12	.79	5.11	6.13	.58	3.43	5.34	.44	2.37	59.00
60.00	8.48	1.11	8.07	7.23	.81	5.27	6.23	.60	3.54	5.43	.46	2.45	60.00
62.00	8.77	1.19	8.63	7.47	.87	5.63	6.44	.64	3.79	5.61	.49	2.62	62.00
64.00	9.05	1.27	9.19	7.72	.92	6.01	6.65	.69	4.04	5.79	.52	2.79	64.00
66.00	9.33	1.35	9.77	7.96	.98	6.39	6.86	.73	4.30	5.97	.55	2.97	66.00
68.00	9.62	1.43	10.39	8.20	1.04	6.78	7.06	.77	4.55	6.15	.59	3.15	68.00
70.00	9.90	1.52	11.00	8.44	1.10	7.19	7.27	.82	4.83	6.33	.62	3.34	70.00
72.00	10.19	1.61	11.66	8.68	1.17	7.60	7.48	.87	5.11	6.52	.66	3.54	72.00
74.00	10.47	1.70	12.31	8.92	1.23	8.03	7.69	.92	5.40	6.70	.70	3.74	74.00
76.00	10.75	1.79	12.97	9.16	1.30	8.46	7.90	.97	5.70	6.88	.73	3.94	76.00
78.00				9.40	1.37	8.91	8.10	1.02	5.99	7.06	.77	4.15	78.00
80.00				9.65	1.44	9.40	8.31	1.07	6.31	7.24	.81	4.37	80.00
82.00				9.88	1.51	9.85	8.52	1.12	6.63	7.42	.85	4.5[illegible]	82.00
84.00				10.13	1.59	10.35	8.73	1.18	6.96	7.60	.90	4.81	84.00
86.00							8.94	1.24	7.30	7.78	.94	5.04	86.00
88.00							9.14	1.30	7.63	7.96	.98	5.28	88.00
90.00							9.35	1.36	7.99	8.15	1.03	5.54	90.00
92.00							9.56	1.42	8.35	8.33	1.08	5.78	92.00
94.00							9.77	1.48	8.72	8.51	1.12	6.04	94.00
96.00							9.98	1.54	9.10	8.69	1.17	6.29	96.00
98.00							10.18	1.61	9.47	8.87	1.22	6.56	98.00
100.00							10.39	1.67	9.87	9.05	1.27	6.83	100.00
102.00										9.24	1.33	7.12	102.00
104.00										9.42	1.38	7.40	104.00
106.00										9.60	1.43	7.68	106.00
108.00										9.78	1.49	7.97	108.00
110.00										9.95	1.54	8.25	110.00
112.00										10.14	1.60	8.57	112.00
114.00										10.32	1.66	8.88	114.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$.
A = Area of pipe in square feet.
V = Velocity in feet per second.
h_v = Velocity head in feet.
n = Coefficient of roughness.
r = Hydraulic radius in feet
s = Friction loss in feet per foot.
H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS
MANNING'S FORMULA n = .013
36", 39", 42", AND 45", DIAMETERS

Sheet 3 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-4

Q	48" A = 12.566			51" A = 14.186			54" A = 15.904			57" A = 17.721			Q
c.f.s	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c.f.s.
26.00	2.07	.07	.32	1.83	.05	.23							26.00
28.00	2.23	.08	.38	1.97	.06	.27							28.00
30.00	2.39	.09	.43	2.11	.07	.31							30.00
32.00	2.55	.10	.49	2.26	.08	.36							32.00
34.00	2.70	.11	.55	2.40	.09	.40							34.00
36.00	2.86	.13	.62	2.53	.10	.45							36.00
38.00	3.02	.14	.69	2.68	.11	.50	2.39	.09	.37	2.14	.07	.27	38.00
40.00	3.18	.16	.77	2.81	.12	.55	2.52	.10	.41	2.26	.08	.31	40.00
42.00	3.34	.17	.85	2.96	.14	.61	2.64	.11	.45	2.37	.09	.34	42.00
44.00	3.50	.19	.93	3.10	.15	.67	2.76	.12	.49	2.48	.10	.37	44.00
46.00	3.66	.21	1.02	3.24	.16	.74	2.89	.13	.54	2.60	.10	.41	46.00
48.00	3.82	.23	1.11	3.38	.18	.80	3.02	.14	.59	2.71	.11	.44	48.00
50.00	3.98	.25	1.21	3.52	.19	.87	3.14	.15	.64	2.82	.12	.48	50.00
52.00	4.14	.27	1.31	3.66	.21	.94	3.27	.17	.69	2.93	.13	.52	52.00
54.00	4.29	.29	1.40	3.80	.22	1.01	3.39	.18	.75	3.04	.14	.56	54.00
56.00	4.45	.31	1.51	3.94	.24	1.09	3.52	.19	.81	3.16	.16	.60	56.00
58.00	4.61	.33	1.62	4.09	.26	1.18	3.64	.21	.86	3.27	.17	.65	58.00
60.00	4.77	.35	1.74	4.22	.28	1.25	3.77	.22	.92	3.38	.18	.69	60.00
62.00	4.93	.38	1.86	4.37	.30	1.34	3.89	.24	.98	3.49	.19	.74	62.00
64.00	5.09	.40	1.98	4.51	.32	1.43	4.02	.25	1.05	3.61	.20	.79	64.00
66.00	5.25	.43	2.10	4.65	.34	1.52	4.15	.27	1.12	3.72	.22	.84	66.00
68.00	5.41	.45	2.23	4.79	.36	1.61	4.27	.28	1.19	3.83	.23	.89	68.00
70.00	5.57	.48	2.37	4.93	.38	1.71	4.40	.30	1.26	3.95	.24	.94	70.00
72.00	5.73	.51	2.51	5.07	.40	1.81	4.52	.32	1.33	4.06	.26	1.00	72.00
74.00	5.89	.54	2.65	5.21	.42	1.91	4.65	.34	1.41	4.17	.27	1.05	74.00
76.00	6.04	.57	2.79	5.35	.44	2.02	4.78	.36	1.49	4.29	.29	1.12	76.00
78.00	6.20	.60	2.94	5.49	.47	2.12	4.90	.37	1.57	4.40	.30	1.17	78.00
80.00	6.36	.63	3.09	5.64	.49	2.24	5.03	.39	1.65	4.51	.32	1.23	80.00
82.00	6.52	.66	3.25	5.78	.52	2.35	5.15	.41	1.73	4.63	.33	1.30	82.00
84.00	6.68	.69	3.41	5.92	.54	2.47	5.28	.43	1.82	4.74	.35	1.36	84.00
86.00	6.84	.72	3.58	6.06	.57	2.59	5.40	.45	1.90	4.85	.37	1.43	86.00
88.00	7.00	.76	3.75	6.20	.60	2.71	5.53	.48	2.00	4.96	.38	1.49	88.00
90.00	7.16	.79	3.92	6.34	.62	2.83	5.65	.50	2.08	5.08	.40	1.57	90.00
92.00	7.32	.83	4.10	6.48	.65	2.96	5.78	.52	2.18	5.19	.42	1.63	92.00
94.00	7.48	.87	4.28	6.62	.68	3.09	5.91	.54	2.28	5.30	.44	1.70	94.00
96.00	7.64	.90	4.46	6.76	.71	3.22	6.03	.57	2.37	5.41	.46	1.78	96.00
98.00	7.79	.94	4.64	6.91	.74	3.37	6.16	.59	2.48	5.53	.47	1.86	98.00
100.00	7.96	.98	4.84	7.05	.77	3.50	6.29	.61	2.58	5.64	.50	1.93	100.00
102.00	8.12	1.02	5.04	7.19	.80	3.64	6.41	.64	2.68	5.76	.52	2.01	102.00
104.00	8.28	1.06	5.24	7.33	.84	3.79	6.54	.66	2.79	5.87	.54	2.09	104.00
106.00	8.44	1.11	5.45	7.47	.87	3.93	6.67	.69	2.91	5.98	.56	2.17	106.00
108.00	8.60	1.15	5.66	7.61	.90	4.08	6.79	.72	3.01	6.10	.58	2.26	108.00
110.00	8.75	1.19	5.85	7.75	.93	4.23	6.91	.74	3.12	6.20	.60	2.33	110.00
112.00	8.91	1.24	6.07	7.90	.97	4.40	7.04	.77	3.24	6.32	.62	2.43	112.00
114.00	9.07	1.28	6.29	8.04	1.00	4.56	7.17	.80	3.36	6.43	.64	2.51	114.00
116.00	9.23	1.32	6.52	8.18	1.04	4.72	7.29	.83	3.47	6.55	.67	2.61	116.00
118.00	9.39	1.37	6.74	8.32	1.07	4.88	7.42	.86	3.60	6.66	.69	2.69	118.00
120.00	9.55	1.42	6.98	8.46	1.11	5.05	7.54	.89	3.71	6.77	.71	2.78	120.00
122.00	9.71	1.46	7.21	8.60	1.15	5.22	7.67	.91	3.84	6.89	.74	2.88	122.00
124.00	9.87	1.51	7.45	8.74	1.19	5.39	7.80	.95	3.97	7.00	.76	2.98	124.00
126.00	10.03	1.56	7.69	8.88	1.22	5.56	7.92	.98	4.10	7.11	.79	3.07	126.00
128.00	10.19	1.61	7.94	9.02	1.26	5.74	8.05	1.01	4.23	7.22	.81	3.17	128.00
130.00				9.16	1.30	5.92	8.17	1.04	4.36	7.33	.84	3.26	130.00
132.00				9.30	1.34	6.10	8.30	1.07	4.50	7.45	.86	3.37	132.00
134.00				9.45	1.39	6.30	8.43	1.10	4.64	7.56	.89	3.47	134.00
136.00				9.59	1.43	6.49	8.55	1.14	4.78	7.68	.92	3.58	136.00
138.00				9.73	1.47	6.68	8.68	1.17	4.92	7.79	.94	3.69	138.00
140.00				9.87	1.51	6.87	8.80	1.20	5.06	7.90	.97	3.79	140.00
142.00				10.01	1.56	7.07	8.93	1.24	5.21	8.01	1.00	3.90	142.00
144.00				10.15	1.60	7.27	9.05	1.27	5.35	8.13	1.03	4.02	144.00
146.00				10.29	1.64	7.47	9.18	1.31	5.51	8.24	1.06	4.13	146.00
148.00				10.43	1.69	7.67	9.31	1.35	5.66	8.35	1.08	4.24	148.00
150.00				10.57	1.73	7.88	9.43	1.38	5.81	8.46	1.11	4.35	150.00
152.00							9.56	1.42	5.97	8.58	1.14	4.48	152.00
154.00							9.68	1.46	6.12	8.69	1.17	4.59	154.00
156.00							9.81	1.50	6.29	8.80	1.21	4.71	156.00
158.00							9.93	1.53	6.44	8.92	1.24	4.84	158.00
160.00							10.06	1.57	6.61	9.03	1.27	4.96	160.00
162.00							10.19	1.61	6.79	9.14	1.30	5.08	162.00
164.00							10.31	1.65	6.95	9.26	1.33	5.21	164.00
166.00							10.44	1.69	7.12	9.37	1.36	5.34	166.00
168.00							10.56	1.73	7.29	9.48	1.40	5.46	168.00
170.00							10.69	1.78	7.47	9.59	1.43	5.59	170.00
172.00							10.81	1.82	7.64	9.71	1.46	5.73	172.00
174.00							10.94	1.86	7.82	9.82	1.50	5.86	174.00
176.00										9.93	1.53	6.00	176.00
178.00										10.05	1.57	6.14	178.00
180.00										10.15	1.60	6.27	180.00
182.00										10.27	1.64	6.41	182.00
184.00										10.38	1.68	6.55	184.00
186.00										10.50	1.71	6.70	186.00
188.00										10.61	1.75	6.85	188.00
190.00										10.72	1.79	6.99	190.00
192.00										10.84	1.83	7.15	192.00
194.00										10.95	1.86	7.29	194.00
196.00										11.06	1.90	7.44	196.00
198.00										11.17	1.94	7.59	198.00
200.00										11.08	1.98	7.47	200.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$

A = Area of pipe in square feet.

V = Velocity in feet per second

h_v = Velocity head in feet

n = Coefficient of roughness.

r = Hydraulic radius in feet

s = Friction loss in feet per foot.

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS
MANNING'S FORMULA n=.013
48", 51", 54", AND 57", DIAMETERS

Sheet 4 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-5

Q	60" A=19.635			63" A=21.648			66" A=23.758			69" A=25.967			Q
c f s	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c f s
50.00	2.55	.10	.36	2.31	.08	.28							50.00
52.00	2.65	.11	.39	2.40	.09	.30							52.00
54.00	2.75	.12	.42	2.49	.10	.33							54.00
56.00	2.85	.13	.46	2.58	.10	.35							56.00
58.00	2.95	.14	.49	2.68	.11	.38							58.00
60.00	3.05	.14	.52	2.77	.12	.40							60.00
62.00	3.15	.15	.56	2.86	.13	.43							62.00
64.00	3.26	.17	.60	2.95	.14	.46							64.00
66.00	3.36	.18	.64	3.05	.14	.49							66.00
68.00	3.46	.19	.68	3.14	.15	.52							68.00
70.00	3.56	.20	.72	3.23	.16	.55							70.00
72.00	3.66	.21	.76	3.32	.17	.58							72.00
74.00	3.76	.22	.80	3.42	.18	.62							74.00
76.00	3.87	.23	.85	3.51	.19	.65							76.00
78.00	3.97	.25	.89	3.60	.20	.69							78.00
80.00	4.07	.26	.94	3.69	.21	.72	3.36	.18	.56	3.08	.15	.44	80.00
82.00	4.17	.27	.98	3.78	.22	.76	3.45	.18	.59	3.15	.16	.46	82.00
84.00	4.28	.28	1.04	3.88	.23	.80	3.53	.19	.62	3.25	.16	.49	84.00
86.00	4.38	.30	1.09	3.97	.24	.83	3.62	.20	.65	3.31	.17	.51	86.00
88.00	4.48	.31	1.14	4.06	.26	.87	3.70	.21	.68	3.39	.18	.54	88.00
90.00	4.58	.33	1.19	4.15	.27	.91	3.79	.22	.71	3.46	.19	.56	90.00
92.00	4.68	.34	1.24	4.25	.28	.96	3.87	.23	.74	3.54	.20	.59	92.00
94.00	4.78	.36	1.29	4.34	.29	1.00	3.95	.24	.78	3.62	.20	.61	94.00
96.00	4.89	.37	1.35	4.43	.31	1.04	4.04	.25	.81	3.69	.21	.64	96.00
98.00	4.99	.39	1.41	4.53	.32	1.09	4.12	.26	.84	3.77	.22	.67	98.00
100.00	5.09	.40	1.47	4.62	.33	1.13	4.21	.28	.88	3.85	.23	.69	100.00
102.00	5.20	.42	1.53	4.71	.35	1.18	4.29	.29	.92	3.93	.24	.72	102.00
104.00	5.30	.44	1.59	4.80	.36	1.22	4.38	.30	.96	4.01	.25	.75	104.00
106.00	5.40	.45	1.65	4.90	.37	1.27	4.46	.31	.99	4.08	.26	.78	106.00
108.00	5.50	.47	1.71	4.99	.39	1.32	4.55	.32	1.03	4.16	.27	.81	108.00
110.00	5.60	.49	1.78	5.08	.40	1.37	4.63	.33	1.07	4.23	.28	.84	110.00
112.00	5.70	.51	1.84	5.17	.42	1.42	4.71	.34	1.11	4.31	.29	.87	112.00
114.00	5.81	.52	1.91	5.27	.43	1.47	4.80	.36	1.15	4.39	.30	.90	114.00
116.00	5.91	.54	1.99	5.36	.45	1.53	4.88	.37	1.19	4.47	.31	.94	116.00
118.00	6.01	.56	2.05	5.45	.46	1.58	4.97	.38	1.23	4.54	.32	.97	118.00
120.00	6.11	.58	2.12	5.54	.48	1.63	5.05	.40	1.27	4.62	.33	1.00	120.00
122.00	6.21	.60	2.19	5.64	.49	1.69	5.14	.41	1.32	4.70	.34	1.04	122.00
124.00	6.32	.62	2.27	5.73	.51	1.74	5.22	.42	1.36	4.78	.35	1.07	124.00
126.00	6.42	.64	2.34	5.82	.53	1.80	5.30	.44	1.40	4.85	.37	1.10	126.00
128.00	6.52	.66	2.41	5.91	.54	1.86	5.39	.45	1.45	4.93	.38	1.14	128.00
130.00	6.62	.68	2.49	6.00	.56	1.91	5.47	.46	1.49	5.00	.39	1.17	130.00
132.00	6.72	.70	2.56	6.10	.58	1.98	5.56	.48	1.54	5.08	.40	1.21	132.00
134.00	6.93	.72	2.65	6.19	.60	2.04	5.64	.49	1.59	5.16	.41	1.25	134.00
136.00	6.93	.75	2.72	6.28	.61	2.10	5.72	.51	1.63	5.24	.43	1.29	136.00
138.00	7.03	.77	2.80	6.38	.63	2.16	5.81	.52	1.68	5.31	.44	1.33	138.00
140.00	7.13	.79	2.88	6.46	.65	2.22	5.89	.54	1.73	5.39	.45	1.37	140.00
142.00	7.23	.81	2.97	6.56	.67	2.29	5.98	.55	1.78	5.47	.46	1.41	142.00
144.00	7.33	.84	3.05	6.65	.69	2.35	6.06	.57	1.83	5.55	.48	1.45	144.00
146.00	7.44	.86	3.14	6.74	.71	2.41	6.15	.59	1.89	5.62	.49	1.48	146.00
148.00	7.54	.88	3.23	6.84	.73	2.49	6.23	.60	1.94	5.70	.50	1.53	148.00
150.00	7.64	.91	3.31	6.93	.75	2.55	6.31	.62	1.99	5.77	.52	1.57	150.00
152.00	7.74	.93	3.40	7.02	.77	2.62	6.40	.64	2.05	5.85	.53	1.61	152.00
154.00	7.84	.96	3.49	7.11	.79	2.69	6.48	.65	2.10	5.93	.55	1.65	154.00
156.00	7.95	.98	3.59	7.21	.81	2.76	6.57	.67	2.16	6.01	.56	1.70	156.00
158.00	8.05	1.01	3.68	7.30	.83	2.83	6.65	.69	2.21	6.08	.58	1.74	158.00
160.00	8.14	1.03	3.76	7.39	.85	2.90	6.73	.70	2.26	6.16	.59	1.79	160.00
162.00	8.25	1.06	3.86	7.48	.87	2.97	6.82	.72	2.32	6.24	.60	1.83	162.00
164.00	8.35	1.08	3.96	7.58	.89	3.06	6.90	.74	2.38	6.32	.62	1.88	164.00
166.00	8.45	1.11	4.05	7.67	.91	3.13	6.99	.76	2.44	6.39	.64	1.92	166.00
168.00	8.56	1.14	4.16	7.76	.94	3.20	7.07	.78	2.50	6.47	.65	1.97	168.00
170.00	8.65	1.17	4.25	7.85	.96	3.28	7.15	.80	2.55	6.54	.67	2.01	170.00
172.00	8.76	1.19	4.36	7.95	.98	3.36	7.24	.81	2.62	6.62	.68	2.06	172.00
174.00	8.86	1.22	4.46	8.04	1.00	3.44	7.32	.83	2.68	6.70	.70	2.11	174.00
176.00	8.96	1.25	4.56	8.13	1.03	3.52	7.41	.85	2.74	6.78	.71	2.16	176.00
178.00	9.07	1.28	4.67	8.22	1.05	3.59	7.49	.87	2.80	6.85	.73	2.21	178.00
180.00	9.16	1.31	4.76	8.31	1.07	3.67	7.57	.89	2.86	6.93	.75	2.26	180.00
182.00	9.27	1.34	4.88	8.41	1.10	3.76	7.66	.91	2.93	7.01	.76	2.31	182.00
184.00	9.37	1.37	4.99	8.50	1.12	3.84	7.74	.93	2.99	7.09	.78	2.37	184.00
186.00	9.47	1.40	5.09	8.59	1.15	3.92	7.83	.95	3.06	7.16	.80	2.41	186.00
188.00	9.58	1.43	5.21	8.68	1.17	4.01	7.91	.97	3.13	7.24	.81	2.47	188.00
190.00	9.67	1.46	5.31	8.77	1.20	4.09	8.00	.99	3.20	7.31	.83	2.52	190.00
192.00	9.78	1.49	5.43	8.87	1.22	4.19	8.08	1.02	3.26	7.39	.85	2.57	192.00
194.00	9.88	1.52	5.54	8.96	1.25	4.27	8.17	1.04	3.34	7.47	.87	2.63	194.00
196.00	9.98	1.55	5.66	9.05	1.27	4.36	8.25	1.06	3.40	7.55	.88	2.68	196.00
198.00	10.08	1.58	5.77	9.15	1.30	4.45	8.33	1.08	3.47	7.63	.90	2.74	198.00
200.00	10.18	1.61	5.89	9.24	1.33	4.54	8.42	1.10	3.54	7.70	.92	2.79	200.00
202.00	10.29	1.65	6.01	9.33	1.35	4.63	8.50	1.12	3.61	7.78	.94	2.85	202.00
204.00	10.39	1.68	6.13	9.42	1.38	4.72	8.59	1.15	3.69	7.86	.96	2.91	204.00
206.00	10.49	1.71	6.25	9.52	1.41	4.82	8.67	1.17	3.76	7.93	.98	2.96	206.00
208.00				9.61	1.43	4.91	8.75	1.19	3.83	8.01	1.00	3.02	208.00
210.00				9.70	1.46	5.01	8.84	1.21	3.91	8.09	1.02	3.08	210.00
212.00				9.79	1.49	5.10	8.92	1.24	3.98	8.16	1.03	3.14	212.00
214.00				9.89	1.52	5.20	9.01	1.26	4.06	8.24	1.05	3.20	214.00
216.00				9.98	1.55	5.30	9.09	1.28	4.13	8.32	1.07	3.26	216.00
218.00				10.07	1.57	5.40	9.18	1.31	4.21	8.40	1.10	3.32	218.00
220.00				10.16	1.60	5.49	9.26	1.33	4.29	8.47	1.11	3.38	220.00
222.00							9.34	1.35	4.36	8.55	1.14	3.44	222.00
224.00							9.43	1.38	4.45	8.63	1.16	3.51	224.00
226.00							9.51	1.40	4.52	8.70	1.18	3.57	226.00
228.00							9.60	1.43	4.61	8.78	1.20	3.63	228.00
230.00							9.68	1.46	4.69	8.86	1.22	3.70	230.00
232.00							9.77	1.48	4.77	8.93	1.24	3.76	232.00
234.00							9.85	1.51	4.85	9.01	1.26	3.82	234.00
236.00							9.93	1.53	4.93	9.09	1.28	3.89	236.00
238.00							10.02	1.56	5.02	9.17	1.31	3.96	238.00
240.00							10.10	1.58	5.10	9.24	1.33	4.02	240.00
245.00										9.44	1.38	4.20	245.00
250.00										9.63	1.44	4.37	250.00
255.00										9.82	1.50	4.54	255.00
260.00										10.01	1.56	4.72	260.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$

A = Area of pipe in square feet.

V = Velocity in feet per second

h_v = Velocity head in feet

n = Coefficient of roughness

r = Hydraulic radius in feet

s = Friction loss in feet per foot

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS

MANNING'S FORMULA n=.013

60", 63", 66", AND 69", DIAMETERS

Sheet 5 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-6

Q c f s	72" A = 28.274 V	h_v	H	75" A = 30.680 V	h_v	H	78" A = 33.183 V	h_v	H	81" A = 35.785 V	h_v	H	Q c f s
80.00	2.83	.12	.35	2.61	.10	.28							80.00
82.00	2.90	.13	.37	2.67	.11	.30							82.00
84.00	2.97	.14	.39	2.74	.12	.31							84.00
86.00	3.04	.14	.41	2.80	.12	.33							86.00
88.00	3.11	.15	.43	2.87	.13	.34							88.00
90.00	3.18	.16	.45	2.93	.13	.36							90.00
92.00	3.25	.16	.47	3.00	.14	.37							92.00
94.00	3.32	.17	.49	3.06	.14	.39							94.00
96.00	3.39	.18	.51	3.13	.15	.41							96.00
98.00	3.46	.19	.53	3.19	.16	.42							98.00
100.00	3.53	.19	.55	3.26	.16	.44							100.00
102.00	3.61	.20	.58	3.32	.17	.46							102.00
104.00	3.68	.21	.60	3.39	.18	.48							104.00
106.00	3.75	.22	.62	3.46	.18	.50							106.00
108.00	3.82	.23	.65	3.52	.19	.52							108.00
110.00	3.89	.24	.67	3.59	.20	.54	3.32	.17	.44				110.00
112.00	3.96	.24	.69	3.65	.21	.56	3.38	.18	.45				112.00
114.00	4.03	.25	.72	3.72	.21	.58	3.44	.18	.47				114.00
116.00	4.10	.26	.74	3.78	.22	.60	3.50	.19	.49				116.00
118.00	4.17	.27	.77	3.85	.23	.62	3.56	.20	.50				118.00
120.00	4.24	.28	.80	3.91	.24	.64	3.62	.20	.52	3.35	.18	.42	120.00
122.00	4.31	.29	.82	3.98	.24	.66	3.68	.21	.54	3.41	.18	.44	122.00
124.00	4.39	.30	.85	4.04	.25	.68	3.74	.22	.56	3.46	.19	.45	124.00
126.00	4.46	.31	.88	4.11	.26	.71	3.80	.22	.57	3.52	.19	.47	126.00
128.00	4.53	.32	.91	4.17	.27	.73	3.86	.23	.59	3.58	.20	.48	128.00
130.00	4.60	.33	.94	4.24	.28	.75	3.92	.24	.61	3.63	.21	.50	130.00
132.00	4.67	.34	.97	4.30	.29	.78	3.98	.25	.63	3.69	.21	.51	132.00
134.00	4.74	.35	1.00	4.37	.30	.80	4.04	.25	.65	3.74	.22	.53	134.00
136.00	4.81	.36	1.03	4.43	.30	.82	4.10	.26	.67	3.80	.22	.55	136.00
138.00	4.88	.37	1.06	4.50	.31	.85	4.16	.27	.69	3.86	.23	.56	138.00
140.00	4.95	.38	1.09	4.56	.32	.87	4.22	.28	.71	3.91	.24	.58	140.00
142.00	5.02	.39	1.12	4.63	.33	.90	4.28	.28	.73	3.97	.25	.60	142.00
144.00	5.09	.40	1.15	4.69	.34	.92	4.34	.29	.75	4.02	.25	.61	144.00
146.00	5.16	.41	1.18	4.76	.35	.95	4.40	.30	.77	4.08	.26	.63	146.00
148.00	5.23	.42	1.21	4.82	.36	.98	4.46	.31	.79	4.14	.27	.65	148.00
150.00	5.30	.44	1.25	4.89	.37	1.00	4.52	.32	.81	4.19	.27	.66	150.00
152.00	5.38	.45	1.29	4.95	.38	1.03	4.58	.33	.84	4.25	.28	.68	152.00
154.00	5.44	.46	1.31	5.02	.39	1.06	4.64	.33	.86	4.30	.29	.70	154.00
156.00	5.52	.47	1.35	5.08	.40	1.06	4.70	.34	.88	4.36	.30	.72	156.00
158.00	5.59	.48	1.39	5.15	.41	1.11	4.76	.35	.90	4.41	.30	.74	158.00
160.00	5.66	.50	1.42	5.22	.42	1.15	4.82	.36	.93	4.47	.31	.76	160.00
162.00	5.73	.51	1.46	5.28	.43	1.17	4.88	.37	.95	4.53	.32	.78	162.00
164.00	5.80	.52	1.49	5.35	.44	1.20	4.94	.38	.97	4.58	.33	.79	164.00
166.00	5.87	.54	1.53	5.41	.46	1.23	5.00	.39	1.00	4.64	.33	.82	166.00
168.00	5.94	.55	1.57	5.48	.47	1.26	5.06	.40	1.02	4.70	.34	.84	168.00
170.00	6.01	.56	1.60	5.54	.48	1.29	5.12	.41	1.05	4.75	.35	.85	170.00
172.00	6.08	.58	1.64	5.61	.49	1.32	5.18	.42	1.07	4.81	.36	.88	172.00
174.00	6.15	.59	1.68	5.67	.50	1.35	5.24	.43	1.09	4.86	.37	.89	174.00
176.00	6.22	.60	1.72	5.74	.51	1.39	5.30	.44	1.12	4.92	.38	.92	176.00
178.00	6.30	.62	1.76	5.80	.52	1.41	5.36	.45	1.15	4.97	.38	.94	178.00
180.00	6.37	.63	1.80	5.87	.54	1.45	5.42	.46	1.17	5.03	.39	.96	180.00
182.00	6.44	.64	1.84	5.93	.55	1.48	5.48	.47	1.20	5.09	.40	.98	182.00
184.00	6.51	.66	1.88	6.00	.56	1.51	5.54	.48	1.22	5.14	.41	1.00	184.00
186.00	6.58	.67	1.92	6.06	.57	1.55	5.60	.49	1.25	5.20	.42	1.03	186.00
188.00	6.65	.69	1.97	6.13	.58	1.58	5.67	.50	1.28	5.25	.43	1.04	188.00
190.00	6.72	.70	2.01	6.19	.60	1.61	5.73	.51	1.31	5.31	.44	1.07	190.00
192.00	6.79	.72	2.05	6.26	.61	1.65	5.79	.52	1.34	5.36	.45	1.09	192.00
194.00	6.86	.73	2.09	6.32	.62	1.68	5.85	.53	1.37	5.42	.46	1.11	194.00
196.00	6.93	.75	2.14	6.39	.63	1.72	5.91	.54	1.39	5.48	.47	1.14	196.00
198.00	7.00	.76	2.18	6.45	.65	1.75	5.97	.55	1.42	5.53	.48	1.16	198.00
200.00	7.07	.78	2.22	6.52	.66	1.79	6.03	.57	1.45	5.59	.49	1.19	200.00
202.00	7.14	.79	2.27	6.58	.67	1.82	6.09	.58	1.48	5.64	.50	1.21	202.00
204.00	7.22	.81	2.32	6.65	.69	1.86	6.15	.59	1.51	5.70	.51	1.23	204.00
206.00	7.29	.82	2.36	6.71	.70	1.90	6.21	.60	1.54	5.76	.52	1.26	206.00
208.00	7.36	.84	2.41	6.79	.71	1.94	6.27	.61	1.57	5.81	.53	1.28	208.00
210.00	7.43	.86	2.46	6.84	.73	1.97	6.33	.62	1.60	5.87	.54	1.31	210.00
212.00	7.50	.87	2.50	6.91	.74	2.01	6.39	.64	1.63	5.92	.55	1.33	212.00
214.00	7.57	.89	2.55	6.98	.76	2.05	6.45	.65	1.66	5.98	.56	1.36	214.00
216.00	7.64	.91	2.60	7.04	.77	2.09	6.51	.66	1.69	6.04	.57	1.38	216.00
218.00	7.71	.92	2.64	7.11	.78	2.13	6.57	.67	1.72	6.09	.58	1.41	218.00
220.00	7.78	.94	2.69	7.17	.80	2.17	6.63	.68	1.76	6.15	.59	1.44	220.00
222.00	7.85	.96	2.74	7.24	.81	2.21	6.69	.70	1.79	6.20	.60	1.46	222.00
224.00	7.92	.98	2.79	7.30	.83	2.24	6.75	.71	1.82	6.26	.61	1.49	224.00
226.00	7.99	.99	2.84	7.37	.84	2.29	6.81	.72	1.85	6.32	.62	1.52	226.00
228.00	8.06	1.01	2.89	7.43	.86	2.33	6.87	.74	1.89	6.37	.63	1.54	228.00
230.00	8.14	1.03	2.95	7.50	.87	2.37	6.93	.75	1.92	6.43	.64	1.57	230.00
232.00	8.21	1.05	3.00	7.56	.89	2.41	6.99	.76	1.95	6.48	.65	1.59	232.00
234.00	8.28	1.06	3.05	7.63	.90	2.45	7.05	.77	1.99	6.54	.67	1.62	234.00
236.00	8.35	1.08	3.10	7.69	.92	2.49	7.11	.79	2.02	6.60	.66	1.65	236.00
238.00	8.42	1.10	3.15	7.76	.94	2.54	7.17	.80	2.05	6.65	.69	1.68	238.00
240.00	8.49	1.12	3.21	7.82	.95	2.58	7.23	.81	2.09	6.71	.70	1.71	240.00
245.00	8.67	1.17	3.35	7.99	.99	2.69	7.38	.85	2.18	6.85	.73	1.78	245.00
250.00	8.84	1.21	3.48	8.15	1.03	2.80	7.53	.88	2.27	6.99	.76	1.86	250.00
255.00	9.02	1.26	3.62	8.31	1.07	2.91	7.68	.92	2.36	7.13	.79	1.93	255.00
260.00	9.20	1.31	3.77	8.47	1.11	3.02	7.84	.95	2.46	7.27	.82	2.01	260.00
265.00	9.37	1.36	3.91	8.64	1.16	3.15	7.99	.99	2.55	7.41	.85	2.09	265.00
270.00	9.55	1.42	4.06	8.80	1.20	3.26	8.14	1.03	2.65	7.55	.89	2.17	270.00
275.00	9.73	1.47	4.21	8.96	1.25	3.38	8.29	1.07	2.75	7.66	.92	2.24	275.00
280.00	9.90	1.52	4.36	9.13	1.29	3.51	8.44	1.11	2.85	7.82	.95	2.32	280.00
285.00	10.08	1.58	4.52	9.29	1.34	3.64	8.59	1.15	2.95	7.96	.98	2.41	285.00
290.00	10.26	1.63	4.69	9.45	1.39	3.76	8.74	1.19	3.06	8.10	1.02	2.49	290.00
295.00							8.89	1.23	3.16	8.24	1.05	2.58	295.00
300.00							9.04	1.27	3.27	8.38	1.09	2.67	300.00
310.00							9.34	1.35	3.49	8.66	1.16	2.85	310.00
320.00							9.64	1.44	3.72	8.94	1.24	3.04	320.00
330.00							9.94	1.53	3.95	9.22	1.32	3.23	330.00
340.00							10.25	1.63	4.20	9.50	1.40	3.43	340.00
350.00										9.78	1.49	3.64	350.00
360.00										10.06	1.57	3.85	360.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$.

