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]

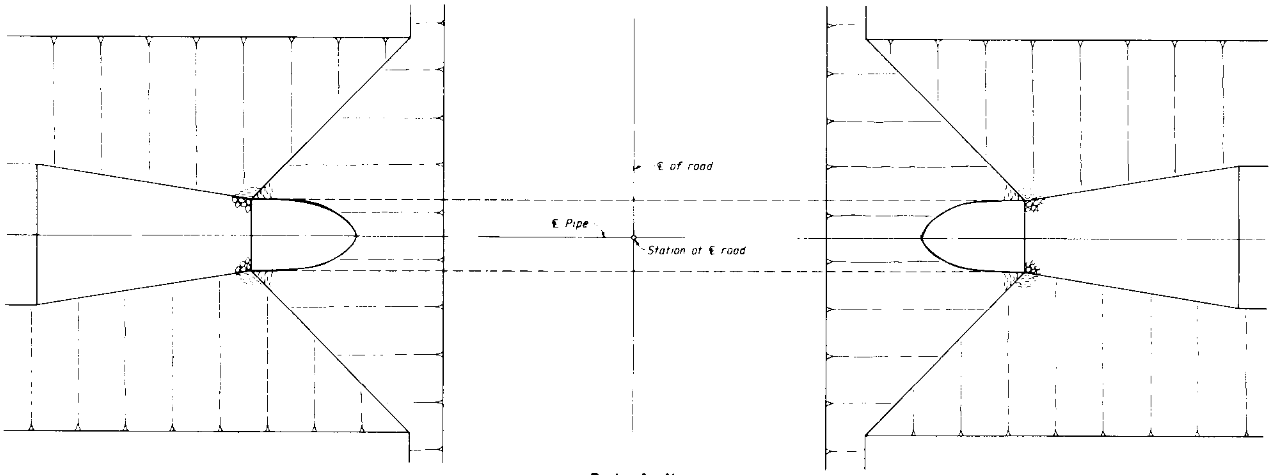
2-2. ***Purpose and Description.***—Road crossings are used to convey canal water under roads or railroads (fig. 2-1). Pipe conduit is generally used for these purposes. In accomplishing these objectives the road crossing conduit may have a straight line profile (see fig. 2-2), as discussed in these 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.

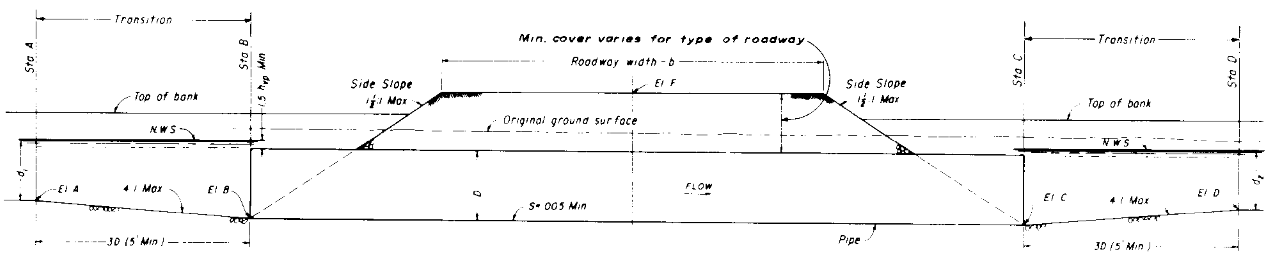
Figure 2-1. Road crossing.

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

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

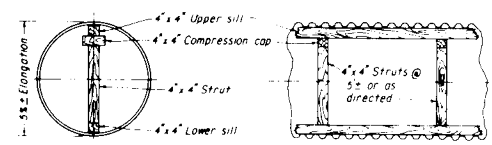


PLAN



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	0785
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.

