FARM IRRIGATION STRUCTURES

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Handbook No. 2

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WATER MANAGEMENT SYNTHESIS PROJECT

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PREFACE

The purpose of this handbook is to provide information on small structures used in irrigated agriculture, primarily for selecting those structures needed to improve on-farm water management. Complete information on design, construction and operation of the multitude of structures that are available is impossible to assemble in one publication. This handbook is intended to emphasize the importance of adequate control and distribution of irrigation water, enumerate some of the successful structures that are available, give a selected amount of design information, show a limited number of examples of design procedures, and give references where more information can be obtained. With the exception of low pressure underground pipeline systems, only surface systems are discussed. Sprinkler and other methods of irrigation are not covered in this handbook.

Information on small canals and structures is given for flows of less than 0.14 cubic meters per second (5 cubic feet per second). Generally the flows will be in the 0.03 to 0.06 m^3/s (1-2 cfs) range. Structures that can be constructed from local materials and with local labor are emphasized. Precast structures and structures constructed from precast brick block and sections are also presented.

In addition to the extensive bibliography listed for small canals and structures, there is a section presenting standard definitions of terms used in the handbook. Appendices given include: 1) preparation of concrete for small jobs; 2) standard designs of farm irrigation structures (including metric conversion factors); 3) standards for pipe irrigation systems; and 4) standards for plastic pipe irrigation systems.

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I. INTRODUCTION

Surface methods of irrigation are still used on most of the 234 million hectares (1972 data) of irrigated,cultivated lands in the world with the remaining lands being irrigated by sprinkler and trickle systems (3). It has been projected that the irrigated area will increase to about 273 million ha by 1990. An estimated 86 million ha of the world's irrigated lands have systems that now need improvement of both the main and on-farm systems for distributing and applying irrigation water. The United States of America presently (1980) has about 21 million ha of irrigated,cultivated lands, of which 70 percent is surface irrigation.

The sprinkle and trickle methods of irrigation use pumps almost entirely. Pumps require a large energy input, and initially, the systems require a large capital outlay. There is limited use of gravity pressure sprinkle and trickle systems. In the near future, the obvious world shortage of cheap fossil fuel energy will probably mean a return to gravity pressure irrigation systems. The rapidly expanding world population will demand an increasing food supply which will also require an increase in production from irrigated agriculture, mostly from surface gravity systems.

Water application efficiencies for surface irrigation systems around the world typically have been quite low, 40-50 percent (11). Water conveyance efficiencies can be quite low also, possibly in the 40-50 percent range, due to canal and ditch seepage, leakage, and spillage. Overall irrigation efficiencies then might range as low as 28-35 percent indicating that 65-72 percent of the water is lost to the individual farm use. Overall, the efficiencies can be materially increased with canal and ditch maintenance and lining, use and improvement of control structures, and improved on-farm water management. Good farm irrigation water management includes all of these concerns as well as reliable water delivery and a regulated flow rate. Improved water control structures, properly used, can materially improve the use and control of irrigation water. It is possible for many surface irrigation systems to have water application efficiencies of 70 to 80 percent and higher.

The primary purpose of this handbook is to present information on small irrigation channels and structures that can be used to improve on-farm water management. Low pressure pipe systems are also considered. The application of this information will result in much less waste of irrigation water with the resultant increase in water for existing crops and for irrigating additional areas. Improvement of water management on individual farms will result in higher crop yields also.

There has been a lack of attention to the design and operation of the irrigation systems at the farm level because government custody usually ends with the secondary canal systems, and farmers, either by organization or individually, operate the balance of the systems. There also has been the assumption that farm irrigation structures should be low cost and therefore, quality has been a secondary consideration.

It is important that the systems and structures be adapted for use in different countries with consideration for availability and existence of materials, skills, labor, financing and customs. Generally, the procedures and structural designs in this handbook are described simply. The structures are usually easy to operate, are reliable and give good, positive control. Some will require more maintenance than others. Structures and linings that require specialized and expensive equipment for installation are not emphasized. Small, low cost structures that can be constructed entirely with local materials and labor are presented.

II. CHANNELS AND STRUCTURES

1. Delivery Channels and Ditches

The channels discussed here are tertiary and quaternary canals, i.e., the canals commonly called farm and field laterals. They supply water to farm or field outlets and turnouts. The larger canals (tertiary) are called farm laterals (USA), *meskas* (Egypt) and minors (Pakistan and India), while the smaller ones (quaternary) are called field laterals and head-ditches (USA) and *marwas* (Egypt), (Figure 1). The channels may be unlined earth or lined with concrete, masonry or asphalt.

a. Channel Design.

In order to determine the channel size required, the maximum discharge, together with the desired shape of the section and an estimate of the channel roughness, must be known. The Manning equation is the most commonly used relationship for determining channel discharge and will be used in this handbook.

$$Q = C A R^{2/3} s^{1/2} / n \qquad (1)$$

where Q = discharge, (L^3/T).

A = cross sectional area of ditch, (L^2).

R = hydraulic radius--area divided by the wetted perimeter, (L).

s = longitudinal slope, (L/L).

n = Manning roughness coefficient ($L^{1/6}$) (same value for both metric and English units).

C = 1.0 when using metric units, 1.49 for English units.

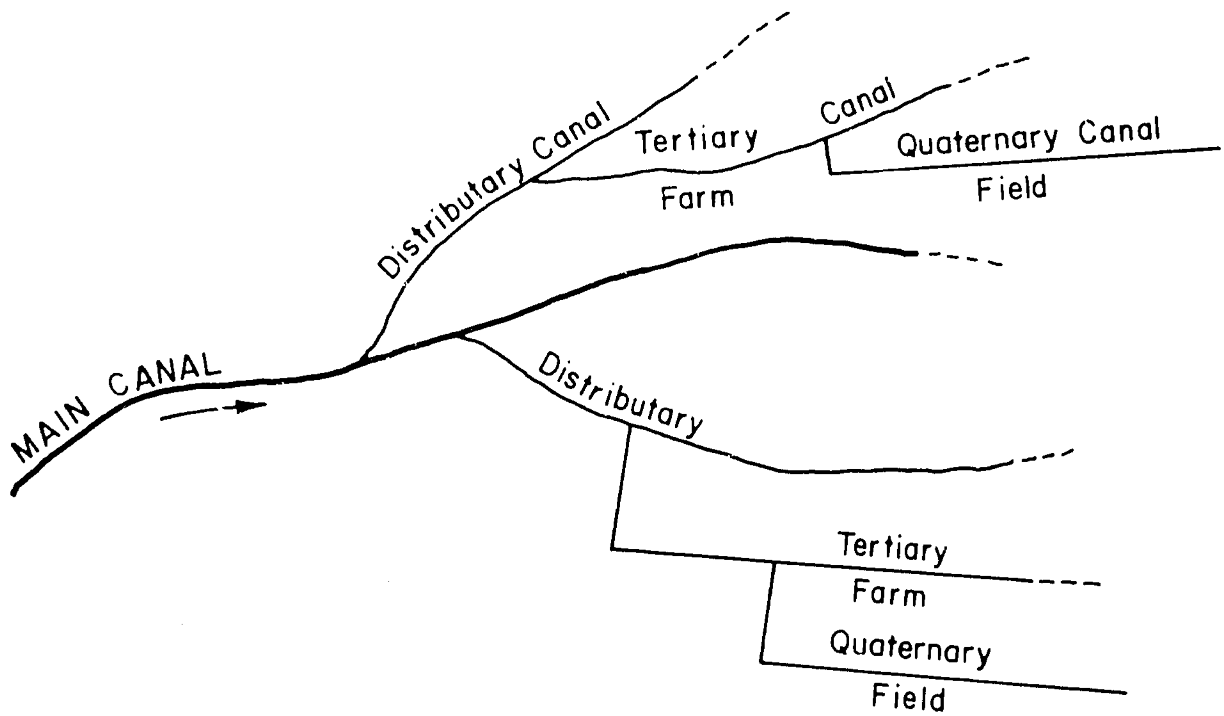


Figure 1. Surface irrigation canal system.

The Manning roughness coefficient, n, for canals varies from 0.010 for smooth concrete to over 0.10 for channels with weeds and brush. Table 1 lists values of n for earthen and lined channels that can be used for design. The value for n should be chosen only after a careful study of the field situation.

The design problem is usually to determine the width and depth required for a given flow with a measured slope in a given material or with a selected lining of a predetermined shape. In other situations an estimate of the discharge is required while knowing the ditch size and slope, with an estimate of the roughness (Manning n from Table 1). Figure 2 gives a solution for Equation 1 that can be used to make estimates of the ditch shape and flow. The following are two examples using Figure 2.

Example 1

Earth canal in clay loam after weathering, clean; n = 0.022 (Table 1).

Assume: Bottom width, B = 0.45 m (1.5 ft)
Longitudinal slope, s = 0.001
Side slope, z = 1.5 (1.5 horizontal to 1 vertical)
Discharge, Q = 0.10 m^3/s (3.5 cfs)

Problem: Determine the depth of flow.

Solution: Solve for the E_m in Figure 2.

$E_m = (Qn/\sqrt{s})/B^{8/3} = [(0.10)(0.022)/(0.032)]/(0.12) = 0.57$

From Figure 2 for z = 1.5, E_m = 0.57, then D/B = 0.60.

Since B = 0.45 m, then

D = 0.27 m (0.89 ft).

Example 2

Brick with vertical wall, mortar trowel finished surface, n = 0.013 (Table 1).

Assume: Bottom width, B = 0.45 m (1.5 ft)
Depth of section, 0.45 m (1.5 ft)
Freeboard, 0.15 m (0.5 ft)
Depth of flow, D = 0.30 m (1.0 ft)
Longitudinal slope, s = 0.001
Side slope, z = 0

Problem: Determine the discharge.

Solution: From Figure 2, for D/B = 0.67 and z = 0 then E_m = 0.28

$E_m = (Qn/\sqrt{s})/B^{8/3}$ or

$Q = E_m\sqrt{s}\,B^{8/3}/n = (0.28)(0.032)(0.12)/0.013$

$Q = 0.083\ m^3/s$ (2.93 cfs)

Table 1. Values of Manning roughness coefficient, *n*, for earthen and lined channels (30).

Type of Channel and Description	Roughness coefficient *n*		
	Minimum	Normal	Maximum
A. Excavated earthen channels			
a. Straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
5. With long grass and weeds	0.030	0.040	0.045
b. Winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest state of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
B. Lined or built-up channels			
a. Cement			
1. Neat, smooth surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
c. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
d. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035

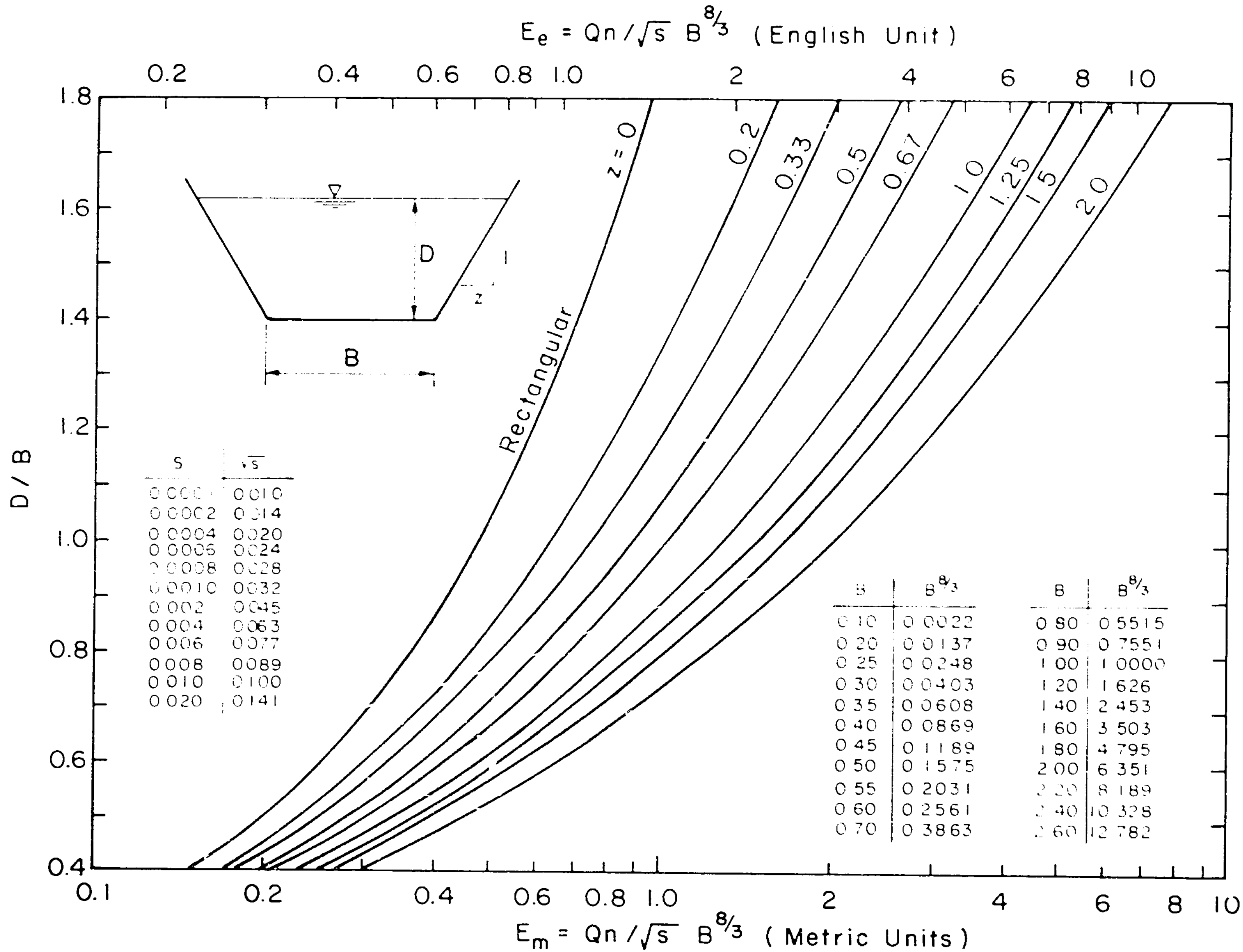


Figure 2. Manning equation solution for determining canal design (30).

Note that the amount of flow is inversely proportional to the roughness, n; i.e., an increase in roughness decreases the discharge in direct proportion; with shape, slope and depth remaining the same. If the discharge remains constant and the roughness increases (such as from growing vegetation), then the depth of flow must increase.

So that the ditch is not overtopped, there should be a freeboard (distance from the maximum water surface to the top of the banks) of at least 15 cm (6 in.) for the small canals. The banks tend to lower with seasoning, aging of the canal, and use of the banks by traffic.

b. Earth Ditches.

Unlined earth ditches are the most common means of conveying irrigation water to the farm fields. Unlined ditches are preferred by many farmers because they can be built cheaply and easily, and maintained with farm equipment. Also, unlined ditches provide flexibility - it is easy to change the layout, increase capacity, or even eliminate them after a rotation and rebuild them the next season. However, they have many disadvantages that make them less desirable than lined ditches or underground pipe.

1. They occupy more land than lined ditches.
2. They usually lose more water due to seepage, leakage and spillage.
3. Rodents can cause leakage.
4. If weed growth is a problem, frequent cleaning is needed.
5. Earth ditches can erode and meander, creating problems in maintaining straight and proper alignment.

The slope for an earth ditch may be as low as 0.00018. (Egyptian irrigation canals generally have slopes ranging from 0.00018 to 0.00020.) However, small slopes result in slow flow velocities, large cross sections, and possible sediment deposition on the bed.

It is customary to use a gradient of 0.001 in many areas. The slope of the ditch should be such that the bed does not erode and the water flows at a self-cleaning velocity; i.e., there is no deposition. A heavy clay soil will allow fairly high velocities without eroding, (Table 2). At times it is necessary to insert drops into the ditch to reduce velocities and prevent scour and erosion. For soils that are normally encountered, the maximum velocities given in Table 2 should not be exceeded. For Example 1, the average velocity for an earth canal in clay loam is 0.43 m/s (1.4 ft/s). For Example 2 and lined ditch, the velocity is 0.61 m/s (2.0 ft/s). Both of these velocities are in the safe range. For unlined ditch side slopes, the lower value (steeper slopes) given in Table 2 should be used for cuts and the higher value (flatter slopes) for canals excavated in a fill section.

The approximate sizing of earth ditches with a side slope of 1½:2 ($z = 1.5$) is given in Table 3 and can be used for preliminary design. With an estimate of slope, roughness factor and desired discharge, several possible ditch sizes can be determined. Conversely, with a known ditch shape (bottom width), roughness and discharge, the required depth and slope can be estimated. By using the Manning Equation (1), tables can be developed similar to Table 3 for other ditch shapes, roughness and slopes.

Ditch locations should be carefully planned to adequately serve the irrigated area. If adjacent fields are being leveled, any needed fill material for the ditch can be easily obtained. Earth ditches can be formed manually or with pulled ditchers. The animal powered V-ditcher shown in Figure 3 can be used to form the ditch. The V-ditcher is run in furrows opened by a moldboard-type plow. Two furrows are made adjacent to each other with the furrow slice thrown in opposite directions. The V-ditcher then moves the soil to form a berm on each side. Usually it is necessary to plow a second or third time to obtain more earth for the banks.

Table 2. Suggested maximum flow velocities and side slopes for lined and unlined channels (5).

Type of Surface	Maximum Flow Velocities m/sec	ft/sec	Side Slopes Range (z)*
Unlined Ditches, seasoned			
Sand	0.3 - 0.7	1.0 - 2.3	3
Sandy loam	0.5 - 0.7	1.6 - 2.3	2-2½
Clay loam	0.6 - 0.9	2.0 - 3.0	1½-2 **
Clays	0.9 - 1.5	3.0 - 5.0	1-2 **
Gravel	0.9 - 1.5	3.0 - 5.0	1-1½
Rock	1.2 - 1.8	4.0 - 6.0	¼-1
Lined Ditches			
Concrete			
Cast-in-place	1.5 - 2.5	5.0-8.2†	¾-1½
Precast	1.5 - 2.0	5.0 - 6.5	0-1½ ††
Brick	1.2 - 1.8	4.0 - 6.0	0-1½ ††
Asphalt			
Concrete	1.2 - 1.8	4.0 - 6.0	1-1½
Exposed membrane	0.9 - 1.5	3.0 - 5.0	1½-2
Buried membrane ‡	0.7 - 1.0	1.6 - 3.3	2
Plastic			
Buried membrane ‡	0.6 - 0.9	2.0 - 3.0	2½

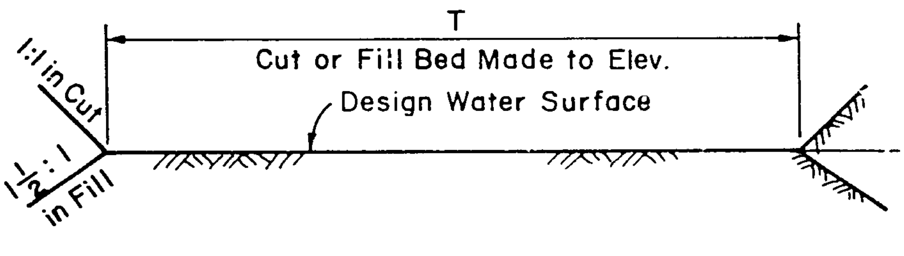
* z is the horizontal unit to one (1) vertical unit.
** Side slopes of 1:1 for small canals in clay and clay loam are common.
† Flows in this velocity range may be supercritical (see definitions) and difficult to control. They are not recommended except for special uses.
†† Small precast and brick channels may have vertical walls (z = 0).
‡ Maximum flow velocities will depend on the cover over the membrane.

Tractor drawn ditchers may be obtained in many different designs and sizes, Figure 4. They may be adjusted manually or hydraulically. Generally the "nose" element is at an angle with the "wings" so that when tilted, a somewhat flat bottom is obtained. The wings are adjustable for different widths. By combining tilt with wing spread, depth and top width can be varied. In use, the first pass is not at full depth unless the earth is reasonably soft. On the second pass the tractor wheels or tracks will compact the earth moved out on the first pass. This will reduce seepage and stabilize the banks. Compaction of the banks and bed by manual or machine tamping or rolling is desirable.

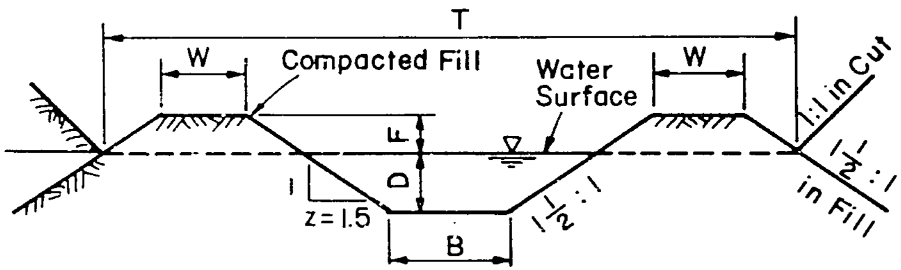
Table 3. Earth irrigation ditch sizes for different slopes, roughness and discharges (33).