A = Area of pipe in square feet.

V = Velocity in feet per second.

h_v = Velocity head in feet

n = Coefficient of roughness.

r = Hydraulic radius in feet.

s = Friction loss in feet per foot

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS

MANNING'S FORMULA n = .013

72", 75", 78", AND 81", DIAMETERS

Sheet 6 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-7

Q c.f.s.	84" A=38.484 V	h_v	H	87" A=41.282 V	h_v	H	90" A=44.179 V	h_v	H	93" A=47.173 V	h_v	H	Q c.f.s.
120.00	3.12	.15	.35	2.91	.13	.29	2.72	.11	.24	2.54	.10	.20	120.00
125.00	3.25	.17	.38	3.02	.14	.31	2.83	.12	.26	2.65	.11	.22	125.00
130.00	3.38	.18	.41	3.15	.15	.34	2.94	.13	.28	2.76	.12	.24	130.00
135.00	3.51	.19	.44	3.27	.17	.37	3.06	.15	.30	2.86	.13	.25	135.00
140.00	3.64	.21	.48	3.39	.18	.39	3.17	.16	.33	2.97	.14	.27	140.00
145.00	3.77	.22	.51	3.52	.19	.42	3.28	.17	.35	3.07	.15	.29	145.00
150.00	3.90	.24	.55	3.63	.21	.45	3.40	.18	.38	3.18	.16	.32	150.00
155.00	4.03	.25	.58	3.76	.22	.48	3.51	.19	.40	3.29	.17	.34	155.00
160.00	4.16	.27	.62	3.88	.23	.52	3.62	.20	.43	3.39	.18	.36	160.00
165.00	4.29	.29	.66	4.00	.25	.55	3.73	.22	.46	3.50	.19	.38	165.00
170.00	4.42	.30	.70	4.12	.26	.58	3.85	.23	.49	3.60	.20	.41	170.00
175.00	4.55	.32	.75	4.24	.28	.62	3.96	.24	.51	3.71	.21	.43	175.00
180.00	4.68	.34	.79	4.36	.30	.65	4.07	.26	.54	3.82	.23	.46	180.00
185.00	4.81	.36	.83	4.48	.31	.69	4.19	.27	.58	3.92	.24	.48	185.00
190.00	4.94	.38	.88	4.60	.33	.73	4.30	.29	.61	4.03	.25	.51	190.00
195.00	5.07	.40	.93	4.72	.35	.77	4.41	.30	.64	4.13	.26	.54	195.00
200.00	5.20	.42	.98	4.84	.36	.81	4.53	.32	.67	4.24	.28	.56	200.00
205.00	5.33	.44	1.03	4.96	.38	.85	4.64	.33	.71	4.35	.29	.59	205.00
210.00	5.46	.46	1.08	5.09	.40	.89	4.75	.35	.74	4.45	.31	.62	210.00
215.00	5.59	.48	1.13	5.20	.42	.93	4.87	.37	.78	4.56	.32	.65	215.00
220.00	5.72	.51	1.18	5.33	.44	.98	4.98	.38	.82	4.66	.34	.68	220.00
225.00	5.85	.53	1.24	5.45	.46	1.02	5.09	.40	.85	4.77	.35	.72	225.00
230.00	5.98	.56	1.29	5.57	.48	1.07	5.21	.42	.89	4.88	.37	.75	230.00
235.00	6.11	.58	1.35	5.69	.50	1.12	5.32	.44	.93	4.98	.38	.78	235.00
240.00	6.24	.60	1.41	5.81	.52	1.16	5.43	.46	.97	5.09	.40	.82	240.00
245.00	6.37	.63	1.47	5.93	.55	1.21	5.55	.48	1.01	5.19	.42	.85	245.00
250.00	6.50	.66	1.53	6.06	.57	1.27	5.66	.50	1.06	5.30	.44	.89	250.00
255.00	6.63	.68	1.59	6.18	.59	1.32	5.77	.52	1.10	5.41	.45	.92	255.00
260.00	6.76	.71	1.65	6.30	.62	1.37	5.89	.54	1.14	5.51	.47	.96	260.00
265.00	6.89	.74	1.72	6.42	.64	1.42	6.00	.56	1.19	5.62	.49	1.00	265.00
270.00	7.02	.77	1.78	6.54	.66	1.48	6.11	.58	1.23	5.72	.51	1.03	270.00
275.00	7.15	.79	1.85	6.66	.69	1.53	6.22	.60	1.28	5.83	.53	1.07	275.00
280.00	7.28	.82	1.92	6.78	.71	1.59	6.34	.62	1.33	5.94	.55	1.11	280.00
285.00	7.41	.85	1.99	6.90	.74	1.64	6.45	.65	1.37	6.04	.57	1.15	285.00
290.00	7.54	.88	2.06	7.02	.77	1.70	6.56	.67	1.42	6.15	.59	1.19	290.00
295.00	7.67	.91	2.13	7.15	.79	1.77	6.68	.69	1.47	6.25	.61	1.23	295.00
300.00	7.80	.94	2.20	7.27	.82	1.83	6.79	.72	1.52	6.36	.63	1.28	300.00
310.00	8.06	1.01	2.35	7.51	.88	1.95	7.02	.76	1.63	6.57	.67	1.36	310.00
320.00	8.32	1.07	2.51	7.75	.93	2.08	7.24	.81	1.73	6.78	.71	1.45	320.00
330.00	8.57	1.14	2.66	7.99	.99	2.21	7.47	.87	1.84	7.00	.76	1.55	330.00
340.00	8.83	1.21	2.82	8.24	1.05	2.35	7.70	.92	1.96	7.21	.81	1.64	340.00
350.00	9.09	1.28	2.99	8.48	1.12	2.49	7.92	.97	2.07	7.42	.85	1.74	350.00
360.00	9.35	1.36	3.17	8.72	1.18	2.63	8.15	1.03	2.19	7.63	.90	1.84	360.00
370.00	9.61	1.43	3.35	8.96	1.25	2.78	8.37	1.09	2.31	7.84	.95	1.94	370.00
380.00	9.87	1.51	3.53	9.20	1.31	2.93	8.60	1.15	2.44	8.05	1.01	2.05	380.00
390.00	10.13	1.59	3.72	9.45	1.39	3.09	8.83	1.21	2.58	8.27	1.06	2.16	390.00
400.00				9.69	1.46	3.25	9.05	1.27	2.71	8.48	1.12	2.27	400.00
410.00				9.93	1.53	3.41	9.28	1.34	2.85	8.69	1.17	2.39	410.00
420.00				10.17	1.61	3.58	9.51	1.40	2.99	8.90	1.23	2.50	420.00
430.00							9.73	1.47	3.13	9.12	1.29	2.63	430.00
440.00							9.96	1.54	3.28	9.33	1.35	2.75	440.00
450.00							10.19	1.61	3.43	9.54	1.41	2.88	450.00
460.00							10.41	1.68	3.58	9.75	1.48	3.01	460.00
470.00							10.64	1.76	3.74	9.96	1.54	3.14	470.00
480.00							10.86	1.83	3.90	10.18	1.61	3.28	480.00
490.00							11.09	1.91	4.07	10.39	1.68	3.42	490.00
500.00							11.32	1.99	4.24	10.60	1.74	3.56	500.00
510.00							11.54	2.07	4.40	10.81	1.81	3.70	510.00
520.00							11.77	2.15	4.58	11.02	1.89	3.84	520.00

NOTES

Based on formula $V=\frac{1.486}{n}r^{\frac{2}{3}}s^{\frac{1}{2}}$
A = Area of pipe in square feet
V = Velocity in feet per second
h_v = Velocity head in feet
n = Coefficient of roughness
r = Hydraulic radius in feet
s = Friction loss in feet per foot.
H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS
MANNING'S FORMULA n=.013
84", 87", 90", AND 93", DIAMETERS

Sheet 7 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-8

Q	96" A = 50.266			102" A = 56.745			108" A = 63.617			114" A = 70.882			Q
c. f. s.	V	h_v	H	V	h_v	H	V	h_v	H	V	h_v	H	c. f. s.
120.00	2.39	.09	.17										120.00
125.00	2.49	.10	.18										125.00
130.00	2.59	.10	.20										130.00
135.00	2.69	.11	.21										135.00
140.00	2.79	.12	.23										140.00
145.00	2.88	.13	.25										145.00
150.00	2.98	.14	.26										150.00
155.00	3.08	.15	.28										155.00
160.00	3.18	.16	.30										160.00
165.00	3.28	.17	.32										165.00
170.00	3.38	.18	.34	3.00	.14	.25							170.00
175.00	3.48	.19	.36	3.08	.15	.26							175.00
180.00	3.58	.20	.38	3.17	.16	.28	2.83	.12	.20				180.00
185.00	3.68	.21	.41	3.26	.17	.29	2.91	.13	.21				185.00
190.00	3.78	.22	.43	3.35	.17	.31	2.99	.14	.23				190.00
195.00	3.88	.23	.45	3.44	.18	.33	3.07	.15	.24				195.00
200.00	3.98	.25	.48	3.52	.19	.34	3.14	.15	.25				200.00
205.00	4.08	.26	.50	3.61	.20	.36	3.22	.16	.26				205.00
210.00	4.18	.27	.53	3.70	.21	.38	3.30	.17	.28				210.00
215.00	4.28	.28	.55	3.79	.22	.40	3.38	.18	.29				215.00
220.00	4.38	.30	.58	3.88	.23	.42	3.46	.19	.31				220.00
225.00	4.48	.31	.60	3.97	.24	.44	3.54	.20	.32				225.00
230.00	4.58	.33	.63	4.05	.25	.45	3.62	.20	.34				230.00
235.00	4.68	.34	.66	4.14	.27	.48	3.70	.21	.35				235.00
240.00	4.77	.35	.69	4.23	.28	.50	3.77	.22	.36				240.00
245.00	4.87	.37	.72	4.32	.29	.52	3.85	.23	.38				245.00
250.00	4.97	.38	.75	4.41	.30	.54	3.93	.24	.40	3.53	.19	.30	250.00
255.00	5.07	.40	.78	4.49	.31	.56	4.01	.25	.41	3.60	.20	.31	255.00
260.00	5.17	.42	.81	4.58	.33	.58	4.09	.26	.43	3.67	.21	.32	260.00
265.00	5.27	.43	.84	4.67	.34	.61	4.17	.27	.45	3.74	.22	.33	265.00
270.00	5.37	.45	.87	4.76	.35	.63	4.24	.28	.46	3.81	.23	.35	270.00
275.00	5.47	.46	.90	4.85	.37	.65	4.32	.29	.48	3.88	.24	.36	275.00
280.00	5.57	.48	.94	4.93	.38	.68	4.40	.30	.50	3.95	.24	.37	280.00
285.00	5.67	.50	.97	5.02	.39	.70	4.48	.31	.52	4.02	.25	.39	285.00
290.00	5.77	.52	1.01	5.11	.41	.73	4.56	.32	.53	4.09	.26	.40	290.00
295.00	5.87	.54	1.04	5.20	.42	.75	4.64	.33	.55	4.16	.27	.41	295.00
300.00	5.97	.55	1.08	5.29	.43	.78	4.72	.35	.57	4.23	.28	.43	300.00
310.00	6.17	.59	1.15	5.46	.46	.83	4.87	.37	.61	4.37	.30	.46	310.00
320.00	6.37	.63	1.23	5.64	.49	.89	5.03	.39	.65	4.51	.32	.49	320.00
330.00	6.57	.67	1.31	5.82	.53	.94	5.19	.42	.69	4.66	.34	.52	330.00
340.00	6.76	.71	1.38	5.99	.56	1.00	5.34	.44	.74	4.80	.36	.55	340.00
350.00	6.96	.75	1.47	6.17	.59	1.06	5.50	.47	.78	4.94	.38	.58	350.00
360.00	7.16	.80	1.55	6.34	.62	1.12	5.66	.50	.83	5.08	.40	.62	360.00
370.00	7.36	.84	1.64	6.52	.66	1.19	5.82	.53	.87	5.22	.42	.65	370.00
380.00	7.56	.89	1.73	6.70	.70	1.25	5.97	.55	.92	5.36	.45	.69	380.00
390.00	7.76	.94	1.82	6.87	.73	1.32	6.13	.58	.97	5.50	.47	.73	390.00
400.00	7.96	.98	1.92	7.05	.77	1.39	6.29	.61	1.02	5.64	.49	.76	400.00
410.00	8.16	1.03	2.02	7.23	.81	1.46	6.44	.64	1.07	5.78	.52	.80	410.00
420.00	8.36	1.09	2.12	7.40	.85	1.53	6.60	.68	1.13	5.93	.55	.84	420.00
430.00	8.55	1.14	2.22	7.58	.89	1.60	6.76	.71	1.18	6.07	.57	.88	430.00
440.00	8.75	1.19	2.32	7.75	.93	1.68	6.92	.74	1.24	6.21	.60	.93	440.00
450.00	8.95	1.24	2.43	7.93	.98	1.76	7.07	.78	1.29	6.35	.63	.97	450.00
460.00	9.15	1.30	2.54	8.11	1.02	1.84	7.23	.81	1.35	6.49	.65	1.01	460.00
470.00	9.35	1.36	2.65	8.28	1.06	1.92	7.39	.85	1.41	6.63	.68	1.06	470.00
480.00	9.55	1.42	2.77	8.46	1.11	2.00	7.55	.89	1.47	6.77	.71	1.10	480.00
490.00	9.75	1.48	2.88	8.64	1.16	2.09	7.70	.92	1.53	6.91	.74	1.15	490.00
500.00	9.95	1.54	3.00	8.81	1.21	2.17	7.86	.96	1.60	7.05	.77	1.20	500.00
510.00	10.15	1.60	3.12	8.99	1.25	2.26	8.02	1.00	1.66	7.20	.80	1.25	510.00
520.00	10.34	1.66	3.24	9.16	1.30	2.35	8.17	1.04	1.73	7.34	.84	1.30	520.00
530.00				9.34	1.35	2.44	8.33	1.08	1.80	7.46	.87	1.35	530.00
540.00				9.52	1.41	2.53	8.49	1.12	1.87	7.62	.90	1.40	540.00
550.00				9.69	1.46	2.63	8.65	1.16	1.94	7.76	.94	1.45	550.00
560.00				9.87	1.51	2.72	8.80	1.20	2.01	7.90	.97	1.50	560.00
570.00				10.04	1.57	2.82	8.96	1.25	2.08	8.04	1.00	1.56	570.00
580.00				10.22	1.62	2.92	9.12	1.29	2.15	8.18	1.04	1.61	580.00
590.00				10.40	1.68	3.02	9.27	1.33	2.23	8.32	1.07	1.67	590.00
600.00				10.57	1.73	3.12	9.43	1.38	2.30	8.46	1.11	1.72	600.00
620.00							9.75	1.48	2.46	8.75	1.19	1.84	620.00
640.00							10.06	1.57	2.62	9.03	1.27	1.96	640.00
660.00							10.37	1.67	2.79	9.31	1.35	2.09	660.00
680.00							10.69	1.77	2.96	9.59	1.43	2.22	680.00
700.00							11.00	1.88	3.14	9.88	1.52	2.35	700.00
720.00										10.16	1.60	2.49	720.00
740.00										10.44	1.69	2.63	740.00
760.00										10.72	1.78	2.77	760.00
780.00										11.00	1.88	2.92	780.00
800.00										11.29	1.98	3.07	800.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$

A = Area of pipe in square feet.

V = Velocity in feet per second

h_v = Velocity head in feet

n = Coefficient of roughness

r = Hydraulic radius in feet

s = Friction loss in feet per foot.

H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS

MANNING'S FORMULA n = .013

96", 102", 108", AND 114", DIAMETERS

Sheet 8 of 9

Table 8-1.–*Friction losses for circular conduits.*–Continued. 103-D-1316-9

Q c.f.s.	120" A=78.540 V	h_v	H	126" A=86.590 V	h_v	H	132" A=95.033 V	h_v	H	138" A=103.869 V	h_v	H	Q c.f.s.
250.00	3.18	.16	.22	2.89	.13	.17	2.63	.11	.13				250.00
260.00	3.31	.17	.24	3.00	.14	.19	2.74	.12	.14				260.00
270.00	3.44	.18	.26	3.12	.15	.20	2.84	.13	.16				270.00
280.00	3.57	.20	.28	3.23	.16	.22	2.95	.14	.17				280.00
290.00	3.69	.21	.30	3.35	.17	.23	3.05	.14	.18				290.00
300.00	3.82	.23	.32	3.46	.19	.25	3.16	.16	.19	2.89	.13	.15	300.00
310.00	3.95	.24	.35	3.58	.20	.27	3.26	.17	.21	2.98	.14	.16	310.00
320.00	4.07	.26	.37	3.70	.21	.28	3.37	.18	.22	3.08	.15	.17	320.00
330.00	4.20	.27	.39	3.81	.23	.30	3.47	.19	.23	3.18	.16	.18	330.00
340.00	4.33	.29	.42	3.93	.24	.32	3.58	.20	.25	3.27	.17	.20	340.00
350.00	4.46	.31	.44	4.04	.25	.34	3.68	.21	.26	3.37	.18	.21	350.00
360.00	4.58	.33	.47	4.16	.27	.36	3.79	.22	.28	3.47	.19	.22	360.00
370.00	4.71	.34	.50	4.27	.28	.38	3.89	.23	.30	3.56	.20	.23	370.00
380.00	4.84	.36	.52	4.39	.30	.40	4.00	.25	.31	3.66	.21	.25	380.00
390.00	4.97	.38	.55	4.50	.31	.42	4.10	.26	.33	3.75	.22	.26	390.00
400.00	5.09	.40	.58	4.62	.33	.45	4.21	.28	.35	3.85	.23	.27	400.00
410.00	5.22	.42	.61	4.73	.35	.47	4.31	.29	.36	3.95	.24	.29	410.00
420.00	5.35	.44	.64	4.85	.37	.49	4.42	.30	.38	4.04	.25	.30	420.00
430.00	5.47	.46	.67	4.97	.38	.52	4.52	.32	.40	4.14	.27	.32	430.00
440.00	5.60	.49	.70	5.08	.40	.54	4.63	.33	.42	4.24	.28	.33	440.00
450.00	5.73	.51	.74	5.20	.42	.57	4.74	.35	.44	4.33	.29	.35	450.00
460.00	5.86	.53	.77	5.31	.44	.59	4.84	.36	.46	4.43	.30	.36	460.00
470.00	5.98	.55	.80	5.43	.46	.62	4.95	.38	.48	4.52	.32	.38	470.00
480.00	6.11	.58	.84	5.54	.48	.64	5.05	.40	.50	4.62	.33	.39	480.00
490.00	6.24	.60	.87	5.66	.50	.67	5.16	.41	.52	4.72	.35	.41	490.00
500.00	6.37	.63	.91	5.77	.52	.70	5.26	.43	.54	4.81	.36	.43	500.00
510.00	6.49	.65	.95	5.89	.54	.73	5.37	.45	.57	4.91	.37	.45	510.00
520.00	6.62	.68	.98	6.01	.56	.76	5.47	.46	.59	5.01	.39	.46	520.00
530.00	6.75	.71	1.02	6.12	.58	.79	5.58	.48	.61	5.10	.40	.48	530.00
540.00	6.88	.74	1.06	6.24	.60	.82	5.68	.50	.64	5.20	.42	.50	540.00
550.00	7.00	.76	1.10	6.35	.63	.85	5.79	.52	.66	5.30	.44	.52	550.00
560.00	7.13	.79	1.14	6.47	.65	.88	5.89	.54	.68	5.39	.45	.54	560.00
570.00	7.26	.82	1.18	6.58	.67	.91	6.00	.56	.71	5.49	.47	.56	570.00
580.00	7.38	.85	1.22	6.70	.70	.94	6.10	.58	.73	5.58	.48	.58	580.00
590.00	7.51	.88	1.27	6.81	.72	.98	6.21	.60	.76	5.68	.50	.60	590.00
600.00	7.64	.91	1.31	6.93	.75	1.01	6.31	.62	.79	5.78	.52	.62	600.00
620.00	7.89	.97	1.40	7.16	.80	1.08	6.52	.66	.84	5.97	.55	.66	620.00
640.00	8.15	1.03	1.49	7.39	.85	1.15	6.73	.70	.89	6.16	.59	.71	640.00
660.00	8.40	1.10	1.59	7.62	.90	1.22	6.94	.75	.95	6.35	.63	.75	660.00
680.00	8.66	1.16	1.69	7.85	.96	1.30	7.16	.80	1.01	6.55	.67	.80	680.00
700.00	8.91	1.23	1.79	8.08	1.01	1.37	7.37	.84	1.07	6.74	.71	.85	700.00
720.00	9.17	1.31	1.89	8.32	1.07	1.46	7.58	.89	1.14	6.93	.75	.89	720.00
740.00	9.42	1.38	2.00	8.55	1.14	1.54	7.79	.94	1.20	7.12	.79	.94	740.00
760.00	9.68	1.46	2.11	8.78	1.20	1.62	8.00	.99	1.27	7.32	.83	1.00	760.00
780.00	9.93	1.53	2.22	9.01	1.26	1.71	8.21	1.05	1.33	7.51	.88	1.05	780.00
800.00	10.19	1.61	2.34	9.24	1.33	1.80	8.42	1.10	1.40	7.70	.92	1.10	800.00
820.00	10.44	1.69	2.45	9.47	1.39	1.89	8.63	1.16	1.47	7.89	.97	1.16	820.00
840.00	10.70	1.78	2.58	9.70	1.46	1.98	8.84	1.21	1.55	8.09	1.02	1.22	840.00
860.00	10.95	1.86	2.70	9.93	1.53	2.08	9.05	1.27	1.62	8.28	1.06	1.28	860.00
880.00	11.20	1.95	2.82	10.16	1.60	2.18	9.26	1.33	1.70	8.47	1.11	1.34	880.00
900.00	11.46	2.04	2.96	10.39	1.68	2.28	9.47	1.39	1.78	8.66	1.16	1.40	900.00
920.00				10.62	1.75	2.38	9.68	1.46	1.86	8.86	1.22	1.46	920.00
940.00				10.86	1.83	2.49	9.89	1.52	1.94	9.05	1.27	1.53	940.00
960.00				11.09	1.91	2.59	10.10	1.58	2.02	9.24	1.33	1.59	960.00
980.00				11.32	1.99	2.70	10.31	1.65	2.11	9.43	1.38	1.66	980.00
1000.00				11.55	2.07	2.81	10.52	1.72	2.19	9.63	1.44	1.73	1000.00
1020.00							10.73	1.79	2.28	9.82	1.50	1.80	1020.00
1040.00							10.94	1.86	2.37	10.01	1.56	1.87	1040.00
1060.00							11.15	1.93	2.46	10.21	1.62	1.95	1060.00
1080.00							11.36	2.00	2.56	10.40	1.68	2.02	1080.00
1100.00							11.57	2.08	2.65	10.59	1.74	2.09	1100.00
1120.00										10.78	1.80	2.17	1120.00
1140.00										10.98	1.87	2.25	1140.00
1160.00										11.17	1.94	2.33	1160.00
1180.00										11.36	2.00	2.41	1180.00
1200.00										11.55	2.07	2.49	1200.00

NOTES

Based on formula $V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}}$.
A = Area of pipe in square feet.
V = Velocity in feet per second
h_v = Velocity head in feet
n = Coefficient of roughness.
r = Hydraulic radius in feet
s = Friction loss in feet per foot.
H = Friction loss in feet per thousand feet.

FRICTION LOSSES FOR CIRCULAR CONDUITS
MANNING'S FORMULA n=.013
120", 126", 132", AND 138", DIAMETERS

Sheet 9 of 9

coefficient for pipe as related to this publication is n = 0.013, except for corrugated-metal pipe where n = 0.024 is assumed. Except for corrugated-metal pipe, the friction loss for all pipe flowing full can also be determined by multiplying the friction loss per 1,000 feet (table 8-1) by 0.001 times the number of feet of pipe.

Entrance and exit losses for pipe structures having concrete inlet and outlet transitions are discussed in section 7-3.

For pipe structures without concrete transitions, entrance and exit losses for a submerged condition are assumed to be $0.5h_{v_p}$ and $1.0h_{v_p}$, respectively, where h_{v_p} is the pipe velocity head for pipe flowing full.

Bend losses can be determined by using the formula:

$$h_b = \zeta \frac{V^2}{2g}$$

The value of Hind's ζ (zeta) can be obtained from figure 8-1.

B. PIPE

8-3. ***Precast Reinforced Concrete Pressure Pipe (figs. 2-5, 2-6, 8-2, and 8-3).***—Precast reinforced concrete pressure pipe (PCP) is used for conveying canal water or storm runoff water under roadways, railroads, other canals, natural drains, channels, depressions, down steep terrain to a lower elevation, and for turnouts through canal banks. All precast concrete pipe which must operate under hydrostatic pressure or pipes used for turnouts, chutes, and cross drain culverts under canals should be PCP which has watertight rubber gasket joints to prevent leakage.

Pipe structures using PCP are limited to combination loadings of hydrostatic heads measured to the centerline of the pipe not exceeding 150 feet with earth cover over the top of the pipe (fill) not exceeding 20 feet. Table 1 in the Standard Specifications [2] shows designs for pipe, 12 through 108 inches in diameter, that will meet requirements for various loading combinations of head and fill. A special design is required for fills exceeding 20 feet. The use of PCP is limited to a maximum head of 150 feet.

8-4. ***Precast Reinforced Concrete Culvert Pipe.***—Precast reinforced concrete culvert pipe (RCCP) is used for conveying canal water or storm runoff water under roadways and railroads. The pipe design may be determined from tables provided by a pipe manufacturer. A typical set of tables is provided in bibliography reference [3].

8-5. ***Corrugated-metal Pipe (figs. 8-4 and 8-5).***—Corrugated-metal pipe (CMP) fabricated from either galvanized steel or aluminum alloy is used for conveying canal water or storm runoff water under roadways and railroads, and also for conveying storm runoff water through canal banks into canals. In use this pipe should have little or no internal hydrostatic pressure. The required gage of CMP for a combination of given height of fill and surcharge load from vehicles may be determined from tables provided by a pipe manufacturer. A typical set of tables is provided in bibliography references [4] and [5].

8-6. ***Asbestos-cement Pressure Pipe.***—Asbestos-cement pressure pipe (AC) may be used as an alternative to PCP pipe for conveying canal water or storm runoff water. This pipe has a connecting collar with rubber gaskets and may be used for hydrostatic heads up to 675 feet in combination with fills up to 20 feet. The selection table for pipe, 4 through 42 inches in diameter, in the Standard Specifications [6] shows the class of pipe required to withstand the loadings imposed for various combinations of head and fill.

8-7. ***Reinforced Plastic Mortar Pressure Pipe.***—Reinforced plastic mortar pressure pipe (RPM) may be used as an alternative to PCP pipe for conveying canal water or storm runoff water. This pipe is composed of resin, sand,

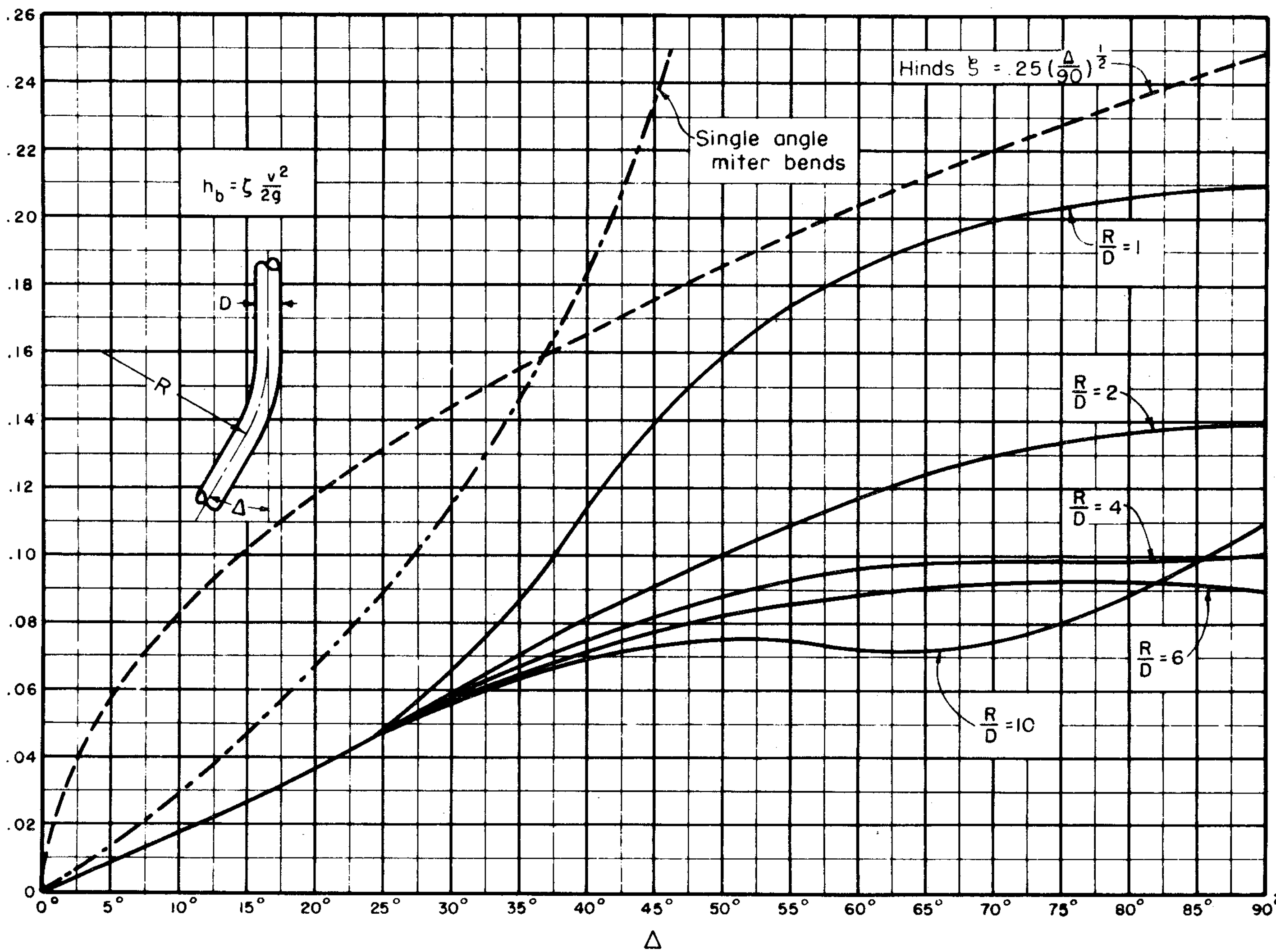


Figure 8-1. **Head loss coefficients for pipe bends. 103-D-1317**

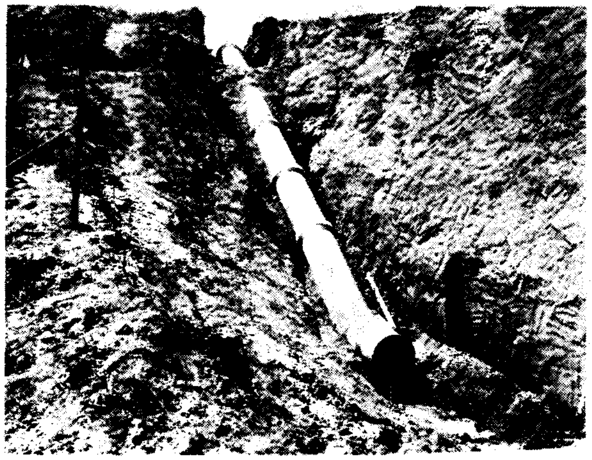


Figure 8-2. Precast reinforced concrete pressure pipe in trench prior to backfilling. P-271-701-3819

Figure 8-3. Precast reinforced concrete pressure pipe used for inverted siphon. P-328-701-8498

and fiberglass reinforcement and may be used for heads up to 450 feet in combination with fills not exceeding 15 feet.

The selection table for pipe, 8 through 48 inches in diameter, included with the standard paragraphs for specifications [7] shows the class of pipe required to withstand various combinations of head and fill.