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

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 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]

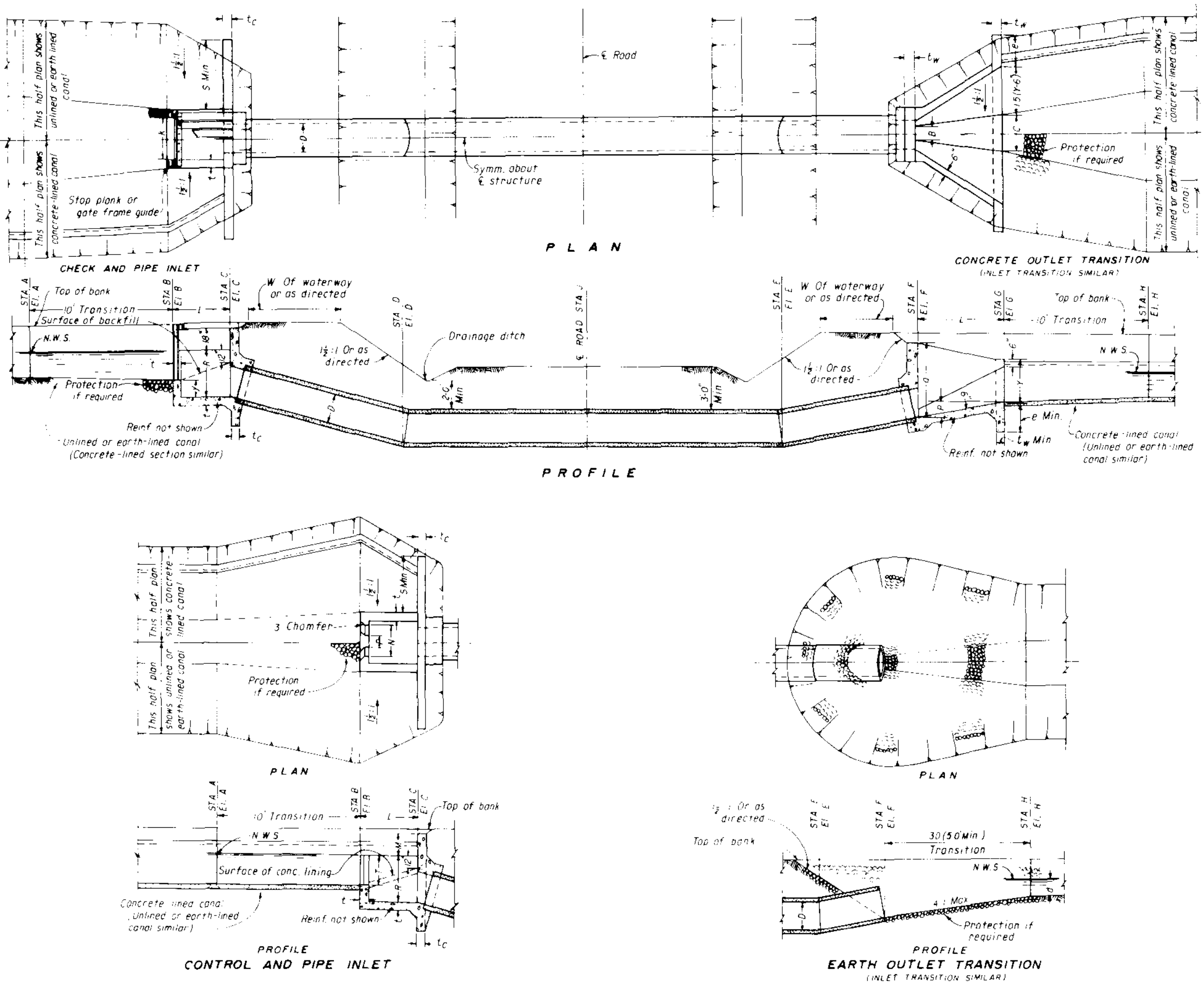
2-7. ***Purpose and Description.***—Inverted siphons figures 2-4, 2-5, and 2-6 (sometimes called sag pipes or sag lines) are used to convey canal water by gravity under roads, railroads, other structures, various types of drainage channels, and depressions. A siphon is a closed conduit designed to run 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.



PIPE DIAMETER SELECTION DATA							
MAX. V=3.5 fps (EARTH TRANSITION)		MAX. V=5.0 fps (CONC. TRANSITION)		MAX. V=10.0 fps (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.