Flat bed before ditch is formed														Finished ditch section															
														s = 0.0005				s = 0.001				s = 0.002				s = 0.003			
														n = 0.03		n = 0.04		n = 0.03		n = 0.04		n = 0.03		n = 0.04		n = 0.03		n = 0.04	
B		D		F		W		T		A		R		Q		Q		Q		Q		Q		Q		Q		Q	
m	ft	m	ft	m	ft	m	ft	m	ft	m²	ft²	m	ft	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s	m³/s	ft³/s
0.15	0.5	0.30	1.0	0.15	0.5	0.30	1.00	2.6	8.5	0.19	2.00	0.15	0.49	0.04	1.4	0.03	1.0	0.05	1.9	0.04	1.5	0.08	2.7	0.06	2.1	0.10	3.4	0.07	2.5
0.30	1.0	0.30	1.0	0.15	0.5	0.46	1.50	3.0	10.0	0.23	2.50	0.16	0.54	0.05	1.8	0.04	1.4	0.07	2.6	0.06	2.0	0.10	3.7	0.08	2.8	0.13	4.5	0.10	3.4
0.46	1.5	0.30	1.0	0.15	0.5	0.61	2.00	3.5	11.5	0.28	3.00	0.18	0.59	0.07	2.3	0.05	1.7	0.09	3.3	0.07	2.5	0.13	4.7	0.10	3.5	0.16	5.7	0.12	4.3
0.61	2.0	0.30	1.0	0.15	0.5	0.76	2.50	4.0	13.0	0.33	3.50	0.19	0.62	0.08	2.8	0.06	2.1	0.11	4.0	0.08	3.0	0.16	5.6	0.12	4.2			0.15	5.2
0.30	1.0	0.37	1.2	0.21	0.7	0.38	1.25	3.4	11.3	0.31	3.36	0.19	0.63	0.08	2.7	0.06	2.0	0.11	3.9	0.08	2.9	0.16	5.5	0.12	4.1			0.14	5.0
0.46	1.5	0.37	1.2	0.21	0.7	0.46	1.50	3.7	12.3	0.37	3.96	0.21	0.68	0.10	3.4	0.07	2.5	0.14	4.8	0.10	3.6			0.14	5.1			0.18	6.2
0.61	2.0	0.37	1.2	0.21	0.7	0.61	2.00	4.2	13.8	0.42	4.56	0.22	0.72	0.12	4.1	0.08	3.1	0.16	5.7	0.12	4.3			0.17	6.1				
0.46	1.5	0.41	1.33	0.24	0.8	0.53	1.75	4.1	13.6	0.43	4.65	0.23	0.74	0.12	4.2	0.09	3.2			0.13	4.5								
0.61	2.0	0.41	1.33	0.24	0.8	0.61	2.00	4.5	14.6	0.49	5.31	0.24	0.78	0.14	4.9	0.11	3.8			0.15	5.3								
0.61	2.0	0.46	1.5	0.30	1.0	0.46	1.50	4.7	15.5	0.59	6.38	0.26	0.86	0.18	6.4	0.14	4.8												
0.91	3.0	0.46	1.5	0.30	1.0	0.61	2.00	5.3	17.5	0.73	7.88	0.29	0.94			0.18	6.3												

A - cross sectional area
R - hydraulic radius
n - Manning's roughness coefficient
  0.03 - soil with gravel
  0.04 - soil with grass
s - slope
Q - ditch flow capacity



Flat Bed before Ditch is Formed



Finished Ditch Section

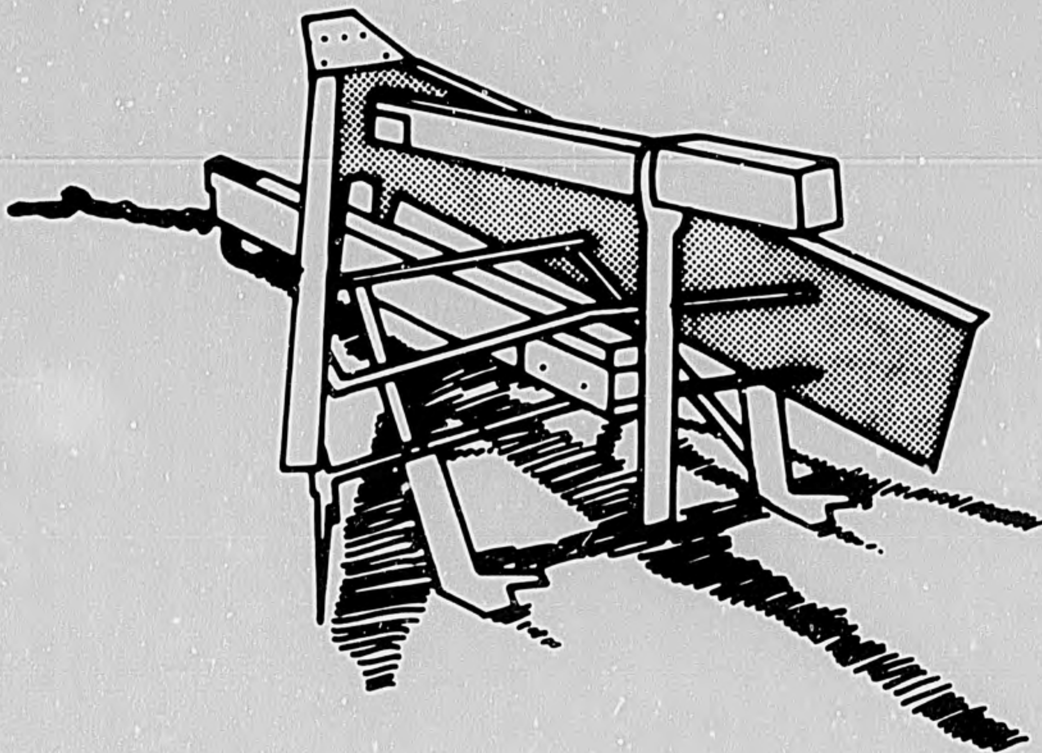


Figure 3. Animal powered V-ditcher

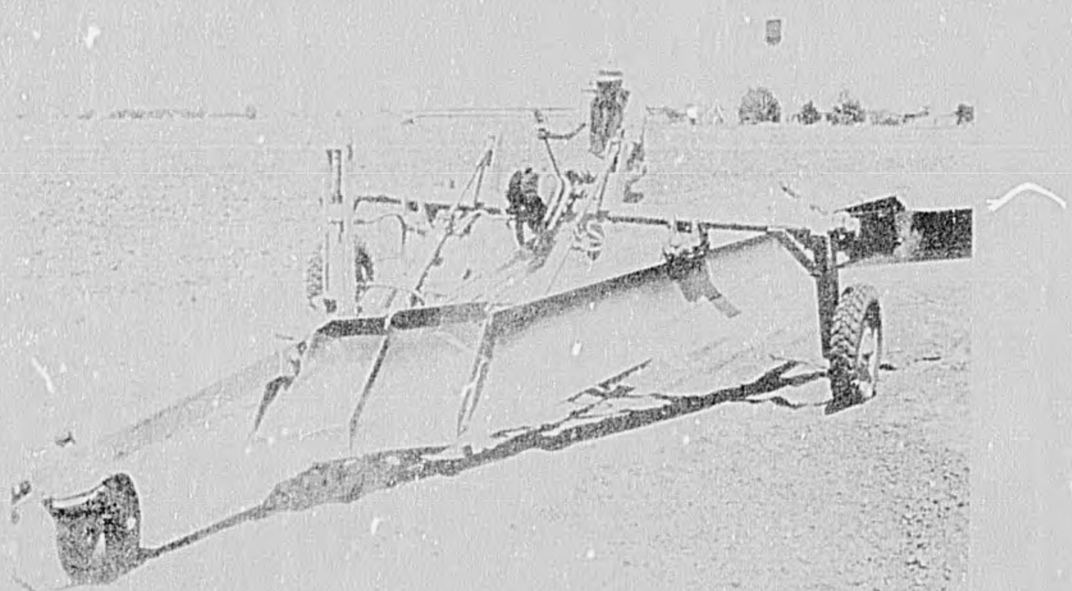


Figure 4. Tractor powered V-ditchers.

Many old ditches become eroded and deteriorated and it is better to remove the old ditches and form new ones. Figure 5 gives a procedure for doing this that will result in a new, more stable channel that will lose less water than the old one. The compaction and forming of the new channel can be done manually or with a machine. Soil which has a high percentage of silt and clay will form the best channel from all standpoints.

The importance of good construction for earth channels depends a great deal on expected ditch usage. Some ditches, such as those run on a contour for grain and rice, are used only one season and then filled in. Other ditches are relatively permanent and should be constructed with more effort and care. Ditches intended for furrow or border irrigation directly from the ditch need substantial banks; and the banks might be higher for using spiles and siphon tubes than for open ditch bank cuts. In this case the top of the banks should be a minimum of 25 cm (10 in.) above the surrounding field surface. Banks must be high enough to allow the water level to be increased by checks if needed. If seepage is excessive, compaction of the banks or deposition of a clay blanket can be tried.

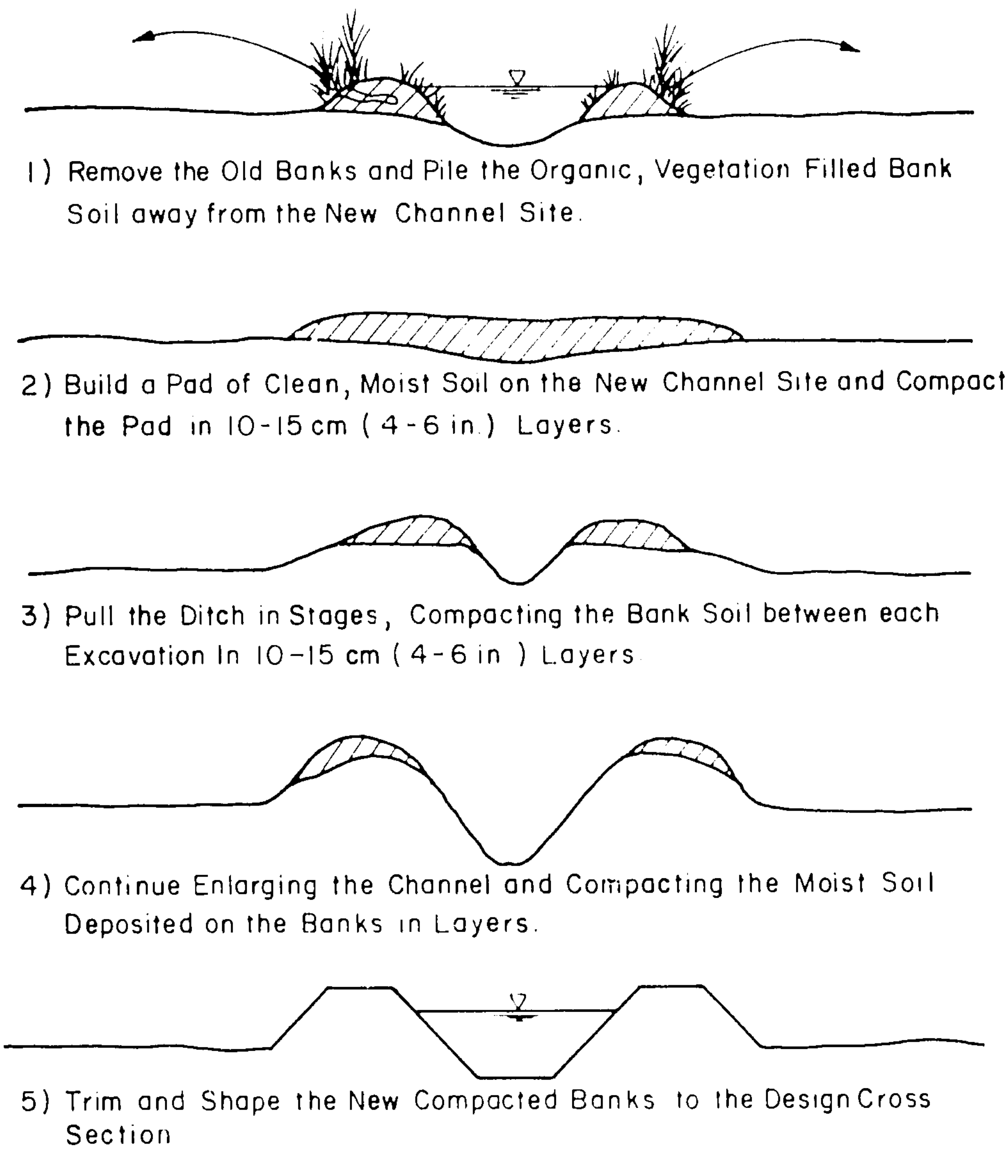


Figure 5. Suggested procedure for mechanical reconstruction of earthen channels with a tractor-drawn scraper, ditcher, and compactor (30).

c. Lined Ditches.

Often farm ditches are lined to reduce water loss and maintenance and to improve water control. Lining can reduce the amount of seepage loss, control weed growth and prevent the ditch from eroding. Lined ditches with sloping sidewalls must have adequate support from earthen banks that should be constructed to an elevation of 10 to 15 cm (4-6 in.) above the top of the lining. These banks must be maintained to protect the lining from damage. Linings also reduce the amount of land occupied by the ditches and may provide some control against damage by rodents or burrowing animals. However, improved water control is the major benefit of lining. There is a reduction in water storage and ponding and water moves through the system at a faster rate.

The most common types of lining for larger canals are concrete and concrete with masonry. Asphaltic linings and compacted soil materials are also being used. Limited use has been made of chemical sealants and plastic membranes. For small canals and ditches, brick or rock masonry linings are common in some areas. The selection and construction of a lining is governed by: 1) the availability of the material, 2) equipment and labor required for installation, 3) size of ditch, and 4) climatic and soil conditions. Chemical composition of the water may be a factor. Many times the initial cost governs the type of ditch lining chosen. However, the availability and need for water should also be considered. Fluctuating water tables and intermittent streams of water can damage linings. Tile drains can be used to lower the water table below the bottom of the canal to prevent damage to the lining.

Livestock can damage some linings and, if necessary, special provisions should be made for livestock watering at suitable locations. Washing of clothes and utensils should be anticipated and provided for in some areas. Vegetation can damage some linings unless steps are taken to control growth. Figure 6 shows some types of small channel linings.

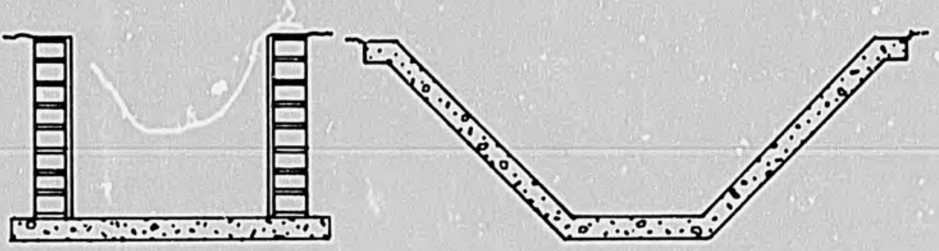
1. *Concrete Lining.* Concrete linings have many fine qualities. When properly constructed, and where site conditions are favorable, the linings will give long service with minimum repair and maintenance cost. They will withstand high water velocities and are resistant to damage from animals, machines and man. Sulphate resistant cement should be used when irrigation water or the soil contains high concentrations of sulphates. Instructions for preparation and mixing of the concrete are given in Appendix 1.

Concrete lining can be placed in many ways:

1. handplaced by plastering on sides and bottom;
2. using forms and pouring alternate panels;
3. pneumatically blown;
4. precast concrete box or part sections; and
5. slipform using heavy equipment (Figure 7*a*, *b*).

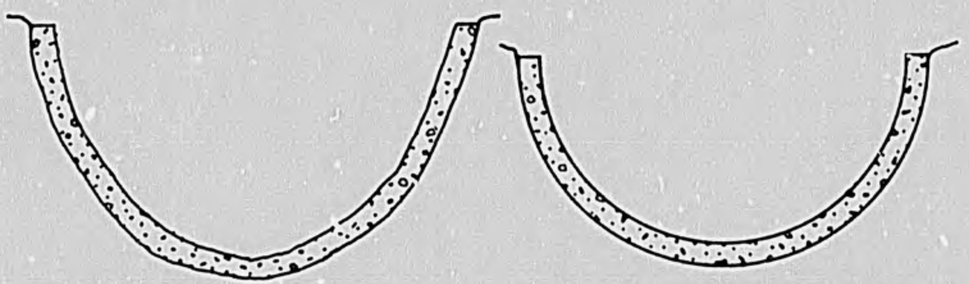
The slipform method requires very careful scheduling of operations and a large amount of equipment, including a transit mixer for the concrete. For small canals, reinforcing steel is generally not necessary and the concrete linings are usually placed without it.

In some areas, particularly India, Turkey and Egypt, precast sections are placed manually in the small farm ditches. Figure 8 shows forms for casting small, rectangular sections. Precast concrete sections are usually manufactured at a central place and hauled to the job site (22, 36). Precast sections may be made box-shaped, semicircular, trapezoidal-shaped or half-parabolic. The semicircular sections are usually made by pipe companies, 1-2 m (3.3-6.6 ft) in length, 46-61 cm (18-24 in.) in diameter. The sections can be placed above the ground surface, supported on cradle-type pedestals when needed to cross low areas. Parabolic sections 20-30 cm (8-12 in.) wide are made in halves in Egypt, and when installed and mortared, can result in lined ditches with a range of top widths. This feature is desirable since the same precast sections can be used for a narrow range of ditch sizes by varying the top width.



a) Rectangular Brick Masonry on a Poured Concrete Pad

b) Poured Concrete - Trapezoidal



c) Sprayed or Hand Placed Cement Mortar, Soil Cement or Asphalt - Parabolic

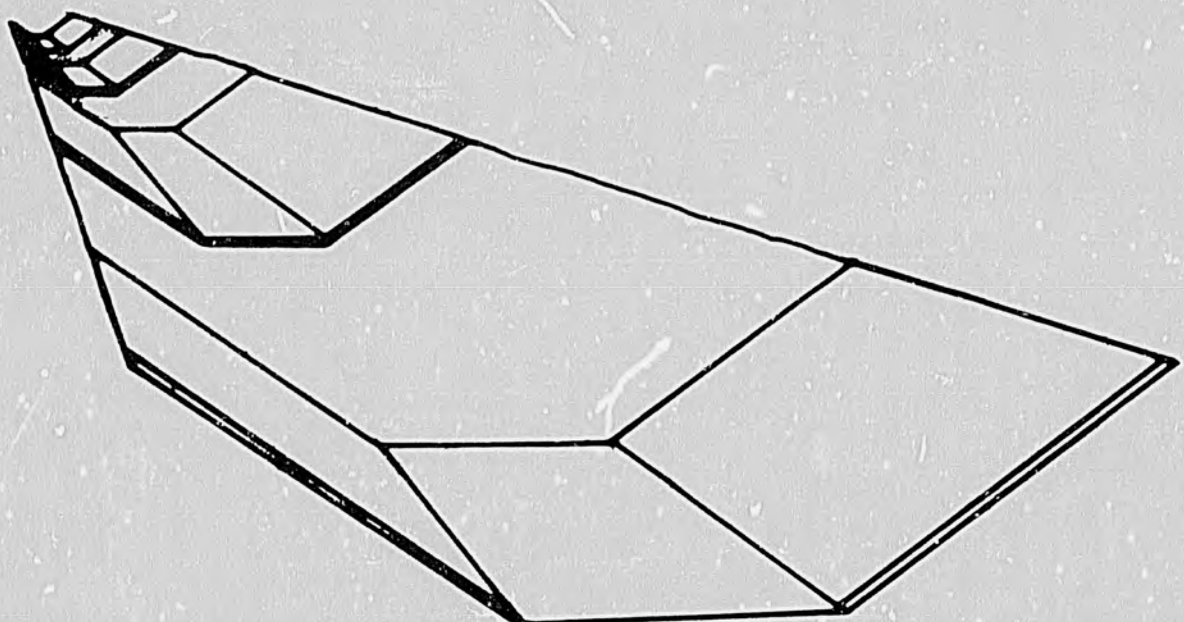
d) Precast Concrete-Semicircular, Parabolic or Rectangular

e) Precast Concrete (SCS photo)

Figure 6. Types of linings for small canals (30).

(a)

(b)



(c)

Figure 7. Concrete lined canals. Lining with slipform (a) and (b), and the alternate panel method (c) (25).

When the concrete lining is hand placed and the ditch is not over 0.6 m (2 ft) deep, the side slopes can be as steep as ¼ horizontal to 1 vertical. Care must be exercised with the concrete mix so it does not slump from the steep sides. When steeper slopes or vertical sidewalls are used, forms are necessary to hold the concrete in place until it sets. Nonreinforced concrete linings with vertical sides can be used for depths up to 0.5 m (1.6 ft). The bottom and sides should have a thickness of at least 15 cm (6 in) and expansion-contraction joints are needed.

The alternate panel method can be used for forming small parabolic and trapezoidal ditches and canals (8, 36). For these sections, guide forms are used and the sections are poured alternately with the finished sections used for forming the intervening ones (Figure 7c). Joints (preformed cracks) spaced at intervals of 1.5 to 3 m (5-10 ft) are needed for the expansion and contraction of the nonreinforced concrete. These cracks are filled with flexible, asphaltic material to prevent water leakage.

To save concrete and to facilitate forming, the soil subgrade should be overexcavated and compacted to the exact shape, grade and alignment of the underside of the lining. A tractor or animal drawn ditcher can do a sufficiently accurate excavation job with a minimum of hand work required. Careful attention must be paid to the foundation of any ditch lining especially when fills are involved. The fill should be carefully made, compacted and wetted prior to placement of concrete. The top of the dike or berm on either side of the ditch should have a minimum width of 0.4-0.5 m (1.3-1.6 ft). The berm should be seeded to grass after construction if possible.

2. *Asphaltic Concrete.* The asphaltic concrete lining has had only limited use. It is similar in installation to cement concrete but has a shorter service life expectancy since it is more subject to mechanical and animal damage. However, this type of lining when properly installed and maintained can give satisfactory performance (28). It can be placed by the slip-form method or hand placed in manners similar to cement concrete. However, the subgrade must be sterilized to prevent vegetation from damaging the lining.

3. *Masonry.* Single layer brick, tile or stone may be used for satisfactory lining of field irrigation channels (23). The bricks, tiles or stones are laid flat on the compacted sides and bottom of the trapezoidal channel and the joints are filled with cement mortar. For a rectangular channel, the bottom may be concrete or masonry with vertical masonry walls. Wall height should be limited to 5-6 courses for brick. Usually the water side of the masonry structure is plastered, particularly if the bricks are not of good quality. The mortar should have a cement-sand ratio of 1:4. The choice of brick, tile or stone depends mainly on its availability and cost. Masonry channels are successfully used in many areas of the world. In some areas, only the sides of the channels are lined, but this is usually not recommended since the bottom may erode and cause the sides to fail.

4. *Asphalt, Plastic and Rubber-like Sheeting.* These lining materials have been used but have not been successful in some areas. Weathering, particularly exposure to the sun, has been a problem. Subgrades must be sterilized with chemicals to prevent vegetative damage. Mechanical and animal damage have contributed to failures. However, plastic sheeting used underneath precast or poured concrete sections has been successful in reducing water losses.