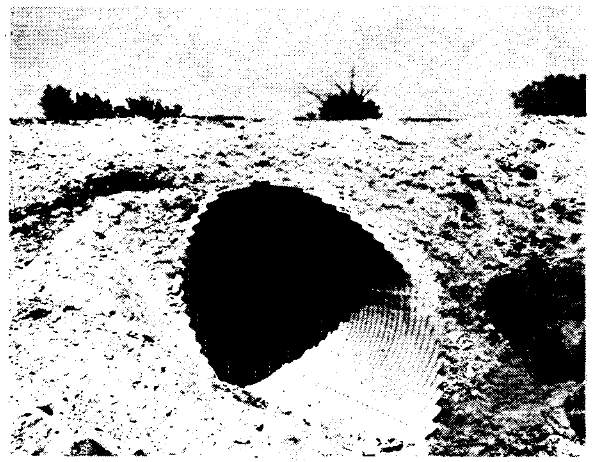


Figure 8-4. Corrugated-metal pipe used for road crossing. P-796-701-1548

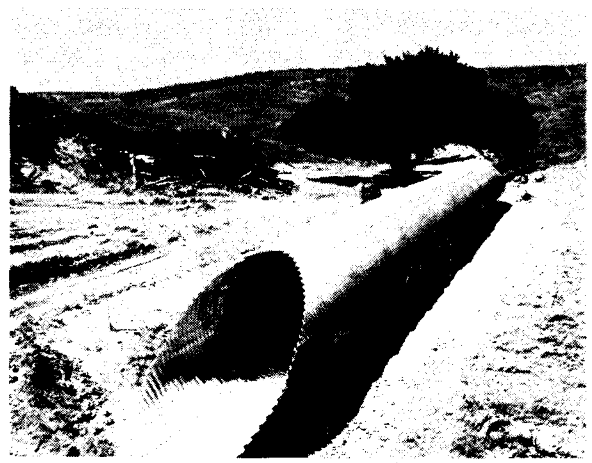


Figure 8-5. Installation of corrugated-metal-pipe road crossing. P-328-701-8759

8-8. ***Welded Steel Pipe (figs. 8-6 and 8-7).***—Welded steel pipe (WSP) is used for carrying canal or storm runoff water over other structures or natural features. Steel pipe is usually used for this purpose because lengths of pipe may be welded together to form a continuous length of pipe with sufficient strength and watertightness for spanning between supports. Stiffeners may or may not be required to obtain the required beam strength.

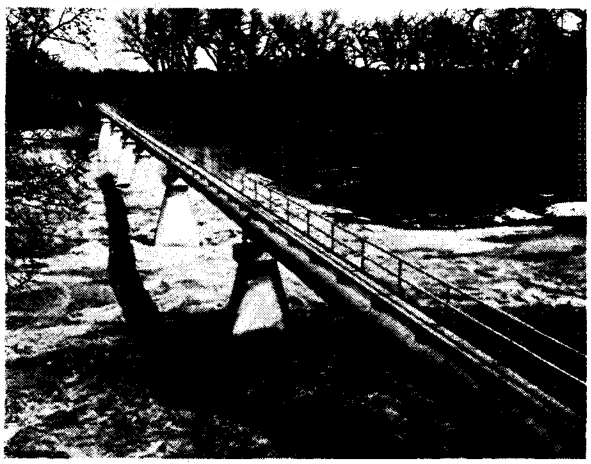


Figure 8-6. Welded steel pipe flume. P-271-701-3855

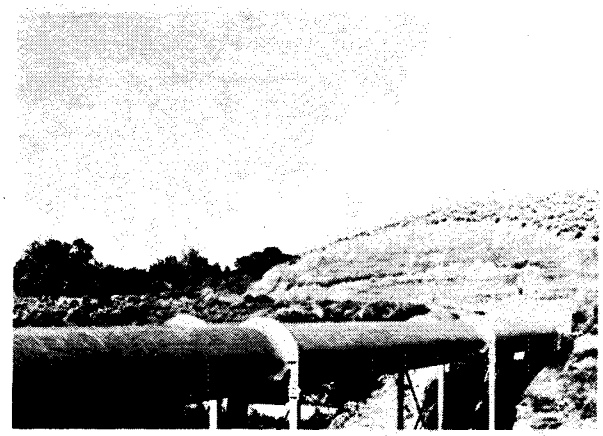


Figure 8-7. Overhead welded steel pipe crossing. P-31-D-22581

The required gage of WSP for a given span length and internal water pressure may be conveniently determined from tables and other information provided by the pipe manufacturer. A typical set of tables is provided in bibliography reference [8].

8-9. *Other Pipe.*—Generally reinforced monolithic concrete pipe, pretensioned concrete pipe, or prestressed concrete pipe are not economical when used for structures discussed in this publication, therefore their design has been omitted.

C. PIPE APPURTENANCES

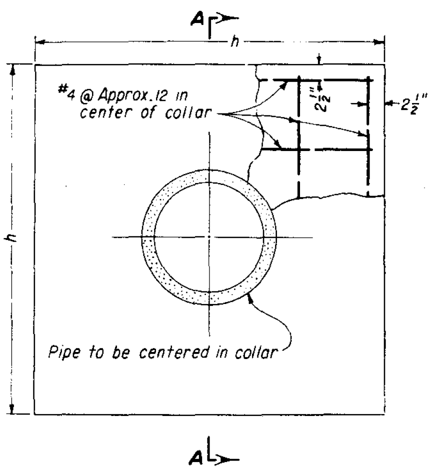
8-10. *Pipe Collars and Percolation.*—Pipe collars are transverse fins (figs. 2-23, 4-19, 4-22, and 8-8) that extend from the pipe into the surrounding earth and function as barriers to percolating water and burrowing rodents.

Collars are often needed to reduce the velocity of water moving along the outside of pipe or through surrounding earth thereby preventing removal of soil particles (piping) at the point of emergence. For any pipe structure where the inlet water surface is significantly higher than a potential point of relief for the percolating water (percolation gradient through earth of 5 to 1 or steeper), the structure should be examined to determine the need for pipe collars.

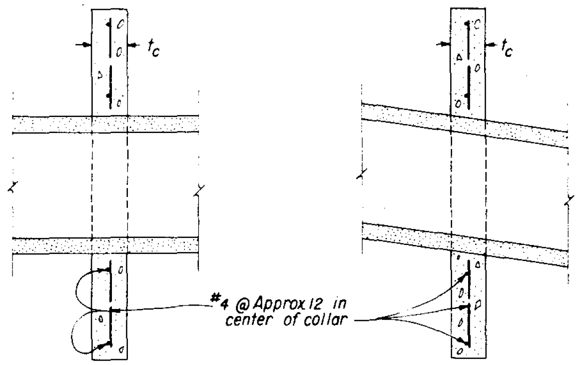
Lane's weighted creep method [9] is commonly used for percolation path studies related to canal structures. Using this method, the length of weighted creep (weighted path) necessary to prevent piping is directly related to the head of water and the type of soil. Recommended weighted-creep ratios (percolation factors) vary with the type of soil due to the soils resistance to percolating water. Lane's weighted-creep ratio is the weighted-creep length divided by the effective head. Effective head is the difference in water surface elevations at the beginning of path and point of relief, and weighted-creep length is the sum of: (1) vertical path distance along the structure (steeper than 45°), (2) one-third of the horizontal path distance along the structure (flatter than 45°), and (3) two times any percolation path distance that short-cuts through the soil.

$$\begin{array}{c}\text{weighted-creep ratio}\\ \text{(percolation factor)}\end{array} = \frac{\text{weighted-creep length}}{\text{difference in water surface elevations}}$$

The weighted-creep ratio should be at least 2.5 to 1 as computed by Lane's weighted-creep method. Larger ratios may be required where warranted by the type of soil or importance and the type of structure.

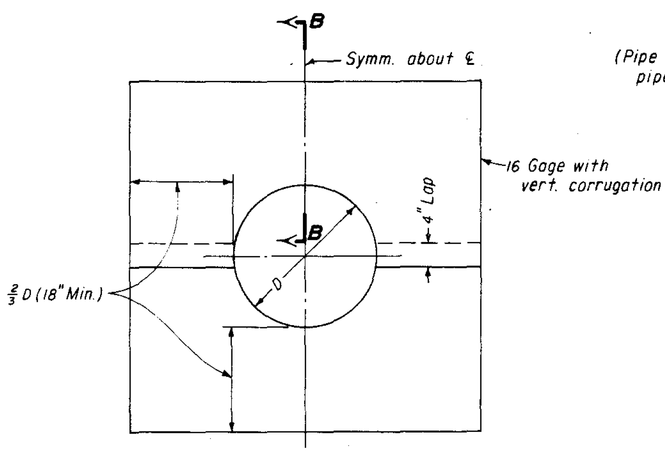


ELEVATION
(Pipe collar for precast concrete pipe, asbestos-cement pipe and reinforced plastic mortar pipe)



LEVEL PIPE SLOPED PIPE

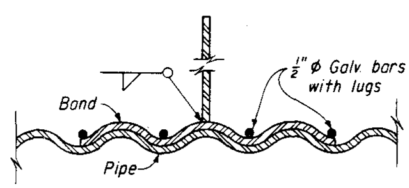
SECTION A-A



ELEVATION
(Pipe collar for corrugated metal pipe)

DIMENSIONS AND ESTIMATED QUANTITIES
(Pipe collar for precast concrete pipe, asbestos-cement pipe and reinforced plastic mortar pipe)

PIPE DIA.	h	t_c	CONC. CU.YD.	REINF. LBS.
18"	5'-0"	6"	0.4	33
21"	5'-3"	6"	0.4	35
24"	5'-6"	6"	0.5	44
27"	6'-3"	6"	0.7	50
30"	7'-0"	6"	0.8	62
36"	8'-6"	6"	1.2	96
42"	9'-3"	8"	1.8	100
48"	10'-0"	8"	2.2	120
54"	11'-6"	8"	2.7	170
60"	12'-0"	8"	2.9	180



SECTION B-B

Figure 8-8. **Pipe collars. 103-D-1318**

Lane's recommended weighted-creep ratios are:

Material	Ratio
Very fine sand or silt	8.5:1
Fine sand	7.0:1
Medium sand	6.0:1
Coarse sand	5.0:1
Fine gravel	4.0:1
Medium gravel	3.5:1
Coarse gravel including cobbles	3.0:1
Boulders with some cobbles and gravel	2.5:1
Soft clay	3.0:1
Medium clay	2.0:1
Hard clay	1.8:1
Very hard clay or hardpan	1.6:1

If the computed weighted-creep ratio is less than that recommended, collars should be added to the pipe structure.

Cross-drainage structure pipe under a canal requires at least one collar around the pipe in the uphill bank and two collars around the pipe in the downhill bank. Two collars are required around the pipe in the upper bank if the invert elevation at the inlet cutoff is below the bottom grade of the canal. See figures 4-26 and 4-28 and chapter IV for locating collars on cross-drainage structures.

8-11. ***Pipe Bends*** *(fig. 8-9)*.—(a) *Purpose and Design Considerations.*—Pipe bends are used both for horizontal changes in pipe alinement and for vertical changes in grade.

Bends in pipe cause directional changes in the flow of water which results in dynamic and static thrusts at the bends.

Dynamic thrust for most pipe structures can be omitted because of the relatively slow design velocities. Static thrust for low heads is also usually insignificant. However, for relatively high heads, the static thrust is significant and stability of the bend should be investigated. Thrust on horizontal bends and on vertical bends which are concave (__/) is resisted by soil pressure. If allowable soil-bearing capacity for these vertical bends or allowable passive pressure for these horizontal bends is exceeded, thrust blocks are provided usually by encasing the bend in concrete. The thrust is thereby spread over a greater surface area resulting in reduced soil pressures. For horizontal bends, resistance to sliding developed by friction is also sometimes included in the bend stability computations. A sliding coefficient of 0.35 is frequently used. For vertical bends which are convex (¯¯\), thrust forces tend to push the pipe out of the ground. Resisting these forces are the weight of the full pipe and the earth cover. Earth cover is usually omitted in the stability computations as such cover may be removed or washed away.

Figure 8-9. **Forms for thrust block on 54-inch-diameter precast concrete pipe. P-499-700-519**

The total static thrust (T) on a bend can be computed by

$$T = 62.4\ H\ A \sin\left(\frac{\Delta}{2}\right) 2$$

where:

T = thrust, in pounds
H = hydrostatic head to centerline of pipe
A = area of pipe opening in square feet
Δ = deflection angle of bend in degrees
62.4 = weight in pounds of a cubic foot of water

For a vertical convex upward bend, the thrust is resisted by the dead weight of the full pipe for a length extending between joints at each end of the bend. If additional weight is required for equilibrium, the pipe bend is usually encased in a concrete thrust block.

Pipe bend energy losses can be determined by using the equation: $h_b = \zeta \frac{V^2}{2g}$ as was previously explained in section 8-2.

(b) *Bends for Precast Concrete Pipe.*—Changes in alinement or grade of precast concrete pipe may be made by one of the following methods:

(1) *Banded mitered pipe.*—Bends formed with mitered pipe are commonly used for pipe structures in small canals. These bends are fabricated by cutting straight pipe units to one-half the required deflection angle and mortaring the two pipe units together as shown on figures 8-10 and 8-11. The class of pipe used in fabricating the bends should not be less than the class of pipe in the adjacent pipe units.

It is often necessary to provide additional weight when using miter bends because of static thrust as previously discussed. If concrete encasement is used with pipe having rubber gasket joints, the distance from the face of the encasement to the pipe joint should be limited to 18 inches for pipe up to 36 inches in diameter and one-half the diameter for pipe larger than 36-inch diameter.

(2) *Bevel-end pipe.*—Bends may also be formed using beveled-end pipe. The deflection of each bevel should not exceed 5°. This limitation permits the use of circular joint-forming rings at the slightly elliptical (beveled) end of the pipe.

(3) *Precast elbows.*—Precast elbows are sometimes used for fabricating pipe bends as shown on figure 8-11. The units may either be curved or contain an angle such that the ends of the elbow units are normal to the longitudinal axis. Elbows should be the same class of pipe with the same type of joint as the adjacent pipe.

(4) *Pulled joints.*—Bends of long-radius may be produced by deflecting (pulling) a sufficient number of pipe units at their joints, provided that the alinement or profile will permit this procedure. A full-laying length of pipe should be used on both sides of each joint.

Where tongue-and-groove mortar joint pipe is laid on a long-radius curve, the joint opening at any point on the inside circumference should not be more than one-half inch wider than a normal joint.

Where pipe with type R-1 joints is laid on a long-radius curve, the distance A (fig. 1 in bibliography reference [2]) should not be less than one-fourth inch.

Pipe with type R-2 joints may be deflected an amount such that distance A (fig. 2 in bibliography reference [2]) is not less than one-eighth inch; however, the grout space on the outside of the pipe should not be less than one-half inch wide and the mortar space on the inside of the pipe should not be less than one-fourth inch wide.

Type R-3 and R-4 joints may be pulled on one side of the pipe so that the joint opening on that side of the pipe is not more than one-half inch wider than the joint opening on the opposite side of the pipe provided that the distance A (fig. 3, 4, 5, and 6 in bibliography reference [2]) is not less than one-fourth inch at any point on the circumference of the joint.

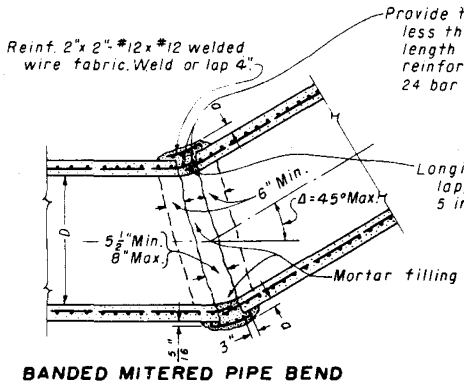
(c) *Bends for Asbestos-cement Pipe.*—Methods used for changes in alinement or grade of asbestos-cement pipe include the following:

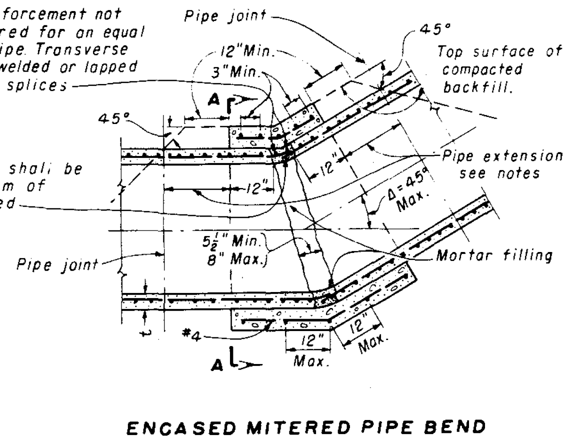
(1) *Mitered pipe.*—Bends may be fabricated by joining mitered asbestos-cement pipe with an approved type of cement and encasing the bend in concrete as shown on figure 8-12. The stability of the bend for static thrust may require additional concrete encasement. The pipe extension beyond the concrete encasement should be limited to 18 inches for pipe up to 36 inches in diameter and one-half the diameter for pipe larger than 36-inch diameter.

The miter bend shown on figure 8-13 may also be used as shown or with cast-iron fittings substituted for the mortar-lined steel pipe.

(2) *Pulled joints.*—Long-radius bends formed by deflecting (pulling) joints may be used where pipe alinement or profile will permit this procedure. The deflection angle between adjacent pipe units should not

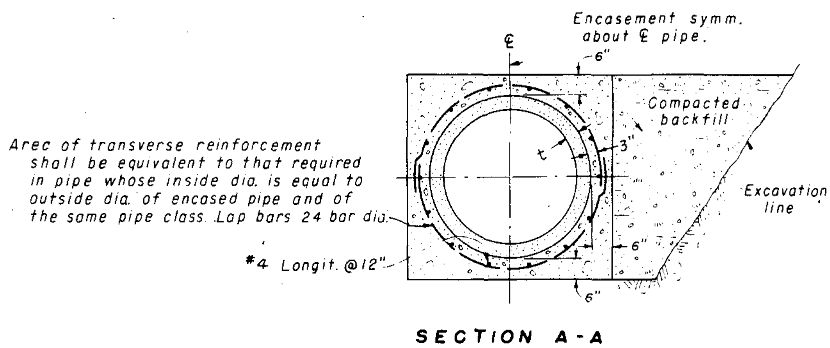
(continued on page 371)





BAND THICKNESS DATA

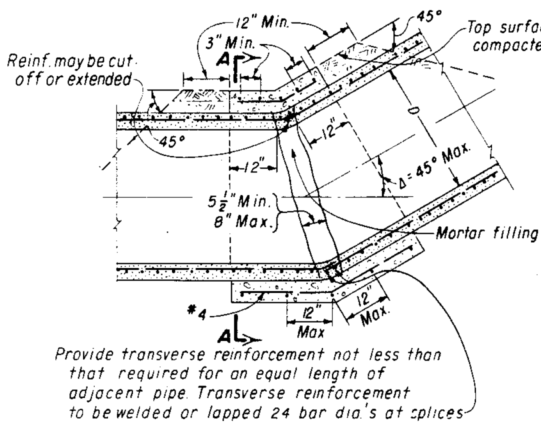
D INCHES	a (MIN.) INCHES
12	1
15	1
18	1
21	1
24	1
27	1
30	1
33	1 1/8
36	1 1/8
39	1 1/4
42	1 3/8
45	1 1/2
48	1 1/2
51	1 3/4
54	1 3/4
57	1 3/4
60	1 3/4
63	2
66	2
69	2
72	2



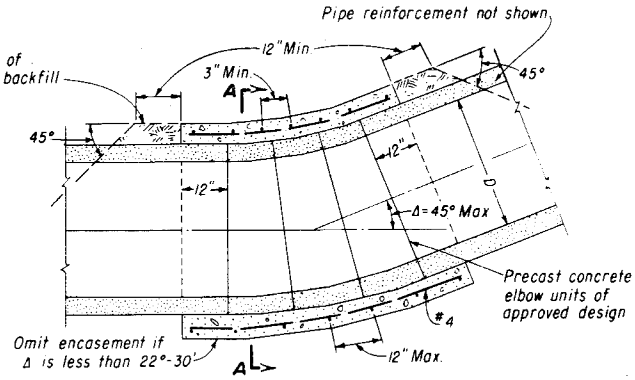
NOTES

Compact backfill to top of encasement at all bends.
Elevation of compacted backfill for banded mitered pipe bends shall be the same as the elevation of compacted backfill specified for the adjacent pipe.
Pipe extension shall be 18" max. when $D \leq 36"$ and $\frac{1}{2}$ D max. when $D > 36"$.
This figure shows two acceptable ways in which pipe bends may be fabricated. The stability of the bend in resisting hydrostatic thrust must be checked. If the thrust is great enough to make these bends unstable, the encasement dimensions of the encased bend may be increased for stability.

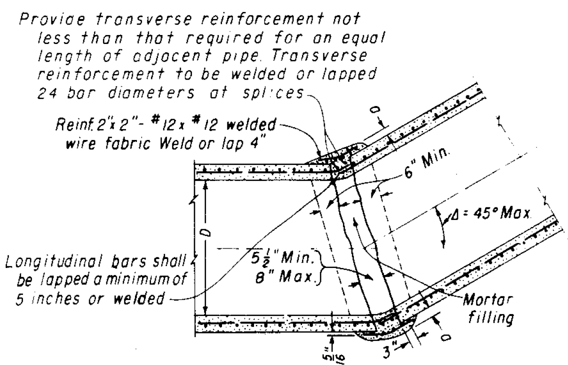
Figure 8-10. **Precast concrete pressure pipe bends. 103-D-1319**



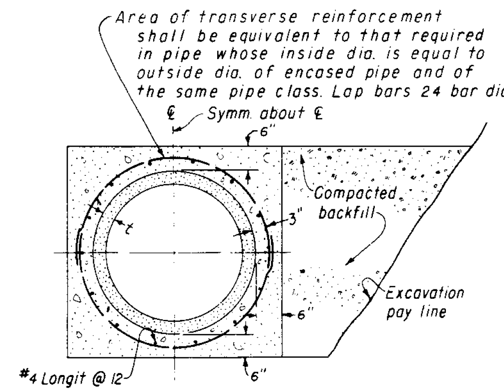
ENCASED MITERED PIPE BEND



ENCASED PRECAST ELBOW BENDS



BANDED MITERED PIPE BEND



SECTION A-A

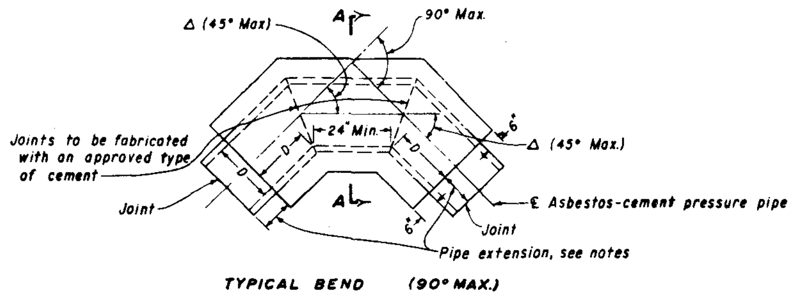
BAND THICKNESS DATA

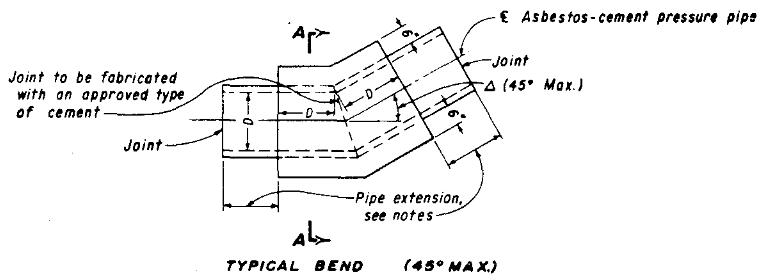
D INCHES	a (MIN.) INCHES	D INCHES	a (MIN.) INCHES
12	1	45	$1\frac{1}{2}$
15	1	48	$1\frac{1}{2}$
18	1	5.	$1\frac{3}{4}$
21	1	54	$1\frac{3}{4}$
24	1	57	$1\frac{3}{4}$
27	1	60	$1\frac{3}{4}$
30	1	63	2
33	$1\frac{1}{8}$	66	2
36	$1\frac{1}{8}$	69	2
39	$1\frac{1}{4}$	72	2
42	$1\frac{3}{8}$		

NOTES

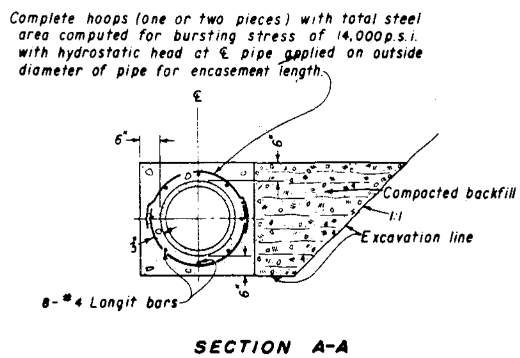
Elevation of compacted backfill for unencased elbow bends and banded pipe bends shall be the same as the elevation of compacted backfill specified for the adjacent pipe.
Compact backfill to top of concrete at all encased bends.
This figure shows three acceptable ways in which pipe bends may be fabricated.

Figure 8-11. **Precast concrete culvert pipe bends. 103-D-1320**





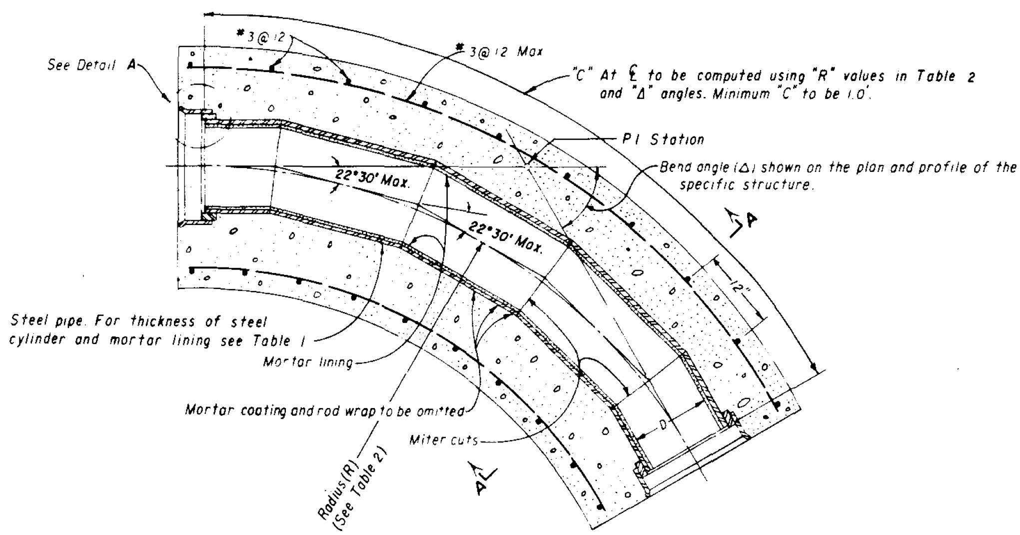
HORIZONTAL OR VERTICAL BENDS



NOTES

Compact backfill to top of encasement at all bends.
Pipe extension shall be 18" max. when D ≤ 36" and ½ D max. when D > 36". Min. extension shall be 8".
For alternative bend fabrication, see Figure 8-13.
Details shown are minimum bend fabrication requirements.
The stability of the bends in resisting hydrostatic thrust must be checked. If the thrust is great enough to make these bends unstable, the encasement dimensions may be increased for stability.

Figure 8-12. Asbestos-cement pressure pipe bends (maximum head 175 feet). 103-D-1321



LONGITUDINAL SECTION
ENCASED HORIZONTAL AND VERTICAL BEND WITH REINFORCEMENT
(OMIT REINFORCEMENT FOR HEADS 50' AND LESS)

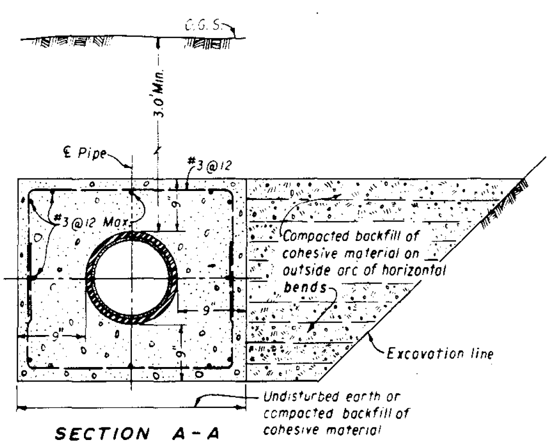
TABLE 1
STEEL PIPE AND MORTAR LINING

D INCHES	STEEL PIPE WALL THICKNESS	LINING THICKNESS F INCHES	LINING THICKNESS G INCHES	D INCHES	STEEL PIPE WALL THICKNESS	LINING THICKNESS F INCHES	LINING THICKNESS G INCHES
12	10 GA.	1/4	1/4	27	1/4"	1/2	1/2
14	10 GA.	3/8	3/8	30	1/4"	1/2	1/2
15	10 GA.	3/8	3/8	33	1/4"	1/2	1/2
16	10 GA.	3/8	3/8	36	5/16"	1/2	1/2
18	3/16"	3/8	3/8	39	5/16"	1/2	1/2
20	3/16"	1/2	1/2	42	5/16"	1/2	1/2
21	3/16"	1/2	1/2	45	3/8"	1/2	3/4
24	3/16"	1/2	1/2	48	3/8"	1/2	3/4

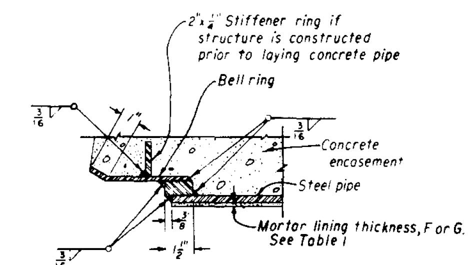
For spun linings or machine applied in-place linings, use "F" for thickness of mortar lining.
For hand applied or pneumatically applied linings, use "G" for thickness of mortar lining.
Hand applied or pneumatically applied linings shall be reinforced with 2 x 4 x 13 gage welded wire fabric for sizes greater than 42".

TABLE 2

MAX. HEAD FEET	MIN. RADIUS (R) HORIZ. BENDS & VERT. BENDS CONCAVE UP	MIN. RADIUS (R) VERT. BENDS CONVEX UP
50	1.5 D	3.5 D
100	3.0 D	6.5 D
150	4.5 D	10.0 D
200	6.0 D	13.0 D
250	7.5 D	16.5 D
300	9.0 D	19.5 D
350	10.5 D	23.0 D
400	12.0 D	26.0 D
450	13.5 D	29.5 D



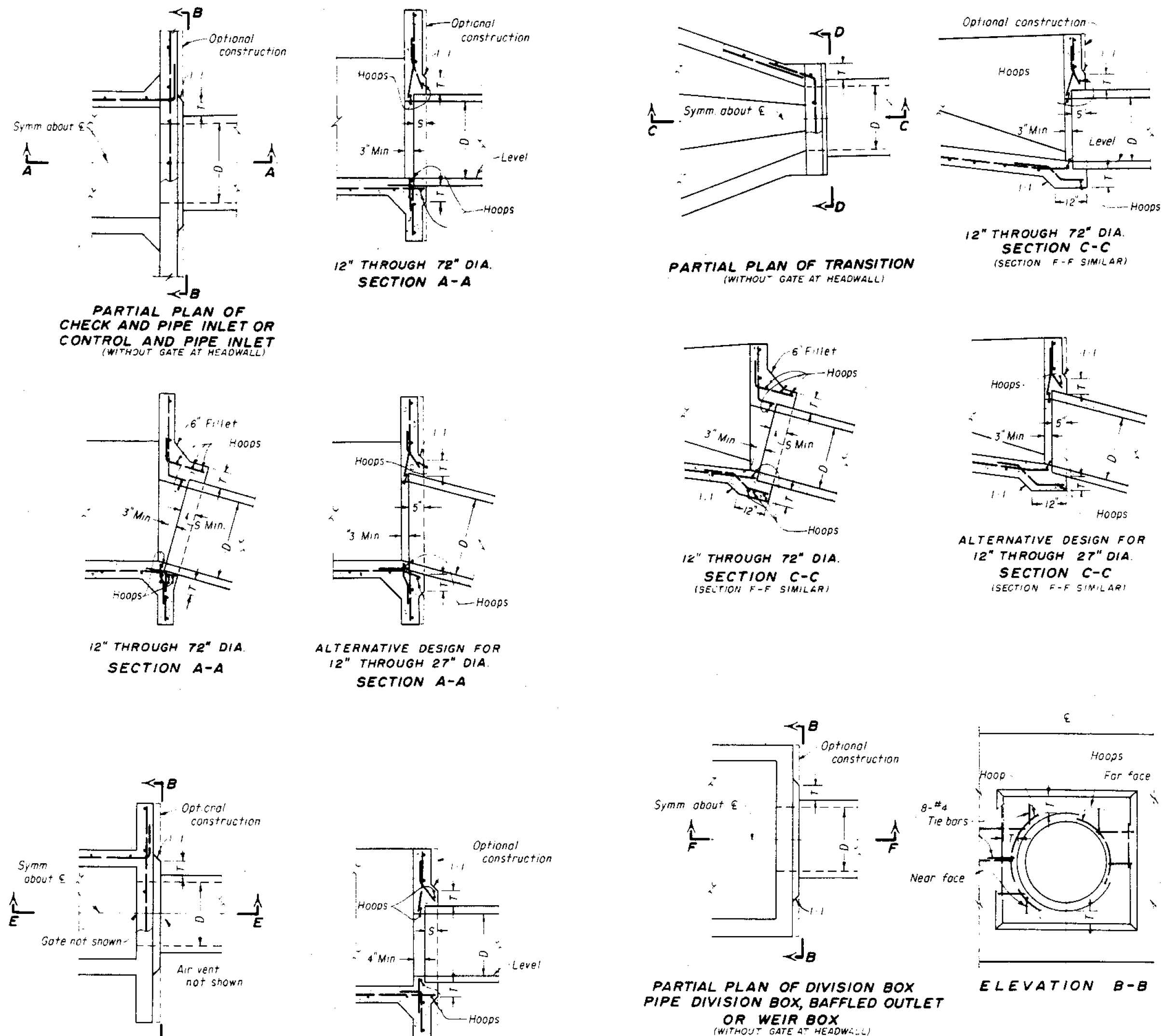
SECTION A-A

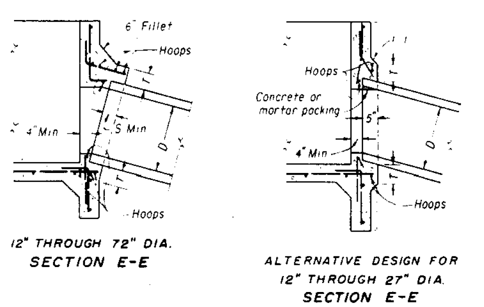


DETAIL A
BELL TO SUIT LINE PIPE FURNISHED

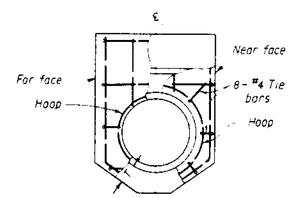
NOTE
For optional asbestos-cement pressure pipe bends, see figure 8-12.