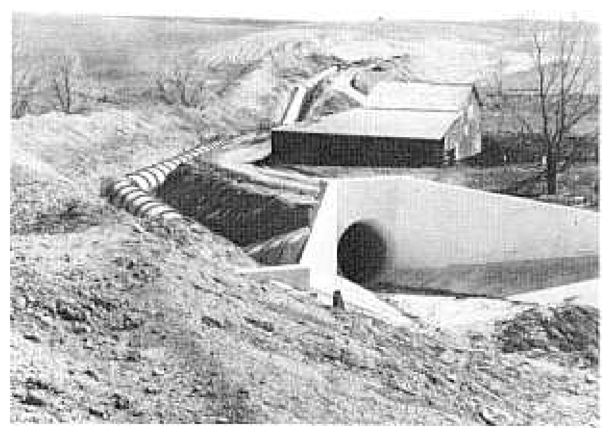


Figure 2-6. **Inverted siphon inlet transition and 60-inch-diameter precast concrete pipe. P 499-700-520.**

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.

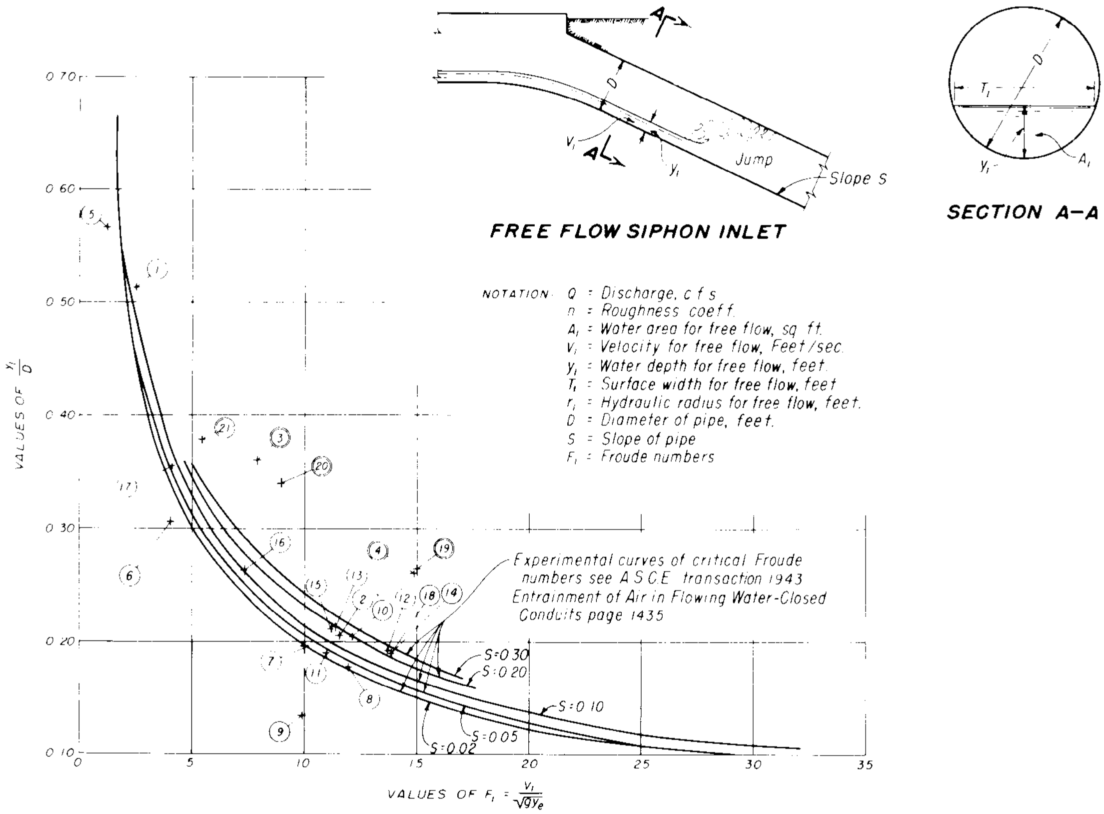
If the computed losses are greater than the difference in upstream and downstream canal water surface, the siphon will probably cause backwater in the canal upstream from the siphon. If backwater exists, the pipe size should be increased or the canal profile revised to provide adequate head.