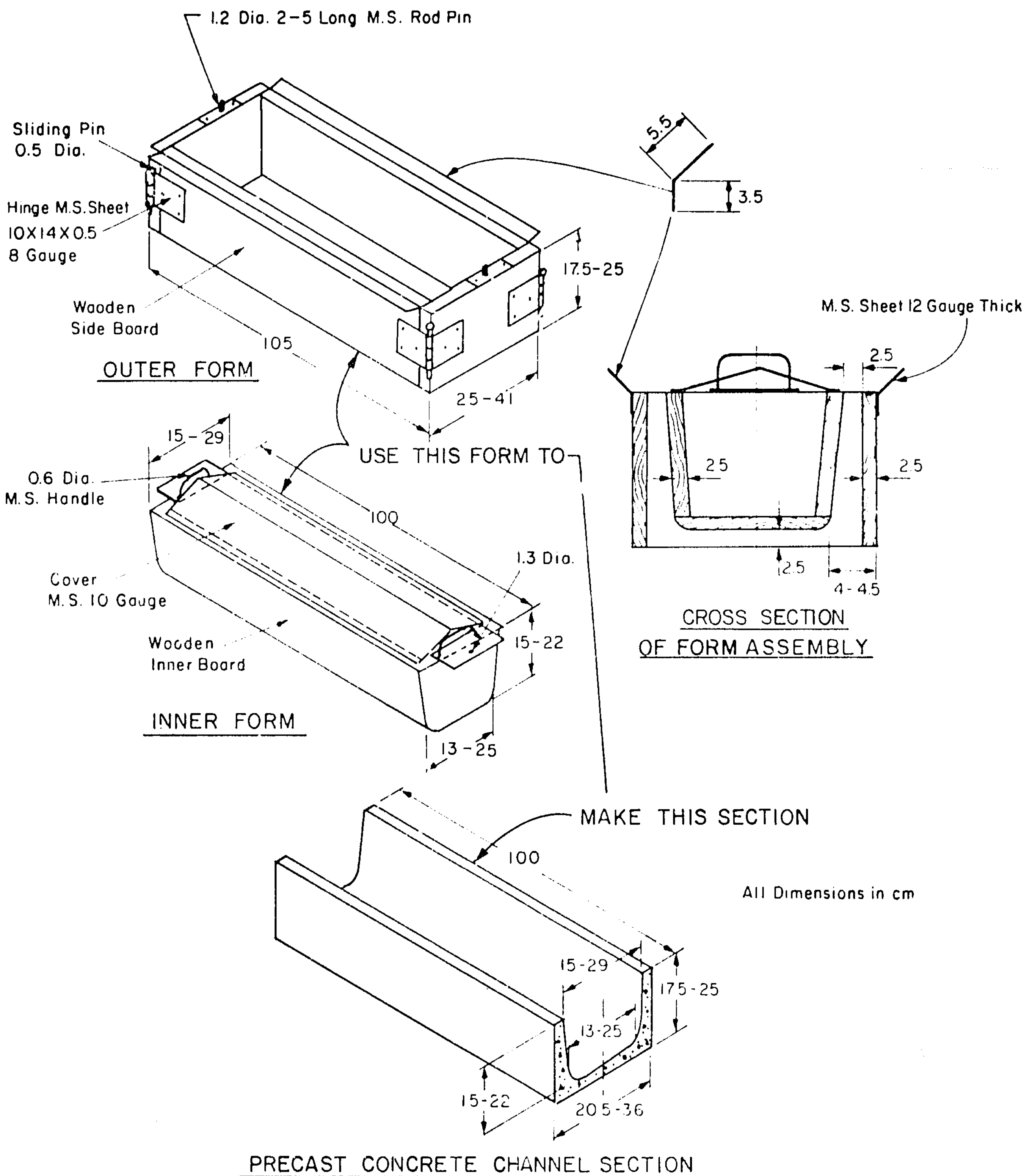


Figure 8. Precast concrete channel sections. Wooden forms used for casting (22).

5. *Chemical Sealants.* Chemical sealants have been used to reduce seepage losses from canals only to a limited extent. Some have been used successfully, but others have failed after a short life.

6. *Earthen Materials.* One of the oldest methods for reducing seepage losses and improving ditches is to remove the porous earth and replace it with clay material. When impervious earthen material is found near the ditch, it may be used to solve the problem. If the material is slightly moist, and if it is placed in 10-15 cm (4-6 in.) layers and rolled and/or compacted, the effectiveness of the lining will be greatly increased. However, some clays are subject to excessive shrinking and cracking upon drying and should not be placed as lining material.

Bentonite clay is sometimes used to line ditches. This material swells greatly when wet and is best adapted to ditches that are not subject to frequent wetting and drying. It is usually mixed with the surface layer of soil with a disk or spike-tooth harrow. It may also be placed as a blanket on the ditch bottom and covered for protection with about 5 cm (2 in) of soil or gravel.

2. Control Structures

Small irrigation structures must be adapted for use in particular areas depending on the availability and cost of materials and labor. The irrigation methods, customs in the area and the irrigation water delivery schedule are factors also. Small, low cost structures that can be built and installed with local labor and materials are desired. Control structures must be easy to operate, relatively leak proof and give good, positive control. Both permanent and temporary-portable structures are discussed.

Physical layout, operational features, and socioeconomic factors should be considered when selecting small, irrigation control structures. The important decision considerations of the physical system of divisors, drops, checks and turnouts include:

1. availability of water requiring division, equitable delivery, and minimizing leakage, spillage and seepage,
2. topography of the area and available hydraulic head for the system,
3. size of area served by each canal, lateral and turnout,
4. amount of time each canal is used and number of times each turnout is used,
5. number of turnouts and the amount of regulation for each,
6. need for regulating flow depths in the system requiring permanent and/or temporary checking,
7. importance of minimizing head loss in the system,
8. need for flow measurement, and
9. availability of materials and labor.

Social and economic considerations are very important for the improved system and include:

1. availability of skills and cost of labor for construction and irrigation,
2. cost of materials,
3. marginal cost or value of water,
4. availability of capital to finance the improved system,
5. organization, or lack thereof, of the water users,
6. level of experience and understanding of the users,
7. cooperative nature of the users,
8. pride in ownership, and
9. potential theft problems of the structures and materials for other uses.

In some areas domestic uses (washing and bathing) and animal access must be considered also.

The layout of a small canal system with different structures is given in Figure 9. It is difficult to separate small irrigation structures into distinct categories since division structures may also serve as checks and/or turnouts. For this publication the primary function of the structures has been used to categorize the structures.

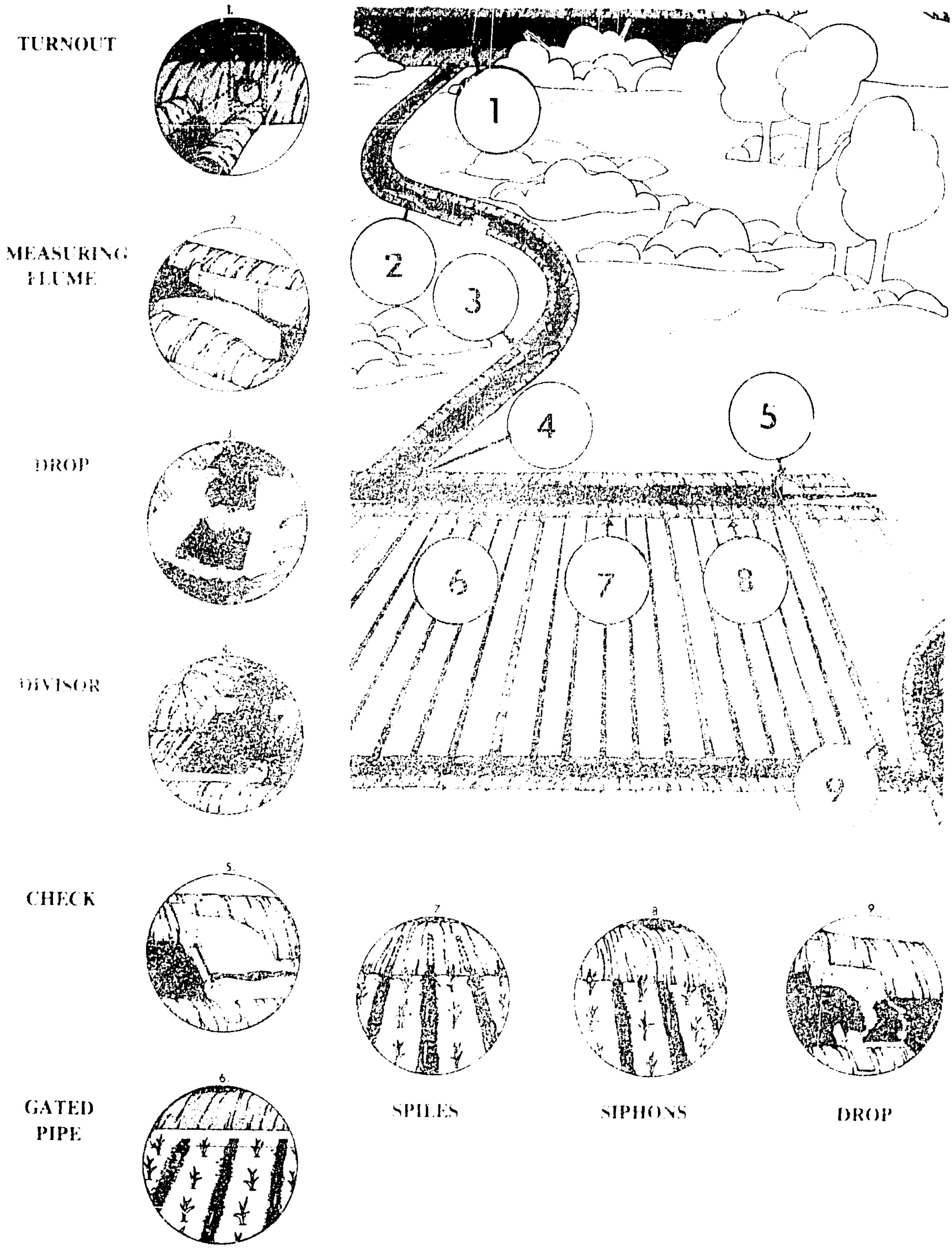


Figure 9. A farm irrigation system.

a. Division Structures.

Divisors (diversion structures) are used to separate a flow into two or more parts. These structures simply divide the flow in a ditch into the desired measured or proportional parts. Ideally, water levels and flow in the divisors are carefully controlled and measured. The dimensions of the openings are not necessarily in the same proportions as the desired division of discharge because of the flow situation. The openings may be made rigid or variable depending on the need for flexibility. However, to be effective as a divisor, the upstream and downstream flow conditions should be similar and standard. Figure 10 shows a *fixed divisor* used to divide the flow between two ditches. Figure 11 is a *divisor* for *four divisions* where the outer two sections are fixed and the middle divisor is movable for adjusting the flow between the middle channels. The divisor shown in Figure 10 does not always accurately divide the flow due to the large pier and the low velocity of flow. Divisors that give accurate proportions divide the flow at a control section where supercritical flow (see definition of terms) exists such as that shown in Figure 11. Flow can be accurately divided without supercritical flow if: 1) there is a long straight approach upstream; 2) there is no backwater effect in the downstream channels, and 3) the flow sections have uniform roughness (3). The divisor shown in Figure 11, although effective as a divider, has the distinct disadvantage of a large hydraulic head loss. Most small irrigation systems need to conserve head and minimize loss, so the divisor in Figure 11 is generally not recommended.

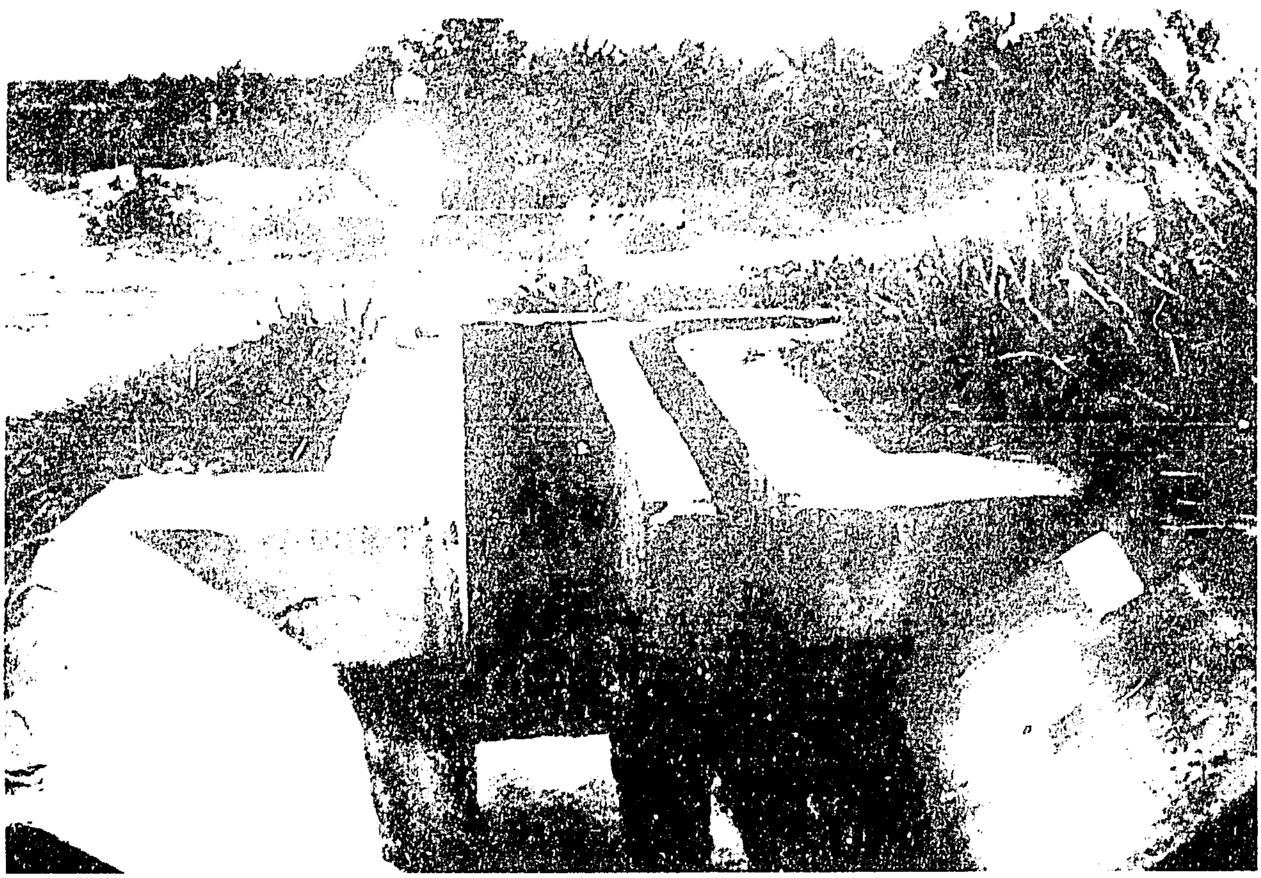


Figure 10. Divisor to distribute the flow between two ditches (4).

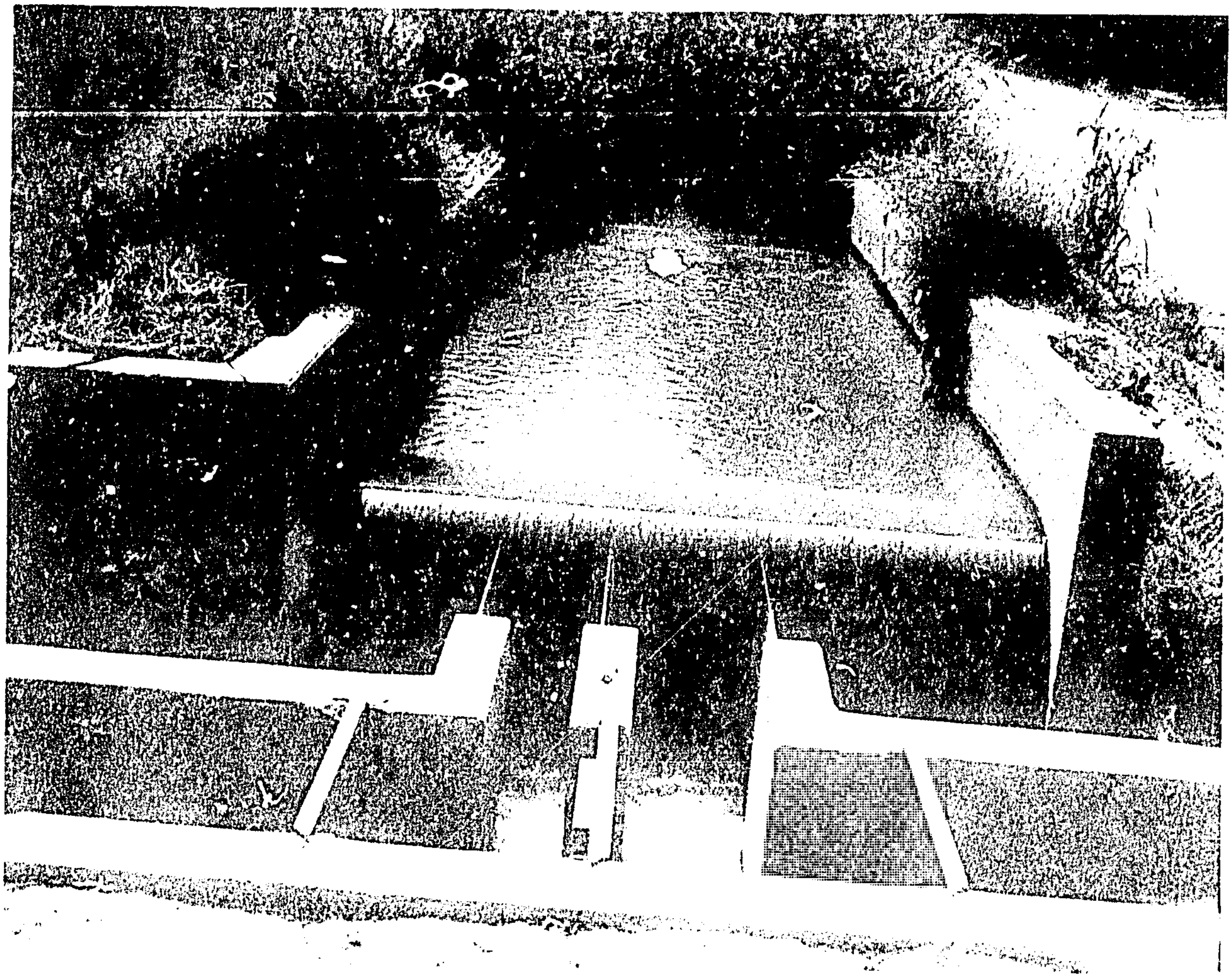


Figure 11. Division box for flow to four ditches (SCS Photo).

In some areas, the amount of water delivered to a system may vary, but all deliveries within the system are to remain proportional to the gross amount delivered. This insures a somewhat equitable distribution without providing a control device at each individual turnout for periodic regulation (29). In areas where deliveries are based on crop acreage or on land irrigated, equitable distribution of available water to users has been attempted. For instance, in India, Pakistan and Egypt, measurement is made at the head of the tertiary canal and equitable distribution serves as a basis for delivery. This method requires that the tertiary canal be designed so that the water level is essentially the same for each turnout (which commonly have no gate control). This water level is difficult to maintain, particularly for the turnouts at the end of the tertiary system. For a system where there are several turnouts, there should be adequate inflow to insure that all water users get their allotted share.

The *semimodular turnout* shown in Figure 12 delivers equitable amounts reasonably well. The entrance is shaped so that an equitable share of the flow is extracted. Exit conditions are such that changes in downstream water levels do not significantly change the discharge over a range of downstream depths. However, the discharge is reduced when the downstream depth exceeds this range. This divisor turnout can be used also as a water measuring device.

There are several types of *barrel or pipe type divisor turnouts* that serve the purpose of equitable division of water based on delivery to the system. Figure 13 shows the pipe outlets that presently are used in Egypt, Pakistan and India. In Egypt, pipe diameters are determined depending on the area to be irrigated and assumed water duty or use. A constant water depth of 25 cm is assumed to exist on the upstream side for all divisor turnouts in the individual system. Submergence on the downstream end of the pipe is assumed but rarely occurs, resulting in flow variations due to depths of flow at the outlet below the top of the pipe.

An *adjustable divisor* used in southern Europe and north Africa is shown in Figure 14. The dividing blade is adjustable and calibrations have been made so that the flow may be proportioned accurately between two channels (6). The design assures that downstream flow conditions do not change the amount and division of flow over a wide range since the flow normally passes to supercritical flow over a crest. This divisor is quite expensive to build.

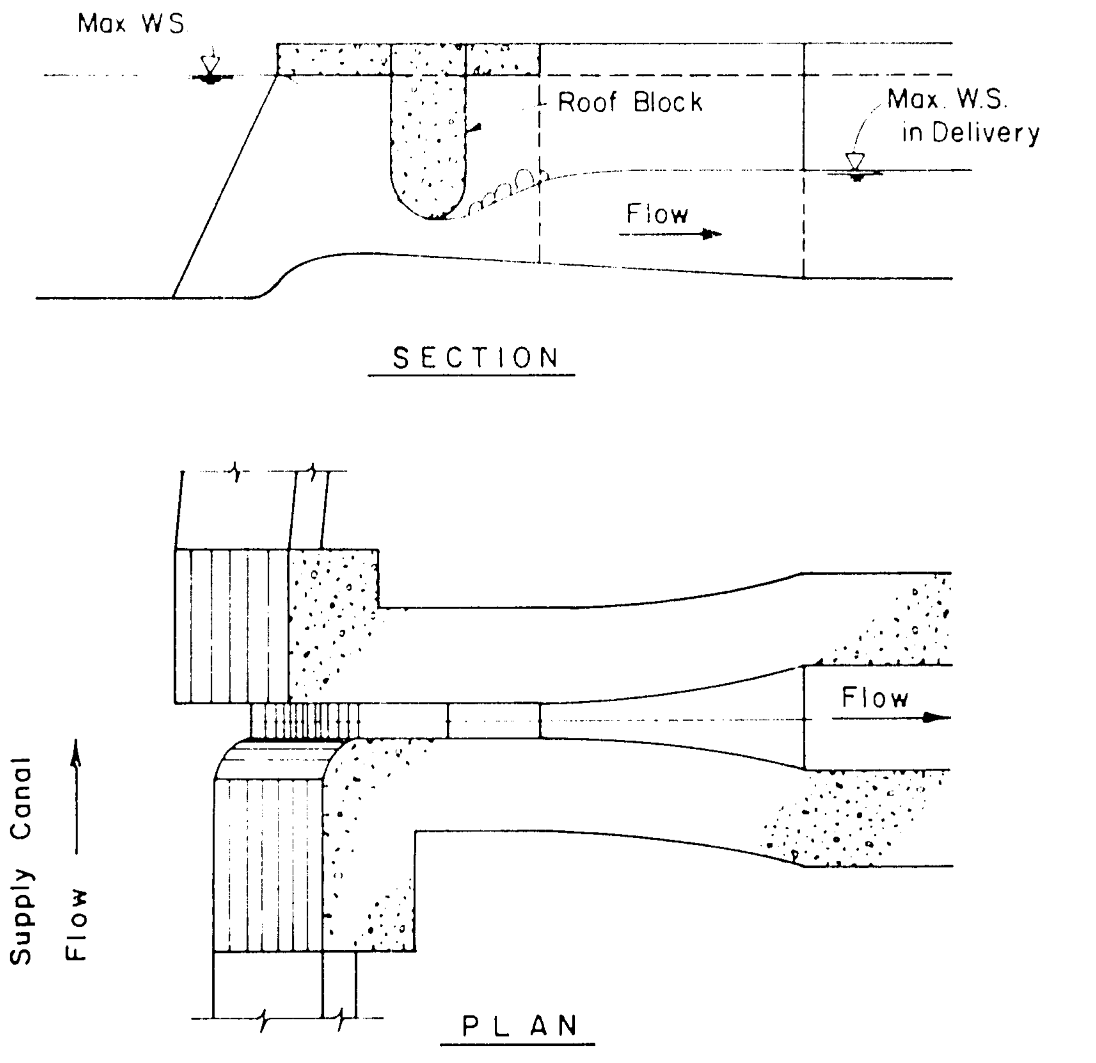


Figure 12. Semimodule divisor used in India and Pakistan (29).