Figure 8-13. **Asbestos-cement pressure pipe and reinforced plastic mortar pressure pipe bends (maximum head 450 feet). 103-D-1322**





D INCHES	S INCHES	T INCHES	HOOP SIZE #
12	4	6	4
15	4	6	4
18	4	6	4
21	4	6	4
24	4	6	4
27	4	6	4
30	4	6	4
33	4	6	4
36	4	6	5
39	$4\frac{1}{2}$	6	5
42	$4\frac{1}{2}$	6	5
45	5	6	5
48	5	6	5
51	$5\frac{1}{2}$	6	5
54	$5\frac{1}{2}$	6	5
57	6	6	6
60	6	6	6
63	$6\frac{1}{2}$	7	6
66	$6\frac{1}{2}$	7	6
69	7	7	6
72	7	7	6



NOTE

Where pressure pipe is used, a pipe joint shall be placed within 18" from outside face of structure for pipe dia $\leq 36"$ and within $\frac{1}{2}$ dia for pipe dia $> 36"$

Figure 8-14. **Precast concrete pipe embedment (asbestos-cement pressure and reinforced plastic pressure pipe similar). 103-D-1323** exceed 5° for 18-inch-diameter pipe and smaller, 4° for 21-inch-diameter pipe, and 3° for 24-inch-diameter pipe and larger. A full-laying length of pipe should be used on both sides of each joint to insure stability against static thrust.

(d) *Bends for Reinforced Plastic Mortar Pipe.*—Changes in alinement or grade may be made by one of the following methods:

(1) *Mitered pipe.*—Miter bends may be fabricated and encased in concrete as shown on figure 8-13.

(2) *Pulled joints.*—Long-radius bends may be formed where alinement or profile will permit by deflecting (pulling) joints a small amount. The joints may be pulled on one side of the pipe so that the joint opening on that side will not be more than one-half inch wider than the joint opening on the other side; however, the distance A (fig. 1 in bibliography reference [7]) should not be less than one-fourth inch at any point in the circumference of the joint. A full-laying length of pipe should be used on each side of the joint to provide stability for static thrust.

(e) *Bends for Corrugated-metal Pipe.*—Changes in alinement or grade of corrugated-metal pipe may be made with standard prefabricated elbow fittings of the same gage as the adjacent pipe. Encasement in concrete may be required to provide bend stability for static thrust.

8-12. ***Pipe Embedment.***—Figure 8-14 shows connections between precast concrete pipe and adjoining structures. To insure good bond between the concrete structure and the precast concrete pipe, the end of the pipe to be embedded should be wire-brushed to remove all laitance and foreign matter and then wetted prior to placing concrete.

The alternative embedment for sloping pipe as shown on the figure is restricted to diameters 27 inches and less. The alternative allows pipe of this size range to be miter-cut and is restricted, by usual practice, to a maximum diameter of 27 inches. This diameter restriction is not applicable if other than precast concrete pipe is used.

When pipe having a rubber gasket joint is used, the pipe joint should be placed within 18 inches from the face of the embedment for 36-inch pipe and smaller and within one-half the pipe diameter for pipe larger than 36 inches. This provision allows small movements, due to unequal settlement of the structure and the pipe, to be absorbed in the joint and minimizes the possibility of cracking the pipe or the pipe embedment.

Embedment of corrugated-metal and welded steel pipe may be similar to that shown on the figure, or the pipe may extend through the headwall with the pipe end flush with the face of the headwall. Steel pipe laid on a steep slope will usually have a steel ring welded to the pipe near the end that will be embedded in the concrete and will provide pipe anchorage with the concrete.

Embedment for a gated structure requires a vertical step in the floor at the headwall as shown on the figure to accommodate installation of the gate frame.

8-13. ***Pipe Joints.***—(a) *Purpose and Design Considerations.*—Pipe joints are used to connect individual pipe units to form a continuous pipeline for a pipe structure. Careful consideration must be given to the type of joint permitted so that the pipe structure will function as intended without unnecessary maintenance. Considerations should include joint leakage from hydrostatic pressure; joint flexibility, especially where some uneven foundations settlement may reasonably be expected; relative importance for the safety of a structure under which the pipe may be placed; exposure of the joint to cold water in the spring months which may cause contraction cracking; and exposure to atmospheric temperature changes which cause contraction-expansion movement.

(b) *Joints for Precast Reinforced Concrete Pressure Pipe.*—These joints use solid rubber gaskets, circular in cross section, and are the only element in the joint to provide watertightness. These joints also provide for slight movements in each pipe unit of the pipe structure which may be caused by foundation settlement, expansion, contraction, or lateral displacement. The joint assemblies should form a continuous watertight conduit having a smooth and uniform interior surface.

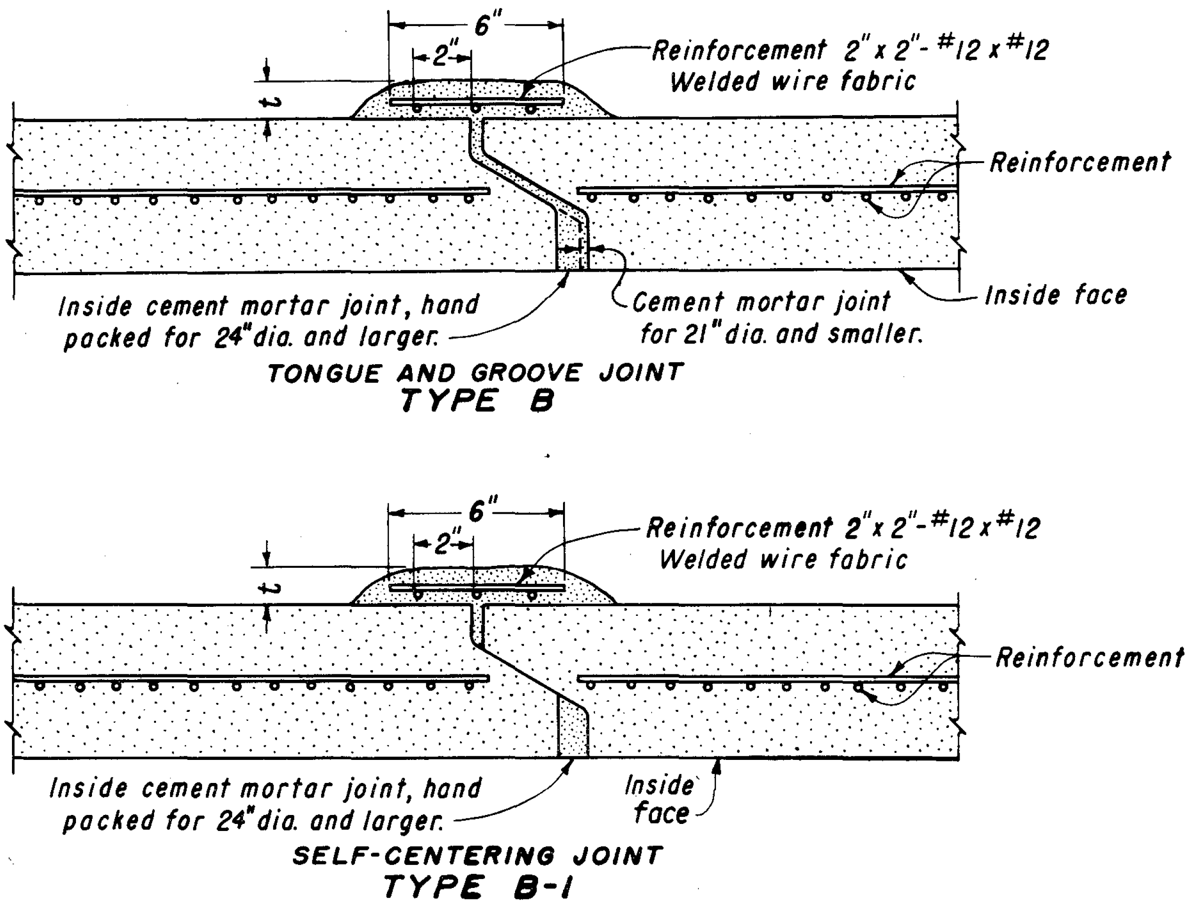
Rubber gasket joints for precast reinforced concrete pressure pipe may be type R-1, type R-2, type R-3, or type R-4 (figures 1, 2, 3, 4, 5, and 6, in bibliography reference [2]).

Type R-2 is used for installations where the pipe is to be jacked. Some cushioning between the joints will reduce the likelihood of breakage due to concentrated pressure. This cushioning can be provided by a piece of 1/2-inch manila rope stuck to the pipe with asphaltic cement, strips of asphalt roofing paper, metal strips, or cement grout inserted around the circumference, in the joints, as each piece of pipe is placed ahead of the jacking form. After the pipe is in final position, the joint may be pointed from the inside with mortar or joint compound.

Requirements for all materials used in type R joints and other detailed requirements of these joints are given in bibliography reference [2], see sec. 8-19.

(c) *Joints for Culvert Pipe.*—Reinforced mortar-banded joints of type B or type B-1 (fig. 8-15) are perhaps the most commonly used joints for culvert pipe installations.

The ends of culvert pipe sections are formed to permit effective jointing which reduces



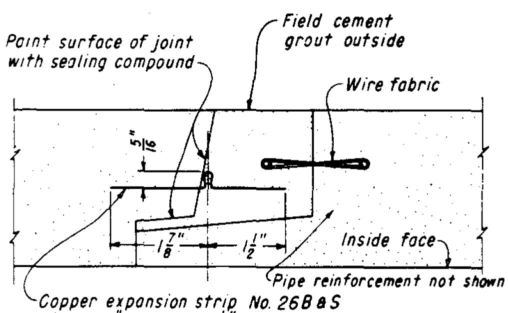
DIA OF PIPE	12" TO 20"	21" TO 24"	27" TO 33"	36" TO 45"	48" TO 51"	54" TO 60"	63" TO 96"
MINIMUM THICKNESS (t)	5/8"	3/4"	1"	1 1/4"	1 1/2"	1 3/4"	2"

Figure 8-15. Precast concrete pipe joints—types B and B-1. 103-D-1324

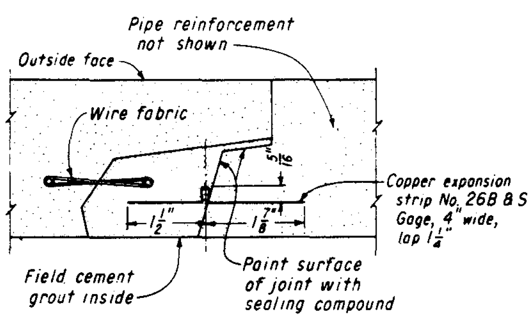
leakage and infiltration to a satisfactory minimum, and produces a continuous and uniform line of pipe.

Type F joints may be substituted for type B or type B-1 joints. The type F joint design and rubber gaskets for this joint should be in accordance with the American Society for Testing amd Materials [10] except that the taper on the tongue and groove should not be more than 3-1/2° measured from a longitudinal trace on the inside surface of the pipe.

Type D joints (fig. 8-16) are used if a copper-seal-type joint is specified where culvert pipe is to be jacked under a railroad or highway. The tongue and groove dimensions should provide an inside or outside annular joint space for filling with grout as shown on the figure. A copper expansion strip is embedded in the tongue or groove end of each pipe unit and protrudes into the annular joint space. The ends of the copper strip are



PIPE DIA.-21"OR SMALLER



PIPE DIA.-24"OR LARGER

Figure 8-16. **Precast concrete pipe joint–copper seal–type D. 103-D-1325**

overlapped 1-1/4 inches and soldered or brazed together along the edges of the lap. Copper for seals should be in accordance with Federal Specifications [11]. At the opposite end of each pipe unit, wire fabric is embedded in the concrete pipe and protrudes into the annular joint space.

(d) *Joints for Corrugated-metal Pipe.*—These joints are made with corrugated coupling (connecting) bands [4] [5] which are furnished by the pipe manufacturer.

(e) *Joints for Asbestos-cement Pressure Pipe.*—These joints are made with coupling sleeves having rubber gaskets circular in cross section. With these couplings, units of pipe may be joined to form a continuous watertight conduit with a smooth and uniform interior surface. The joint assemblies provide for slight movements which may be caused by expansion, contraction, settlement, or lateral displacement. Detailed requirements of these joints are given in bibliography reference [6], see sec. 8-19.

(f) *Joints for Reinforced Plastic Mortar Pipe.*—These joints also use solid, circular rubber gaskets to serve as the only element of the joint to provide watertightness. The joint assemblies form and provide a continuous watertight conduit with smooth and uniform interior surface and provide for slight movements due to expansion, contraction, settlement, or lateral displacement. Detailed requirements of these joints are given in bibliography reference [7], see sec. 8-19.

(g) *Joints for Welded Steel Pipe.*—These joints are made by welding or by using sleeve-type couplings [12] without pipe stops. These couplings provide for expansion-contraction and other slight pipe movement. Resilient gaskets in the couplings effect a flexible, watertight joint assembly.

8-14. *Pipe Safety Barrier.*—Pipe safety barriers made of barbed wire strung on a frame are placed on pipes crossing over waterways to discourage the use of the pipe as a pedestrian walkway. Figure 8-17 shows details of the barrier and a means of fastening it to steel or corrugated-metal pipe.

The need for a barrier is dependent on the accessibility of the site to the general public. Past experiences indicate that regardless of the

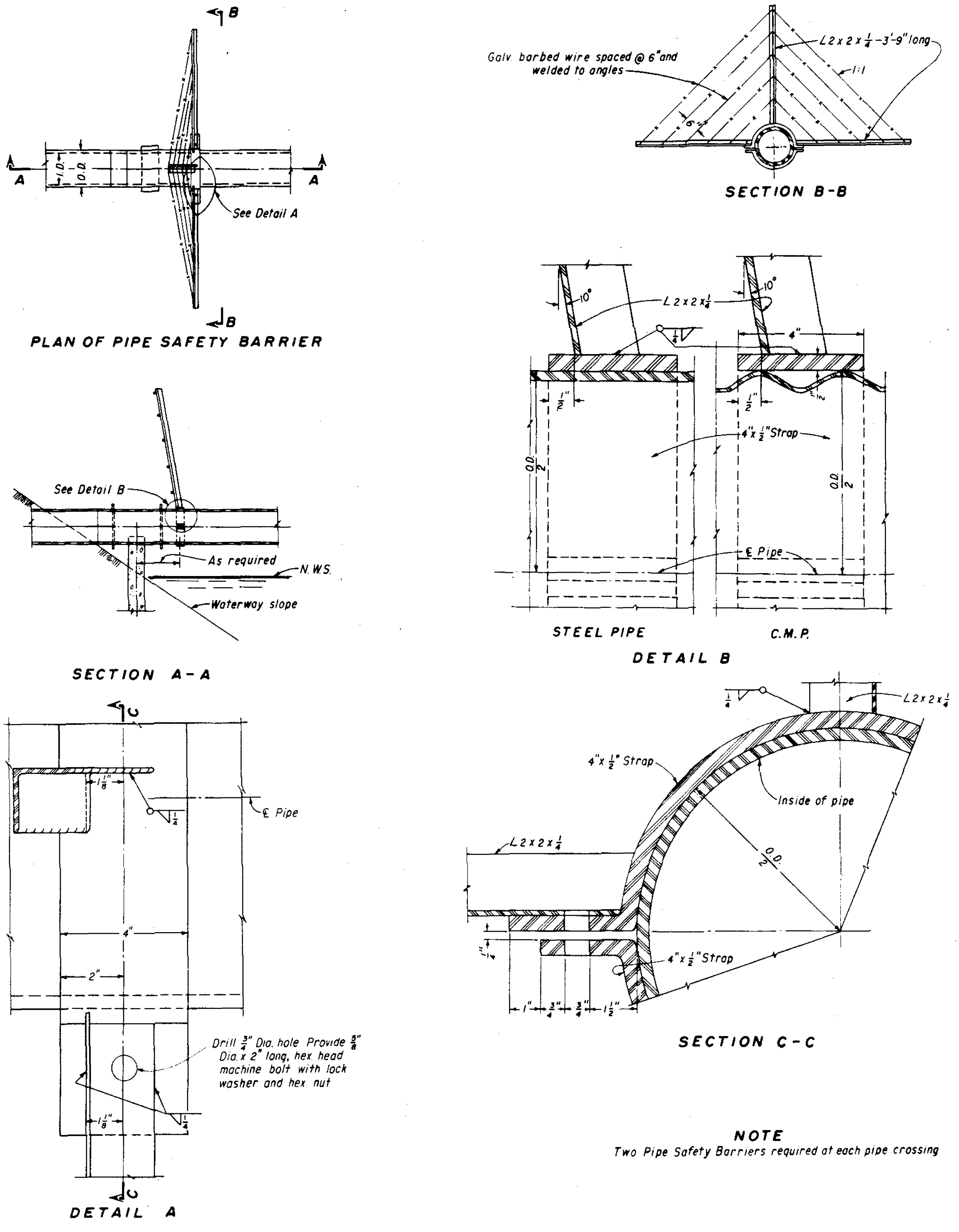


Figure 8-17. **Pipe safety barrier. 103-D-1326**

proximity of other pedestrian access over the waterway, a pipe barrier should be used on all pipe crossings except those in remote areas and for those crossing over fenced waterways.

8-15 ***Blowoff Structures (figs. 8-18 and 8-19).***—Blowoff structures are provided at or near the low point of relatively long inverted siphons to permit draining the pipe for inspection and maintenance or wintertime shutdown. These structures consist essentially of a valved steel pipe tapped into the siphon barrel. Blowoffs may also be used in an emergency in conjunction with wasteways for evacuating water from canals. Short siphons may be dewatered by pumping from either end of the siphon, and therefore, a blowoff is not required.

Details of blowoff installations vary with the size of the siphon pipe and the topography near the siphon low point. The drain arrangement associated with the blowoff installation is also determined by the topography. Where feasible, blowoff structures should be located to permit the water to flow directly into the natural drainage channel. A 6-inch blowoff is commonly used with siphon pipe to accommodate a reasonable time for draining. If annual wintertime draining is not required, breaking into pipe smaller than 24-inch diameter for emergency draining is an alternative to providing a blowoff.

A blowoff can be tapped into the siphon at or near the top of the pipe which will permit the draining of water from the pipe by gravity to the invert elevation of the blowoff discharge pipe. Draining can then be completed by pumping. A pipe drain may be tapped into the siphon near the bottom of the pipe which will permit draining the entire pipe by gravity.

To drain the entire pipe by gravity, the topography should permit the discharge pipe to be relatively short and laid on a steep slope without bends. This is essential to prevent clogging of the pipe.

Draining by the combination gravity and pump method is usually required. After the siphon has been drained by gravity down to the discharge pipe elevation, the blind flange is then removed from the vertical pipe and a pump suction line is inserted into the siphon, and the remaining water can then be drained by pumping.

The blowoff appurtenances should ordinarily be enclosed in a covered well extending above the adjacent original ground surface and protected by earthwork to near the top. Figure 8-18 shows a typical 6-inch blowoff installation tapped into the top of precast concrete pipe.

A manhole should be included with a blowoff on siphons one-half mile in length and longer and 36 inches and larger in diameter, to provide an intermediate access point for inspection and maintenance. A typical manhole and blowoff combination structure is shown on figure 8-19.

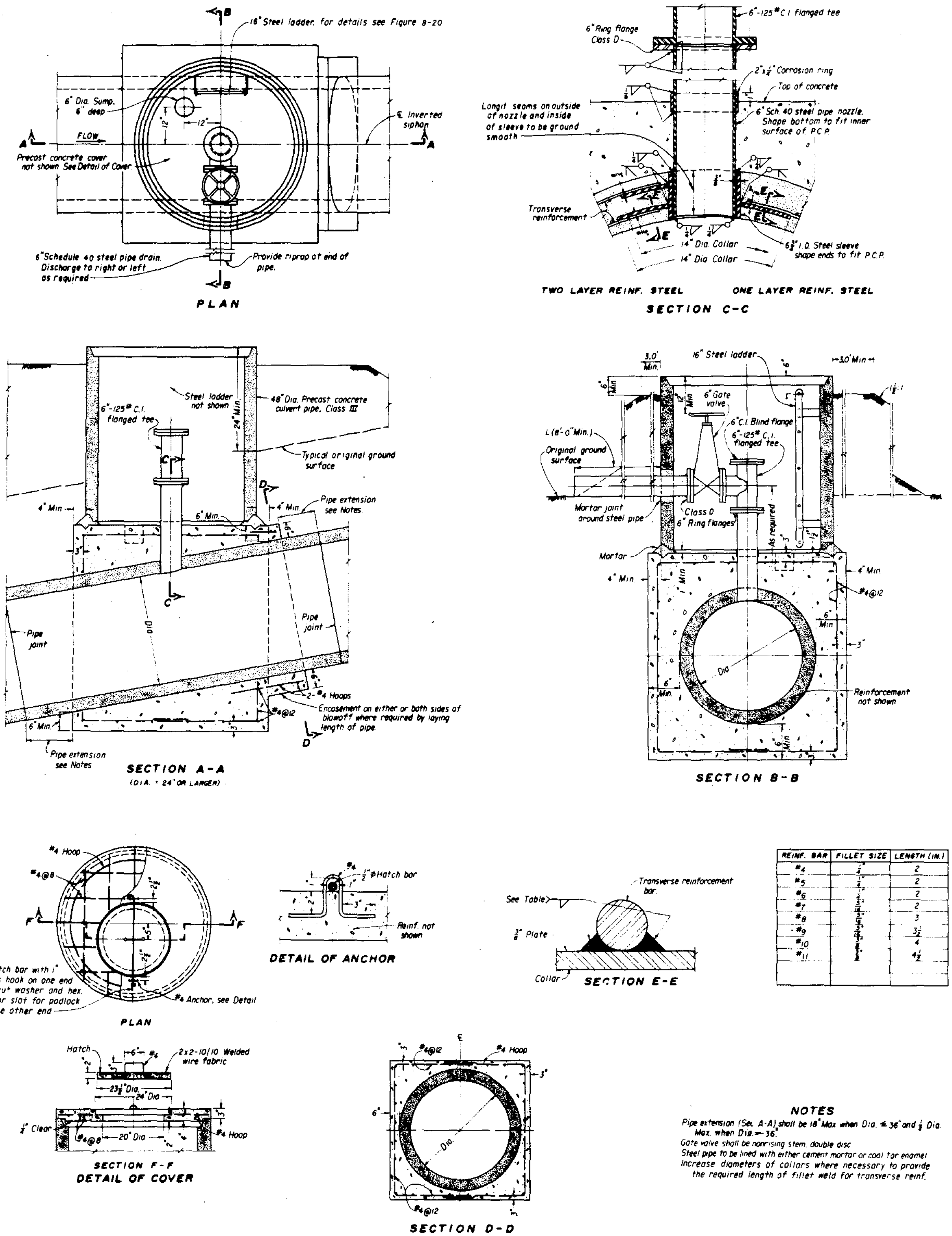
The hydraulic head on the blowoff will determine the class of fittings such as valves, tees, flanges, rings, and also the size of required welds.

Blowoff and manhole assemblies for asbestos-cement and reinforced plastic mortar pipe require steel pipe fittings, concrete encasement, and covered wells similar to those shown for precast concrete pressure pipe. An inline section of mortar-lined steel pipe is used to provide connections for the blowoff and manhole nozzles. Joints at connections of the mortar-lined steel pipe and AC or RPM pipe should be as shown on figure 8-13.

8-16. ***Air Vents.***—Air vents should be provided at all locations along a pipe structure where air may accumulate or where a partial vacuum may occur.

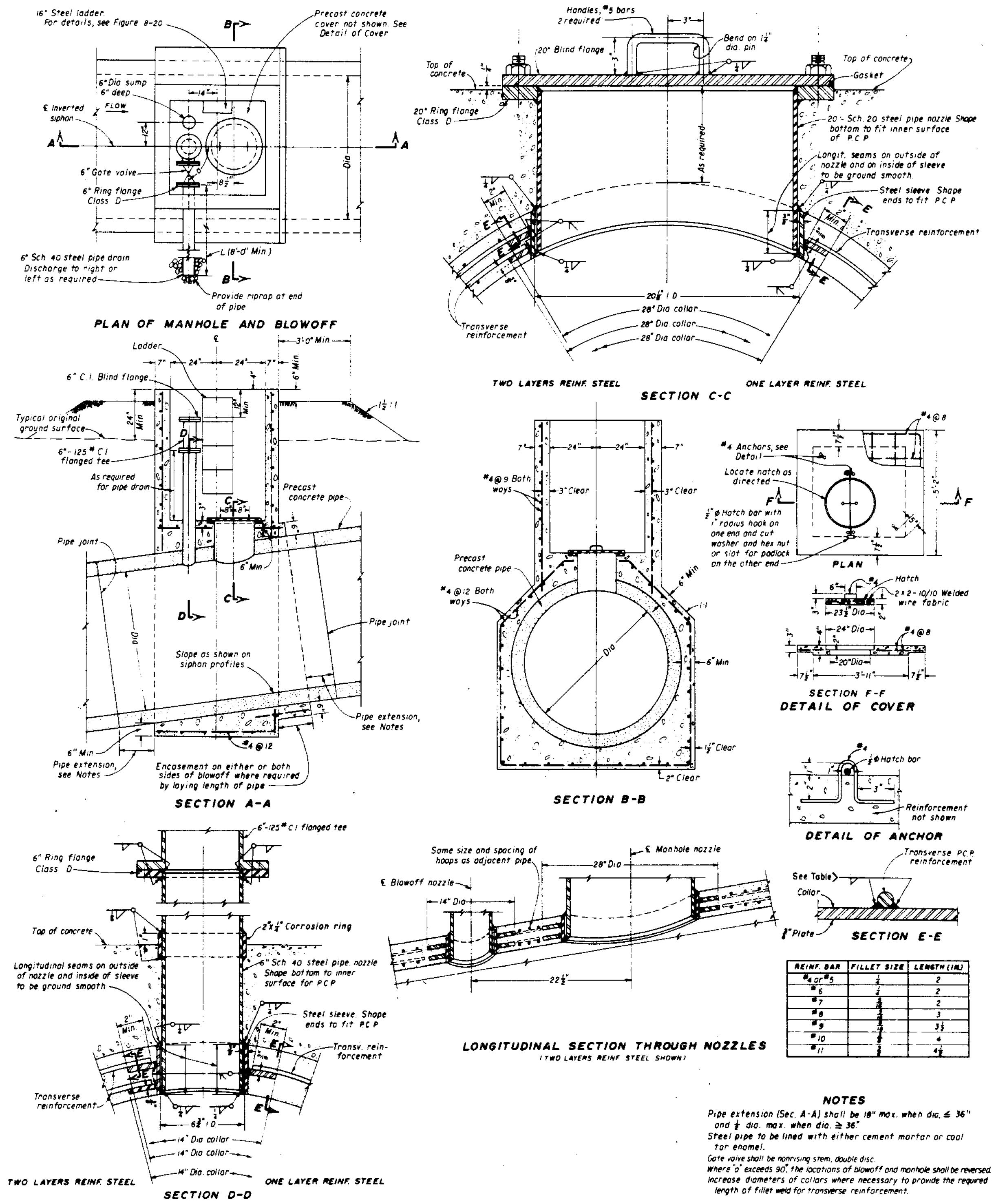
One of the more common uses of an air vent as related to structures for an open irrigation system is to prevent formation of subatmospheric pressures immediately downstream from a gate. High water velocity issuing through a partial gate opening can entrain air immediately downstream of the gate and cause a partial vacuum. This condition can create very undesirable water surface surging resulting in unsteady flow; can cause blowbacks of air and water of sufficient force to damage the structure; and can cause detrimental unequal air pressure loads on the gate leaf. An air vent having a diameter of one-sixth the pipe diameter (or 4 inches minimum) and located as shown on figure 3-15 has given satisfactory performance in preventing these adverse conditions.

The probability of an air and water blowback occurring in a long, inverted siphon



REINF. BAR	FILLET SIZE	LENGTH (IN.)
#4	1/4"	2
#5	1/4"	2
#6	1/4"	2
#7	5/16"	2
#8	5/16"	3
#9	5/16"	3½
#10	3/8"	4
#11	3/8"	4½

Figure 8-18. **Blowoff for precast concrete pipe. 103-D-1327**



REINF. BAR	FILLET SIZE	LENGTH (IN.)
#4 or #5	1/4	2
#6	1/4	2
#7	5/16	2
#8	3/8	3
#9	7/16	3½
#10	1/2	4
#11	1/2	4½

NOTES

Pipe extension (Sec. A-A) shall be 18" max. when dia. ≤ 36" and ½ dia. max. when dia. ≥ 36"
Steel pipe to be lined with either cement mortar or coal tar enamel.
Gate valve shall be nonrising stem, double disc.
Where "a" exceeds 90°, the locations of blowoff and manhole shall be reversed.
Increase diameters of collars where necessary to provide the required length of fillet weld for transverse reinforcement.

Figure 8-19. **Manhole and blowoff for concrete pressure pipe. 103-D-1328**

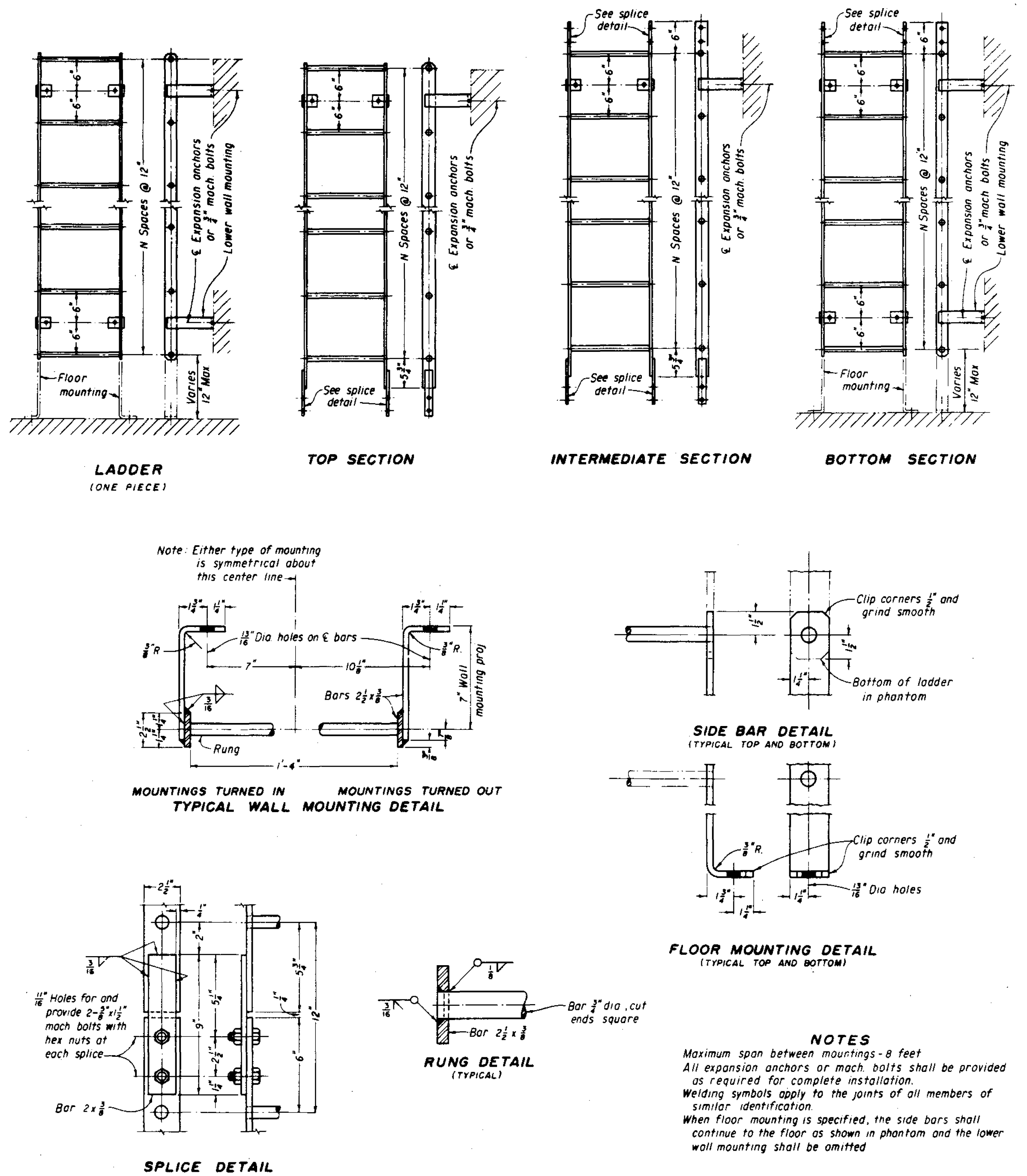


Figure 8-20. **16-inch steel ladder. 103-D-1329**

with a free-flow inlet may be determined by the procedure shown on figure 2-7. Siphon design should be adjusted if necessary to reduce this probability. These blowbacks are caused by air being entrained in the water at different locations such as the hydraulic jump then accumulating in the downstream portion of the jump until the pressure and volume of the resulting bubble is sufficient to cause it to rise and explode into the atmosphere. The process is repetitive and may produce sufficient violent force to damage the structure; therefore, design effort should be made to avoid this condition. Air vents are discussed here only as a corrective measure in the event a blowback condition develops.