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 1673+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. $F_1 = \frac{V_1}{\sqrt{g y_e}}$

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

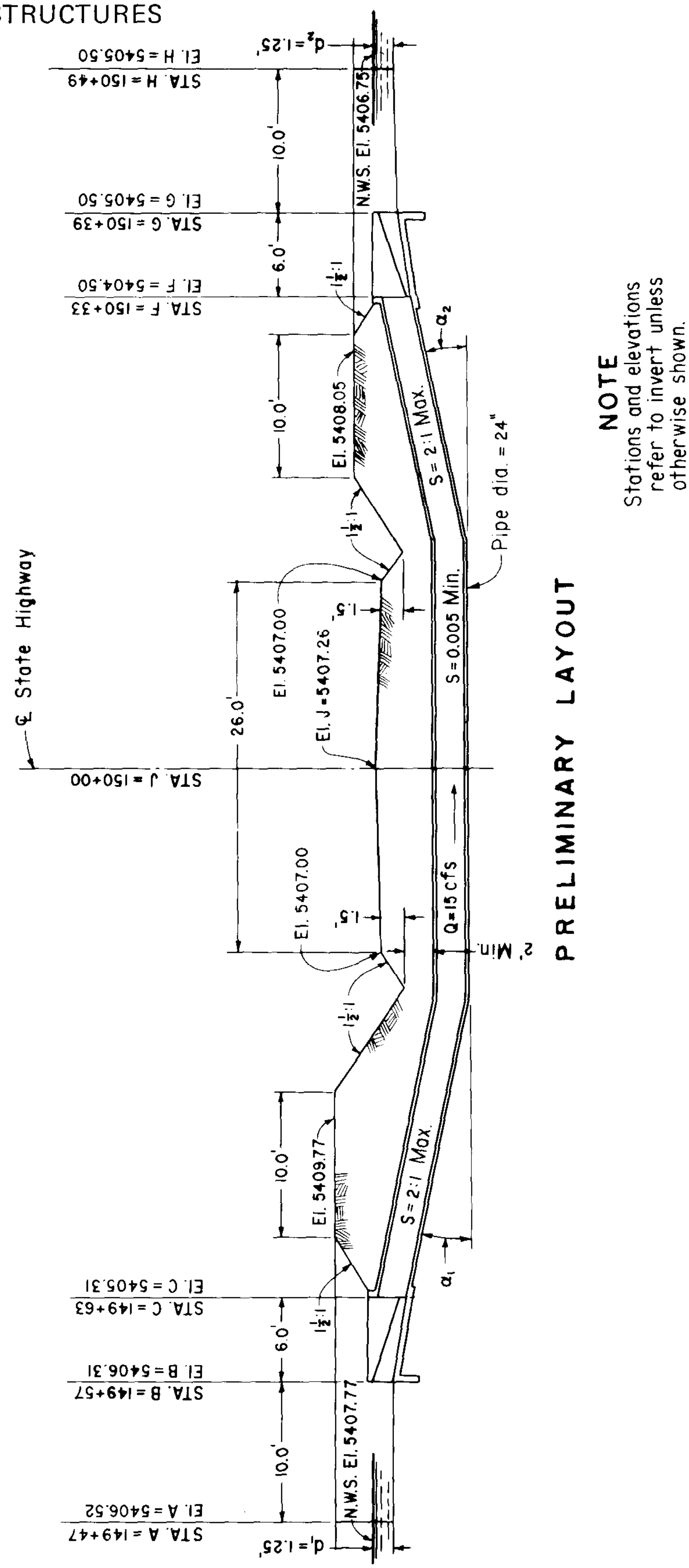


Figure 2-8. Preliminary layout of inverted siphon. 103-D-1256

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 = 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°.

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

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,

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

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.