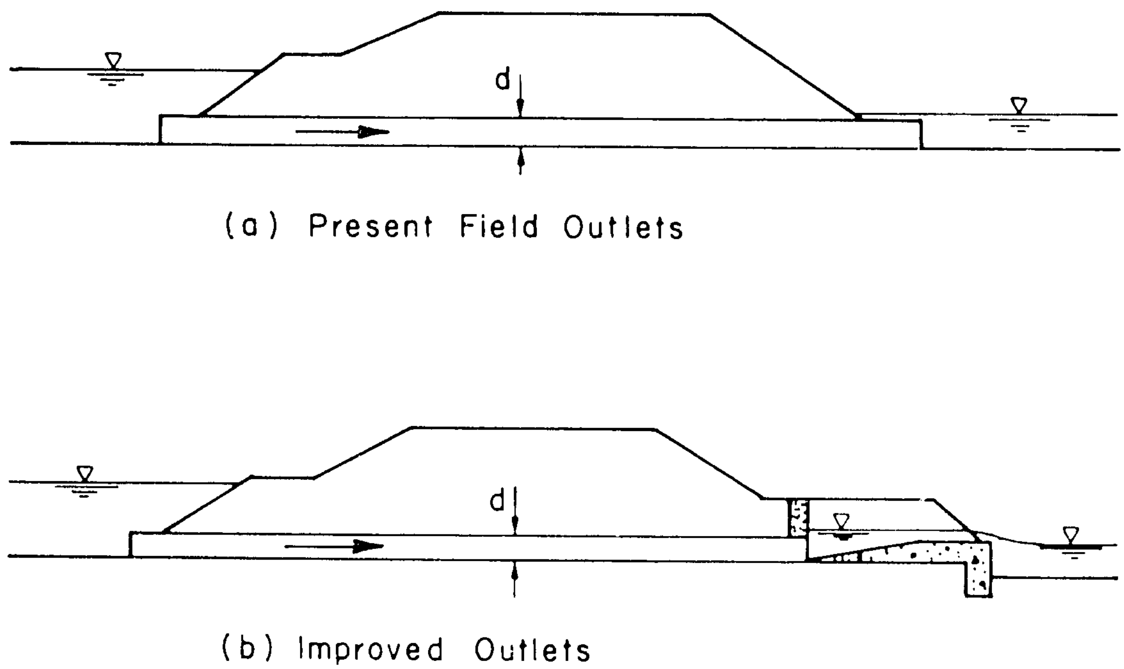


Figure 13. Divisor-turnouts used in Egypt and India.

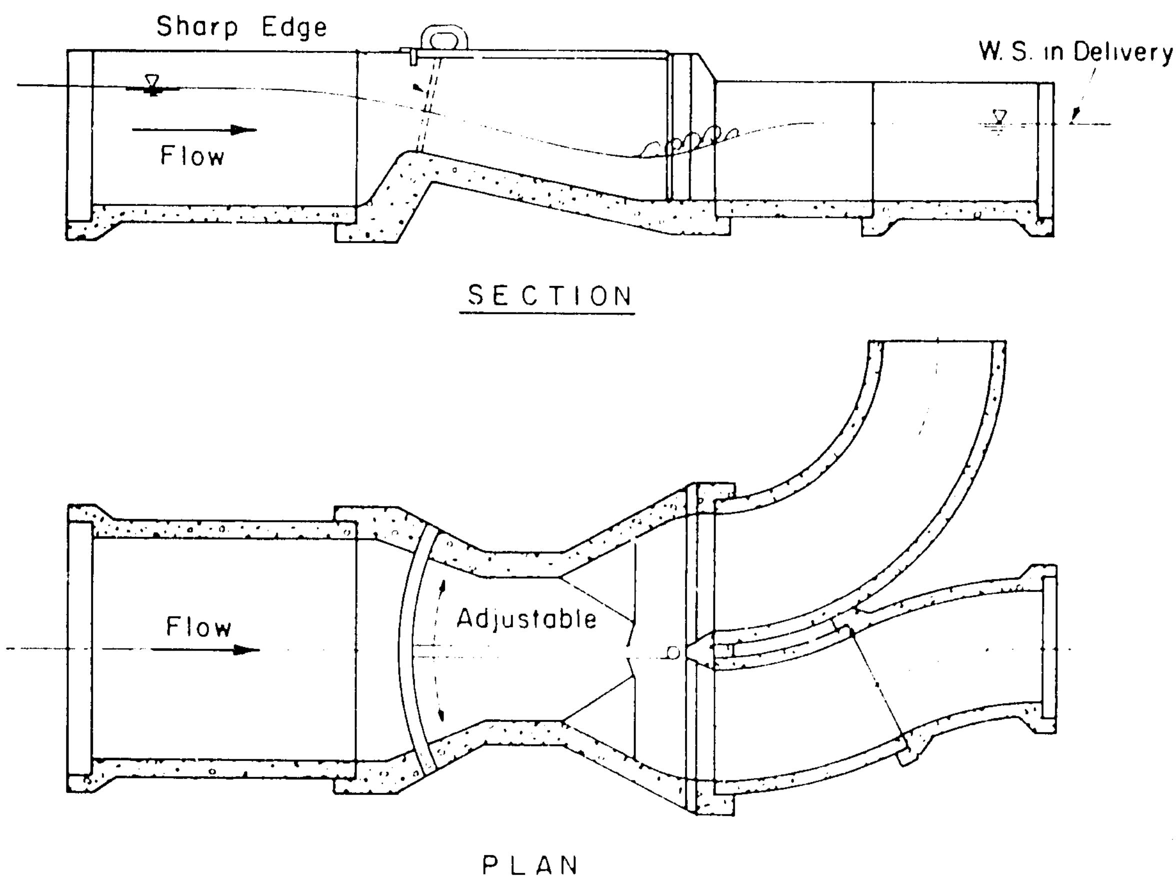


Figure 14. French-type proportional divisor (6).

A simple type of proportional divisor is shown in Figure 15 and has an adjustable divisor board. This divisor can be made semiportable (movable) from wood or metal and permanent from concrete or masonry. Its usefulness is limited except as a means for temporarily dividing the flow. The device requires calibration for different settings of the divisor board and different flow conditions. Another design for an adjustable divisor is shown in Plate 1, Appendix 2.

Several designs for *two and three way small divisors* are shown in Figures 16-18. These structures can be constructed in place to the inside dimensions shown using concrete, concrete blocks, brick, or rock. The designs have been modified to provide for an overflow section above the gate or stop log slots to assure that the water will not overflow onto the ditch banks, but that excess flow will spill into the downstream ditch. They are most often used to divert the full incoming flow into one ditch or another.

In some areas, structures with formed gate slots and sliding gates are a source of problems. The slots become chipped and broken. Gates and stop logs become lost or stolen. When in use they jam, leak and become clogged with sediment and debris. An alternative to the formed gate slots is angle iron guides which are fastened to the inner face of the structures in the same position as the slots. Since these guides are fastened to the surface, the area for flow would be reduced by the width of the two angle iron pieces. Sheet metal liners for formed gate slots as shown on Plate 2, Appendix 2, have been successful also.

Another alternative for structures like Figure 18 would be to eliminate gate slots entirely and use *portable* gates. These gates would be placed in trapezoidal or rectangular sections and would completely or partially obstruct the flow as shown in Figure 19. The advantages are ease of insertion and removal in the ditches. The disadvantages are lack of adjustment for depth and possibility of being removed for other uses.

A recent development of the FAO/World Bank Cooperative Programme is small irrigation structures fabricated from concrete components produced on extrusion machines (9). Figure 20 shows a *divisor* constructed from the *prefabricated* sections now being produced in India. Two sizes of trapezoidal and rectangular sections are being produced by extrusion. Hollow blocks with gate grooves are also being made.

A combination pump outlet and division structure is shown in Plate 3, Appendix 2. The wood (or metal) baffle is necessary to diffuse and still the flow from the pipe.

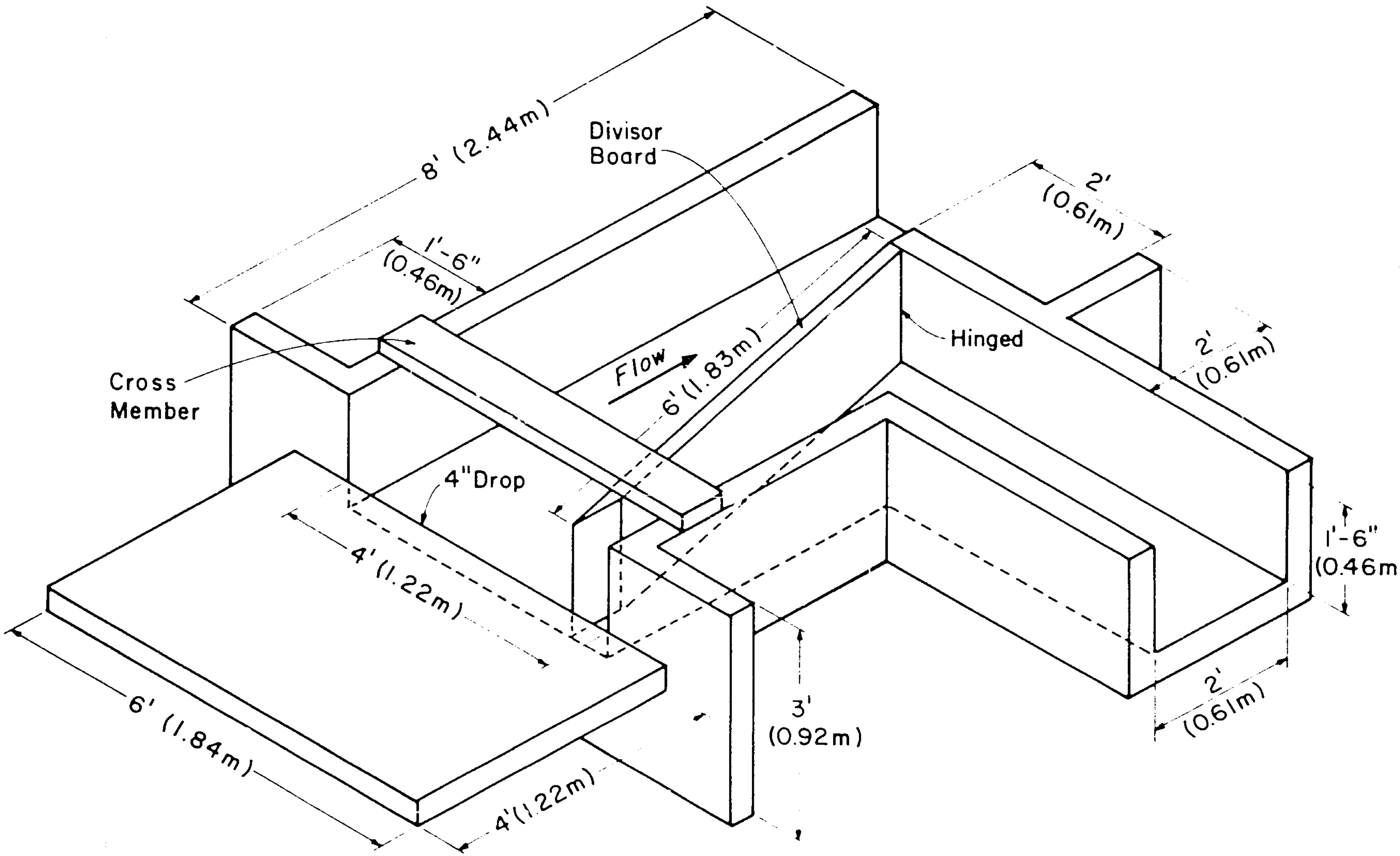


Figure 15. Adjustable proportional divisor (see Plate 1, Appendix 2).

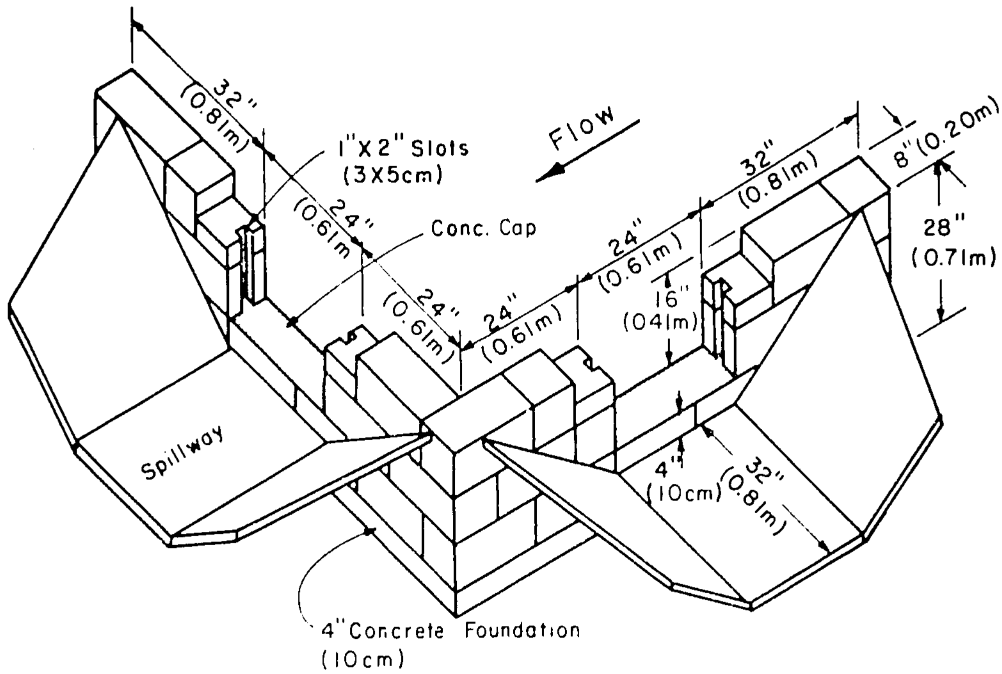


Figure 16. Two-way concrete block divisor (12).

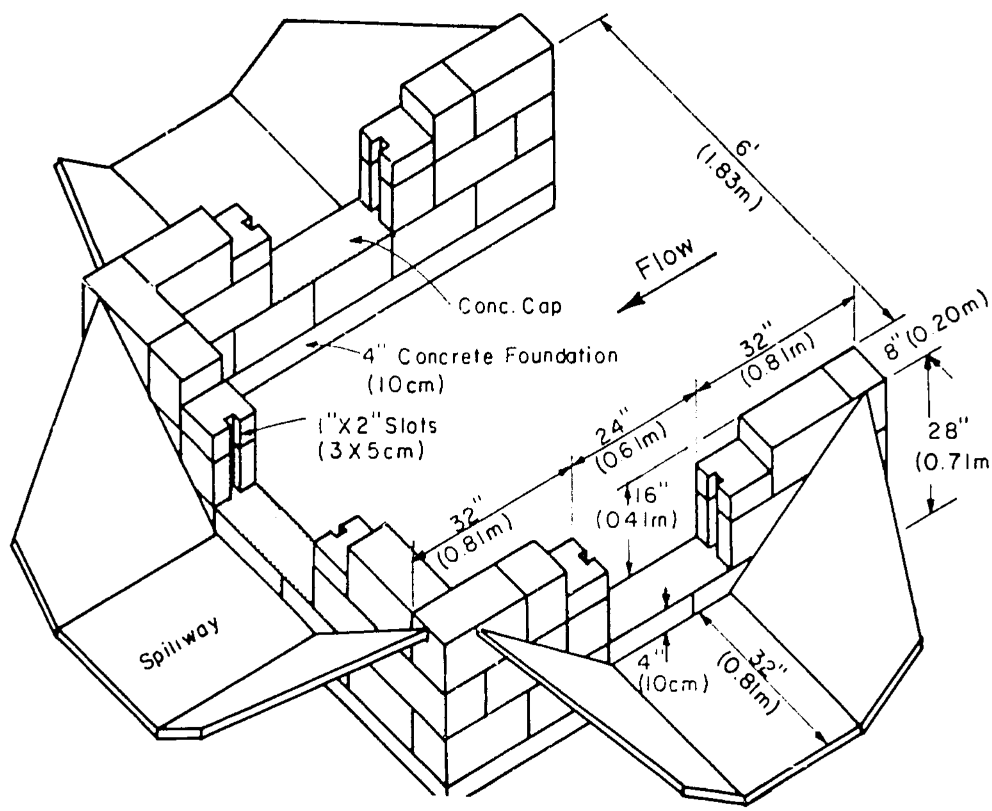


Figure 17. Three-way divisor (12).

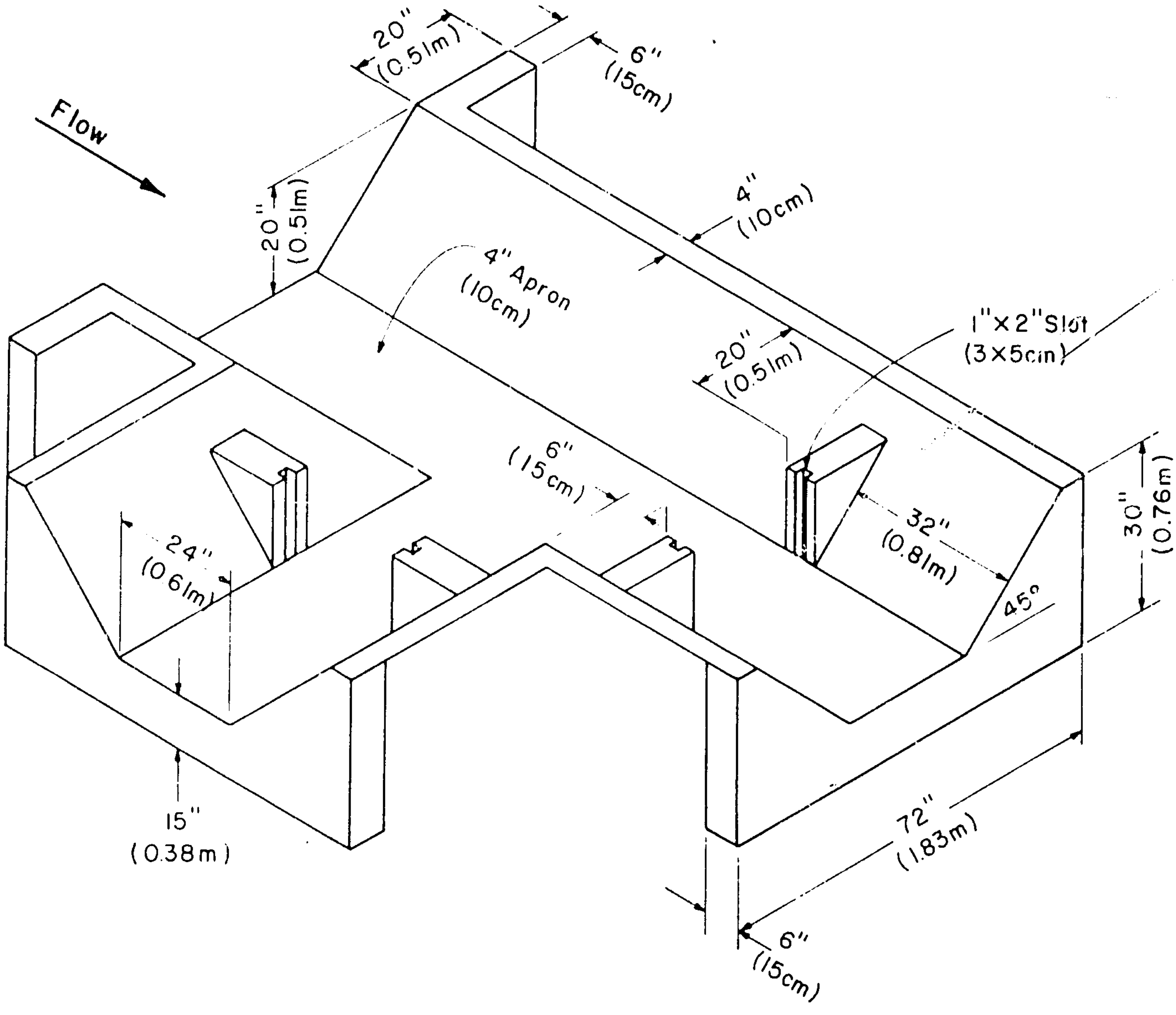


Figure 18. Concrete trapezoidal two-way divisor (14). (See Plate 2, Appendix 2).