Air vents are also commonly located at high points along a pipe profile to allow accumulated air to escape and also to permit entry of air when the pipe is being drained. Air release is essential to avoid trapping air bubbles which can reduce or completely stop the flow of water. Air entry into a pipe while being drained is essential to equalize air pressures inside and outside of the pipe which otherwise might collapse the pipe. Figure 8-21 shows the installation requirements of a concrete pipe air vent used for the above purposes. Six-inch-diameter air vents are commonly used with inline pipe diameters as large as 21 inches while 12-inch air vents are installed in larger diameter pipe. The pipe vent should extend at least 24 inches above the hydraulic gradient. Air valves are sometimes used in place of pipe vents.

Installation of air vents in asbestos-cement, and reinforced plastic mortar pipe may be similar to figure 8-21 except tee fittings may be used.

8-17. *Step-type Pipe Tapers (fig. 8-22).*—These tapers may be used to transition from one pipe size to another, provided the hydraulic head is 50 feet or less and provided the head loss in the taper is permissible. If the head is greater than 50 feet or if the head loss associated with this taper is too great it may be necessary to use a gradual pipe taper instead of the step type.

Head losses for step-type pipe tapers may be assumed to be 0.25 times the change in velocity head through the taper for an increasing diameter and 0.10 times that change for a reducing diameter. If taper is gradual the above losses may be reduced by one-half.

8-18. *Pipe Earthwork.*—Excavation required for pipe installation in an open irrigation system is generally such that the use of a motorized trenching machine would seldom be advantageous. For example, steep topography at chutes, drops, and inverted siphon sites, or generally shallow cuts at road crossings and turnouts, which are usually relatively short, are not particularly conducive to machine trenching. Also, pipe installation in an open irrigation system is generally a minor part of the total work. As a result, side slope excavation pay lines used are 1 to 1 for earth and 1/4 to 1 for rock. These temporary excavation slopes will generally be stable during the time required for construction. Unusual soil conditions may, however, make it desirable or necessary to use flatter slopes.

Figure 8-23 shows the excavation, backfill, and bedding for precast concrete and asbestos-cement pipe. To be assured that the distribution of foundation pressures approximates the distribution assumed in the design of concrete pressure pipe and approximate field strength requirements of concrete culvert pipe and asbestos-cement pipe, special attention must be given to the bedding during field installation. Backfill should be compacted to a vertical distance of three-eighths the outside diameter of the pipe above the bottom of the pipe to achieve these conditions. If the backfill is consolidated by surface compaction, an earth cradle to fit the bottom of the pipe is needed. The depth of the cradle should be 0.05 D_o (outside diameter). The cradle can be omitted if free-draining, noncohesive backfill material is used and compaction of the backfill is accomplished by saturation and internal vibration. Experience has shown that this method will fill the small spaces around the bottom of the pipe for pipe laid on unshaped bedding and on pipe slopes not exceeding 30 percent (0.30). In rock or other unsuitable foundation material, the trench should be overexcavated to provide a compacted cushion of selected material to furnish a uniform bedding for the pipe.

Figure 8-24 shows the excavation, backfill, and bedding for reinforced plastic mortar pipe (RPM). Because RPM pipe is relatively flexible,

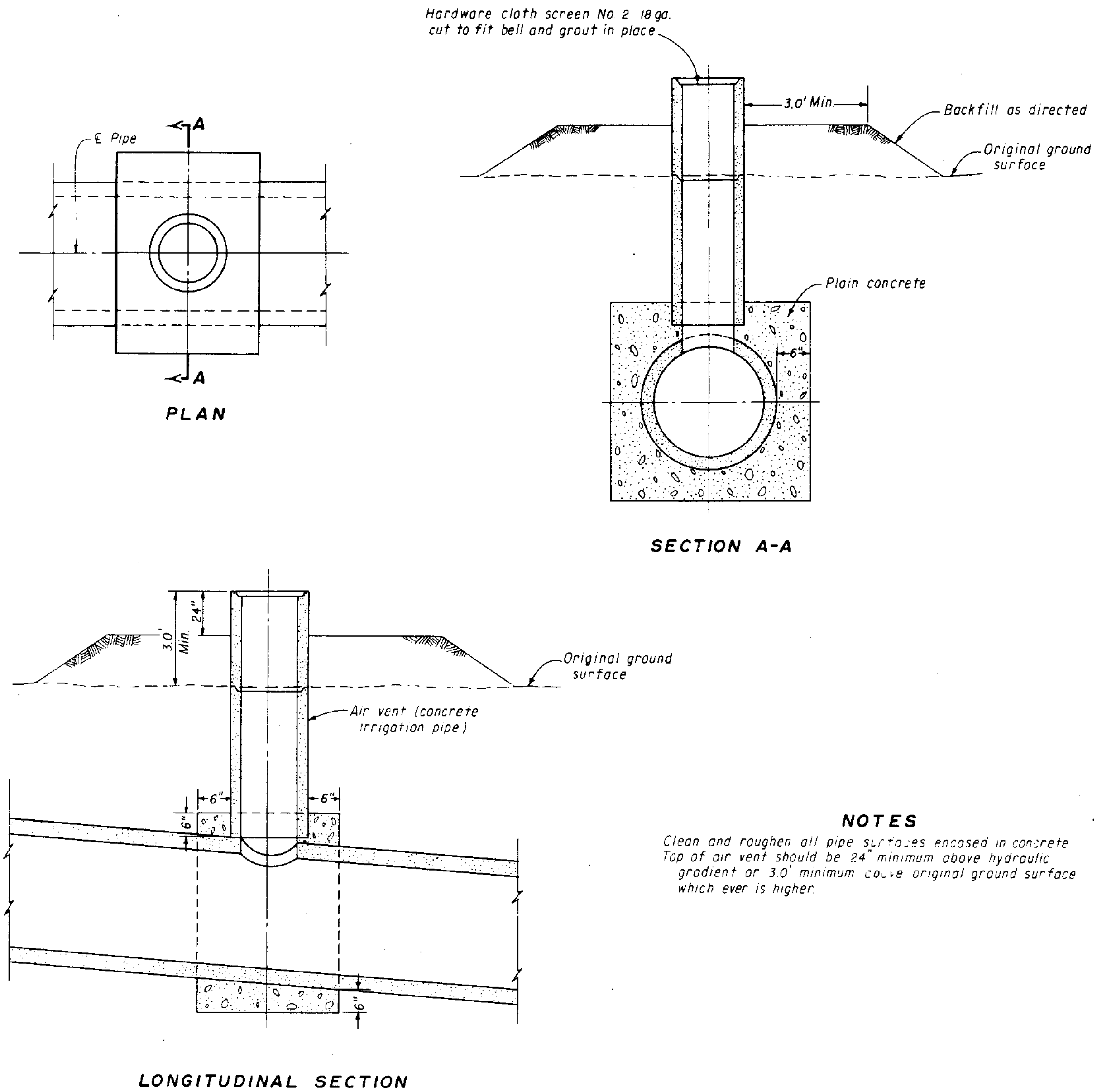
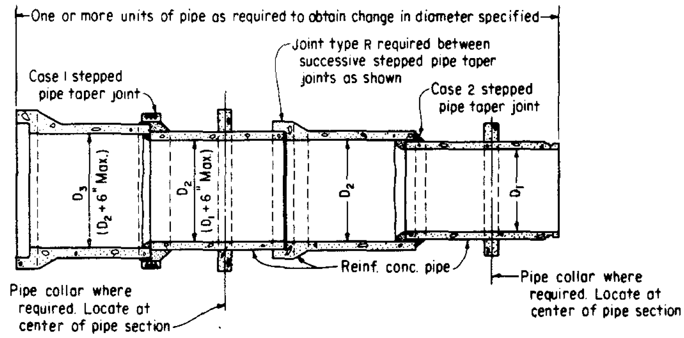


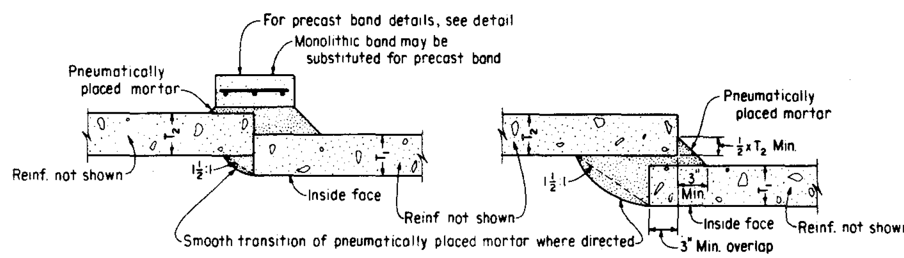
Figure 8-21. **Concrete pipe air vent. 103-D-1330**

the backfill should be compacted to a vertical distance of 0.7 D_0 above the bottom of the pipe. With this earth support on the sides of the pipe, earth pressures around the pipe will adjust and approach the more favorable condition of uniform pressure around the pipe as the pipe deflects from external loading. Excavation and bedding requirements are otherwise the same as previously discussed for precast concrete and asbestos-cement pipe.

For corrugated-metal pipe, excavation should carefully follow the alinement and grade to provide a firm and uniform bearing for the entire length of the pipe. In rock or other unsuitable material, the foundation should be overexcavated to the same requirements used for the other types of pipe as previously discussed. In open irrigation systems, corrugated-metal pipe is often used for such structures as drain inlets into canals and drain

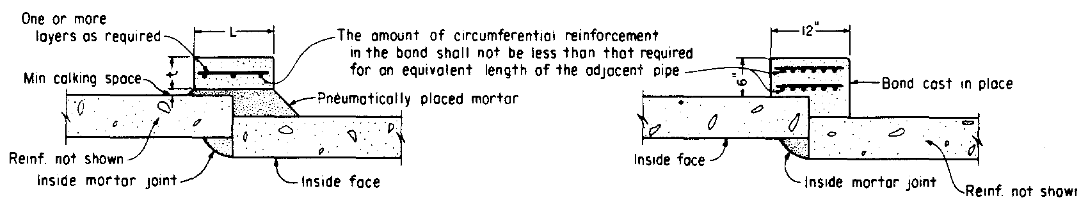


CONCRETE STEP TYPE TAPERS
(PIPE REINFORCEMENT NOT SHOWN)



CASE I **CASE 2**

TYPICAL JOINT AT CHANGE OF PIPE SIZE



PRECAST BAND DETAIL **MONOLITHIC BAND**

TABLE FOR PRECAST BAND DIMENSIONS

DIA. OF PIPE	6" TO 15"	18"	21"	24" TO 45"	48" TO 63"	66" TO 72"	78"	84"	90"	96"
Min thickness (t)	1½"	1⅞"	2¼"	2½"	2¾"	4"	5½"	6"	6½"	7"
Min. length (L)	6"		6"	7½"			9"			
Min. calking space	⅝"		¾"							

NOTES

Step type tapers shall be used in pipe where head is 50' or less except that step type tapers may be replaced by precast or steel tapers at the contractors option.
All fitting to be the same class of pipe as the adjacent line as shown on profile.
Clean and roughen surface of pipe in contact with mortar and concrete.

Figure 8-22. Step-type pipe taper. 103-D-1331 culverts under roadway approaches to bridges over the canal. For pipe which is to be placed within canal or roadway embankment, it is commonly required that the embankment material be placed to an elevation approximately one-fourth the diameter of the pipe above the prescribed pipe invert and then excavated carefully to provide a firm and uniform bedding for the pipe. The backfill should be compacted to a vertical distance of 0.7 D_o above the bottom of the pipe as is shown for RPM pipe on figure 8-24.

Backfill over the top of the pipe and above the compacted backfill required for properly bedding the pipe is usually uncompacted

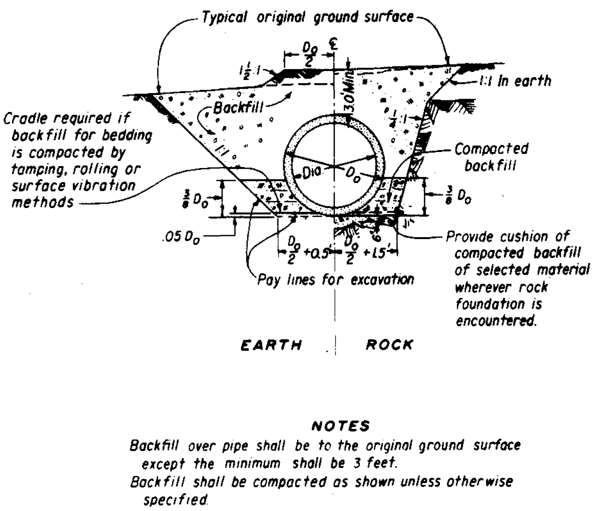


Figure 8-23. **Pipe earthwork details—precast concrete or asbestos-cement pipe—earth or rock excavation. 103-D-1332**

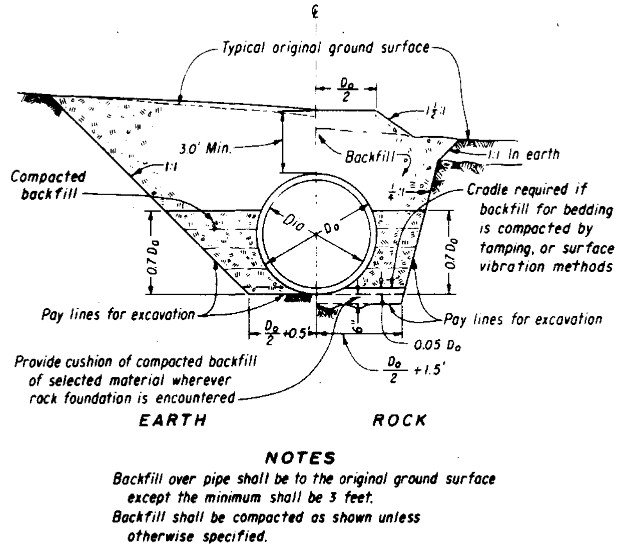


Figure 8-24. **Pipe earthwork details—reinforced plastic mortar pipe—earth and rock excavation. 103-D-1333**

except at roadway crossings and where pipe extends through or under canal sections. For roadway crossings all backfill to the top of the roadway should be compacted. For structures such as turnouts, drain inlets, cross drain culverts, or any other pipe structure which extends through or under canal sections, the backfill should be compacted to the highest of: (1) 1 foot minimum over the top of the pipe and the collars, (2) original ground surface, or (3) to top of compacted embankment if canal embankment has been compacted prior to construction of the pipe structure. Pipe damage from construction equipment travel is thereby avoided, and seepage along the pipe is reduced.

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Safety

H. J. WARREN[1]

9-1. *General.*—The Bureau of Reclamation is acutely aware of the safety hazards associated with open waterways, and incorporates protective features [1][2] into the design of new facilities and modifies existing irrigation features for the protection of the public. Similar preventive measures have been undertaken for animals but on a smaller scale.

This chapter presents the design and selection of some waterway safety devices, and their classification and use.

9-2. ***Hazard Exposure Classification.***—The type of safety features included on an irrigation structure depends on many factors, so it is impractical to standardize designs. However, some general rules on safety are useful to the designer in determining if safety devices are needed. One of the most important considerations is the number of people which will be exposed to the hazard through operation, recreation, or living nearby. This consideration has prompted Reclamation to classify hazard exposures as follows [2]:

Class A.—Those canals adjacent to schools and recreational areas, such as playgrounds, subject to frequent visits by children

Class B.—Those canals nearby or adjacent to urban areas or highways and subject to frequent visits by the public

Class C.—Those canals nearby or adjacent to farms or highways which could be subject to visits by children seeking recreation, such as swimming

Class D.—Those canals far removed from any dwelling subject to infrequent visits by operating personnel and an occasional sportsman

Class E.—Those canals that would be a hazard to domestic animals

Class F.—Those canals that would be an extreme hazard to big game animals

The type of protection used on or around canal structures is dependent on the class of exposure.

[1]General Engineer, Engineering Reference Branch, Bureau of Reclamation.

[2]Numbers in brackets refer to items in the bibliography, sec. 9-4.

A. SAFETY DEVICES

9-3. *Types.*—Many types of safety devices are used on canal structures. These safety devices can be separated into two groups:

(1) Those which limit access or deter people and animals from getting into the canal

(2) Those which are escape devices in the event a person enters the canal either deliberately or by accident

Fencing, guardrails, warning signs, and pipe safety barriers are safety devices which limit or deter people and animals from entering a canal system. Of these, fencing is probably the most effective deterrent; however, warning signs are heeded by some people and guardrails are used to keep people and vehicles from accidentally falling or driving into canals. Pipe safety barriers present enough of an obstacle to reduce the temptation for people to use the pipe crossings over a canal as a walkway.

Safety nets, safety cables, safety racks, and safety ladders are devices which aid people to escape from a canal. A combination of these devices and those which deter access are usually constructed at the upstream ends of hazardous structures where a fatality would surely occur if a person entered the structure.

(a) *Fencing.*—A fence is the most common and effective safety device to deter people and animals from entering a canal. Reclamation uses six types of safety fences. Selection of the appropriate type fence is dependent on the hazard exposure classification. Requirements of these fences, constructed on or near the canal right-of-way, are described in table 9-1.

Standard design drawings for different types of fences are given in figures 9-1 through 9-5.

Fences are used in conjunction with safety nets or safety cables around inlets to siphons 30 inches in diameter and larger for Class A, B, or C exposures. The fence should meet the requirements of the pertinent exposure classification. It should be constructed across the transition headwall and extend along the sides of the transition to the anchor posts of the safety net or cable as shown in figure 9-6. Sometimes a short length of the fence is constructed to the water's edge as shown in figure 9-7 [3].

Fences or guardrails should be constructed along each side of chutes (fig. 9-8), rectangular inclined drops and stilling pools (fig. 9-9) having a 4-foot depth or more in Class A, B, or C exposure. Fence or guardrail might also be used on each side of a bench flume to prevent people from stumbling into the flume when the flume is set into the ground flush with the top of the wall.

(b) *Guardrails.*—There are two types of

Table 9-1.—*Safety Fence Classification.* 103-D-1196

HAZARD EXPOSURE CLASSIFI-CATION	TYPE OF SAFETY FENCE	TOTAL HEIGHT FT.-IN.	FENCING FABRIC				POSTS		TOPRAIL
			CHAIN-LINK HEIGHT FT.-IN.	WIRE-MESH HEIGHT FT.-IN.	WOVEN-WIRE HEIGHT FT.-IN.	BARBED-WIRE NUMBER OF STRANDS	CENTER TO CENTER SPACING (FT.)	MATERIAL	
A	School	7'-0"	6'-0"	——	——	3	10	Steel	yes
B	Urban	5'-0"	4'-0"	——	——	3	10	Steel	yes
C	Rural	5'-0"	——	4'-0"	——	2	12	Steel	——
	Rural	5'-0"	——	4'-0"	——	2	16	Wood	——
D	None - Unless required by right-of-way agreement								
E	Stock	4'-0"	——	——	——	4	16	Wood	——
	Stock	4'-0"	——	——	2'-8"	3	16	Wood	——
	Stock	4'-0"	——	——	——	4	12	Steel	——
	Stock	4'-0"	——	——	2'-8"	3	12	Steel	——
F	Deer	8'-0"	——	7'-0"	——	3	16	Wood	——