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

$$\begin{aligned} H_L &= 1.1[0.4\,(0.35 - 0.07) + 72 \times 0.0044 \\ &\quad + (0.04 \times 0.35)\,2 \\ &\quad + 0.7(0.35 - 0.07)] \\ &= 1.1[0.11 + 0.32 + 0.03 + 0.20] \\ &= 1.1\,(0.66) = 0.73 \text{ foot} \end{aligned}$$

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:

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

(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

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

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

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

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).*—

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

(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

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

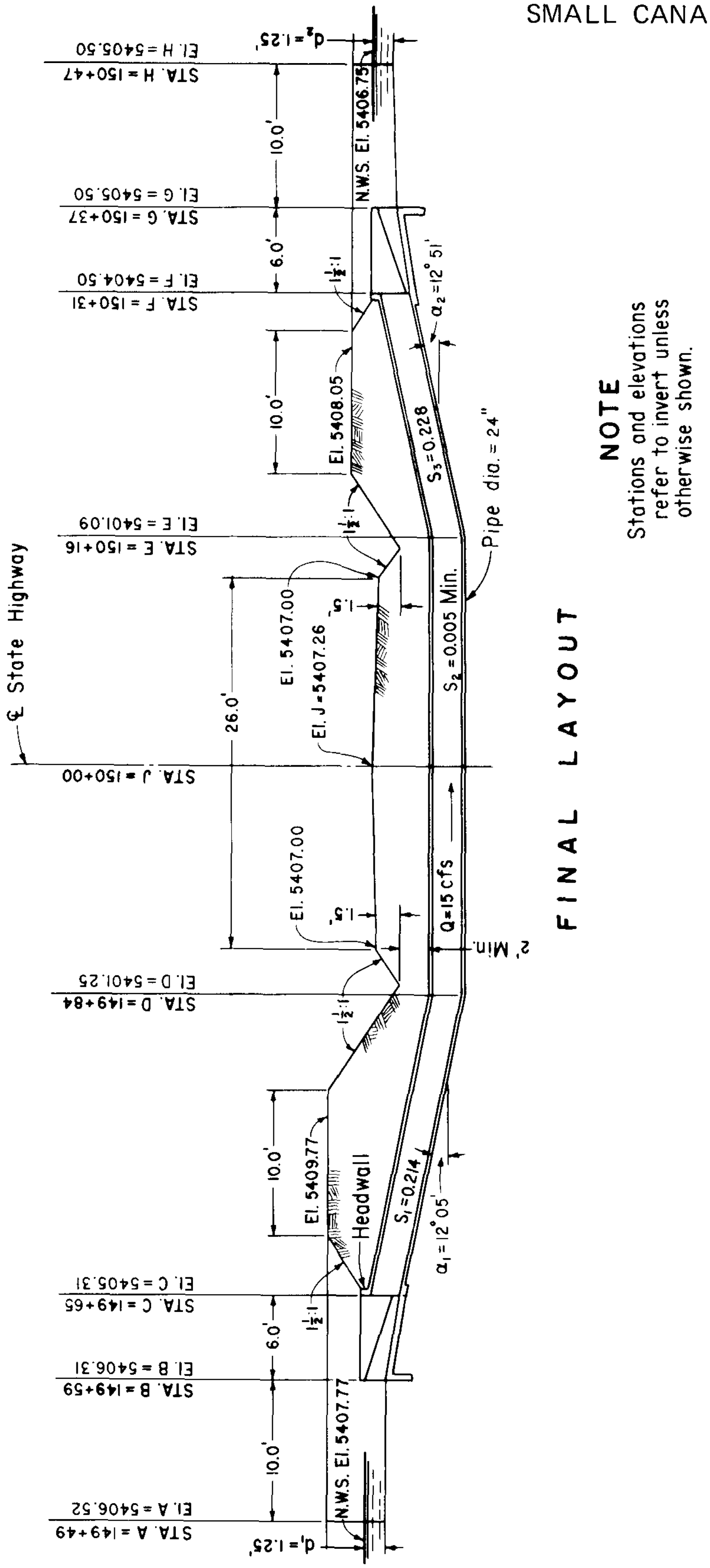


Figure 2-9. **Final layout of inverted siphon. 103-D-1257**

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

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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.