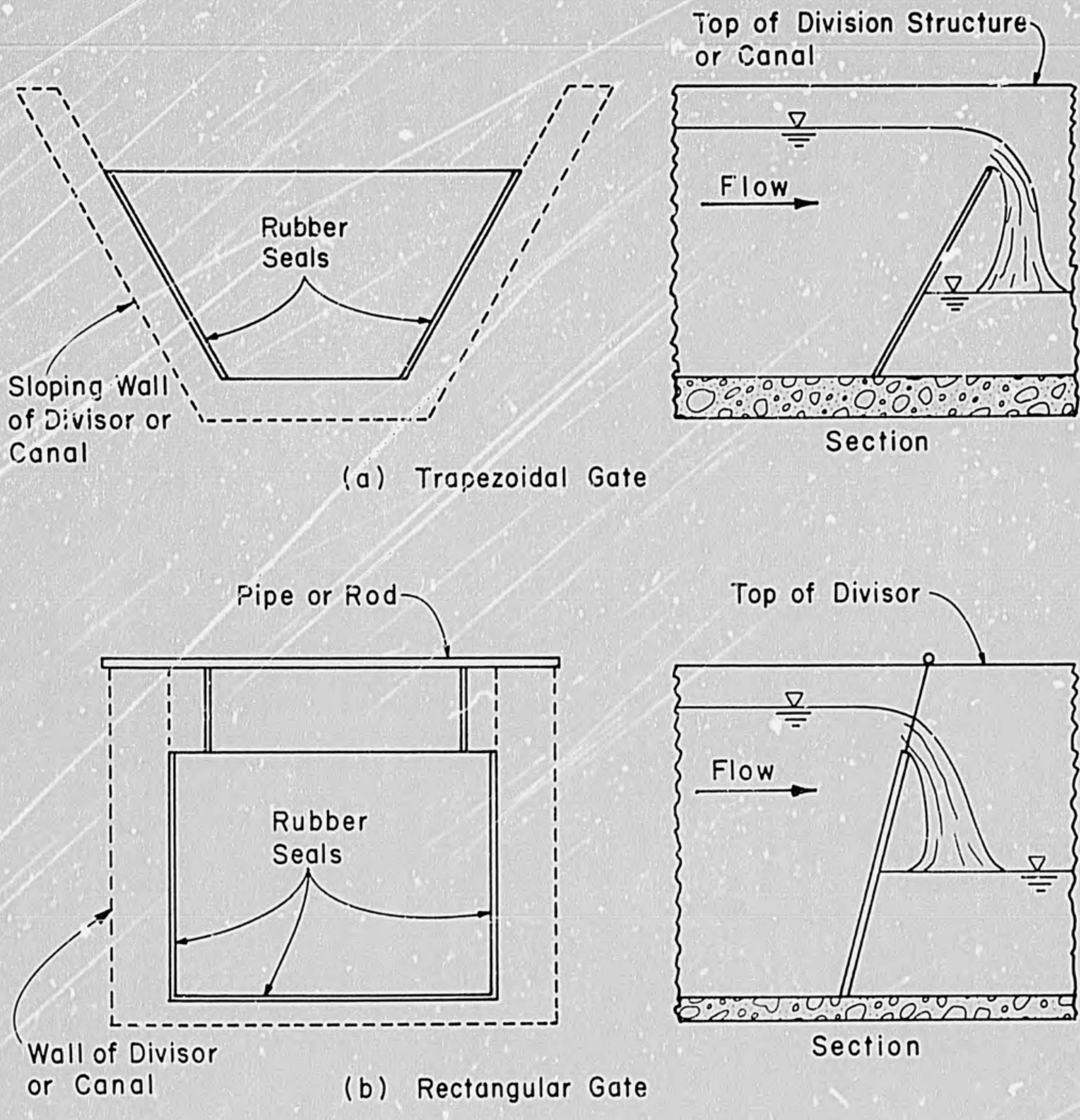


Figure 19. Portable gates for divisor structures.

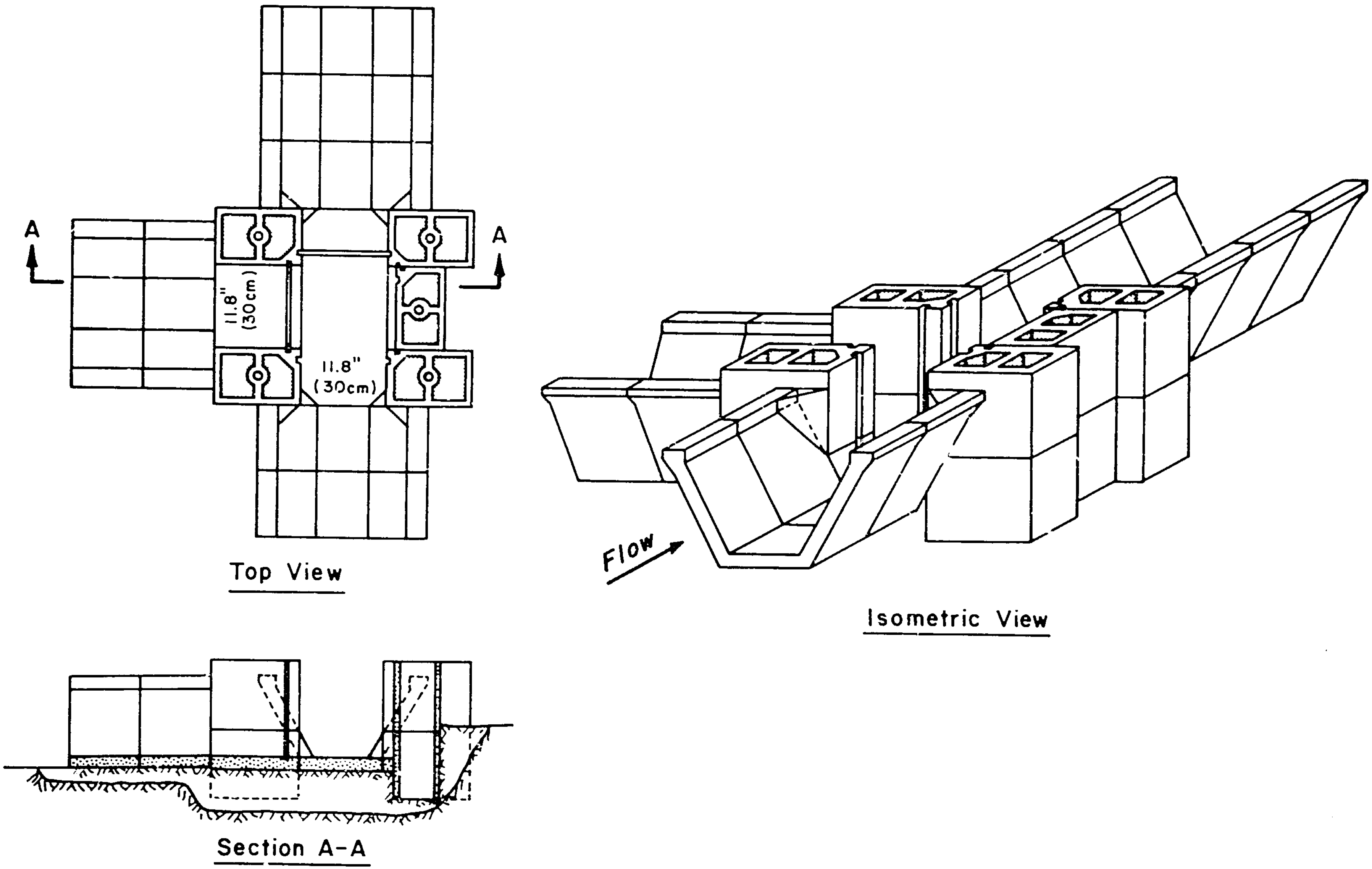


Figure 20. Division structure using extruded, prefabricated sections (9).

b. Drops (Grade Control Structures).

Drops are required to reduce channel grade whenever the natural grade would result in erosive flow velocities in unlined earth channels. Drops are used to 1) control the upstream water velocities to reduce erosion, 2) drop the flow to a lower level, 3) dissipate the excess energy, and 4) control downstream erosion. Drops are particularly needed for newly constructed channels to reduce erosive velocities. After a period of time, the banks and bed may stabilize as the banks become sodded with grass, and drops may no longer be needed.

For a steeply sloping channel, erosion can be controlled by conveying water from one level to another in a stairstep manner with drops (Figure 21). Drops in series are generally spaced so that the difference in water surface elevation at each drop is in the range of 0.3-0.6 m (1.0-2.0 ft). The drop spacing may need to be modified if there a' ˃ irrigation turnouts in the s.ries requiring a prescribed depth of water between two drops.

Figure 22 shows selected types of *drop structures* that can be constructed from concrete, concrete block, brick or stone. These structures are quite effective but may be quite expensive to build, particularly from formed concrete. Where water is to be diverted from one ditch to a lower ditch or field, a drop like that shown in Figure 23 can be used. This structure is a combined turnout, check and drop. The roughness blocks (teeth) in the downstream section are used to accelerate the dissipation of energy and allow use of a shorter and shallower stilling basin. Provision for erosion control in the earth channel to the left of the structure must be provided. Figure 24 shows a concrete drop and check structure under construction (*a*) and after several years of operation (*b*). Rock riprap can be used for a distance downstream from the structure to prevent bank erosion. In some areas there may be objections to water impounding in the downstream apron because of mosquito breeding. An opening in the lower sill is often provided for drainage during non-flow periods. Plates 4, 5, and 6, Appendix 2, give designs for trapezoidal drop structures for drops ranging from 0.3-0.9 m (1-3 ft).

Figure 21. Drop structures used for grade control (4).

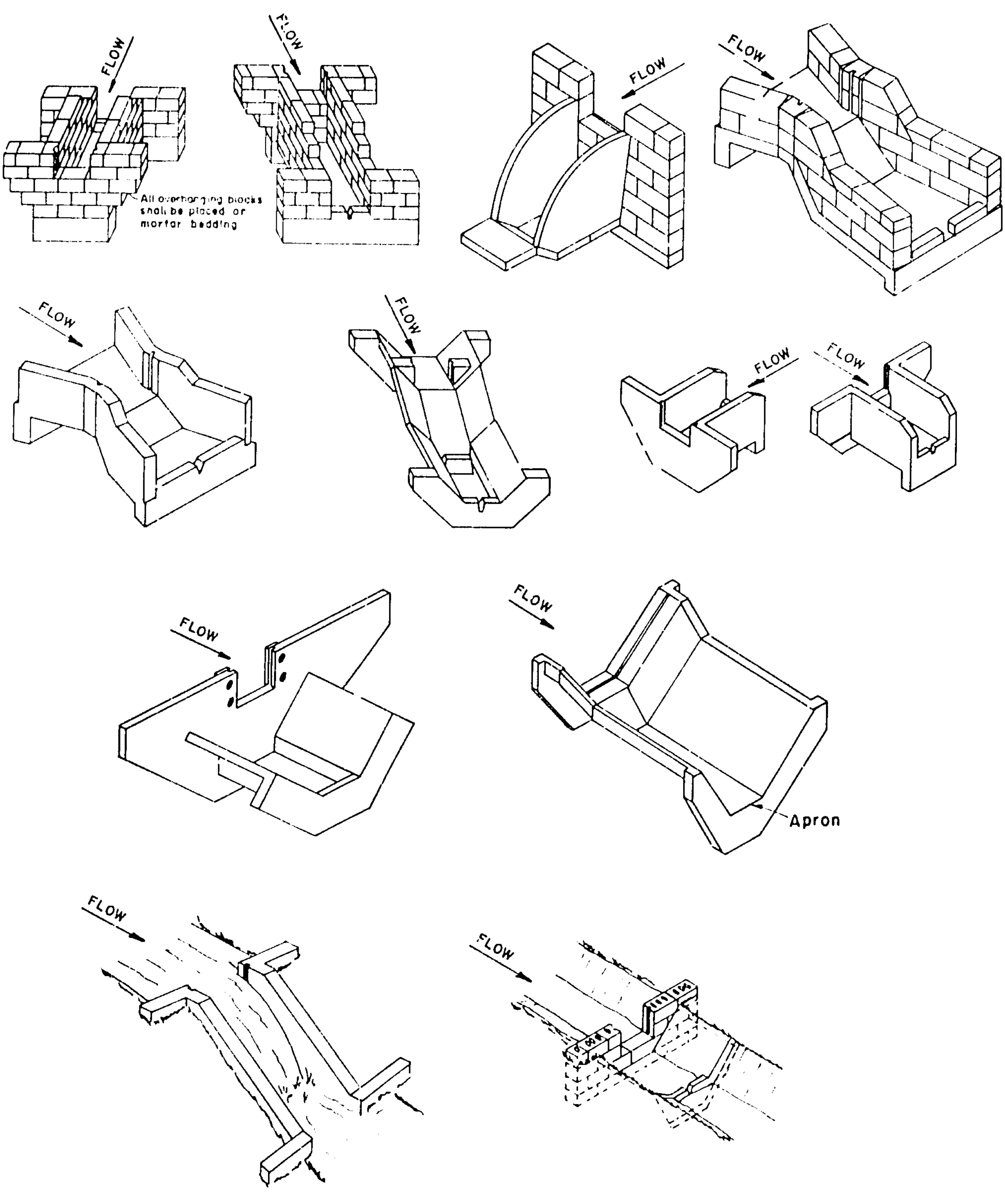


Figure 22. Examples of small drop structures.

The *drop-check structure* shown in Figure 25 can be constructed of concrete, precast concrete, or brick masonry. The wall thickness is usually 7.5 to 10 cm (3-4 in). For the small irrigation ditches, a structure with an opening of 60 cm (2.0 ft) should generally be used. Optional side walls for the stilling basin are shown. The drop and check structure can be easily constructed with standard concrete block following the directions stated on Figure 26. The length of the stilling pool (*L*) should be at least twice the fall height (*H*). Rock protection is placed at the end of the stilling pool to compensate for the relatively short pool.

Concrete drop structures that have been developed over a long period of years by the USDA Soil Conservation Service are shown in Appendix 2. Trapezoidal chute drops for ditch elevation changes of 0.30-0.91 m (1.0-3.0 ft) are given in Plates 4, 5, 6. Vertical, rectangular basin drops are given in Plates 7, 8, 9, 10 for elevation changes of 0.15 to 0.61 m (0.5-2.0 ft). In most cases, the smaller structures with drops of 0.30 m (1.0 ft) or less do not require reinforcing steel for construction. Adequate cutoff walls are provided at either end of the structures to prevent seepage and leakage which might cause the structures to be undermined. These structures have many operational advantages, but disadvantages include high cost and a great amount of labor to construct.

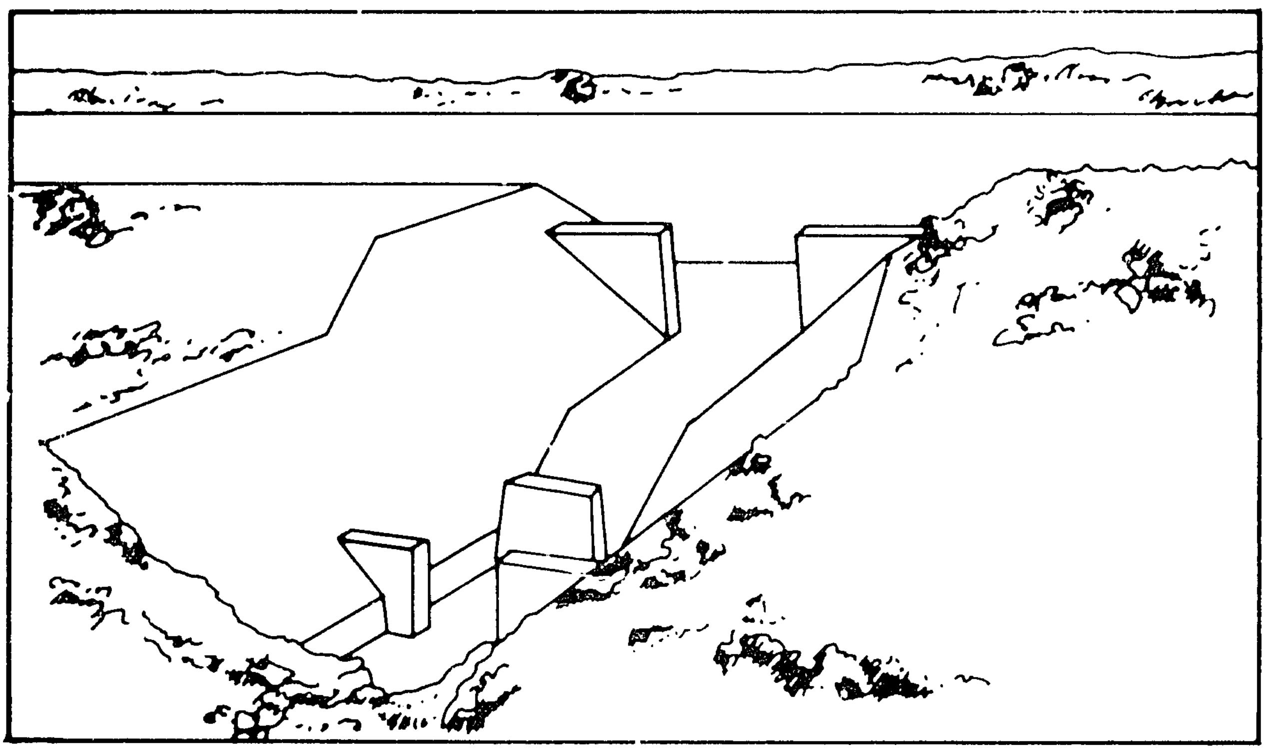
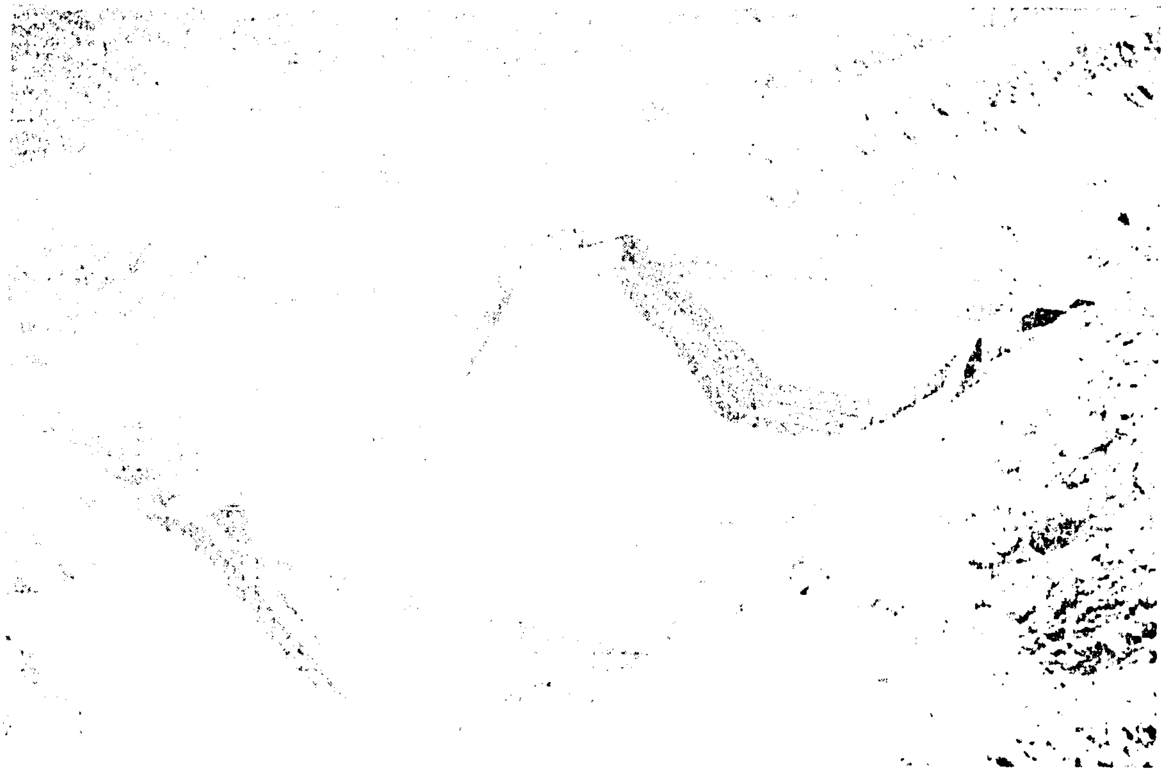


Figure 23. Drop structure combined with turnout.



Note: Steel reinforcement must be supported on wire bridges or small pieces of concrete so that it is above the *ground surface*.

Figure 24*a*. Forms for trapezoidal drop structure ready for pouring concrete (SCS Photo).

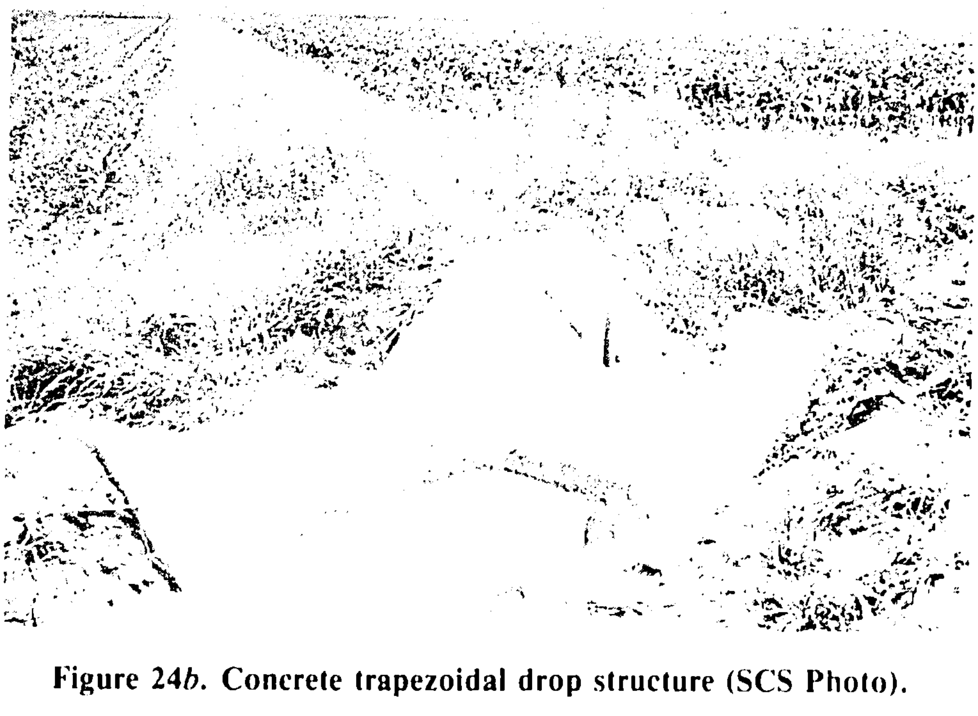


Figure 24*b*. Concrete trapezoidal drop structure (SCS Photo).

Drive-thru irrigation drops made from concrete blocks and concrete are given in Plates 11 and 12 (Appendix 2). These drops are termed drive-through since one set of wheels of tractor-powered ditch cleaning equipment can pass through the structures. Since these structures are long in length, wing walls are not necessary to control the seepage path.

In those areas where wood is available, drop structures like that shown in Plate 13 can be used. For long life, the wood should be treated with a preservative. Sheet metal drops can be made to the same basic dimensions.

Drop-check structures made from the *extruded concrete sections* are shown in Figure 27 (9). The opening in the rectangular check portion of the structure is 30 cm (12 in) wide and the drop does not usually exceed 30 cm (12 in).