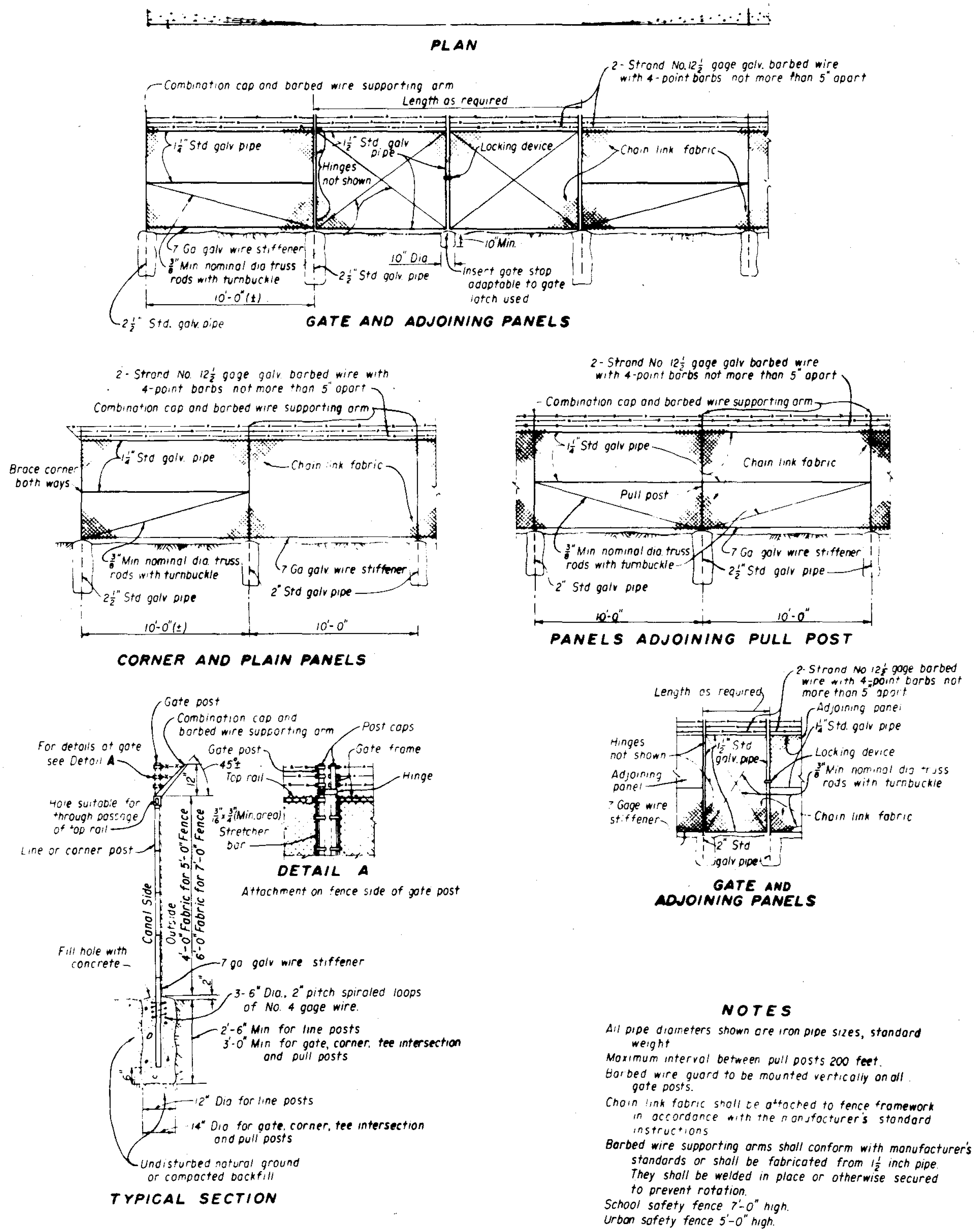


Figure 9-1. **School and urban fence. 103-D-1197**

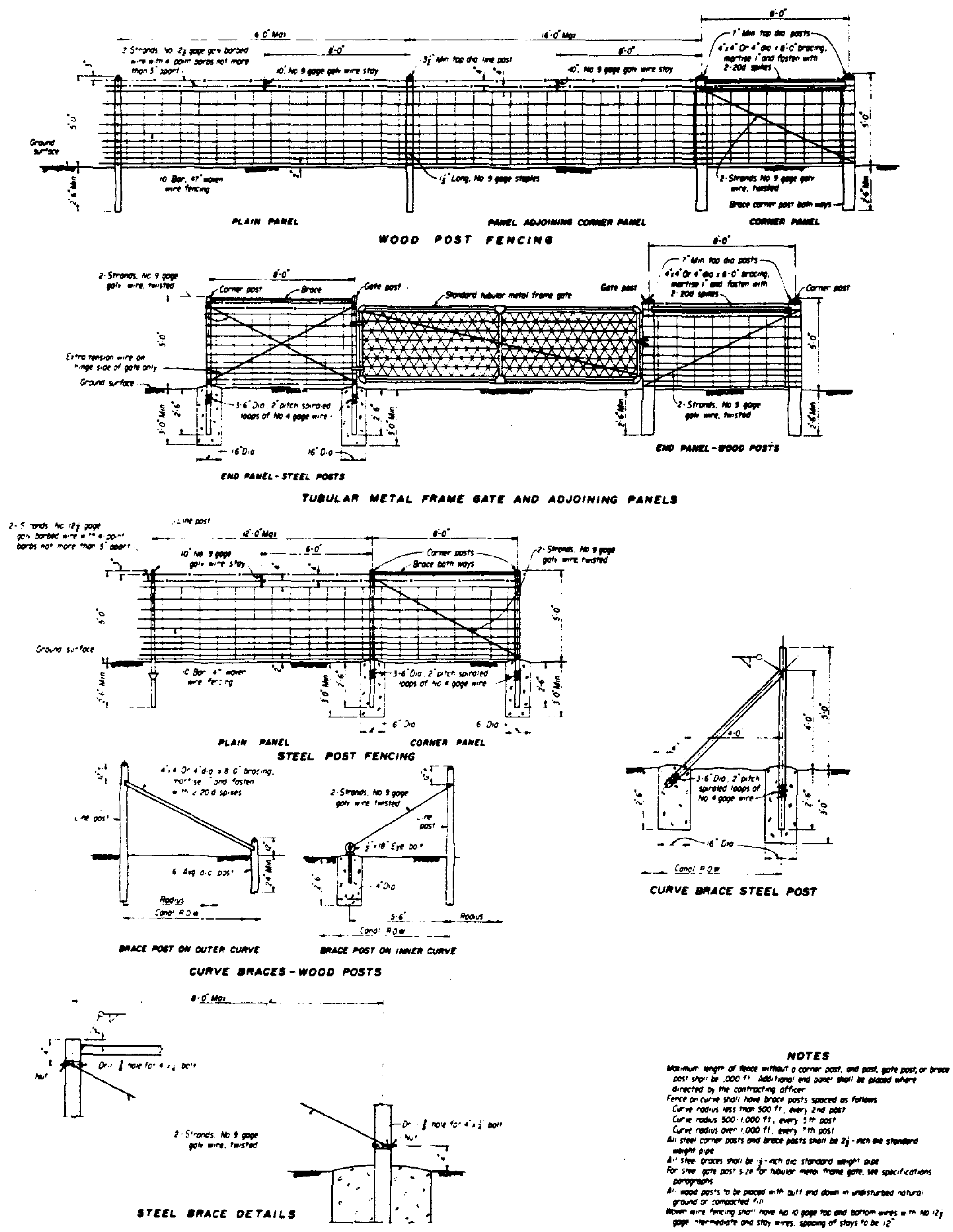


Figure 9-2. Rural fence. 103-D-1198

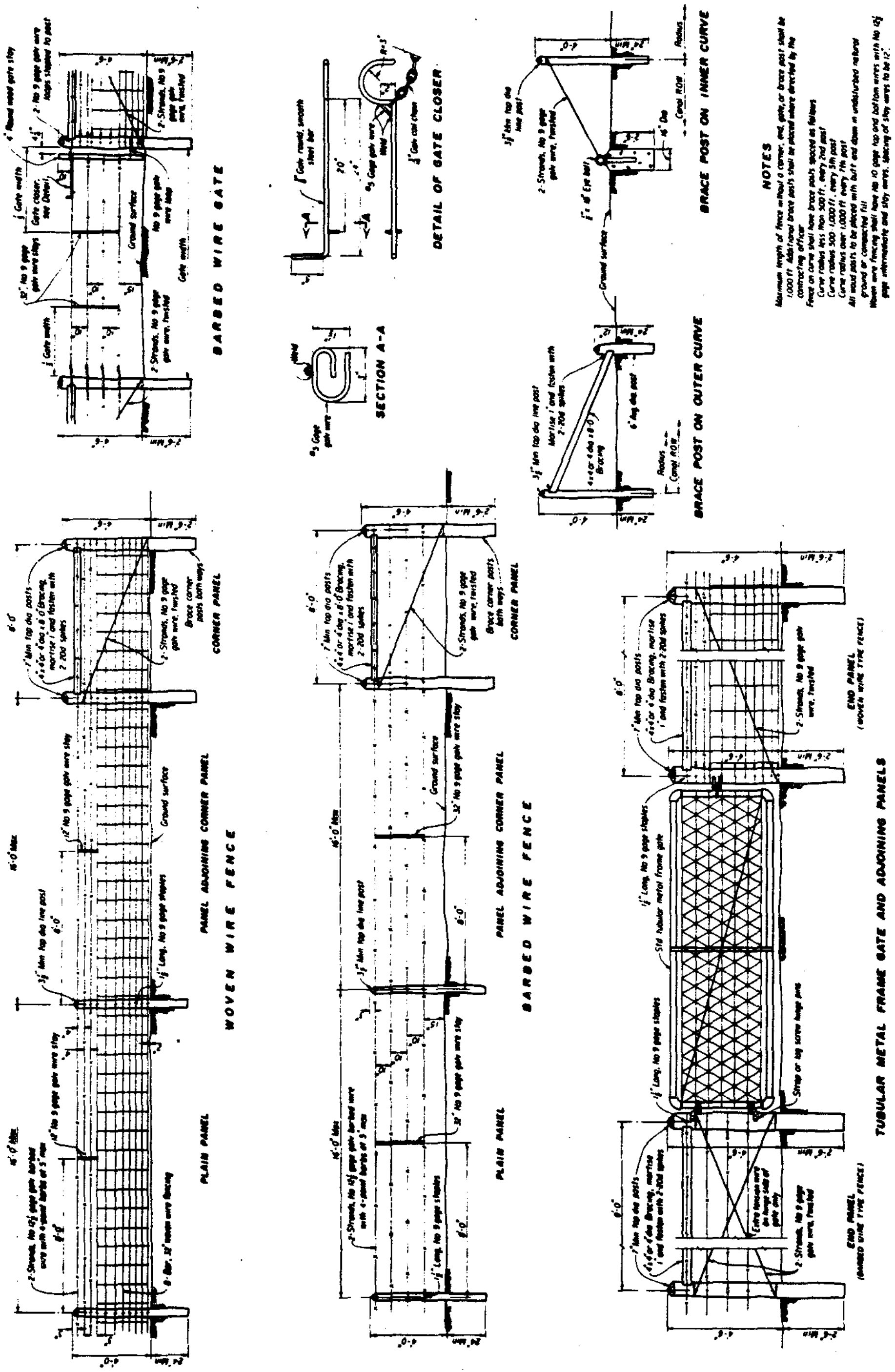


Figure 9-3. Stock fence with wood posts. 103-D-1199

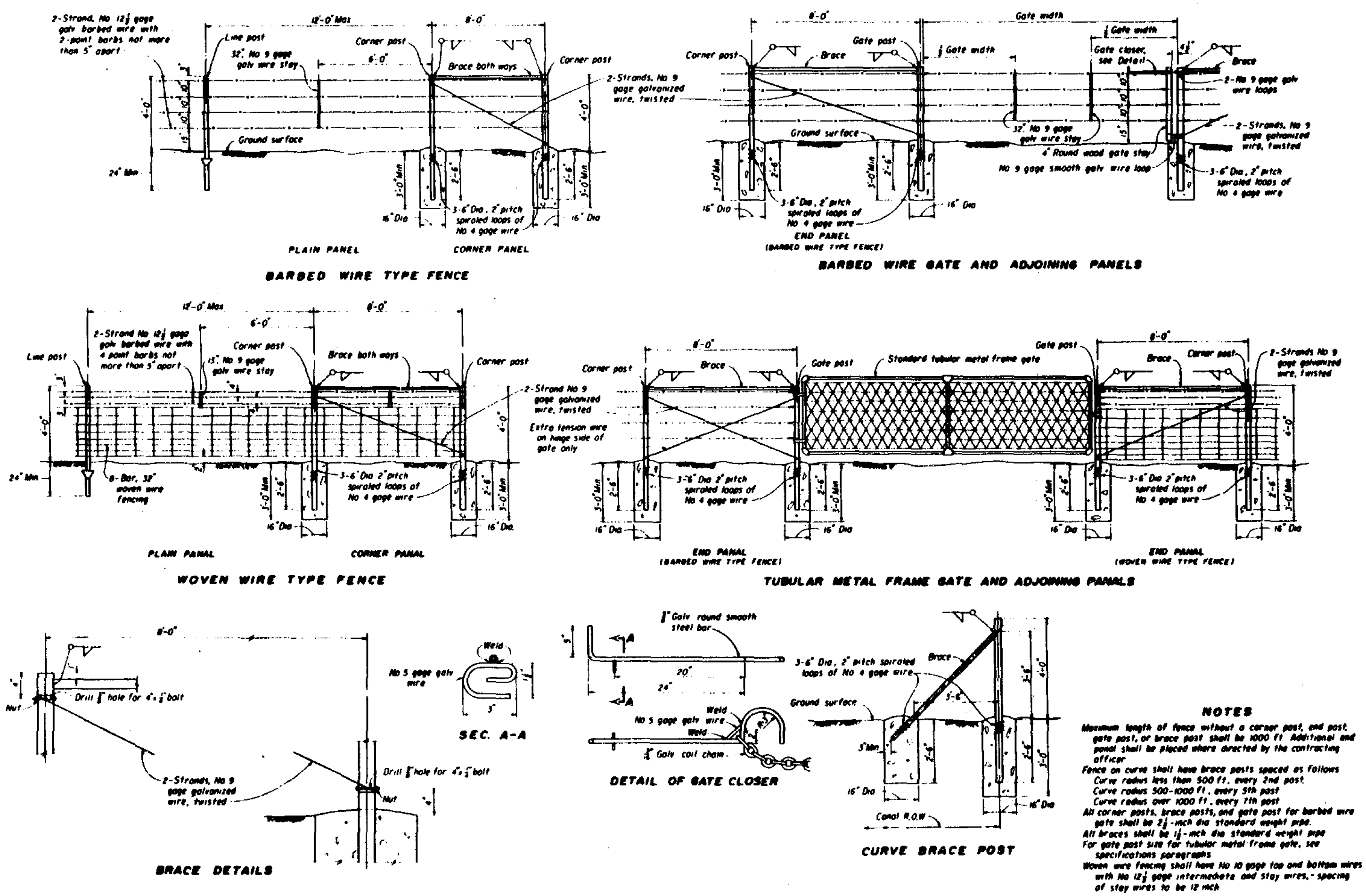
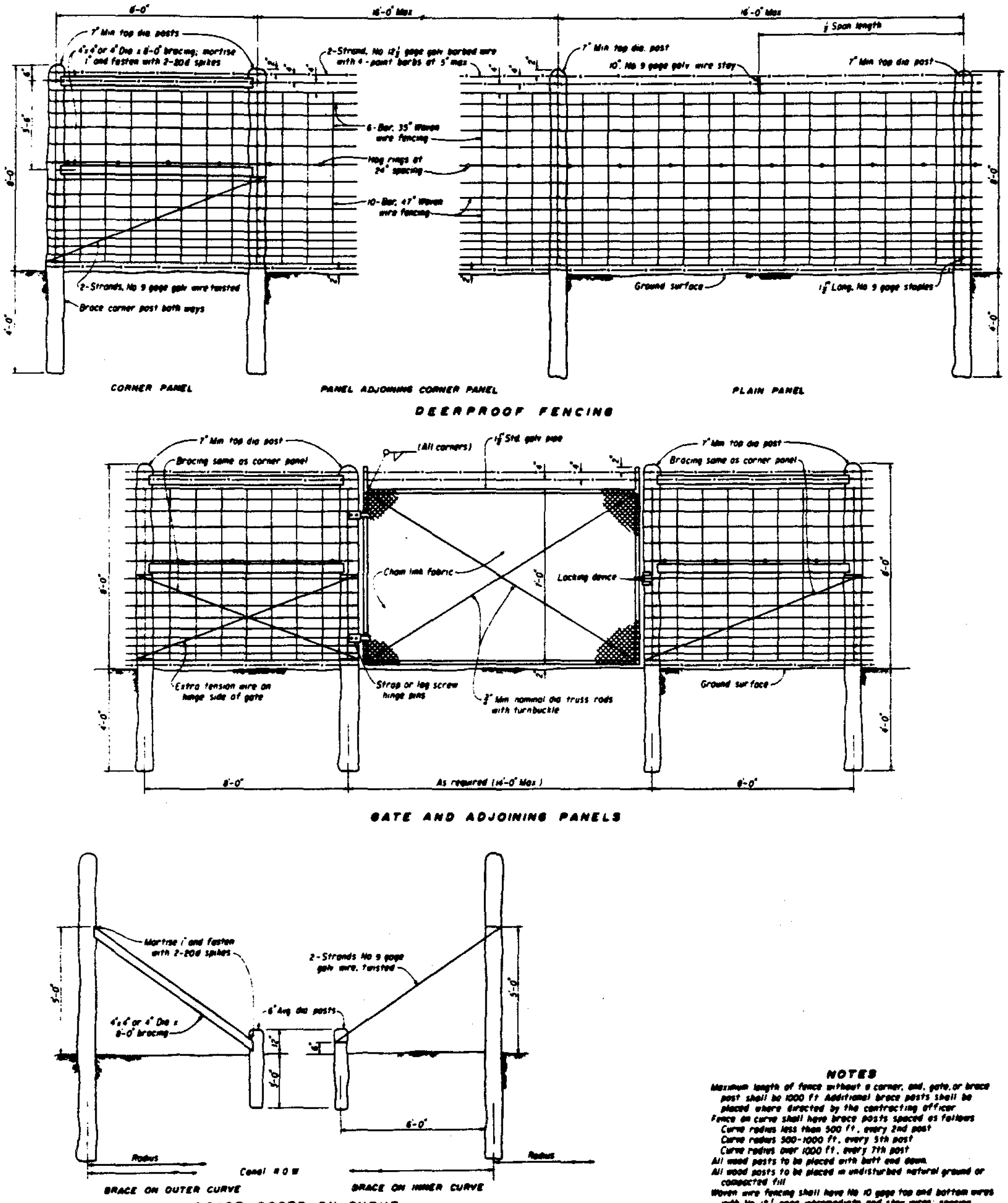


Figure 9-4. **Stock fence with steel posts. 103-D-1200**



NOTES

Maximum length of fence without a corner, end, gate, or brace post shall be 1000 ft. Additional brace posts shall be placed where directed by the contracting officer.

Fence on curve shall have brace posts spaced as follows:
- Curve radius less than 500 ft., every 2nd post
- Curve radius 500-1000 ft., every 5th post
- Curve radius over 1000 ft., every 7th post

All wood posts to be placed with butt end down.

All wood posts to be placed in undisturbed natural ground or compacted fill.

Woven wire fencing shall have No 10 gage top and bottom wires with No 12½ gage intermediate and stay wires; spacing of stays shall be 12".

Chain link fabric shall be attached to gate framework in accordance with the manufacture's standard instructions.

Figure 9-5. **Deer fence. 103-D-1201**

Figure 9-6. **Warning sign, safety cable, and safety fence at siphon inlet. 805-236-14106**

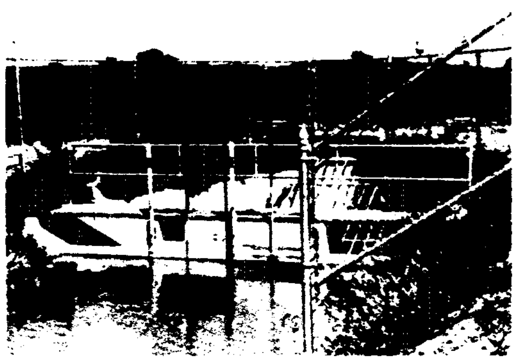


Figure 9-7. **Safety fence, handrail, and safety rack at siphon inlet. P328-700-103 NA**

guardrails: (1) Those used to keep vehicles from entering the canal along the canal bank at dangerous turns; and (2) Those used to prevent pedestrians from falling into the canal at structures. Only the latter type, which is identified as pipe handrail, will be discussed here.

Handrails are usually constructed of 1-1/2-inch-diameter metal pipes in combination with metal posts set in concrete or attached to a wall mounting. They are 42 inches high and have two or three horizontal pipe rails depending on the exposure classification. Two-rail handrail (fig. 9-10) will suffice for

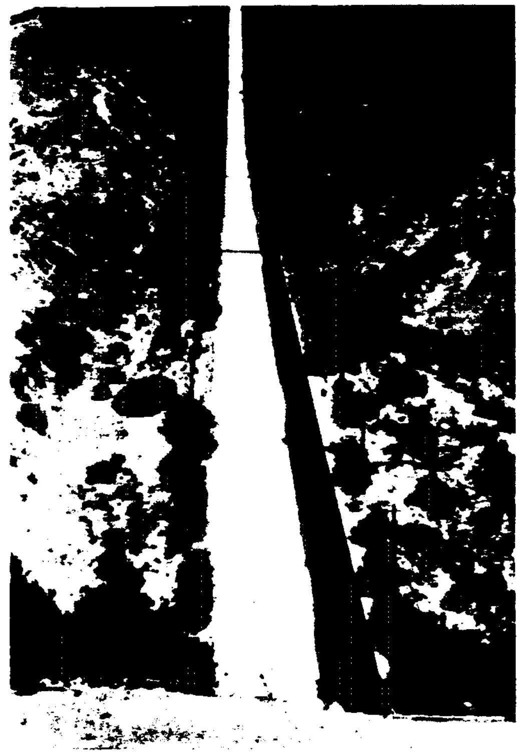


Figure 9-8. **Safety fence along a chute.**

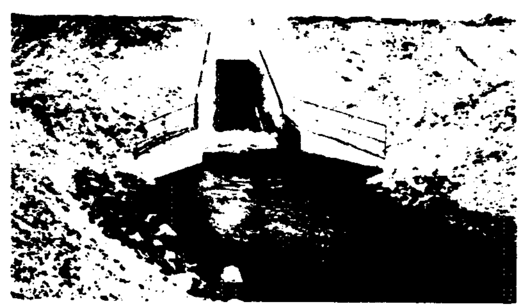


Figure 9-9. **Typical pipe handrail installation along sides of inclined drop and stilling pool.**

most canal structures having a Class C or D exposure. For Class A or B exposure, a three-rail handrail (fig. 9-11) is recommended. Sometimes chain link fence is used in conjunction with three-rail handrail to prevent

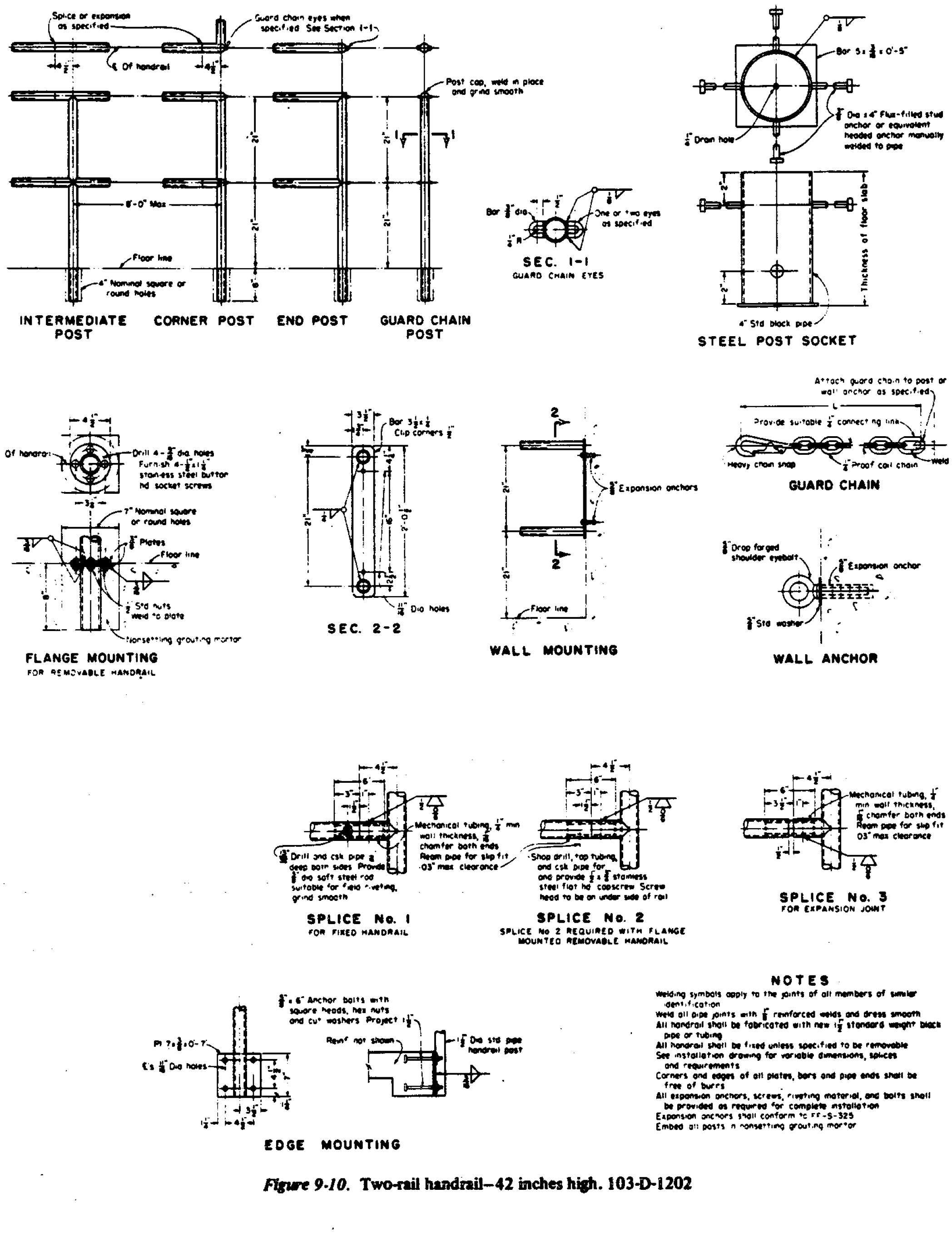


Figure 9-10. **Two-rail handrail–42 inches high. 103-D-1202**

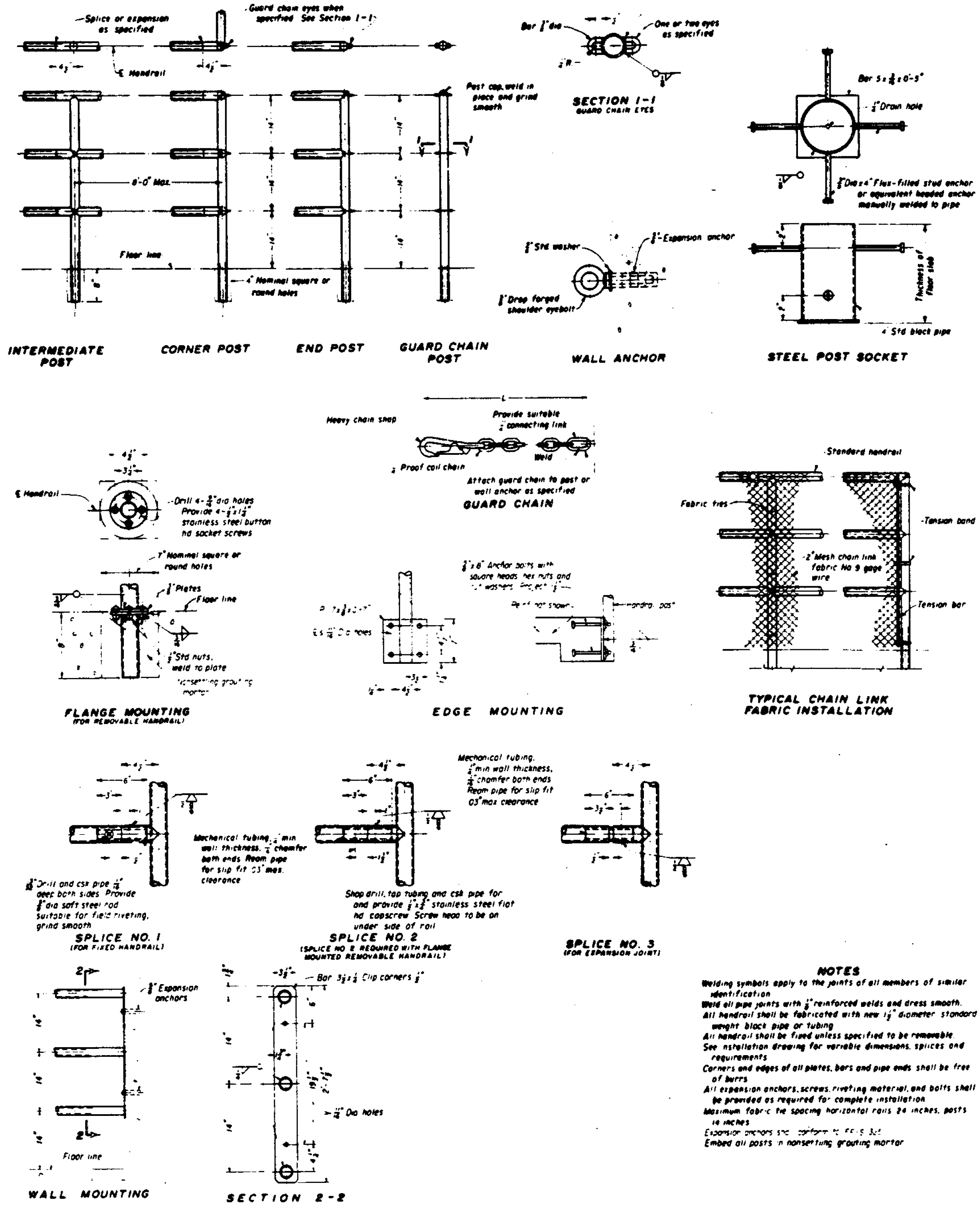


Figure 9-11. Three-rail handrail—42 inches high. 103-D-1203

children from crawling or slipping between the rails and falling into a structure.

Pipe handrails are used in conjunction with safety racks around the inlet transitions of all siphons 30 inches and larger in diameter for Class A, B, or C exposure unless safety fence and safety nets or cables are used as previously described. Pipe handrails are also required around the outlet transition of all siphons 30 inches in diameter and larger for Class A, B, or C exposure as well as around the inlet transition for Class D exposure. These handrails are constructed across the transition headwall and extend the length of the transition on both sides of the canal. Pipe handrails are also used along operating platforms and walkways of many irrigation structures. Generally, handrails are required where the top of the walkway or operating platform is more than 3-1/2 feet above the downstream floor of the structure. Only a downstream handrail is required when the operating deck is 3-1/2 to 5 feet above the downstream floor (fig. 3-10). Handrails are required on both sides of the walkway if the walkway is 5 feet or more above the floor (fig. 9-12).

If a check structure is constructed monolithically with or immediately upstream from a siphon inlet whose diameter is 36 inches or larger, or a steep chute of similar size, handrails should be built on both the upstream and downstream side of the operating platform (fig. 9-12).

Oftentimes the handrails on structures interfere with handling stop planks or cleaning trash from trashracks. In these cases removable pipe rails or guard chains as shown in figures 9-10 and 9-11 are used in place of permanent type rails.

(c) *Warning Signs.*—Oftentimes the public is not aware of the dangers associated with structures in a canal system. To alert them of these dangers, well-worded warning signs advising of specific dangers, should be installed near the structure in a conspicuous place as shown in figure 9-6. No trespassing signs serve a useful purpose, but signs pointing to a specific danger are more effective. In areas where a large number of people speak a language different from the native language, warning signs should be written in dual languages. For instance, in southwestern United States warning signs are written in both English and Spanish.

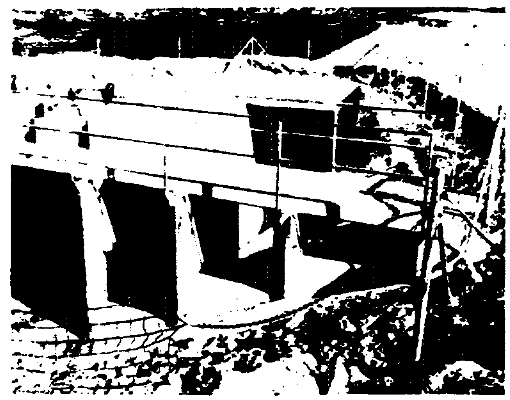


Figure 9-12. **Check inlet with safety fence, safety net, and having handrails on both sides of walkway. P328-701-7147**

(d) *Pipe Safety Barrier.*—Pipe safety barriers which have been previously shown in figure 8-17, are safety devices installed on pipes crossing over canals. Their purpose is to discourage people from trying to walk on the pipe and then accidentally falling into the canal.

Pipe safety barriers should be used on all pipe crossings except those in remote areas and those within fenced waterways.

(e) *Safety Nets.*—Safety nets are escape devices suspended across the canal immediately upstream of siphons, checks, and similar structures. They provide an opportunity for the person in the canal to save his life by grabbing the net and pulling himself along the net to the safety ladder where he can escape from the canal.

Many different types of safety nets have been used. Some have been made of rope netting (fig. 9-13); others have been made from galvanized wire mesh enclosed in a wood frame and suspended from a cable (fig. 9-14); and some were simply wire rope suspended across the canal with rope or chain drop lines hanging from it.

Figure 9-15 shows the design of a typical safety net and cable currently preferred on

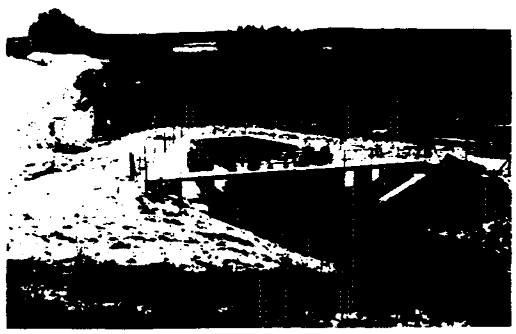


Figure 9-13. **Rope safety net upstream of checked siphon inlet. Note handrails on both sides of walkway. P328-701-7223**

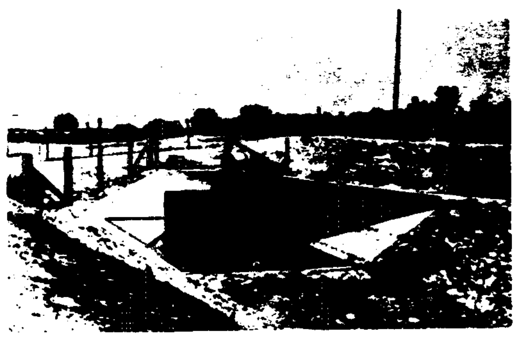


Figure 9-14. **Wire mesh safety net at siphon inlet. P328-701-4241**

Reclamation projects. This device is used in canals (fig. 9-16) where water depth is greater than 3 feet. If the water depth is less than 3 feet, the chain tie, suspenders, and rungs can be omitted.

(f) *Safety Cables.*—Another type of safety device used to aid people in escaping from the canal is safety cables. Safety cables consist of a number of floats attached to a cable that is placed across the canal immediately upstream of siphons, checks, or similar structures (fig. 9-17). Safety cables serve the same purpose as safety nets but are simpler to build and less expensive. They are usually used on large canals where excessive spans might limit the use of safety nets or safety racks. Figure 9-18 shows a safety cable currently used on Reclamation canals. The components of this safety cable are commercially available. The cable assembly includes a 5/8-inch-diameter yellow polypropylene rope with a minimum tensile strength of 5,800 pounds. The floats are high-density polyethylene shells filled with 2-pound-density polyurethane foam. The shells should be at least 0.1 inch thick and contain an ultraviolet inhibitor and orange color pigment incorporated into the polyethylene during manufacture. The polyurethane foam should have a minimum of 95 percent closed cells and should not absorb more than 10 percent water by weight when subjected to a 6-inch head of water for 24 hours. The ends of the float should have a counter bore molded at least 1-1/2 inches into the float to provide a wearing surface for the rope. Short pieces of rope are braided into the cable at each end of the floats to hold them in place.

(g) *Safety Racks.*—Safety racks are barriers placed across inlets to inverted siphons, pipe chutes, and pipe drops to prevent people from being drawn into the structure. They also provide a means for a person to climb out of the canal. Safety racks and guardrails in combination can be used in lieu of fencing and safety nets or cables at the inlet transitions to siphons 30-inch diameter and larger in Class A, B, or C exposure areas. They are generally used in small canals where the canal is relatively free from weeds and debris.

Since safety racks are designed in a number of ways to fit the various types and sizes of inlet transitions, the Bureau of Reclamation does not have standard design drawings for safety racks. Figure 9-19 is a design drawing of a safety rack placed on an inlet transition with a check. Figure 9-20 shows a safety rack on a warped transition. In comparing the two safety racks there are obvious differences, but both types accomplish the same purpose.

Generally, safety racks are made from standard steel galvanized pipe, bolted and welded together to form a grille that is attached to the headwall of an inlet transition. The sloping steel pipes are usually 1-1/2 inches in diameter with 9 inches of clear spacing between pipes. The welded frame, on which the sloping pipes are bolted, may be 2-inch

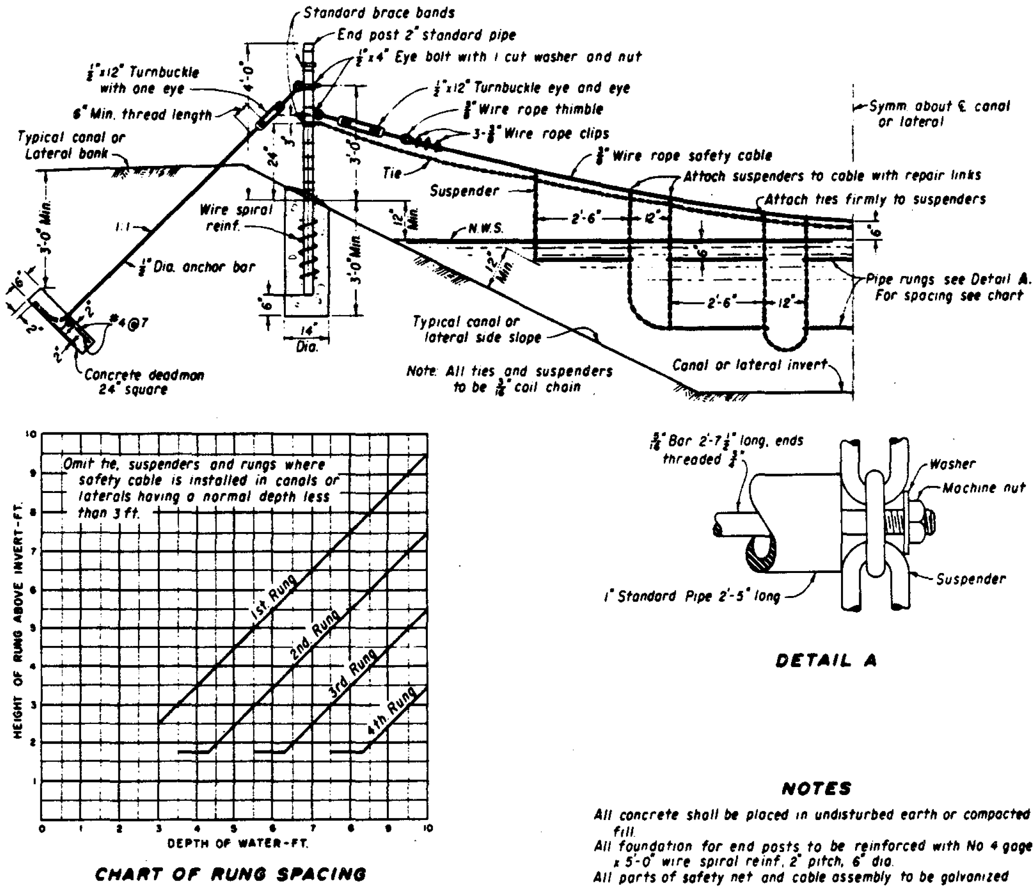


Figure 9-15. **Safety net and cable. 103-D-1204**

Figure 9-16. **Safety net and cable at siphon inlet. PX-D-72613**

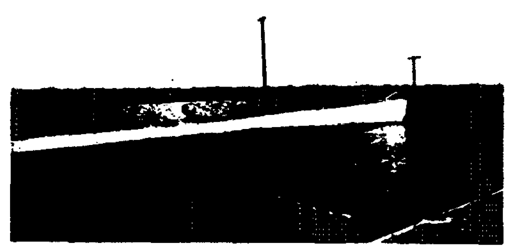


Figure 9-17. **Safety cable with floats upstream of a siphon inlet. P214-D-37162**

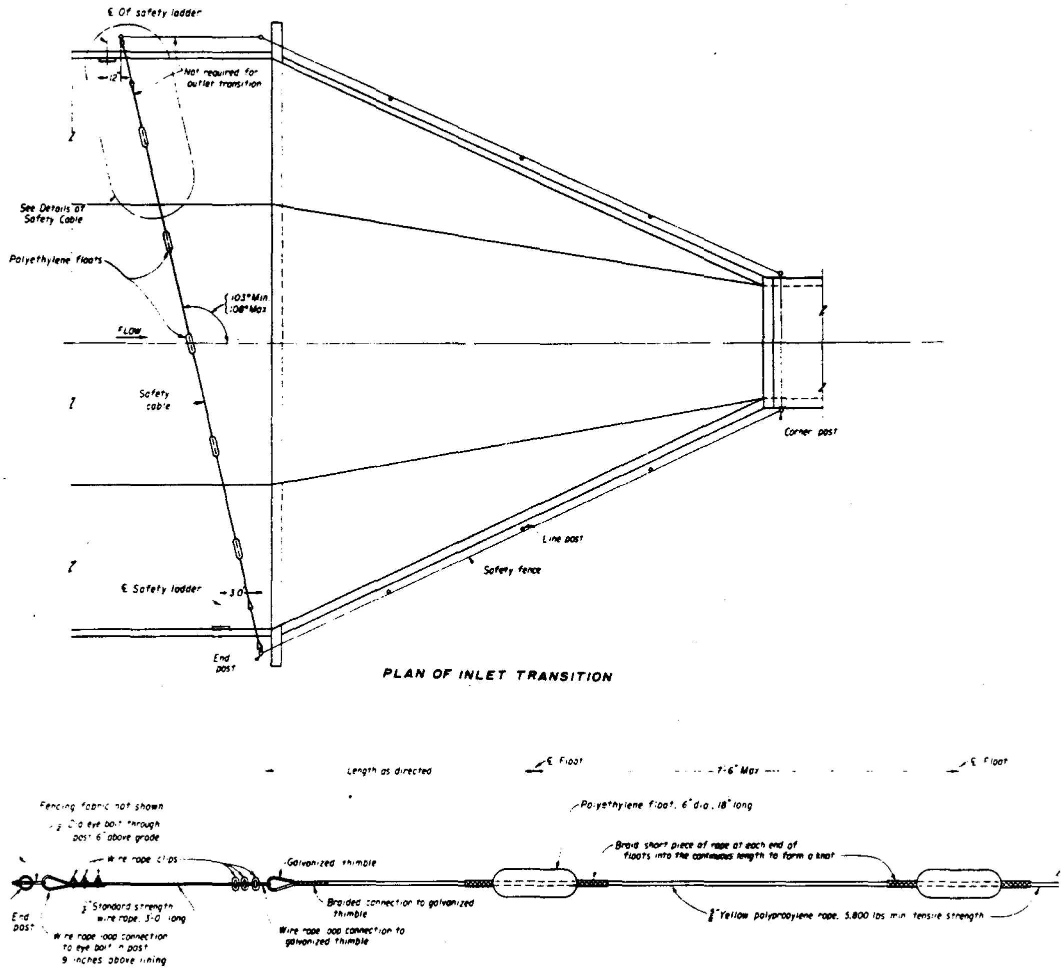


Figure 9-18. **Safety cable with floats. 103-D-1205**

pipe or larger depending on the span of the safety rack.

The horizontal pipes give the rack rigidity and provide steps for aiding a person to escape; however, these pipes do catch weeds and make cleaning the rack difficult. Safety racks should be placed on a 3 to 1 slope or flatter so a person may pull himself from the canal.