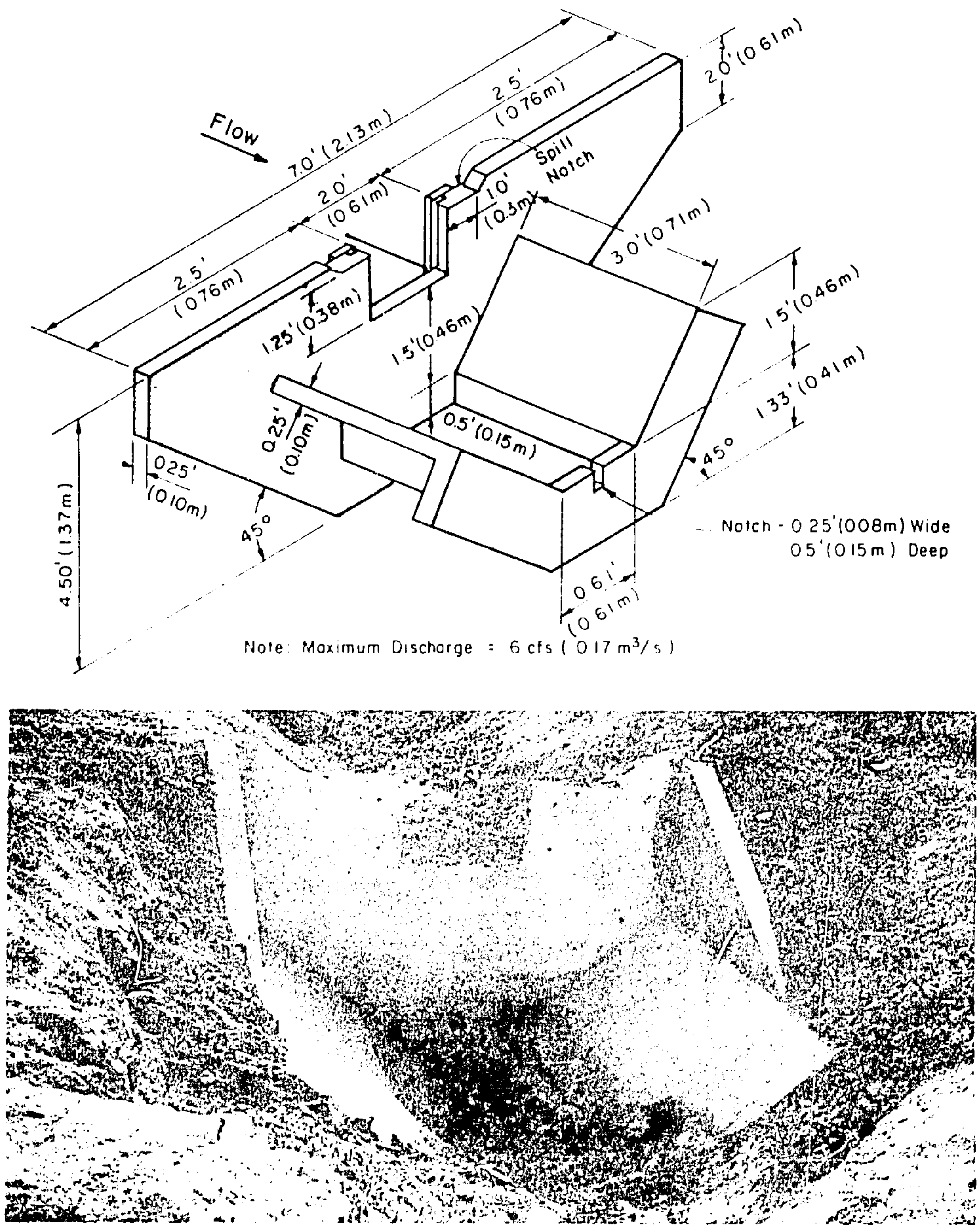


Figure 25. Concrete drop-check (21, 33). Optional sidewalls are shown (SCS Photo).

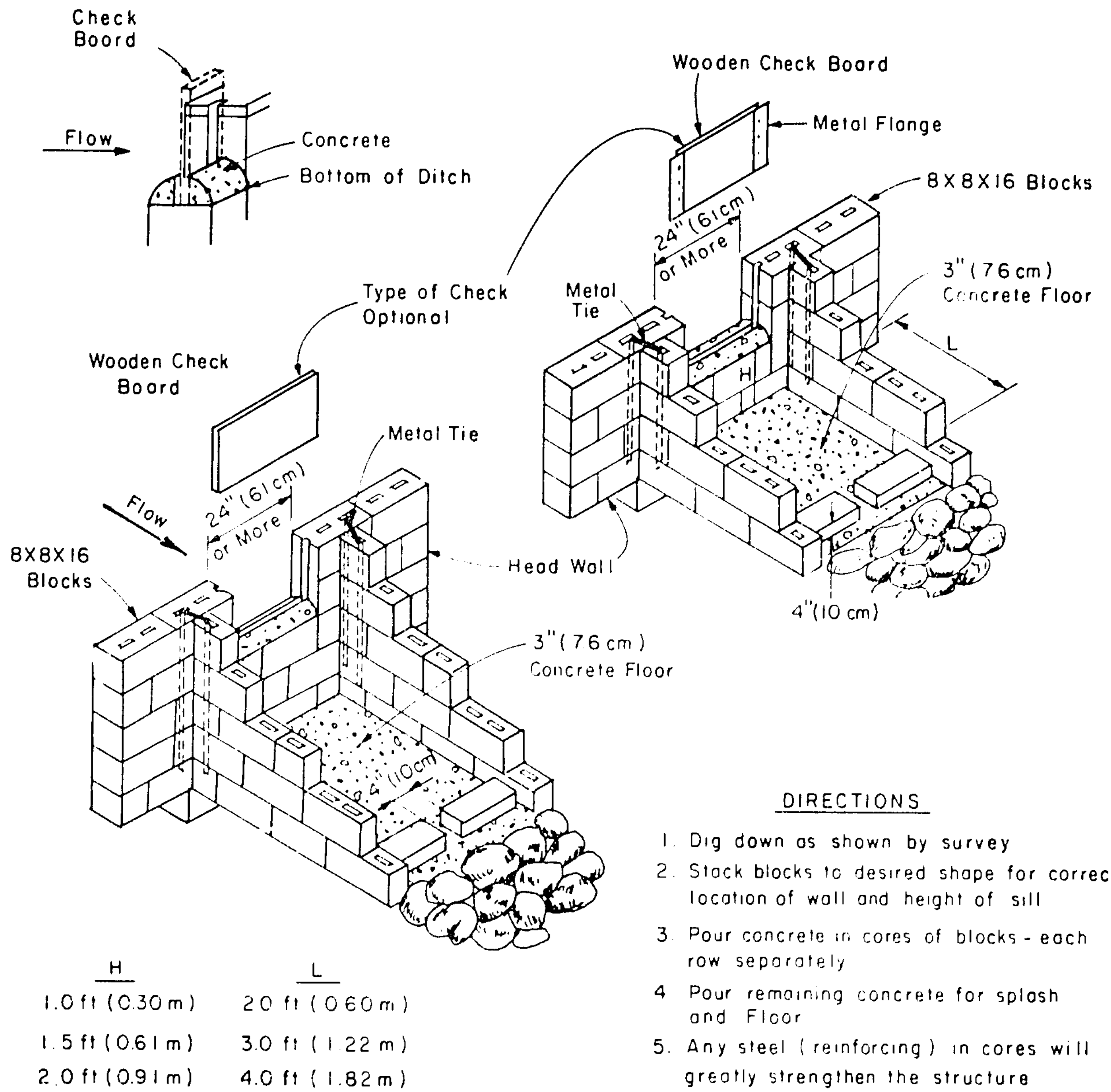


Figure 26. Concrete block drop and check structure (21).

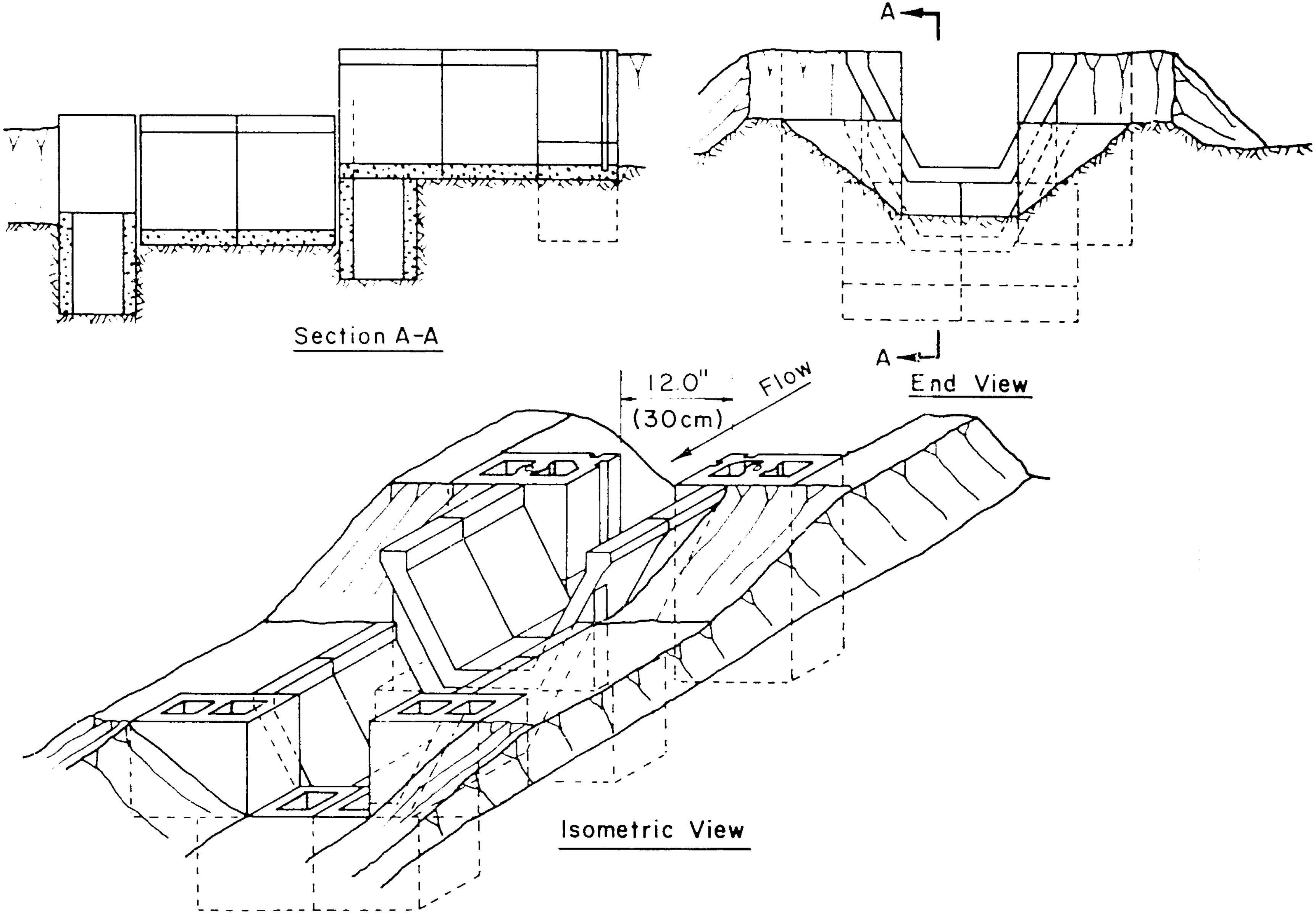


Figure 27. Drop-check structure using extruded concrete sections (9).

In a study made on the performance of small drop-check structures (16) several conclusions were made to aid future designs.

1. Commercial prefabricated structures generally did not have adequate stilling basins.
2. End sills and low tail water depth caused excessive scour due to water cascading over the end sills. The scour holes should be lined with rock or gravel.
3. Wide basins performed better for downstream scour prevention.
4. Trapezoidal basins operated successfully only with high tailwater. (Properly placed blocks in the basin would improve operation.)
5. A nonaerated nappe (see definition of terms) from the drop structure resulted in better stilling.
6. Headwall structure with adequate cutoff depth and width with gravel-lined basin or plunge pool was most effective and most economical.

The structure described as a *headwall* with *gravel-lined basin* was very effective as a drop. Only the vertical wall of the structure shown in Figure 26 is used and can be concrete or masonry. Precast concrete walls are quite common and economical, but are very heavy and require hoists to place. For a masonry wall, the thickness should usually be 30 cm (12 in.), unreinforced concrete, 20 cm (8 in.) and reinforced concrete, 10 cm (4 in.). Slots, either cast into the sides of the opening or with channel iron fastened to the edge, are used with stop logs to adjust elevation of the drop opening. The width of the gravel-lined basin should be about twice the headwall opening crest length. The length of the basin should be 2 to 3 times the difference in upstream and downstream water levels (fall height *H*) at normal ditch capacity. Initially the basin can be dug to an inverted cone shape (V) with the depth below the downstream ditch bottom being about the same as the difference in upstream and downstream water surfaces at normal ditch discharges. Large gravel or crushed rock up to 2.5 to 5.0 cm (1-2 in) in size in a layer about 5.0 cm (2 in) thick should be used for the stilling pool. During operation, the flow will adjust the size and shape of the pool for a stabilized stilling pool (Figure 28).

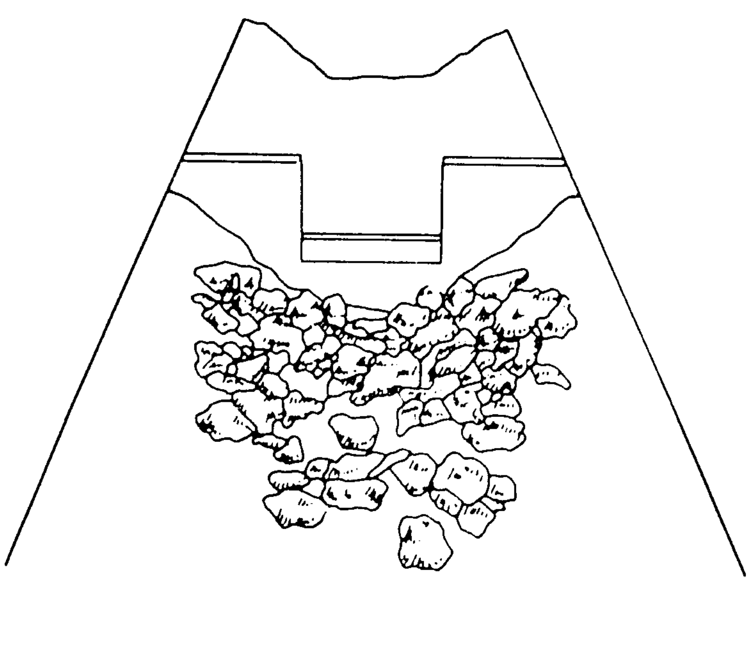


Figure 28. Drop structure with gravel/rock stilling basin (16).

A *pipe drop* structure is shown in Figure 29. Capacities range from 65 l/s (2.3 ft^3/s) for 25.4 cm (10 in.) diameter pipe, to 154 l/s (5.5 ft^3/s) for 38.1 cm (15 in.) pipe, both 3.3 m (11 ft) long operating with a 0.3 m (1 ft) water depth. The pipe diameter is sized specifically to maintain an upstream water level. The change in water surface elevation upstream to downstream of the pipe structure ranges from 0.30-0.91 m (1-3 ft) depending on the design. Concrete, asbestos cement and baked clay pipe may be used as well as corrugated metal pipe. Riprap protection from erosion is usually needed on the downstream side. Disadvantages are that the pipe entrance is easily plugged with debris and the pipe alternately primes and breaks when the flow is less than the design flow, causing surging. One advantage is that the structure can be used as a road crossing, possibly using a somewhat longer length of pipe. Different designs of pipe drops are given in Plates 14, 15, and 16, Appendix 2.

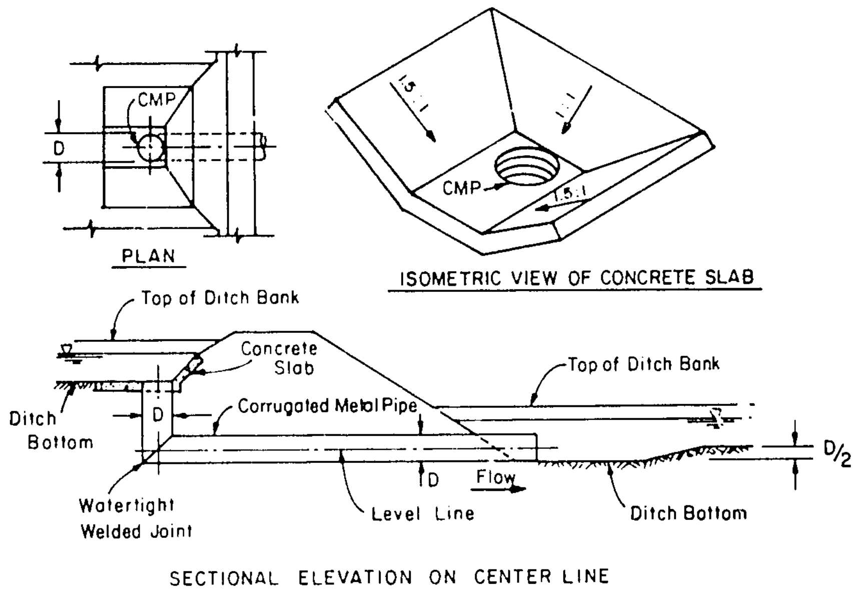


Figure 29. Typical pipe drop structure (3, 21, 33).

Chutes as drops can be cheaply constructed where rock is locally available. Chutes as shown in Figure 30 are suitable where falls (*H*) of 1.5 to 3 m (5-10 ft) are encountered. Fairly large rock, 15-20 cm (6-8 in) in diameter can be used, placed in two layers. The rock layer extends above the waterline at the ditch sides. The rough rock face of the chute assists in dissipating the energy of the falling water. A shallow stilling basin is desirable.

It should be noted that many of the drops can also be used as water measuring weirs. Drops like those shown in Figures 25 and 26 can be adapted for water measurement by carefully measuring the width of the opening and the upstream water depth. This function is discussed in the section on measurement of flow.

c. Checks.

Irrigation ditch checks may be permanent or temporary, portable or stationary. They may check the entire flow or allow a portion of it to pass. The primary purpose of a check is to increase the ditch storage and water surface elevation. Many times, checks and drops are combined into one structure so that a discussion of drops also applies to checks as indicated in the foregoing section on drops.

A variety of checks are used in both lined and unlined ditches. The checks are fitted with checkboards, slide gates or other means for releasing a portion of the flow while maintaining a desired water level. It is highly recommended that checks have an overflow provision so that the ditch will not be overtopped if the check is inadvertently left closed when turnouts are closed. This can be accomplished by providing a weir overflow section on the check at an elevation lower than the ditch bank.

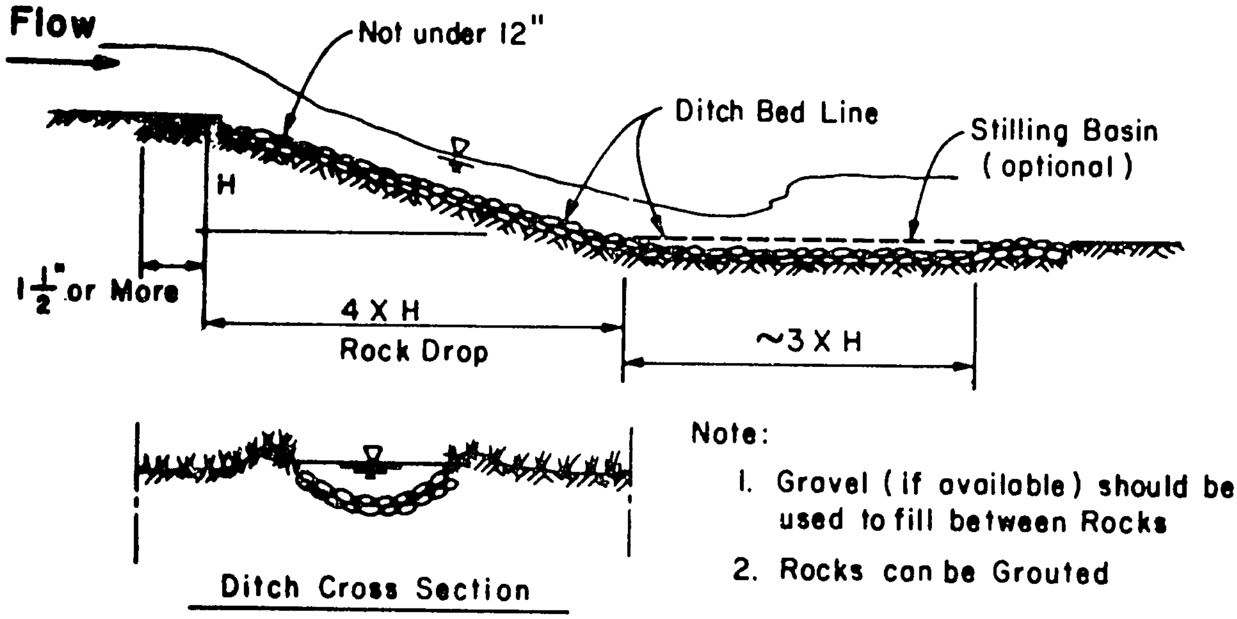


Figure 30. Sloping rock drop structure (21).

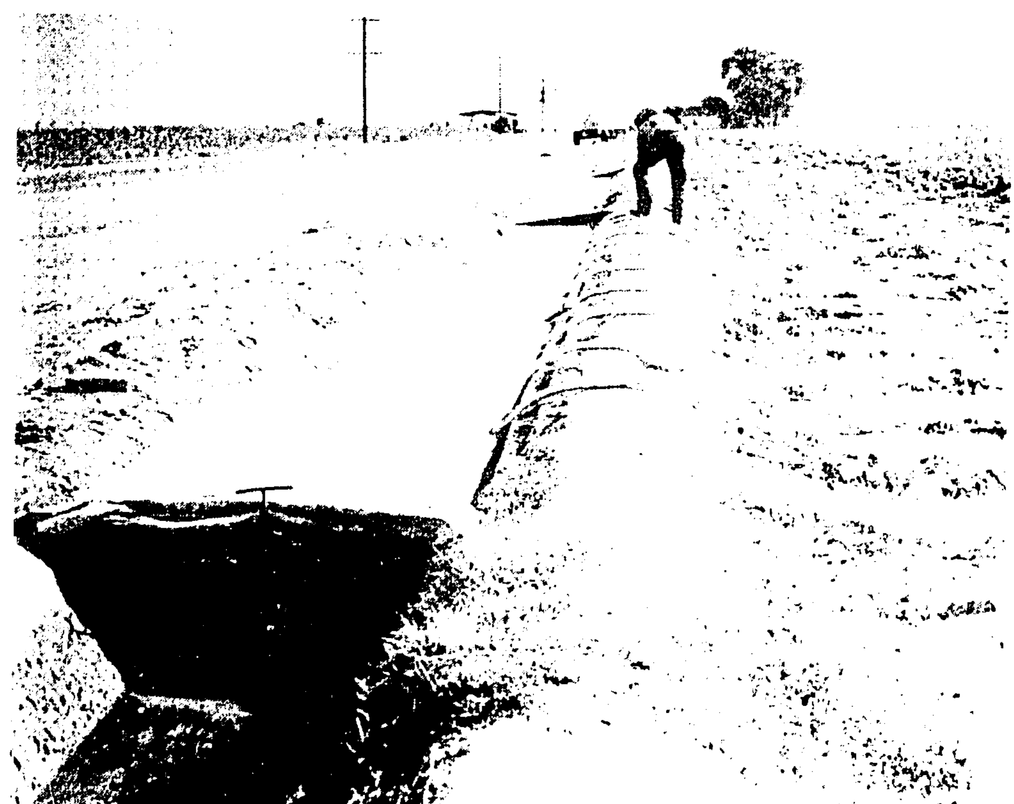


Figure 31. Ditch check in lined ditch with siphon tubes.

Several different types of ditch checks utilizing various methods to control the flow are shown in Figure 31, 32, 33 and 34. Figure 34 is a *simple check structure* combined with a *turnout*. Adequate erosion protection for the bypassed flow must be provided downstream from a check in the form of a basin, rock riprap, or a paved apron. Checks may be used for measuring irrigation flows using standard weir or orifice relationships (6, 35). It should be kept in mind that when checked for irrigating, the water surface in the ditch should typically be 10-15 cm (4-6 in.) above the field ground surface (possibly more for spiles and siphons). Also a freeboard for the canal bank of 15 cm (6 in.) above the water surface must be maintained. This results in the top of ditch banks being 25-30 cm (10-12 in.) above the surrounding ground surface.

Several *simple checks* that can be made from wood, sheet metal, concrete or masonry are shown in Figure 33. The width is determined for the rectangular opening by the width of the ditch, bank elevation, and maximum flow that the structure will pass. However, the opening is usually 50-60 cm (20-24 in.) wide with the bottom within a few centimeters (5-10) (2-4 in.) of the bottom of the ditch. Checkboards in widths of 5-10 cm (2-4 in.) are sometimes used to give incremental depths when required during an irrigation. Checks with rectangular or circular openings are sized using the orifice equation (6, 35) with maximum flow and a small difference (5-10 cm) (2-4 in.) in the depth upstream and downstream from the check. In all cases the cutoff wall for the checks should project to a depth below the ditch bottom of 20 cm (8 in.) for clay soils and 30 cm (12 in.) for sandy soils. The wall should extend into each bank 20-30 cm (8-12 in.) at the elevation of maximum water level. With careful backfilling and compaction, the check should not "wash out."

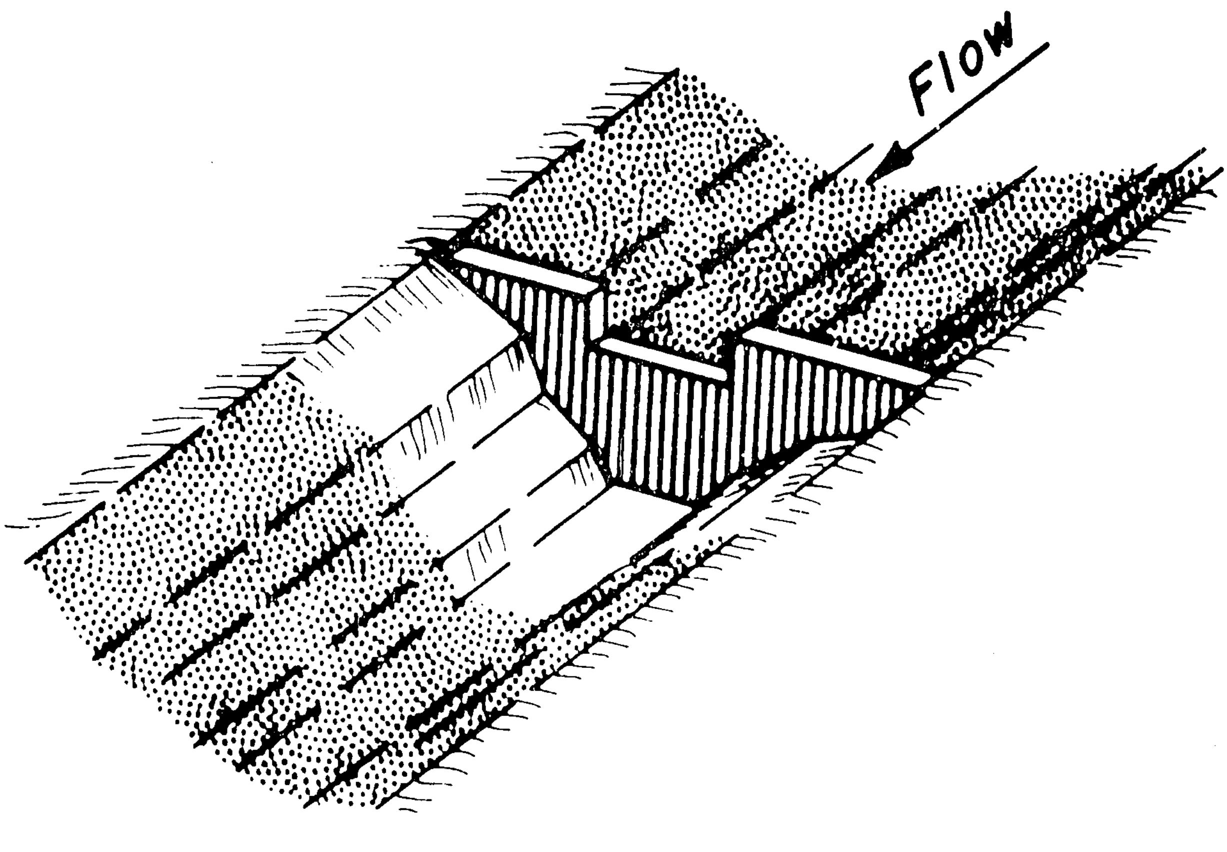


Figure 32. Small concrete ditch check (21).

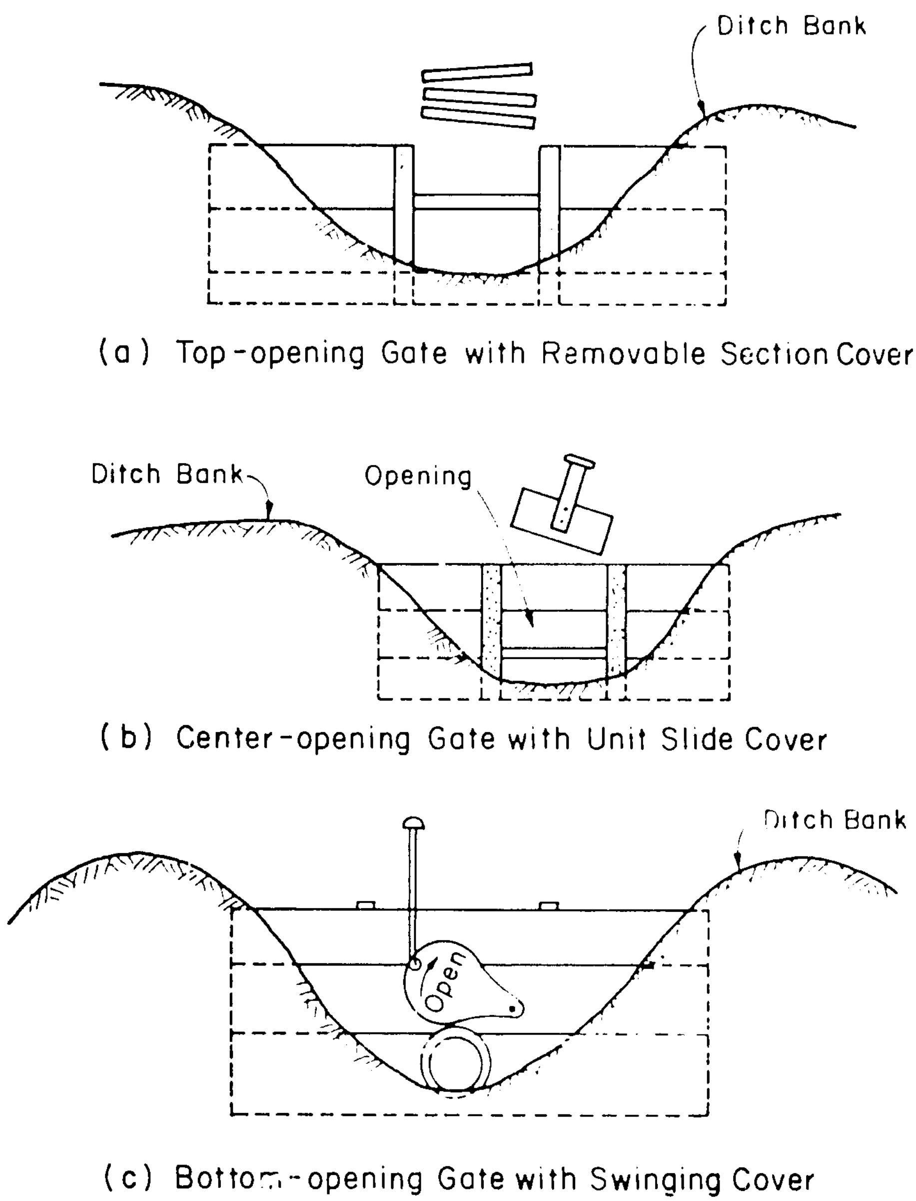


Figure 33. Wooden ditch checks with different openings (19).

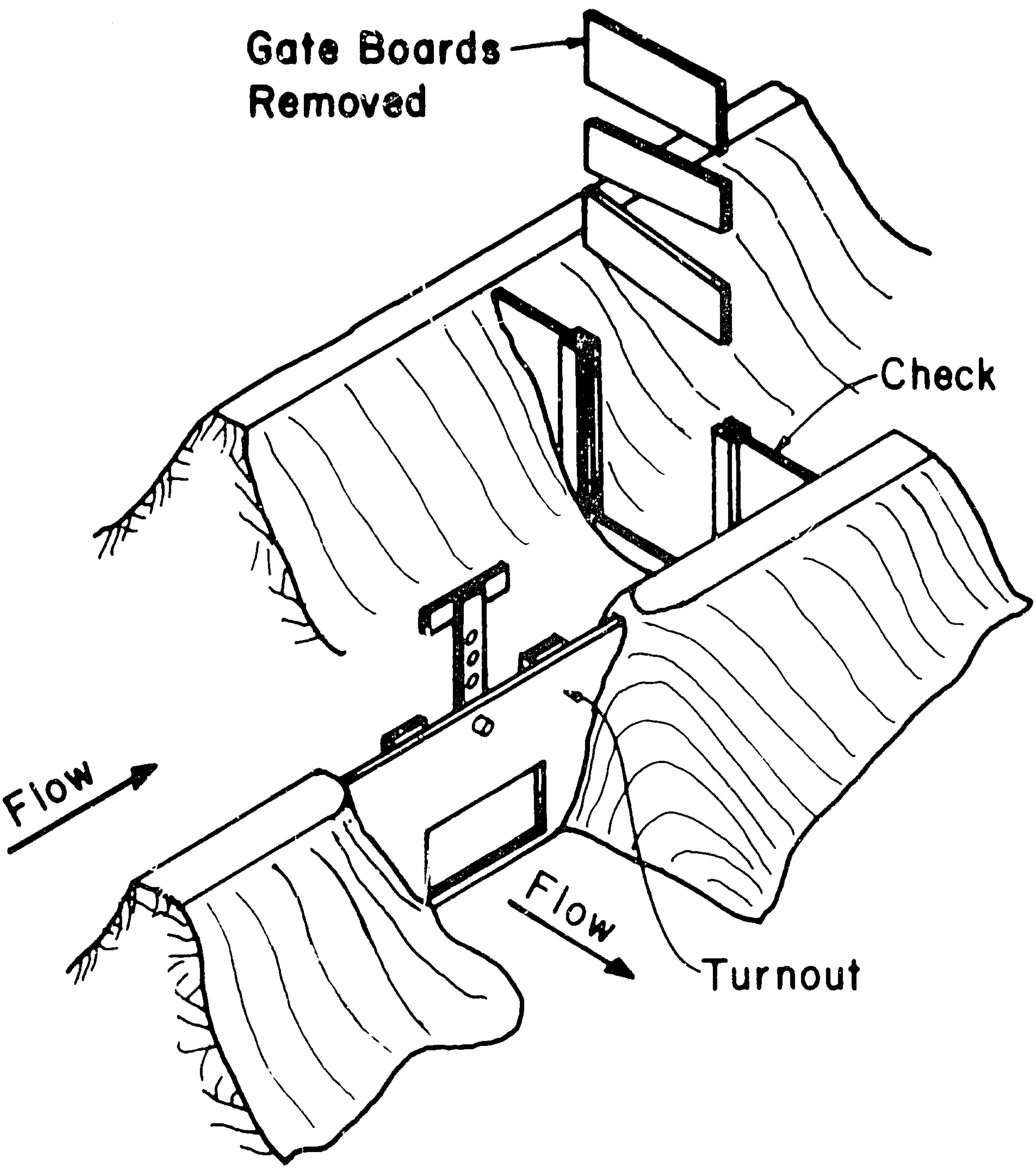


Figure 34. Wood, single-wall check with turnout (19).

Checks that can be made from *concrete blocks* or *brick masonry* set on a concrete foundation are given in Figures 35 and 36. In both cases, the bottom of the opening should be set at the level of the upstream ditch bottom. The depth and width of structures will vary depending on the ditch but usually for small ditches they are built to the dimensions given in Figures 35 and 36. The rubble or masonry spillway basin shown in Figure 35 should be set so that the bottom is 5-7 cm (2-3 in.) below the original ditch bottom. The gravel lined plunge pool described in the section on drops can be used with a check like Figure 36 if downstream erosion is a problem. The design of permanent, cast-in-place concrete checks is given in Plate 17, Appendix 2.

Portable irrigation dams are checks used to raise the water level in ditches for direct irrigation. They can be made by the irrigators or purchased. Figure 37 shows a canvas check with a sleeve for discharging part of the water. The sleeve can be tied with a drawstring to control the flow to be bypassed. A pipe or long, strong stick is used across the top. The canvas has a long section upstream that is anchored in the soil and partly covered to prevent leakage. Plastic and rubber sheeting can be used instead of the canvas cloth as shown in Figure 38. The overflow section can be raised or lowered by turning and anchoring the crossmember.

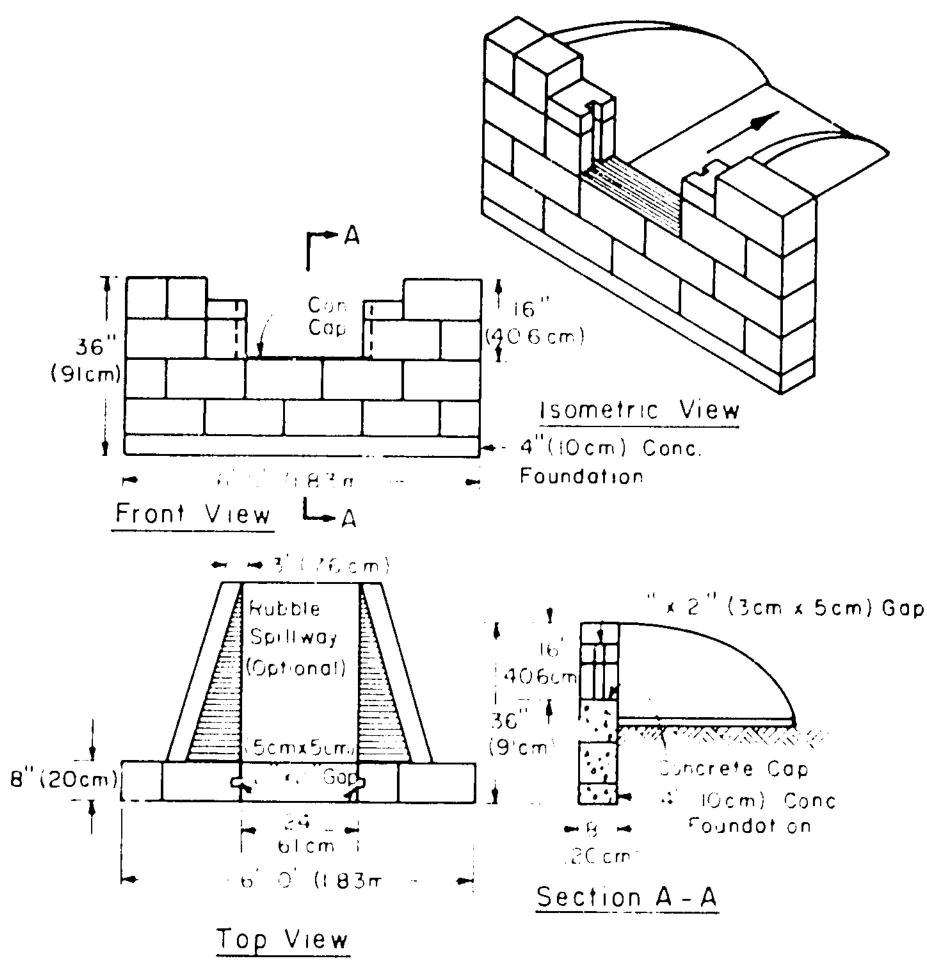


Figure 35. Concrete block check with apron for erosion control (12).

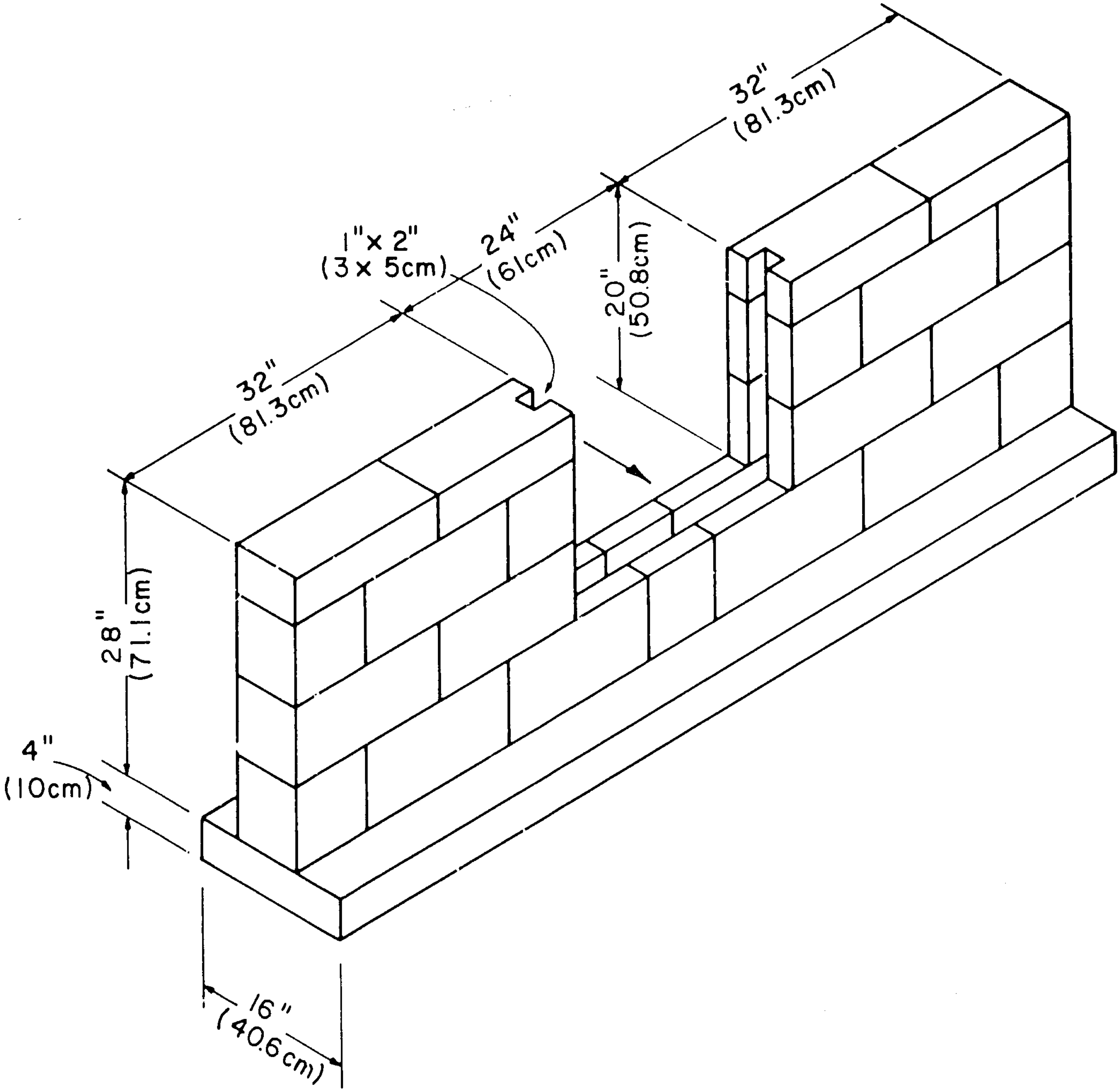
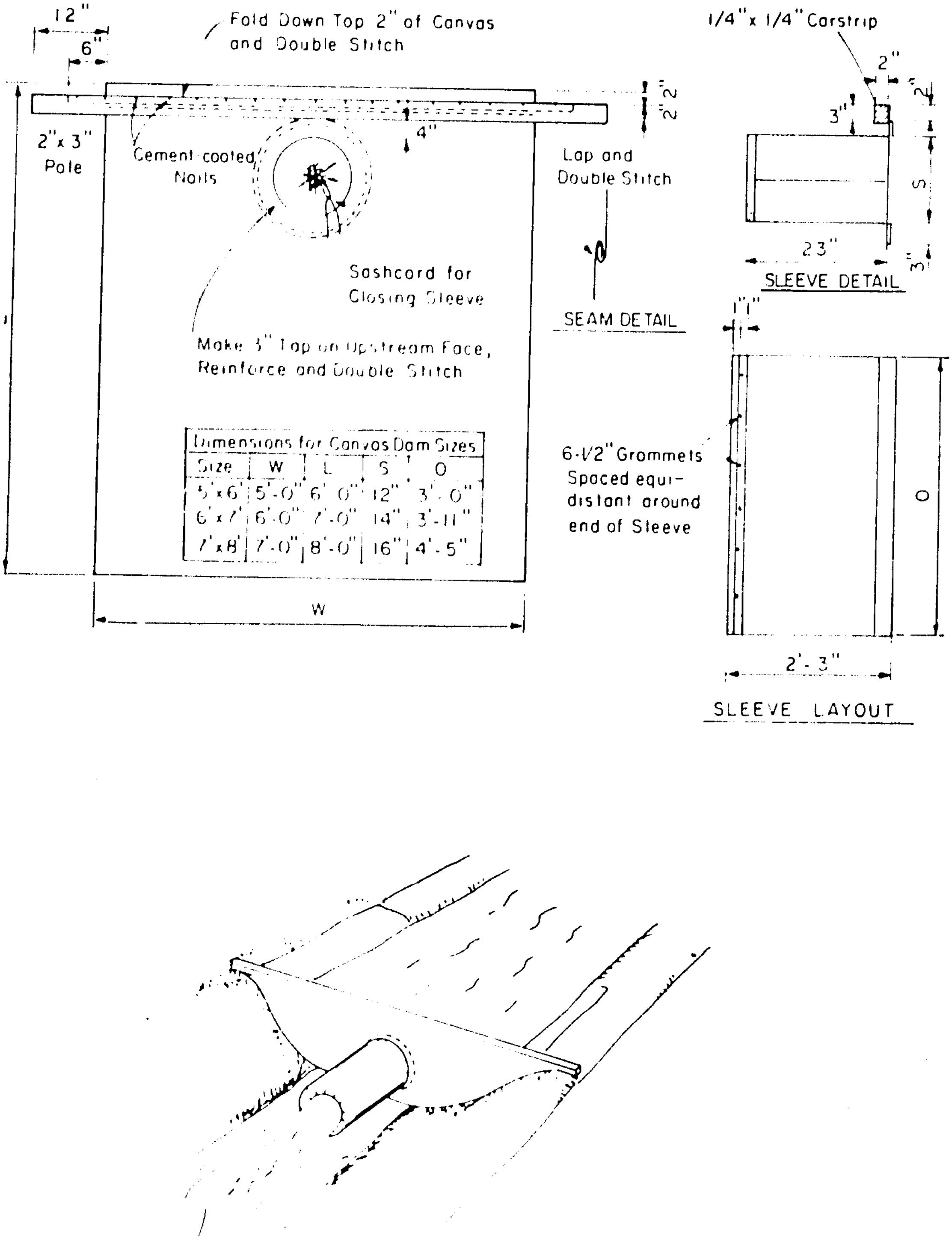


Figure 36. Concrete block check structure (14).