(h) *Safety Ladders.*—Ladders are the most commonly used escape devices in a concrete-lined canal. Their effectiveness, however, depends on the ability of the person in the canal to get to them. An accelerating velocity usually occurs where canal water enters a canal structure. For this reason safety nets or safety cables should be installed in

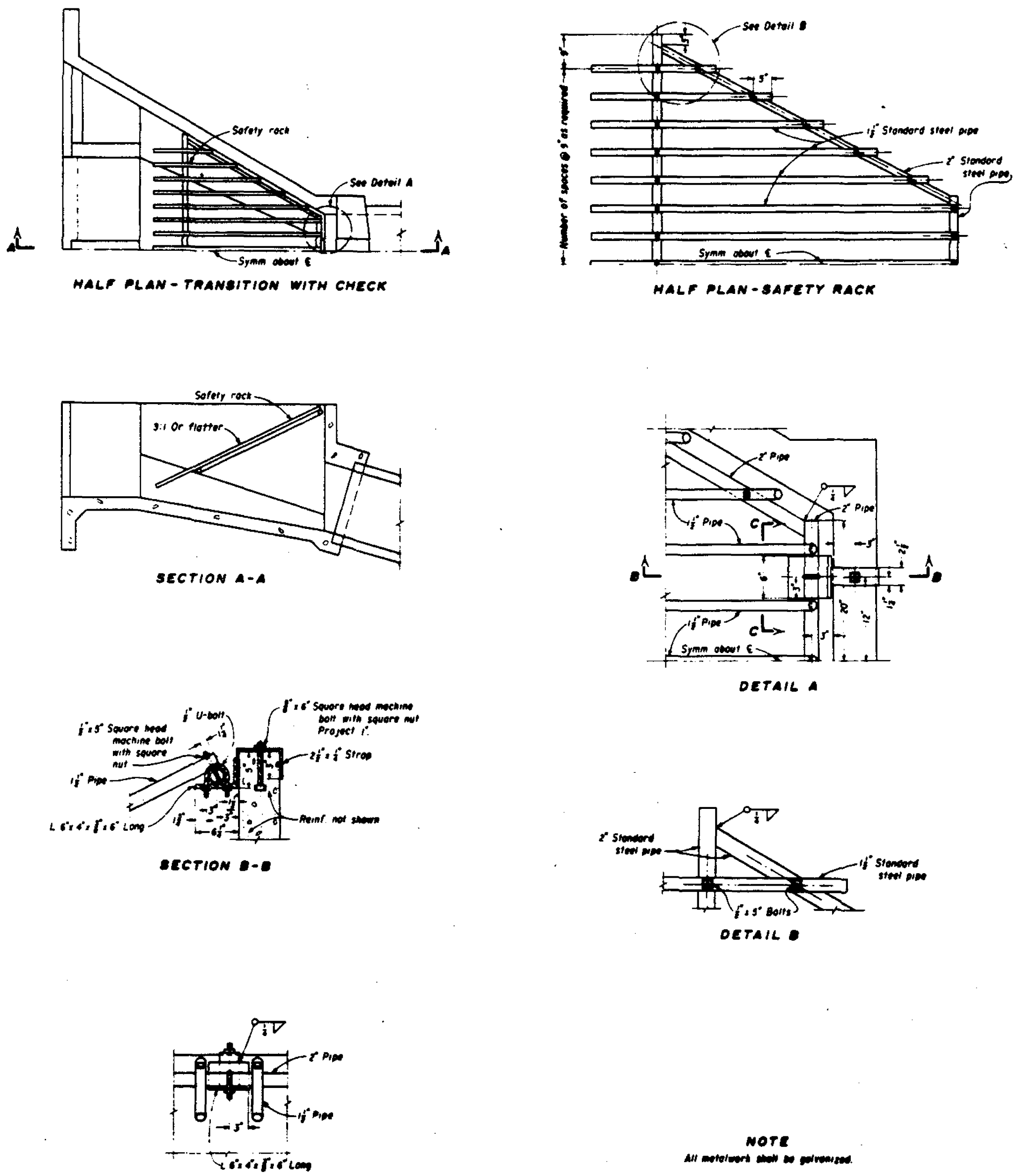


Figure 9-19. **Safety rack. 103-D-1206**

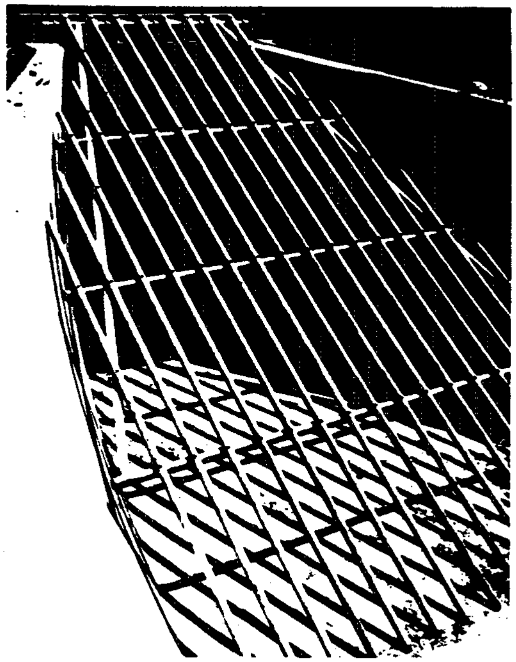


Figure 9-20. **Safety rack on inlet transition. P328-701-9467**

combination with ladders and located a short distance upstream of the canal structure (fig. 9-21). This allows the person to grasp the net or cable and pull himself to the ladder where he can usually escape from the canal.

Figure 9-21. **Safety ladder and safety cable with floats upstream of a siphon inlet. PX-D-49120**

In concrete-lined canals where the vertical lining height is 30 inches or greater, ladders are required immediately upstream of all siphons, checks, check drops, pipe drops, and chutes. Safety ladders are also required to be installed at 500-foot intervals in bench flumes where walls are 36 inches and higher. Figure 9-22 is a design drawing of a ladder used in a concrete-lined canal or in a bench flume. Generally, safety ladders are used only in hard-surface-lined canals.

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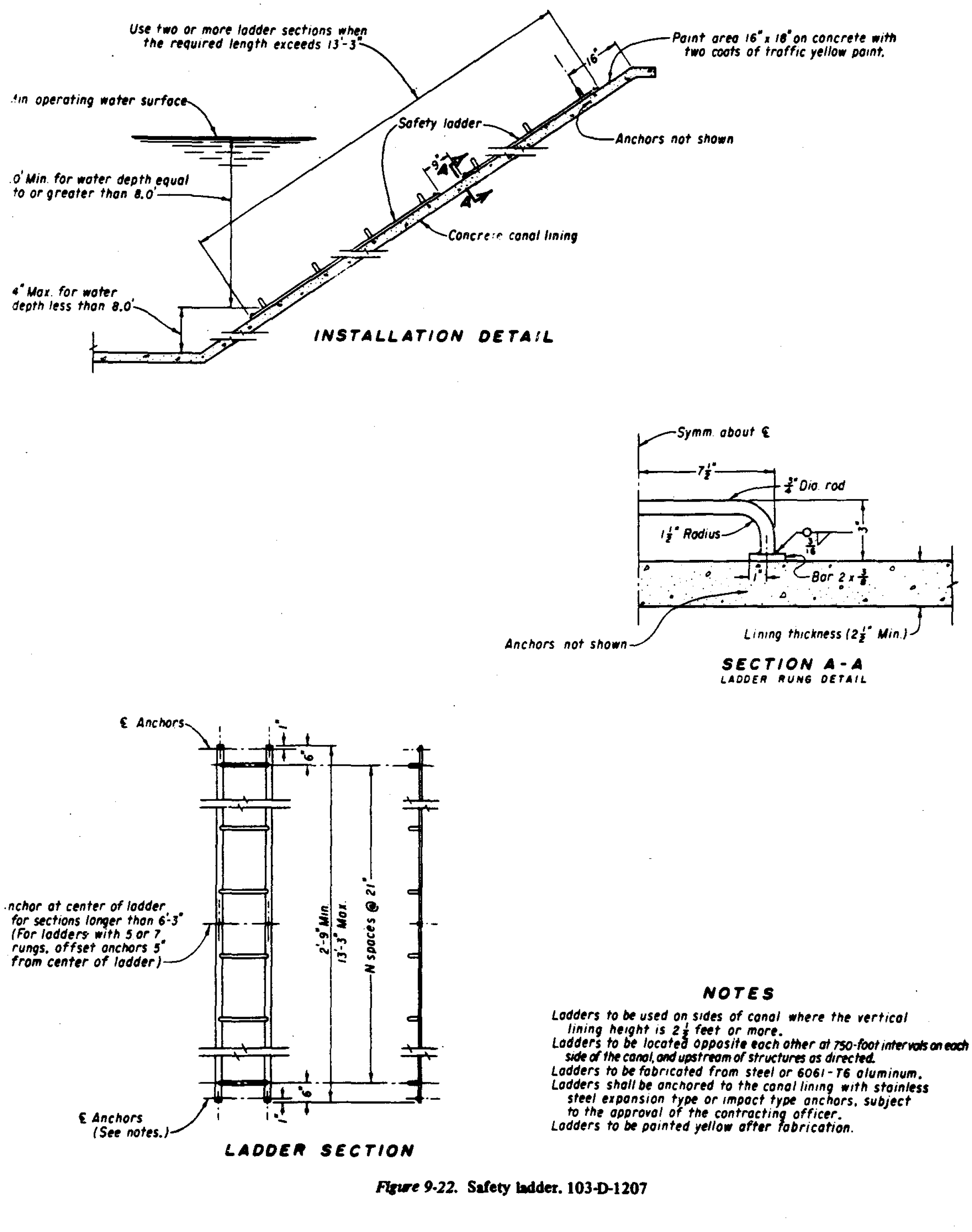

Figure 9-22. **Safety ladder. 103-D-1207**

Glossary of Terms

A-1. ***List of Symbols and Abbreviations.***—The following is a list of symbols and abbreviations used in this publication. Commonly used mathematical symbols and abbreviations are not listed.

Symbol or abbreviation	Definition
A	Area; cross-sectional area of flow in an open channel or pipe; cross-sectional area of a pipe or an orifice
AASHO	American Association of State Highway Officials
AC	Asbestos-cement pipe
Adj	Adjustable
Approx	Approximate
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
Avg	Average
b	Bottom width of a channel
C	Coefficient of discharge
℄	Centerline
cc	Center to center
CF	Cold finish
cfs	Cubic feet per second
CHO	Constant-head orifice

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Symbol or abbreviation	Definition
CI	Cast iron
CMP	Corrugated-metal pipe
conc	Concrete
crs	Centers
csk	Countersink
cu	Cubic
CWS	Control water surface
D	Diameter of a pipe; height of a siphon throat opening; vertical drop of invert
d	Depth of flow in an open channel or pipe flowing part full
d_c	Critical depth
d_n	Normal depth
D_o	Outside diameter of a pipe
Dia, ϕ	Diameter; diameter of a bar
Dim	Dimension
DOC	Designers' operating criteria
Dwg	Drawing
DWS	Delivery water surface
E	Energy
E_c	Critical energy
E_s	Specific energy
EG	Energy gradient
El	Elevation
ΔEl	Difference in elevations

Symbol or abbreviation	Definition
F	Froude number; difference in energy elevations; difference in water surface elevations; difference in invert elevations
f_c	Allowable extreme fiber compressive stress in concrete
f'_c	Specified compressive strength of concrete
f_s	Allowable tensile stress in reinforcement steel
f_y	Specified yield strength of reinforcement steel
Fb, f_b	Freeboard
fig	Figure
fps	Feet per second
Frm	Frame
ft	foot or feet
ft^2	Square feet
g	Gravitational acceleration
Ga	Gage
galv	Galvanized
H,h	Hydrostatic head; head on the centerline of a free-flow orifice or head across a submerged orifice; head across a structure; head on a weir crest; head at the centerline of a pipe; head on a siphon spillway; atmospheric pressure head
ΔH	Difference in water surface elevations; differential pressure head
ΣH	Summation of lateral forces acting parallel to assumed sliding plane
h_b	Head loss in a pipe bend
h_f	Head loss due to friction

Symbol or abbreviation	Definition
h_v	Velocity head
Δh_v	Difference in velocity heads
h_{v_c}	Velocity head for flow at critical flow
hd	Head
hex	Hexagon, hexagonal
Ht	Height
ID	Inside diameter of a pipe
In	Inch or inches
Inv	Invert
ΔInv	Difference in invert elevations
K	Head loss coefficient; acceleration factor for trajectory of flowing water
L	Length; length of a weir crest; length of a pipe; length of a structure; length of a transition
L_w	Length of weighted creep for percolating water
Lbs	Pounds
LF	Load factor
Longit	Longitudinal
$\underline{M}$	Montuori number
mach	Machine (machine bolt)
max	Maximum
min	Minimum
Misc	Miscellaneous
n	Manning's roughness coefficient
ΣN	Summation of forces acting normal to assumed sliding plane

Symbol or abbreviation	Definition
No, N, #	Number
NWS	Normal water surface
O&M	Operation and maintenance
OD	Outside diameter of pipe
OGS	Original ground surface
PC	Point of curvature
PCP	Precast reinforced concrete pressure pipe
PF	Percolation factor
PI	Point of intersection
Pl	Plate
proj	Projection; project
psf	Pounds per square foot
psi	Pounds per square inch
PT	Point of tangency
Q	Discharge; rate of flow in cubic feet per second
q	Discharge per foot of width in cubic feet per second
R	Hydraulic radius; radius of a circular curve; radius of a pipe bend; radius of a pipe
r	Hydraulic radius
RCCP	Precast reinforced concrete culvert pipe
Reinf	Reinforcement
req'd	Required
RI	Rectangular inclined (rectangular inclined drop)
ROW	Right-of-way
RPM	Reinforced plastic mortar pressure pipe

Symbol or abbreviation	Definition
s	Slope of energy gradient in the Manning equation (friction loss in feet per foot); invert slope of channel or pipe
s_a	Average slope of energy gradient between two points
s_b	Slope of profile of channel bottom
s_c	Critical slope
s_f	Friction slope; head loss due to friction in feet per foot
sq	Square
S:S, ss	Side slope of channel, horizontal to vertical ratio
S4S	Dressed lumber, surfaced on four sides
spc	Spaces
Sta	Station
Stnd	Standard
Str, Struct	Structure
Symm	Symmetrical
T	Thrust acting on a pipe bend; top width of water surface in an open channel
t	Time in seconds; concrete thickness
TO	Turnout
Transv	Transverse
V	Velocity
$\underline{V}$	Vedernikov number
V_c	Critical velocity
V_t	Theoretical velocity
ΔV	Difference in velocities

Symbol or abbreviation	Definition
ΣV	Summation of vertical forces
vert	Vertical
W	Throat width of Parshall flume; length of weir crest; baffled outlet basin width; roadway width
w	Unit weight
wp	Wetted perimeter
WS	Water surface
ΔWS	Difference in water surface elevations
WSP	Welded steel pipe
X	A coordinate for defining a trajectory profile
Y	A coordinate for defining a trajectory profile
$\bar{y}$	Depth from water surface or hydraulic gradient to center of gravity of a water prism cross section
yds	Yards
Z	Difference in invert elevations between two points

GREEK LETTERS

Greek letter	Greek name	Greek letter	Greek name
α	Alpha	μ	Mu
β	Beta	π	Pi
Δ, δ	Delta	ρ	Rho
ζ	Zeta	Σ	Sigma
θ	Theta	ϕ	Phi

Conversion Factors

B-1. *International System (SI metric)/U.S. Customary Conversion Factors*

To convert from	To	Multiply by	To convert from	To	Multiply by
LENGTH					
angstrom units	nanometers (nm)	0.1	feet	millimeters	304.8
	micrometers (μm)	1×10^{-4}		meters	0.3048
	millimeters (mm)	1×10^{-7}		inches	12
	meters (m)	1×10^{-10}		yards (yd)	0.333 333
	mils	$3.937\ 01 \times 10^{-6}$	yards	meters	0.9144
	inches (in)	$3.937\ 01 \times 10^{-9}$		inches	36
micrometers	millimeters	1×10^{-3}		feet	3
	meters	1×10^{-6}	meters	millimeters	1×10^{3}
	angstrom units (A)	1×10^{4}		kilometers (km)	1×10^{-3}
	mils	0.039 370		inches	39.3701
	inches	$3.937\ 01 \times 10^{-5}$		yards	1.093 61
millimeters	micrometers	1×10^{3}		miles (mi)	$6.213\ 71 \times 10^{-4}$
	centimeters (cm)	0.1	kilometers	meters	1×10^{3}
	meters	1×10^{-3}		feet	$3.280\ 84 \times 10^{3}$
	mils	39.3701		miles	0.621 371
	inches	0.039 370	miles	meters	$1.609\ 34 \times 10^{3}$
	feet (ft)	$3.280\ 84 \times 10^{-3}$		kilometers	1.609 34
centimeters	millimeters	10		feet	5.28×10^{3}
	meters	0.01		yards	1.76×10^{3}
	mils	393.701	nautical miles (nmi)	kilometers	1.853 24
	inches	0.393 701		miles	1.151 55
	feet	0.032 808			
inches	millimeters	25.4			
	meters	0.0254			
	mils	1×10^{3}			
	feet	0.083 333			
AREA					
square millimeters	square centimeters (cm^2)	0.01	square meters	hectares	1×10^{-4}
	square inches (in^2)	1.55×10^{-3}		square feet	10.7639
square centimeters	square millimeters (mm^2)	100		acres	$2.471\ 05 \times 10^{-4}$
	square meters (m^2)	1×10^{-4}		square yards (yd^2)	1.195 99
	square inches	0.155	acres	square meters	$4.046\ 86 \times 10^{3}$
	square feet (ft^2)	$1.076\ 39 \times 10^{-3}$		hectares	0.404 686
square inches	square millimeters	645.16		square feet	4.356×10^{4}
	square centimeters	6.4516	hectares	square meters	1×10^{4}
	square meters	6.4516×10^{-4}		acres	2.471 05
	square feet	$6.944\ 44 \times 10^{-3}$	square kilometers	square meters	1×10^{6}
square feet	square meters	0.092 903		hectares	100
	hectares (ha)	$0.092\ 903 \times 10^{-4}$		square feet	$1.076\ 39 \times 10^{7}$
	square inches	144		acres	247.105
	acres	$2.295\ 68 \times 10^{-5}$		square miles (mi^2)	0.386 102
square yards	square meters	0.836 127	square miles	square meters	$2.589\ 99 \times 10^{6}$
	hectares	$8.361\ 27 \times 10^{-5}$		hectares	258.999
	square feet	9		square kilometers (km^2)	2.589 99
	acres	$2.066\ 12 \times 10^{-4}$		square feet	$2.787\ 84 \times 10^{7}$
				acres	640

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VOLUME—CAPACITY

To convert from	To	Multiply by	To convert from	To	Multiply by
cubic milli-	cubic centimeters (cm^3)	1×10^{-3}	cubic miles	cubic dekameters	$4.168\ 18 \times 10^{6}$
meters	liters (l)	1×10^{-6}		cubic kilometers (km^3)	4.168 18
	cubic inches (in^3)	$6.102\ 37 \times 10^{-5}$		acre-feet	3.3792×10^{6}
cubic centi-	liters	1×10^{-3}	cubic yards	cubic meters	0.764 555
meters	milliliters (ml)	1		cubic feet	27
	cubic inches	0.061 023	cubic meters	liters	1×10^{3}
	fluid ounces (fl. oz)	0.033 814		cubic dekameters	1×10^{-3}
milliliters	liters	1×10^{-3}		gallons	264.172
	cubic centimeters	1		cubic feet	35.3147
cubic inches	milliliters	16.3871		cubic yards	1.307 95
	cubic feet (ft^3)	$5.787\ 04 \times 10^{-4}$		acre-feet	$8.107\ 13 \times 10^{-4}$
liters	cubic meters	1×10^{-3}	acre-feet	cubic meters	$1.233\ 48 \times 10^{3}$
	cubic feet	0.035 315		cubic dekameters	1.233 48
	gallons	0.264 172		cubic kilometers	$1.233\ 48 \times 10^{-6}$
	fluid ounces	33.8140		cubic feet	4.356×10^{4}
gallons	liters	3.785 41		gallons	$3.258\ 51 \times 10^{5}$
	cubic meters	$3.785\ 41 \times 10^{-3}$	cubic deka-	cubic meters	1×10^{3}
	fluid ounces	128	meters	cubic feet	$3.531\ 47 \times 10^{4}$
	cubic feet	0.133 681		acre-feet	0.810 713
cubic feet	liters	28.3169		gallons	$2.641\ 72 \times 10^{5}$
	cubic meters (m^3)	0.028 317	cubic kilometers	cubic dekameters	1×10^{6}
	cubic dekameters (dam^3)	$2.831\ 69 \times 10^{-5}$		acre-feet	$8.107\ 13 \times 10^{5}$
	cubic inches	1.728×10^{3}		cubic miles (mi^3)	0.239 913
	cubic yards (yd^3)	0.037 037			
	gallons (gal)	7.480 52			
	acre-feet (acre-ft)	$2.295\ 68 \times 10^{-5}$			

ACCELERATION

To convert from	To	Multiply by	To convert from	To	Multiply by
feet per second	meters per second	0.3048	G's (standard	meters per second	9.806 65
squared	squared (m/s^2)		gravitational	squared	
	G's	0.031 081	acceleration)	feet per second squared	32.1741
meters per	feet per second	3.280 84			
second	squared (ft/s^2)				
squared	G's	0.101 972			

VELOCITY

To convert from	To	Multiply by	To convert from	To	Multiply by
feet per second	meters per second (m/s)	0.3048	kilometers per	meters per second	0.277 778
	kilometers per hour	1.097 28	hour	feet per second	0.911 345
	(km/h)			miles per hour	0.621 371
	miles per hour (mi/h)	0.681 82	miles per hour	kilometers per hour	1.609 34
meters per	kilometers per hour	3.6		meters per second	0.447 04
second	feet per second (ft/s)	3.280 84		feet per second	1.466 67
	miles per hour	2.236 94	feet per year	millimeters per	$9.665\ 14 \times 10^{-6}$
			(ft/yr)	second (mm/s)	

FORCE

To convert from	To	Multiply by	To convert from	To	Multiply by
pounds	newtons (N)	4.448 22	newtons	pounds	0.224 809
kilograms	newtons	9.806 65	dynes	newtons	1×10^{-5}
(force)	pounds (lb)	2.204 62			

MASS

To convert from	To	Multiply by	To convert from	To	Multiply by
grams	kilograms (kg)	1×10^{-3}	short tons	kilograms	907.185
	ounces (avdp)	0.035 274		metric tons (t)	0.907 185
ounces (avdp)	grams (g)	28.3495		pounds (avdp)	2×10^{3}
	kilograms	0.028 350	metric tons	kilograms	1×10^{3}
	pounds (avdp)	0.0625	(tonne or	pounds (avdp)	$2.204\ 62 \times 10^{3}$
pounds (avdp)	kilograms	0.453 592	megagram)	short tons	1.102 31
	ounces (avdp)	16	long tons	kilograms	$1.016\ 05 \times 10^{3}$
kilograms	kilograms (force)-second	0.101 972		metric tons	1.016 05
	squared per meter			pounds (avdp)	2.24×10^{3}
	($kgf \cdot s^2/m$)			short tons	1.12
	pounds (avdp)	2.204 62			
	slugs	0.068 522			
slugs	kilograms	14.5939			

TEMPERATURE

To convert from	To	Multiply by	To convert from	To	Multiply by
degrees Celsius	kelvin (K)	$t_K = t_{°C} + 273.15$	degrees Rankine	kelvin	$t_K = t_{°R}/1.8$
degrees Fahrenheit	degrees Celsius	$t_{°C} = (t_{°F} - 32)/1.8$			
degrees Fahrenheit	kelvin	$t_K = (t_{°F} + 459.67)/1.8$			

VOLUME PER UNIT TIME FLOW

To convert from	To	Multiply by	To convert from	To	Multiply by
cubic feet per second	liters per second (l/s)	28.3169	acre-feet per day	cubic meters per second	0.014 276
	cubic meters per second (m^3/s)	0.028 317		cubic dekameters per day	1.233 48
	cubic dekameters per day (dam^3/d)	2.446 57		cubic feet per second	0.504 167
	gallons per minute (gal/min)	448.831	cubic dekameters per day	cubic meters per second	0.011 574
	acre-feet per day (acre-ft/d)	1.983 47		cubic feet per second	0.408 735
	cubic feet per minute (ft^3/min)	60		acre-feet per day	0.810 713
gallons per minute	cubic meters per second	$6.309\ 02 \times 10^{-5}$	cubic meters per second	acre-feet per day	70.0456
	liters per second	0.063 090		cubic feet per second	35.3147
	liters per minute	3.785 41		gallons per minute	$1.585\ 03 \times 10^{4}$
	cubic dekameters per day	$5.450\ 96 \times 10^{-3}$		liters per second	1×10^{3}
	cubic feet per second (ft^3/s)	$2.228\ 01 \times 10^{-3}$		million gallons per day	22.8245
	acre-feet per day	$4.419\ 19 \times 10^{-3}$	million gallons per day (mgd)	cubic meters per second	0.043 813

FORCE PER UNIT AREA PRESSURE—STRESS

To convert from	To	Multiply by	To convert from	To	Multiply by
pounds per square inch	kilopascals (kPa)	6.894 76	kilopascals	newtons per square meter (N/m^2)	1×10^{3}
	[1]meters-head	0.703 091		[2]mm of Hg	7.498 55
	[2]mm of Hg	51.7007		[1]meters-head	0.101 975
	[1]feet of water	2.306 73		[2]inches of Hg	0.295 218
	pounds per square foot (lb/ft^2)	144		pounds per square foot	20.8855
	std. atmospheres	0.068 046		pounds per square inch	0.145 039
pounds per square foot	kilopascals	0.047 880		std. atmospheres	$9.869\ 26 \times 10^{-3}$
	[1]meters-head	$4.882\ 60 \times 10^{-3}$	kilograms (f) per square meter	kilopascals	$9.806\ 65 \times 10^{-3}$
	[2]mm of Hg	0.359 033		[2]mm of Hg	0.073 556
	[1]feet of water	0.016 019		pounds per square inch	$1.422\ 73 \times 10^{-3}$
	pounds per square inch	$6.944\ 44 \times 10^{-3}$	millibars (mbar)	kilopascals	0.1
	std. atmospheres	4.7254×10^{-4}	bars	kilopascals	100
short tons per square foot	kilopascals	95.7605	std. atmospheres	kilopascals	101.325
	pounds per square inch (lb/in^2)	13.8889		[2]mm of Hg	760
[1]meters-head	kilopascals	9.806 38		pounds per square inch	14.70
	[2]mm of Hg	73.5334		[1]feet of water	33.90
	[1]feet of water	3.280 84			
	pounds per square inch	1.422 29			
	pounds per square foot	204.810			
[1]feet of water	kilopascals	2.988 98			
	[1]meters-head	0.3048			
	[2]mm of Hg	22.4130			
	[2]inches of Hg	0.882 401			
	pounds per square inch	0.433 514			
	pounds per square foot	62.4259			

[1]Column of H_2O (water) measured at 4 °C.
[2]Column of Hg (mercury) measured at 0 °C.

MASS PER UNIT VOLUME DENSITY AND MASS CAPACITY

To convert from	To	Multiply by
pounds per cubic foot	kilograms per cubic meter (kg/m^3)	16.0185
	slugs per cubic foot ($slug/ft^3$)	0.031 081
	pounds per gallon (lb/gal)	0.133 681
pounds per gallon	kilograms per cubic meter (kg/m^3)	119.826
	slugs per cubic foot	0.232 502
pounds per cubic yard	kilograms per cubic meter	0.593 277
	pounds per cubic foot (lb/ft^3)	0.037 037
grams per cubic centimeter	kilograms per cubic meter	1×10^3
	pounds per cubic yard	$1.685\ 55 \times 10^3$
ounces per gallon (oz/gal)	grams per liter (g/l)	7.489 15
	kilograms per cubic meter	7.489 15
kilograms per cubic meter	grams per cubic centimeter (g/cm^3)	1×10^{-3}
	metric tons per cubic meter (t/m^3)	1×10^{-3}
	pounds per cubic foot (lb/ft^3)	0.062 430
	pounds per gallon	$8.345\ 40 \times 10^{-3}$
	pounds per cubic yard	1.685 56
long tons per cubic yard	kilograms per cubic meter	1.328 94
ounces per cubic inch (oz/in^3)	kilograms per cubic meter	$1.729\ 99 \times 10^3$
slugs per cubic foot	kilograms per cubic meter	515.379

VISCOSITY

To convert from	To	Multiply by
centipoise	pascal-second (Pa · s)	1×10^{-3}
	poise	0.01
	pounds per foot-hour (lb/ft · h)	2.419 09
	pounds per foot-second (lb/ft · s)	$6.719\ 69 \times 10^{-4}$
	slug per foot-second (slug/ft · s)	$2.088\ 54 \times 10^{-5}$
pascal-second	centipoise	1×10^3
	pounds per foot-hour	$2.419\ 09 \times 10^3$
	pounds per foot-second	0.671 969
	slug per foot-second	0.020 885
pounds per foot-hour	pascal-second	$4.133\ 79 \times 10^{-4}$
	pounds per foot-second	$2.777\ 78 \times 10^{-4}$
	centipoise	0.413 38
pounds per foot-second	pascal-second	1.488 16
	slug per foot-second	0.031 081
	centipoise	$1.488\ 16 \times 10^3$
centistokes	square meters per second (m^2/s)	1×10^{-6}
	square feet per second (ft^2/s)	$1.076\ 39 \times 10^{-5}$
	stokes	0.01
square feet per second	square meters per second	0.092 903
	centistokes	$9.290\ 30 \times 10^4$
stokes	square meters per second	1×10^{-4}
rhe	1 per pascal-second (1/Pa · s)	10

Appendix C

Computer Program

C-1. *Program 223-E3WSP.*–

A COMPUTER PROGRAM TITLED 223-E3WSP HAS BEEN DEVELOPED BY THE BUREAU OF RECLAMATION TO DETERMINE WATER SURFACE PROFILES IN RECTANGULAR, TRAPEZOIDAL, CIRCULAR AND "MODIFIED HORSESHOE" SHAPED CHANNELS. THE PROGRAM WILL COMPUTE BACKWATER CURVES IN RECTANGULAR OR TRAPEZOIDAL CHANNELS AND WILL COMPUTE THE CONJUGATE DEPTH IN TRAPEZOIDAL AND RECTANGULAR STILLING BASINS. A "MODIFIED HORSESHOE" SHAPED CHANNEL IS DEFINED AS A SHAPE WITH A SEMI-CIRCULAR TOP, VERTICAL SIDES (ANY HEIGHT) AND A BOTTOM WHICH MAY BE FLAT OR HAVE THE SHAPE OF AN INVERTED "V". THE HEIGHT OF THE "V" CANNOT EXCEED THE HEIGHT OF THE SIDES. THE PROGRAM COMPUTES THE VELOCITY, THE WATER DEPTH, AND THE ENERGY GRADE ELEVATION AT EACH SECTION.

WHEN COMPUTING THE WATER SURFACE PROFILE, THE DEPTH OF WATER IS ASSUMED TO BE NORMAL TO THE SLOPE OF THE FLOOR AND THE DISTANCE BETWEEN SECTIONS IS EQUAL TO THE SLOPE DISTANCE. WHEN COMPUTING A BACKWATER CURVE, THE DEPTH OF WATER IS ASSUMED TO BE VERTICAL AND THE DISTANCE BETWEEN SECTIONS IS EQUAL TO THE HORIZONTAL DISTANCE. THE PROGRAM PROVIDES FOR CHANGES IN SHAPE AT ANY

SECTION BUT DOES NOT INCLUDE A LOSS CAUSED BY A CHANGE IN SHAPE. A TRIAL AND ERROR METHOD IS USED, THEREFORE THE INPUT DATA SHOULD BE REASONABLE. IF THE DEPTH IS EXTREMELY SHALLOW WITH A HIGH VELOCITY, THE NUMBER OF TRIALS BECOMES EXCESSIVE. A SIMILIAR SITUATION MAY OCCUR IF THE DISTANCE BETWEEN SECTIONS IS UNREASONABLY LONG. IF IT IS ANTICIPATED THAT THE RATE OF CHANGE IN DEPTH OR VELOCITY IS GREAT, THE SECTIONS SHOULD BE CLOSER THAN THEY WOULD BE NORMALLY.

THE PROGRAM IS LIMITED TO DEPTHS LESS THAN CRITICAL DEPTH WHEN COMPUTING A WATER SURFACE PROFILE AND TO DEPTHS GREATER THAN CRITICAL DEPTH WHEN COMPUTING BACKWATER CURVES.

IT IS ASSUMED THAT IF THE DEPTH OF WATER EXCEEDS 0.9 TIMES THE HEIGHT OF A CIRCULAR OR "MODIFIED HORSESHOE" SECTION, THE FLOW WILL BECOME UNSTABLE. WHEN THIS CONDITION OCCURS, A MESSAGE IS PRINTED STATING " THE DEPTH IS GREATER THAN 0.9 TIMES THE HEIGHT OF THE STRUCTURE." EXECUTION OF THE PROGRAM WILL THEN CONTINUE PROVIDED THAT THE DEPTH OF WATER IS LESS THAN CRITICAL DEPTH.

WHEN COMPUTING THE WATER SURFACE PROFILE A CHECK IS MADE TO DETERMINE IF A SOLUTION IS POSSIBLE. IF THE HEAD AVAILABLE IS INSUFFICIENT TO OVERCOME THE LOSSES THAT WILL BE INCURRED, AN APPROPRIATE MESSAGE WILL BE PRINTED OUT AND THE EXECUTION WILL BE TERMINATED.

INPUT

THE INPUT REQUIRED IS :

(1) CHANNEL SHAPE OF THE INITIAL SECTION. ENTER EITHER:

(A) RECTANGULAR,

(B) TRAPEZOIDAL,

(C) CIRCULAR, OR

(D) "MODIFIED HORSESHOE" (MUST BE ENCLOSED IN QUOTES).

(2) ASSUMED DISCHARGE IN C. F. S.

(3) MANNING'S "N" VALUE.

(4) INITIAL DEPTH (IF A BACKWATER CURVE IS BEING COMPUTED, THE INITIAL DEPTH MUST BE ABOVE CRITICAL DEPTH. IF A WATER SURFACE PROFILE IS BEING COMPUTED THE INITIAL DEPTH MUST BE BELOW CRITICAL.)

(5) INITIAL STATION (IF A BACKWATER CURVE IS BEING COMPUTED, THIS STATION MUST BE THE DOWNSTREAM STATION).

(6) INITIAL INVERT ELEVATION.

(7) INITIAL SLOPE OF THE INVERT (VERTICAL OVER HORIZONTAL).

(8) ADDITIONAL DATA REQUIRED TO DESCRIBE THE INITIAL SECTION:

(A) FOR RECTANGULAR SECTIONS, ENTER:

INITIAL WIDTH

(B) FOR TRAPEZOIDAL SECTIONS, ENTER:

INITIAL WIDTH, AND

INITIAL SLOPE OF SIDES (HORIZONTAL OVER VERTICAL).

(C) FOR CIRCULAR SECTIONS, ENTER:

INITIAL DIAMETER

(D) FOR "MODIFIED HORSESHOE" SECTIONS, ENTER:

INITIAL WIDTH,

INITIAL HEIGHT TO SPRINGLINE, AND

INITIAL HEIGHT OF "V" BOTTOM (IF BOTTOM
IS FLAT ENTER 0).

(9) DIMENSIONS OF EACH SECTION USED MUST BE ENTERED. THE DATA MUST BE IN THE SAME SEQUENCE AS THE DIRECTION OF FLOW (REVERSE OF THE DIRECTION OF FLOW WHEN COMPUTING A BACKWATER CURVE). THE FIRST ITEM IN THE FOLLOWING LISTS (R,T,C, OR MH) DESIGNATES THE SHAPE OF THE SECTION.

(A) FOR RECTANGULAR SECTIONS ENTER:

R,
STATION,
INVERT ELEVATION,
WIDTH, AND
SLOPE OF INVERT (VERTICAL OVER HORIZONTAL)

(B) FOR TRAPEZOIDAL SECTIONS ENTER:

T,
STATION,
INVEPT ELEVATION,
WIDTH,
SLOPE OF THE SIDES (HORIZONTAL OVER
VERTICAL), AND
SLOPE OF THE INVERT,(VERTICAL OVER THE
HORIZONTAL)

(C) FOR CIRCULAR SECTIONS ENTER:

C,
STATION,
INVERT ELEVATION,

DIAMETER, AND

SLOPE OF THE INVERT (VERTICAL OVER HORIZONTAL).

(D) FOR "MODIFIED HORSESHOE" SECTIONS ENTER:

MH,

STATION,

INVERT ELEVATION,

WIDTH,

HEIGHT TO THE SPRINGLINE,

HEIGHT OF THE "V" BOTTOM, AND

SLOPE OF THE INVERT (VERTICAL OVER HORIZONTAL)

(10) THE LAST DATA ENTRY SHOULD BE THE WORD "END".

OUTPUT

THE OUTPUT WILL CONSIST OF:

(1) INITIAL DATA,

(A) DISCHARGE

(B) MANNING'S N

(C) INITIAL DEPTH

(D) INITIAL WIDTH

(2) STATION,

(3) INVERT ELEVATION,

(4) VELOCITY,

(5) ENERGY GRADIENT,

(6) DEPTH,

(7) CONJUGATE WATER DEPTH AND VELOCITY IF REQUESTED, AND

(8) INPUT DATA IF REQUESTED.

A LISTING OF THE PROGRAM AND SAMPLE RESULTS ARE SHOWN IN THIS APPENDIX. INPUT DATA WAS TAKEN FROM THE CHUTE STRUCTURE USED AS A DESIGN EXAMPLE IN CHAPTER II. STATIONS USED IN THE EXAMPLE HAVE BEEN TRUNCATED TO MEET REQUIREMENTS OF THE PROGRAM.

DISCLAIMER STATEMENT

CØMPUTER PRØGRAMS DEVELØPED BY THE BUREAU ØF RECLAMATIØN ARE SUBJECT TØ THE FØLLØWING CØNDITIØNS. CØNSULTING SERVICE AND ASSISTANCE WITH CØNVERSIØNS TØ ØTHER CØMPUTER SYSTEMS CANNØT BE PRØVIDED. NØ WARRANTY AS TØ THE ACCURACY, USEFULNESS, ØR CØMPLETENESS ØF THE PRØGRAMS IS EXPRESSED ØR IMPLIED.

PRØGRAM 223-E3WSP

```
10    REM WATER SURFACE PRØFILE PRØGRAM (223-E3WSP)  SYSTEM-BASIC
100   PRINT "WHAT IS THE NAME ØF THE DATA FILE";
110   INPUT F$
120   FILE#1=F$
130   PRINT "IS THIS A WATER SURFACE PRØFILE CØMPUTATIØN--YES ØR NØ";
140   INPUT G$
150   PRINT
170   FØR I=1 TØ 10
180   INPUT#1,A$
190   IF A$="RECTANGULAR" THEN 250
200   IF A$="TRAPEZØIDAL" THEN 250
210   IF A$="CIRCULAR" THEN 250
220   IF A$="MØDIFIED HØRSESHØE" THEN 250
230   PRINT TAB(10);A$
240   NEXT I
250   PRINT
280   IF G$="NØ" THEN 310
290   PRINT "WATER SURFACE PRØFILE  -  ";A$;" SECTIØN"
300   GØ TØ 320
310   PRINT "BACKWATER CURVE  -";A$;" SECTIØN"
320   INPUT#1,Q,N,D1,Z1,E1,T1
330   K=Q↑2/32.16
350   PRINT
360   PRINT"DISCHARGE=";Q;"C.F.S."
370   PRINT "MANNINGS N=";N
380   PRINT "INITIAL DEPTH=";D1;"FT."
390   IF A$="RECTANGULAR" THEN 2000
400   IF A$="TRAPEZØIDAL" THEN 4000
410   IF A$="CIRCULAR" THEN 6000
420   IF A$="MØDIFIED HØRSESHØE" THEN 8000
430   GØ TØ 2370
```

```
440   :##+##.##     #####.##      ###.###      #####.###     ###.##
450   Z3=INT(Z1/100)
460   Z4=Z1-Z3*100
470   IF Z4<10 THEN 500
480   PRINT USING 440,Z3,Z4,E1,V1,E,D1
490   GØ TØ 560
500   IF Z4<1 THEN 540
510   :##+0#.##     #####.##      ###.###      #####.###     ###.##
520   PRINT USING 510,Z3,Z4,E1,V1,E,D1
530   GØ TØ 560
540   :##+00.##     #####.##      ###.###      #####.###     ###.##
550   PRINT USING 540,Z3,Z4,E1,V1,E,D1
560   RETURN
570   PRINT
580   PRINT "STATIØN","ELEVATIØN","VELØCITY","E. G. ELEV.","DEPTH"
590   PRINT
600   RETURN
2000  REM THIS BLØCK (2000-3499) IS USED FØR CØMPUTATIØN ØF
2010  REM   RECTANGULAR SECTIØNS
2020  INPUT#1, W1
2030  PRINT "FIRST WIDTH=";W1;"FT."
2040  D3 =((Q/W1)↑2/32.16)↑.3333
2050  IF G$="YES" THEN 2080
2060  IF D3<D1 THEN 2100
2070  GØ TØ 2350
2080  IF D3>D1 THEN 2100
2090  GØ TØ 2320
2100  GØSUB 570
2110  B1=ATN(ABS(T1))
2120  A1 = D1*W1
2130  V1 = Q/(D1*W1)
2140  H1 = V1↑2/64.32
2150  E = E1 + D1*CØS(B1) + H1
2160  R1 = A1/(2*D1+W1)
2170  S1 = V1↑2*N↑2/(1.486↑2*R1↑(4/3))
2180  GØSUB 440
2190  INPUT#1,B$
2200  IF B$="MH" THEN 8980
2210  IF B$ = "T" THEN 4840
2220  IF B$ = "END" THEN 12020
2230  IF  B$ = "C" THEN 7010
2240  INPUT#1, Z2,E2,W2,T2
2250  M1=Ø1=Ø2=Ø3=Ø4=0
2260  P=Z2-Z1
2270  IF P=0 THEN 2370
2280  L9=SQR((E2-E1)↑2+P↑2)
2290  IF P<0 THEN 2970
2300  IF G$="YES" THEN 2410
2310  GØ TØ 2370
2320  PRINT "INITIAL DEPTH IS GREATER THAN CRITICAL DEPTH"
2330  PRINT "CRITICAL DEPTH =";D3
2340  STØP
2350  PRINT "INITIAL DEPTH IS LESS THAN CRITICAL DEPTH"
2360  GØ TØ 2330
2370  PRINT "ERRØR IN DATA"
2380  STØP
2390  T2=0
2400  L9=P
2410  D3 =((Q/W2)↑2/32.16)↑.3333
2420  B2=ATN(ABS(T2))
2430  X = E-E2
2440  D2 = D3
2450  A2 = D2*W2
```

```
2460  V2 = Q/A2
2470  GØ TØ 2520
2480  V2 = SQR(64.32*H)
2490  D2 = (Q/V2)/W2
2500  A2 = D2 * W2
2510  V2=Q/A2
2520  R2 = A2/(2*D2+W2)
2530  S2 = V2↑2*N↑2/(1.486↑2*R2↑(4/3))
2540  H3=L9*(S1+S2)/2
2550  H2 = V2↑2/64.32
2560  H4=D2*CØS(B2)+H2+H3
2570  IF M1>0 THEN 2670
2580  M1=1
2590  H=H8=H2
2600  IF P<0 THEN 2900
2610  IF H4<X THEN 2680
2620  IF D2*CØS(B2)+H2<X THEN 6500
2630  PRINT
2640  PRINT "NØ SØLUTIØN PØSSIBLE-"
2650  PRINT "CØNTRØL IS DØWNSTREAM ØF ABØVE STATIØN"
2660  GØ TØ 2330
2670  IF H4 >= X+.005 THEN 2690
2680  IF H4 > X-.005 THEN 2790
2690  GØ SUB 3520
2700  IF P>0 THEN 2760
2710  IF H<D3 THEN 2740
2720  D2=H
2730  GØ TØ 2450
2740  D2=D3
2750  GØ TØ 2450
2760  IF H>H8 THEN 2480
2770  H=H8
2780  GØ TØ 2480
2790  D1 = D2
2800  Z1 = Z2
2810  E1 = E2
2820  W1 = W2
2830  S1=S2
2840  V1=V2
2850  E=E-H3
2860  GØ TØ 2180
2870  PRINT TAB(6);"CHANGE ØF SECTIØN TØ RECTANGULAR"
2880  Y=Z9=0
2890  GØ TØ 2240
2900  IF X-H3<D2+H2 THEN 2930
2910  IF B$="T" THEN 4640
2920  GØ TØ 2670
2930  Z3=INT(Z1/100)
2940  Z4=Z1-100*Z3
2950  PRINT "CØNTRØL IS UPSTREAM ØF STATIØN";Z3;"+";Z4
2960  STØP
2970  IF G$="YES" THEN 2370
2980  IF B$="R" THEN 2390
2990  IF B$="T" THEN 5030
3500  REM THIS BLØCK (3500-3999) IS USED FØR INCREMENTING
3510  REM EITHER THE VELØCITY HEAD ØR DEPTH
3520  IF P<0 THEN 3550
3530  IF B$="R" THEN 3590
3540  IF B$="T" THEN 3590
3550  H=D2
3560  IF P<0 THEN 3590
3570  IF H4<X THEN 3630
3580  GØ TØ 3600
```

```
3590  IF H4>X THEN 3630
3600  IF Ø1>0 THEN 3680
3610  H=H+1
3620  RETURN
3630  IF Ø4>0 THEN 3780
3640  IF Ø2>0 THEN 3720
3650  H=H-.1
3660  Ø1=Ø1+1
3670  RETURN
3680  IF Ø3>0 THEN 3750
3690  H=H+.01
3700  Ø2=Ø2+1
3710  RETURN
3720  H=H-.001
3730  Ø3=Ø3+1
3740  RETURN
3750  H=H+.0001
3760  Ø4=Ø4+1
3770  RETURN
3780  IF B$="R" THEN 2790
3790  IF B$="T" THEN 4760
3800  IF B$="C" THEN 6460
3810  IF B$="MH" THEN 8900
4000  REM THIS BLØCK(4000-5999) IS USED FØR CØMPUTATIØN ØF
4010  REM   TRAPEZØIDAL SECTIØNS
4020  INPUT#1, W1,Z
4030  PRINT "FIRST SIDE SLØPE=";Z;"TØ 1"
4040  PRINT "FIRST WIDTH=";W1;"FT."
4050  IF G$="YES" THEN 4070
4060  T1=0
4070  W3=W1
4080  T1=ABS(T1)
4090  B1=ATN(T1)
4100  Y=SQR((Z+Z*T1↑2)↑2+T1↑2)
4110  GØSUB 4860
4120  IF G$="YES" THEN 4150
4130  IF D3<D1 THEN 4170
4140  GØ TØ 2350
4150  IF D3>D1 THEN 4170
4160  GØ TØ 2320
4170  GØSUB 570
4180  V1=Q/(D1*W1+D1↑2*Y)
4190  H1 = V1↑2/64.32
4200  E=E1+D1*CØS(B1)+H1
4210  R1=(D1*W1+D1↑2*Y)/(W1+2*SQR(D1↑2+(Y*D1)↑2))
4220  S1 = V1↑2*N↑2/(1.486↑2*R1↑(4/3))
4230  GØSUB 440
4240  INPUT#1, B$
4250  IF B$ = "R" THEN 2870
4260  IF B$="C" THEN 7010
4270  IF B$="MH" THEN 8980
4280  IF B$="END" THEN 12020
4290  INPUT#1, Z2,E2,W2,Z9,T2
4300  X=E-E2
4310  P=Z2-Z1
4320  IF P=0 THEN 2370
4330  IF P<0 THEN 2970
4340  IF G$="YES" THEN 4360
4350  GØ TØ 2370
4360  T2=ABS(T2)
4370  L9=SQR((E2-E1)↑2+P↑2)
4380  M1=Ø1=Ø2=Ø3=Ø4=0
4390  W3=W2
```

```
4400  D3=0
4410  B2=ATN(T2)
4420  Y=SQR((Z9+Z9*T2↑2)↑2+T2↑2)
4430  GØSUB 4860
4440  V2=Q/A3
4450  D2=D3
4460  GØ TØ 4530
4470  A2=D2*W2+D2↑2*Y
4480  V2=Q/A2
4490  GØ TØ 4530
4500  V2 = SQR(64.32*H)
4510  D2 = (-W2+SQR(W2↑2+4*Y*Q/V2))/(2*Y)
4520  GØ TØ 4470
4530  R2=(D2*W2+D2↑2*Y)/(W2+2*SQR(D2↑2+(Y*D2)↑2))
4540  S2 = V2↑2*N↑2/(1.486↑2*R2↑(4/3))
4550  H3=L9*(S1+S2)/2
4560  H2 = V2↑2/64.32
4570  H4=D2*CØS(B2)+H2+H3
4580  IF M1>0 THEN 4640
4590  M1=1
4600  H=H8=H2
4610  IF P<0 THEN 2900
4620  IF H4<X THEN 4650
4630  GØ TØ 2620
4640  IF H4>=X+.005 THEN 4660
4650  IF H4 > X-.005 THEN 4760
4660  GØ SUB 3520
4670  IF P>0 THEN 4730
4680  IF H<D3 THEN 4710
4690  D2=H
4700  GØ TØ 4470
4710  D2=D3
4720  GØ TØ 4470
4730  IF H>H8 THEN 4500
4740  H=H8
4750  GØ TØ 4500
4760  D1 = D2
4770  Z1 = Z2
4780  E1 = E2
4790  W1 = W2
4800  V1=V2
4810  E=E-H3
4820  S1=S2
4830  GØ TØ 4230
4840  PRINT TAB(6);"CHANGE ØF SECTIØN TØ TRAPEZØIDAL"
4850  GØ TØ 4290
4860  D3=D3+1.0
4870  GØSUB 4990
4880  IF K1<K THEN 4860
4890  D3=D3-.1
4900  GØSUB 4990
4910  IF K1>K THEN 4890
4920  D3=D3+.01
4930  GØSUB 4990
4940  IF K1<K THEN 4920
4950  D3=D3-.001
4960  GØSUB 4990
4970  IF K1>K THEN 4950
4980  RETURN
4990  T3=W3+2*D3*Y
5000  A3=W3*D3+D3↑2*Y
5010  K1=A3↑3/T3
5020  RETURN
```

```
5030  T2=B2=0
5040  L9=P
5050  GØ TØ 4380
6000  REM THIS BLØCK (6000-7999) IS USED FØR CØMPUTATIØN ØF
6010  REM   CIRCULAR SECTIØNS
6020  IF G$="NØ" THEN 6980
6030  INPUT#1, D8
6040  PRINT "DIAMETER IS";D8;"FT."
6050  PRINT
6060  B1=ATN(ABS(T1))
6070  B2 = B1
6080  R8 = D8/2
6090  GØSUB 6530
6100  D3=D2
6110  IF D1>D3 THEN 2320
6120  GØSUB 570
6130  D2 =D1
6140  GØSUB 6790
6150  E = E1 + D2*CØS(B2) + H2
6160  D1 = D2
6170  V1 = V2
6180  S1 = S2
6190  GØSUB 440
6200  IF D1<.9*D8 THEN 6230
6210  PRINT "DEPTH IS GREATER THAN .9 TIMES"
6220  PRINT "THE HEIGHT ØF THE STRUCTURE"
6230  INPUT#1, B$
6240  IF B$="MH" THEN 8980
6250  IF B$="R" THEN 2870
6260  IF B$="T" THEN 4840
6270  IF B$ = "END" THEN 13000
6280  INPUT#1, Z2,E2,D8,T2
6290  P=Z2-Z1
6300  B2=ATN(ABS(T2))
6310  IF P=0 THEN 2370
6320  IF P<0 THEN 6960
6330  X = E-E2
6340  R8 = D8/2
6350  GØ SUB 6530
6360  D3=D2
6370  IF H4>X THEN 2620
6380  H=H8=D3
6390  Ø1=Ø2=Ø3=Ø4=0
6400  GØSUB 3520
6410  D2=H
6420  GØ SUB 6790
6430  IF H4>X+.005 THEN 6400
6440  IF H4>X-.005 THEN 6460
6450  GØ TØ 6400
6460  Z1 = Z2
6470  E1 = E2
6480  E=E-H3
6490  GØ TØ 6160
6500  PRINT
6510  PRINT "SECTIØNS TØØ FAR APART"
6520  STØP
6530  H5 = 1.70141E38
6540  D2 = .01
6550  GØSUB 6790
6560  IF D2 + H2 > H5 THEN 6640
6570  H5 = D2+H2
6580  D2 = D2 + .1*D8
6590  IF D2<D8 THEN 6630
```

```
6600  D2=.9999*D8
6610  GØSUB 6790
6620  GØ TØ 6640
6630  GØ TØ 6550
6640  H5 = H2 + D2
6650  D2 = D2 - .01*D8
6660  GØSUB 6790
6670  IF D2 + H2 > H5 THEN 6690
6680  GØ TØ 6640
6690  H5 = D2 + H2
6700  D2 = D2 + .001*D8
6710  GØSUB 6790
6720  IF D2 + H2 > H5 THEN 6740
6730  GØ TØ 6690
6740  H5 = D2 + H2
6750  D2 = D2 - .0001*D8
6760  GØSUB 6790
6770  IF D2 + H2 < H5 THEN 6740
6780  RETURN
6790  IF D2 > R8 THEN 6830
6800  IF D2 < R8 THEN 6850
6810  B6 = 90/57.29577774
6820  GØ TØ 6860
6830  B6=180/57.29577774-ATN(SQR(R8↑2-(D2-R8)↑2)/(D2-R8))
6840  GØ TØ 6860
6850  B6 = ATN(SQR(R8↑2-(R8-D2)↑2)/(R8-D2))
6860  B8 = 2*B6
6870  A2 = R8↑2*(B6-.5*SIN(B8))
6880  P2 = R8*B8
6890  V2 = Q/A2
6900  R2 = A2/P2
6910  S2=V2↑2*N↑2/(1.486↑2*R2↑(4/3))
6920  H2 = V2↑2/64.32
6930  H3=SQR((E2-E1)↑2+(Z2-Z1)↑2)*(S1+S2)/2
6940  H4 = D2*CØS(B2) + H2 + H3
6950  RETURN
6960  PRINT
6970  IF G$="YES" THEN 2370
6980  PRINT "BACKWATER CURVES CANNØT BE"
6990  PRINT "CØMPUTED IN CIRCULAR SECTIØNS"
7000  STØP
7010  PRINT TAB(6);"CHANGE ØF SECTIØN TØ CIRCULAR"
7020  GØ TØ 6280
8000  REM THIS BLØCK (8000-9999) IS USED FØR CØMPUTATIØN ØF
8010  REM  HØRSESHØE SECTIØNS
8020  IF G$="NØ" THEN 8470
8030  INPUT#1, W1,D7,X1
8040  PRINT "FIRST WIDTH=";W1;"FT."
8050  PRINT "FIRST HEIGHT ØF SIDE=";D7;"FT."
8060  R4=W1/2
8070  D8=W1
8080  GØSUB 10000
8090  IF D3<D1 THEN 2320
8100  GØSUB 570
8110  B1=ATN(ABS(T1))
8120  IF D1 > D7 THEN 8210
8130  IF D1 > X1 THEN 8180
8140  L1 = D1*R4/X1
8150  A1 = D1*L1
8160  R1 = A1/(2*(D1+SQR(L1↑2+D1↑2)))
8170  GØ TØ 8300
8180  A1 = W1*D1-W1*X1/2
8190  R1 = A1/(2*D1+SQR(X1↑2+R4↑2)*2)
```

```
8200  GØ TØ 8300
8210  IF D1-D7 <> 0 THEN 8240
8220  B6 = 90/57.29577774
8230  GØ TØ 8250
8240  B6 = ATN(SQR(R4↑2 - (D1-D7)↑2)/(D1-D7))
8250  B8 = 2*B6
8260  A1=W1*D7+3.141593*D8↑2/8-.5*R4↑2*(B8-SIN(B8))-R4*X1
8270  P1 =2*D7+SQR(X1↑2+R4↑2)*2+1.570796*D8-R4*B8
8280  R1 = A1/P1
8290  S1=V1↑2*N↑2/(1.486↑2*R1↑(4/3))
8300  V1 = Q/A1
8310  H1 = V1↑2/64.32
8320  E = E1 + D1*CØS(B1) + H1
8330  GØSUB 440
8340  IF D1<.9*(D7+R4) THEN 8370
8350  PRINT "DEPTH IS GREATER THAN .9 TIMES "
8360  PRINT "THE HEIGHT ØF THE STRUCTURE"
8370  INPUT#1, B$
8380  IF B$ = "R" THEN 2870
8390  IF B$="T" THEN 4840
8400  IF B$="C" THEN 7010
8410  IF B$="END" THEN 13000
8420  INPUT#1, Z2,E2,W2,D7,X1,T2
8430  P=Z2-Z1
8440  IF P=0 THEN 2370
8450  IF P>0 THEN 8500
8460  IF G$="YES" THEN 2370
8470  PRINT "BACKWATER CURVES CANNØT BE"
8480  PRINT "CØMPUTED IN MØDIFIED HØRSESHØE SECTIØNS"
8490  STØP
8500  D8 = W2
8510  X=E-E2
8520  R4 = D8/2
8530  GØSUB 10000
8540  B2=ATN(ABS(T2))
8550  D2=D3
8560  M1=Ø1=Ø2=Ø3=Ø4=0
8570  GØ TØ 8580
8580  IF D2 > D7 THEN 8670
8590  IF D2 > X1 THEN 8640
8600  L2 = D2*R4/X1
8610  A2 = D2*L2
8620  R2 = A2/(2*(D2+SQR(L2↑2+D2↑2)))
8630  GØ TØ 8750
8640  A2 = D2*W2-W2*X1/2
8650  R2 = A2/(2*D2+SQR(X1↑2+R4↑2)*2)
8660  GØ TØ 8750
8670  IF (D2-D7) <> 0 THEN 8700
8680  B6 = 90/57.29577774
8690  GØ TØ 8710
8700  B6 = ATN(SQR(R4↑2-(D2-D7)↑2)/(D2-D7))
8710  B8 = 2*B6
8720  A2=W2*D7+3.141593*D8↑2/8-.5*R4↑2*(B8-SIN(B8))-X1*W2/2
8730  P2=2*D7+1.570796*D8-R4*B8+SQR(X1↑2+R4↑2)*2
8740  R2 = A2/P2
8750  V2 = Q/A2
8760  S2 = V2↑2*N↑2/(1.486↑2*R2↑(4/3))
8770  H3=SQR((E2-E1)↑2+(Z2-Z1)↑2)*(S1+S2)/2
8780  H2 = V2↑2/64.32
8790  H4=D2*CØS(B2)+H2+H3
8800  IF M1>0 THEN 8850
8810  M1=1
8820  H=H8=D3
```

```
8830  IF H4<X THEN 8860
8840  GØ TØ 2620
8850  IF H4>=X+.01 THEN 8870
8860  IF H4 >= X - .01 THEN 8900
8870  GØSUB 3520
8880  D2=H
8890  GØ TØ 8580
8900  V1=V2
8910  D1=D2
8920  S1=S2
8930  Z1 = Z2
8940  H1=H2
8950  E1 = E2
8960  E=E-H3
8970  GØ TØ 8330
8980  PRINT TAB(6);"CHANGE ØF SECTIØN TØ MØDIFIED HØRSESHØE"
8990  GØ TØ 8420
10000 REM THIS BLØCK (10000-11999) IS USED FØR CØMPUTATIØN ØF
10010 REM CRITICAL DEPTH IN A MØDIFIED HØRSESHØE SECTIØN
10020 K=Q↑2/32.16
10030 IF X1=0 THEN 10180
10040 K2= (.5*Q)↑2/32.16
10050 D3=X1
10060 A3=D3*R4/2
10070 K1=A3↑3/R4
10080 IF K1<K2 THEN 10180
10090 D3=D3-.01*X1
10100 T3=D3*R4/X1
10110 A3=T3*D3/2
10120 K1=A3↑3/T3
10130 IF K1>K2 THEN 10090
10140 R3=A3/(D3+SQR(T3↑2+D3↑2))
10150 V3=Q/(2*A3)
10160 S2=V3↑2*N↑2/(1.486↑2*R3↑(4/3))
10170 RETURN
10180 D3=D7
10190 GØSUB 10420
10200 IF K1<K THEN 10310
10210 D3=D3-.1*D7
10220 GØSUB 10420
10230 IF K1>K THEN 10210
10240 D3=D3+.01*D7
10250 GØSUB 10420
10260 IF K1<K THEN 10240
10270 D3=D3-.001*D7
10280 GØSUB 10420
10290 IF K1>K THEN 10270
10300 GØSUB 10450
10310 D3=.999*(R4+D7)
10320 D3=D3-.1*R4
10330 GØSUB 10480
10340 IF K1>K THEN 10320
10350 D3=D3+.01*R4
10360 GØSUB 10480
10370 IF K1<K THEN 10350
10380 D3=D3-.001*R4
10390 GØSUB 10480
10400 IF K1>K THEN 10380
10410 GØSUB 10540
10420 A3=D3*D8-X1*R4
10430 K1=A3↑3/D8
10440 RETURN
10450 R3=A3/(2*(D3+SQR(X1↑2+R4↑2)))
```

```
10460 V3=Q/A3
10470 GØ TØ 10160
10480 B6=ATN(SQR(R4↑2-(D3-D7)↑2)/(D3-D7))
10490 B8=2*B6
10500 A3=D8*D7+3.141593*D8↑2/8-.5*R4↑2*(B8-SIN(B8))-X1*D8/2
10510 T3=2*R4*SIN(B6)
10520 K1=A3↑3/T3
10530 RETURN
10540 P3=2*D7+1.570796*D8-R4*B8+SQR(X1↑2+R4↑2)*2
10550 R3=A3/P3
10560 GØSUB 10460
10570 RETURN
12000 REM THIS BLØCK (12000-12999) IS USED FØR THE CØMPUTATIØN ØF
12010 REM THE CØNJUGATE DEPTH(HYDRAULIC JUMP)
12020 IF P<0 THEN 13000
12030 PRINT
12040 PRINT "DØ YØU WANT THE CØNJUGATE DEPTH CØMPUTED--YES ØR NØ";
12050 INPUT C$
12060 PRINT
12070 IF C$ = "NØ" THEN 13000
12080 G = 32.16
12090 Y9=Y
12100 IF Z9=0 THEN 12230
12110 FØR D2=D3+1 TØ 100
12120 GØSUB 12280
12130 IF X>Y THEN 12150
12140 NEXT D2
12150 D2 = D2-.1
12160 GØSUB 12280
12170 IF X<Y THEN 12190
12180 GØ TØ 12150
12190 D2 = D2+.01
12200 GØSUB 12280
12210 IF X>Y THEN 12250
12220 GØ TØ 12190
12230 D2=-D1/2+SQR(D1↑2/4+2*V1↑2*D1/G)
12240 V2=Q/(D2*W1)
12250 PRINT TAB(6);"D2 = ";D2;" FT"
12260 PRINT TAB(6);"V2 = ";V2;" FPS"
12270 GØ TØ 13000
12280 Y=Q*V1/G+W1*D1↑2/2+Y9*D1↑3/3
12290 V2=Q/(W1*D2+Z9*D2↑2)
12300 X=Q*V2/G+W1*D2↑2/2+Z9*D2↑3/3
12310 RETURN
13000 REM THIS BLØCK(13000-13999) IS USED TØ
13010 REM LIST DATA FRØM THE DATA FILE
13020 PRINT
13030 PRINT "DØ YØU WANT THE INPUT DATA PRINTED ØUT--YES ØR NØ";
13040 INPUT C$
13050 IF C$="NØ" THEN 14000
13060 RESTØRE #1
13080 PRINT
13090 INPUT #1,A$
13100 IF A$="RECTANGULAR" THEN 13160
13110 IF A$="TRAPEZØIDAL" THEN 13190
13120 IF A$="CIRCULAR" THEN 13220
13130 IF A$="MØDIFIED HØRSESHØE" THEN13250
13140 PRINT A$
13150 GØ TØ 13090
13160 INPUT #1,Q,N,D1,Z1,E1,T1,W1
13170 PRINT A$;Q;N;D1;Z1;E1;T1;W1
13180 GØ TØ 13270
13190 INPUT #1,Q,N,D1,Z1,E1,T1,W1,Z
```

```
13200 PRINT A$;Q;N;D1;Z1;E1;T1;W1;Z
13210 GØ TØ 13270
13220 INPUT #1,Q,N,D1,Z1,E1,T1,D8
13230 PRINT A$;Q;N;D1;Z1;E1;T1;D8
13240 GØ TØ 13270
13250 INPUT #1,Q,N,D1,Z1,E1,T1,W1,D7,X1
13260 PRINT A$;Q;N;D1;Z1;E1;T1;W1;D7;X1
13270 INPUT #1,B$
13280 IF B$="R" THEN 13330
13290 IF B$="T" THEN 13360
13300 IF B$="C" THEN13390
13310 IF B$="MH" THEN 13420
13320 IF B$="END" THEN 13450
13330 INPUT #1,Z2,E2,W2,T2
13340 PRINT B$;Z2;E2;W2;T2
13350 GØ TØ 13270
13360 INPUT #1,Z2,E2,W2,Z9,T2
13370 PRINT B$;Z2;E2;W2;Z9;T2
13380 GØ TØ 13270
13390 INPUT #1,Z2,E2,D8,T2
13400 PRINT B$;Z2;E2;D8;T2
13410 GØ TØ 13270
13420 INPUT #1,Z2,E2,W2,D7,X1,T2
13430 PRINT B$;Z2;E2;W2;D7;X1;T2
13440 GØ TØ 13270
13450 PRINT B$
14000 END
```

EXAMPLE ØUTPUT FRØM PRØGRAM 223-E3WSP

```
DESIGN EXAMPLE CHUTE FRØM CHAPTER II

WATER SURFACE PRØFILE  -  RECTANGULAR SECTIØN

INITIAL DATA:

DISCHARGE        35      C.F.S.
MANNING'S N       0.010
INITIAL DEPTH     1.615  FT.
FIRST WIDTH       3      FT.
```

STATIØN	ELEVATIØN	VELØCITY	E. G. ELEV.	DEPTH
35+78.00	3703.00	7.224	3705.426	1.615
37+48.00	3689.12	22.908	3697.786	.509
38+05.00	3684.47	22.626	3692.942	.516
39+40.00	3670.28	24.531	3680.106	.476
40+46.42	3659.10	24.417	3668.845	.478
41+68.00	3652.73	19.495	3659.233	.598
42+00.00	3651.05	19.466	3657.528	.600
42+13.00	3650.37	19.527	3656.777	.479
42+16.00	3650.04	19.788	3656.581	.452
42+19.00	3649.38	20.538	3656.357	.417
42+23.82	3647.06	23.305	3655.810	.339
42+35.00	3642.06	27.024	3653.641	.259

```
DØ YØU WANT THE CØNJUGATE DEPTH CØMPUTED--YES ØR NØ?YES

D2 = 3.30285  FT.
V2 = 2.11938  FPS.
```

```
DØ YØU WANT THE INPUT DATA PRINTED ØUT--YES ØR NØ?YES

                    INPUT DATA


DESIGN EXAMPLE CHUTE FRØM CHAPTER II
RECTANGULAR 35  .01  1.615  3578  3703  0  3

R   3748       3689.12     3         .08163
R   3805       3684.47     3         .08163
R   3940       3670.28     3         .1051
R   4046.42    3659.10     3         .1051
R   4168       3652.73     3         .05241
R   4200       3651.05     3         .05241
R   4213       3650.37     3.74      .05241
R   4216       3650.04     3.91      .05241
R   4219       3649.38     4.09      .05241
R   4223.82    3647.06     4.43      .50000
R   4235       3642.06     5         .50000
                   END
```

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