Dimensions for Canvas Dam Sizes

Size	W	L	S	O
5'x6'	5'-0"	6'-0"	12"	3'-0"
6'x7'	6'-0"	7'-0"	14"	3'-11"
7'x8'	7'-0"	8'-0"	16"	4'-5"

Figure 37. Portable canvas check with discharge sleeve (21).

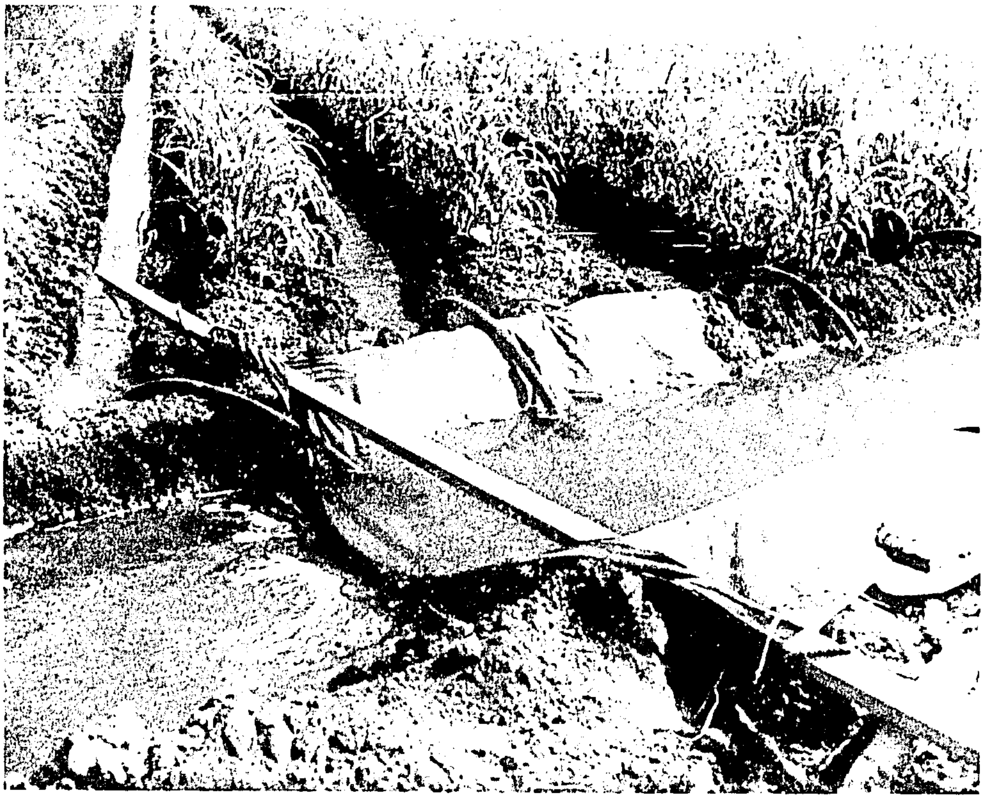


Figure 38. Flexible, portable ditch check.

Rigid and flexible portable check dams are shown in Figures 39 and 40. The rigid dam in Figure 39 is made from metal and driven into the soil. Flow can then be passed downstream through a gate. The series of dams in Figure 40*a* are flexible and made from canvas. Siphon tubes are being used to irrigate the area served between each dam. The metal dam in Figure 40*b* checks the flow for discharge through the upstream gate onto borders. When the ditch is in operation, this gate is held in place in the trapezoidal lined ditch by the water pressure.

d. Turnout, Outlets.

When irrigation water is delivered from the distributary or secondary canal to the tertiary canal, a gated turnout of some design is used. Turnout structures are sometimes similar to check structures but are placed into ditch banks to permit water to be removed from the ditch. A rotation system of water delivery to the tertiary canals is common where the water is on for a prescribed number of days and off for a period. Variable amounts of flow are delivered during the "on" period depending on availability. When the tertiary system has been designed to deliver equal amounts of water, divisors used as turnouts are used as shown in Figures 12 and 13. These divisor-turnouts are commonly used in Pakistan, India and Egypt.

Turnouts and outlets are also used on the tertiary canals (farm laterals) for water release to the quaternary canals (field ditches). They have gates or stop logs for individual flow control (see Plate 18, Appendix 2). These devices may also serve as divisors as shown in Figures 17, 18 and 20; drop structures, Figures 23, 24, 25, 26 and 27; and check structures, Figures 34, 35 and 36.

Small control structures and devices such as gated outlets are used to deliver the water from the field ditches (quaternary canals) to the farm fields. Devices used for this purpose include siphon tubes, spiles (straight tubes), slots in lined canals, and bank cuts in earth canals. For basin irrigation, regular outlet structures like Plate 16, Appendix 2, are used to deliver water to the field.

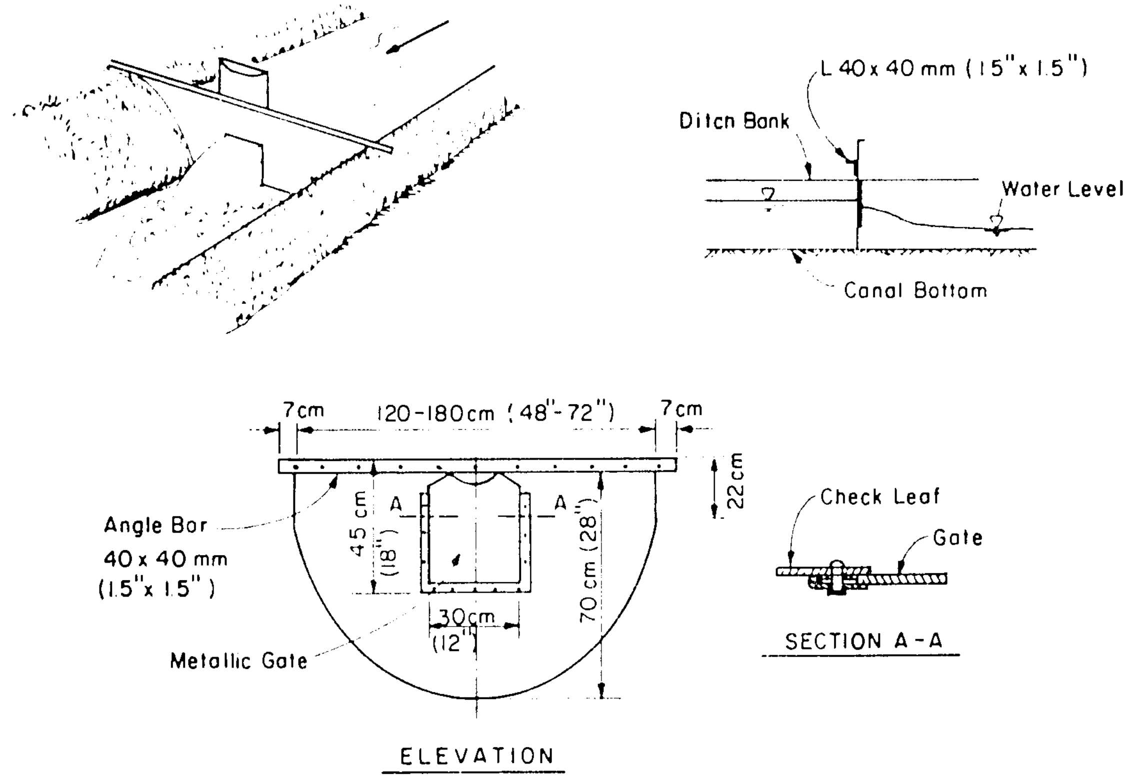


Figure 39. Portable metal check (21).

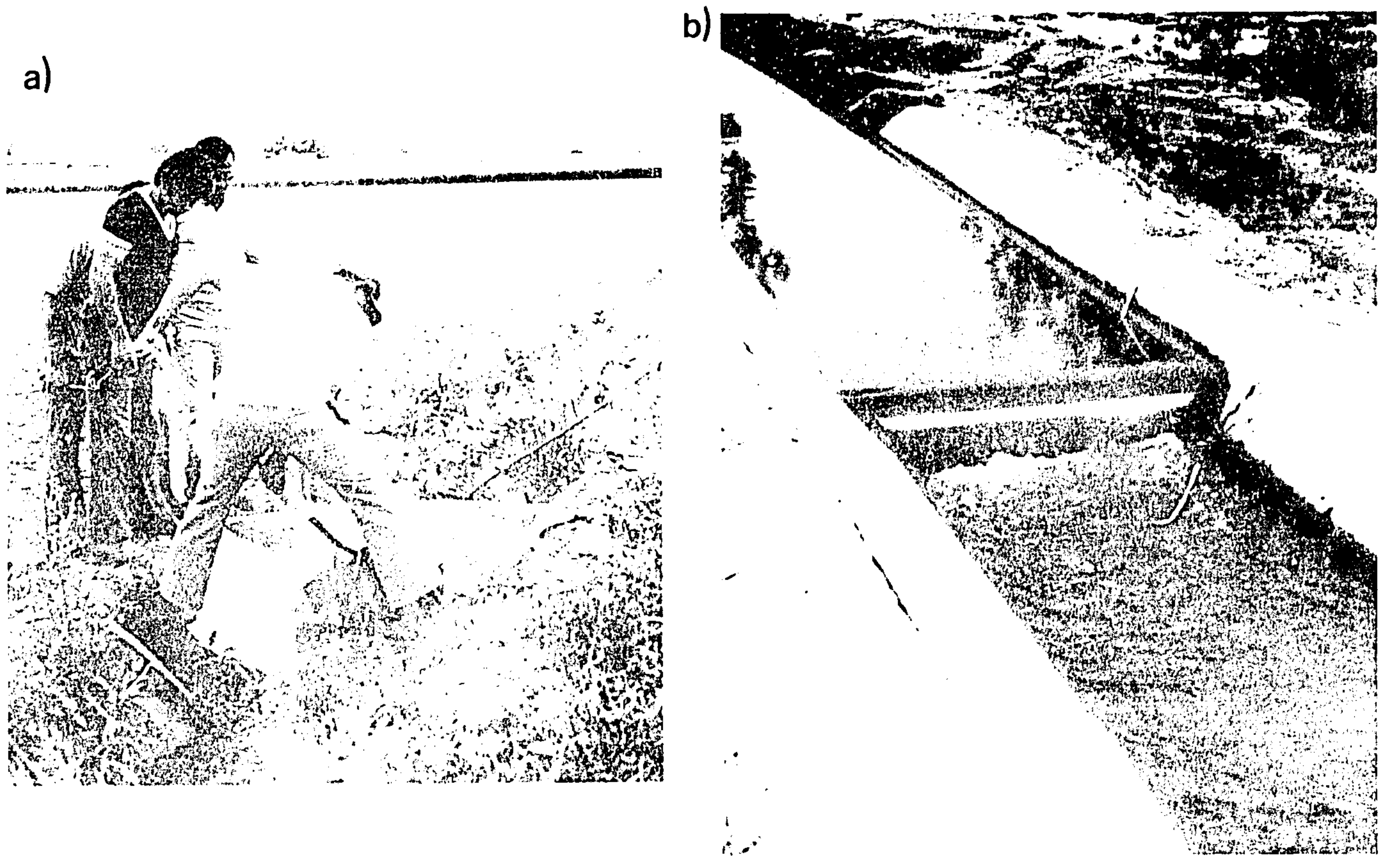


Figure 40. Two types of portable checks (25).

Canal or ditch gates attached to pipe (Figure 41) are the most common type of turnout. The gate may be attached to a concrete or masonry headwall or directly to a steel, concrete or plastic pipe which protrudes through the ditch bank (see Plate 19, Appendix 2). Usually the gate is on the upstream side, but can be placed on the downstream end of the pipe for ease of operation and ditch cleaning. There are many commercially designed ditch gates in use. The movable plate may be square or round and the movement can be controlled with an adjustable rod or hand wheel, as shown in Figure 41.

One type of *canal gate* has been used as a turnout and measuring device in the U.S.A.(35). The discharge end of the pipe has an elbow turned upward so that the pipe flows full. Two stilling wells with point gages or recorders are used to determine the hydraulic head in the canal and downstream from the pipe gate. With the difference in the two heads and the area of gate opening measured by the rise in the gate stem, the amount of flow is determined based on previous calibrations.

Small pipe gates are commonly used for turnout control in field outlets on lined ditches (Figures 42*a* and 43). The gates are mounted vertically in rectangular shaped concrete or masonry lined ditches and along the sidewall for trapezoidal linings. Gates, attached to pipe, can be mounted either vertically or at an angle, and can be used in earth ditches. In most cases, rock riprap will be needed at the pipe discharge end to control erosion. Basic dimensions and flow ranges are shown in Figure 43.

Outlet boxes are shown in Figures 42*b*, 44 and 45. These turnouts are generally made from wood but sheet metal can be used. They are semiportable and may be reset by the irrigator as needed. The floor should be set at or below the field surface--up to 15 cm below--to minimize erosion. These boxes are particularly adapted for the large flows needed in border and basin irrigation.

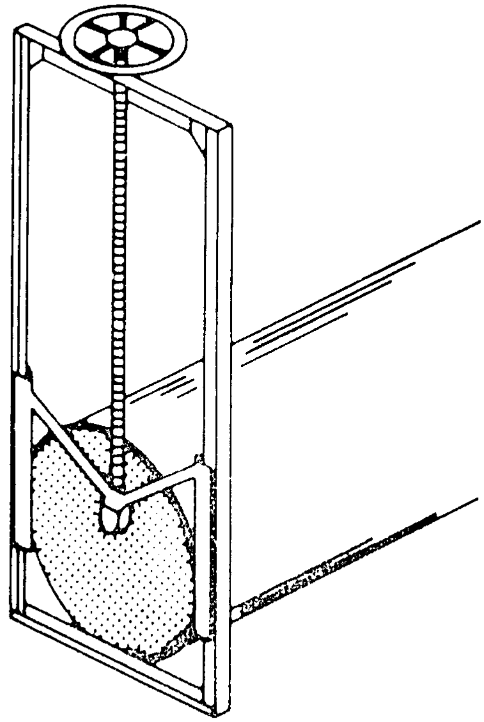
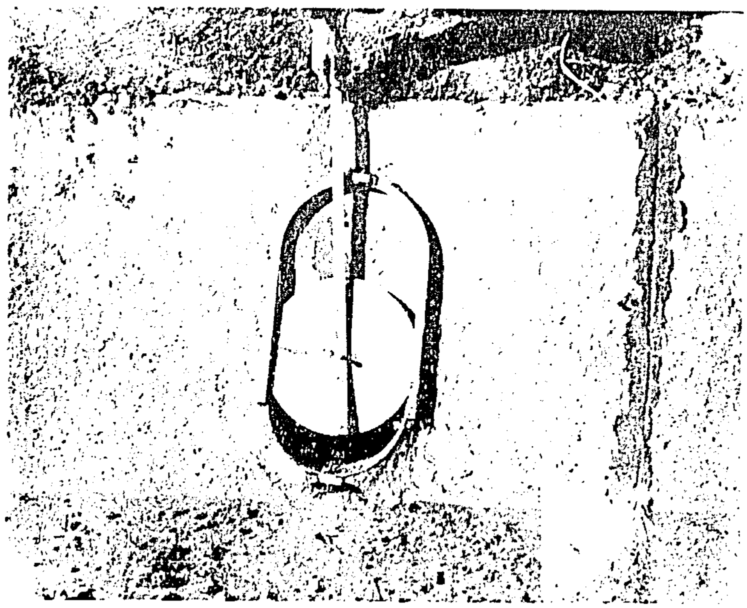


Figure 41. Gated pipe outlet.

A *concrete block turnout* ditch constructed from extruded blocks for an unlined ditch is shown in Figure 46 (9). The turnout is combined with a check structure and utilizes block with formed gate grooves set on a 5 cm (2 in.) thick concrete slab. Erosion protection is needed downstream from the turnout.

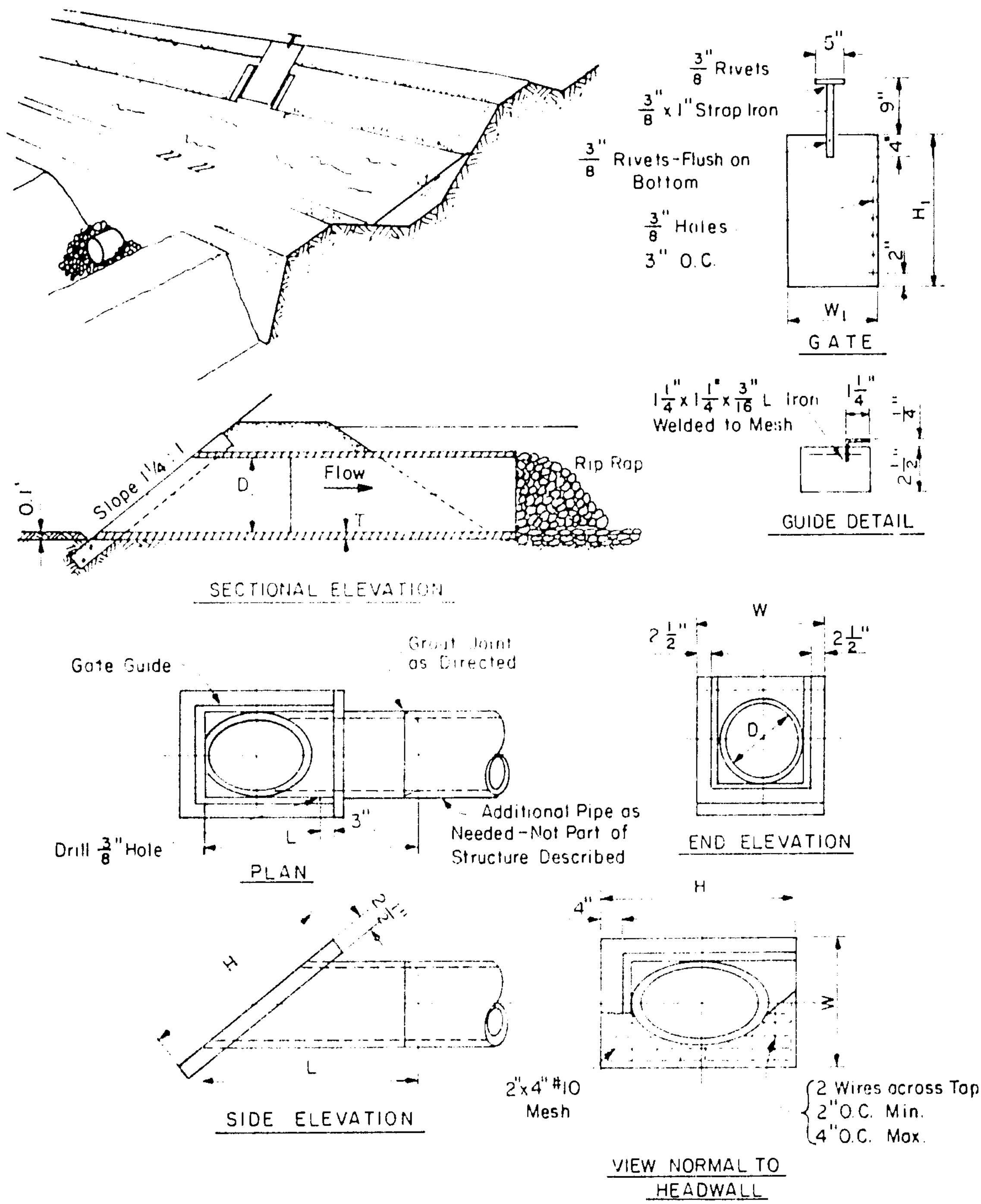
Neyrpic orifice modules (6, 21), used as intakes for secondary and tertiary canals as well as farm turnouts are shown in Figure 47. The module is a metering device with movable slides which can be opened singly or in multiples to obtain the desired flow. Typically, the small distributor (module) has compartments for 5, 10, 15 and 30 l/s flow for a maximum flow of 60 l/s (2.1 cfs). For correct operation, the parent canal should operate within a prescribed depth with only small fluctuation. To maintain the near constant operating depth upstream from the module, constant upstream or downstream level gates are used in the parent canal as shown in Figure 47. The advantages of this system are: 1) automatic and relatively easy operation, 2) preset discharge amounts, and 3) not easy to tamper with. The disadvantages are: 1) relatively expensive, 2) subject to clogging with debris, and 3) requires an almost constant depth of water which in turn requires depth control.



(a)

(b)

Figure 42. Commonly used turnouts for farm irrigation ditches (SCS Photo) (25).



D		H		W		H_1		W_1		Capacity Range	
in	cm	in	cm	in	cm	in	cm	in	cm	cfs	ℓ/s
6	15.2	22	56	17	43	14	36	11	28	0.7 - 1.0	20 - 28
8	20.3	25	64	17	43	18	46	11	28	1.2 - 1.7	34 - 48
10	25.4	29	74	21	53	22	56	15	38	1.6 - 2.4	45 - 58
12	30.5	33	84	21	53	25	64	15	38	2.4 - 3.1	58 - 88

Figure 43. Concrete pipe turnout (34).

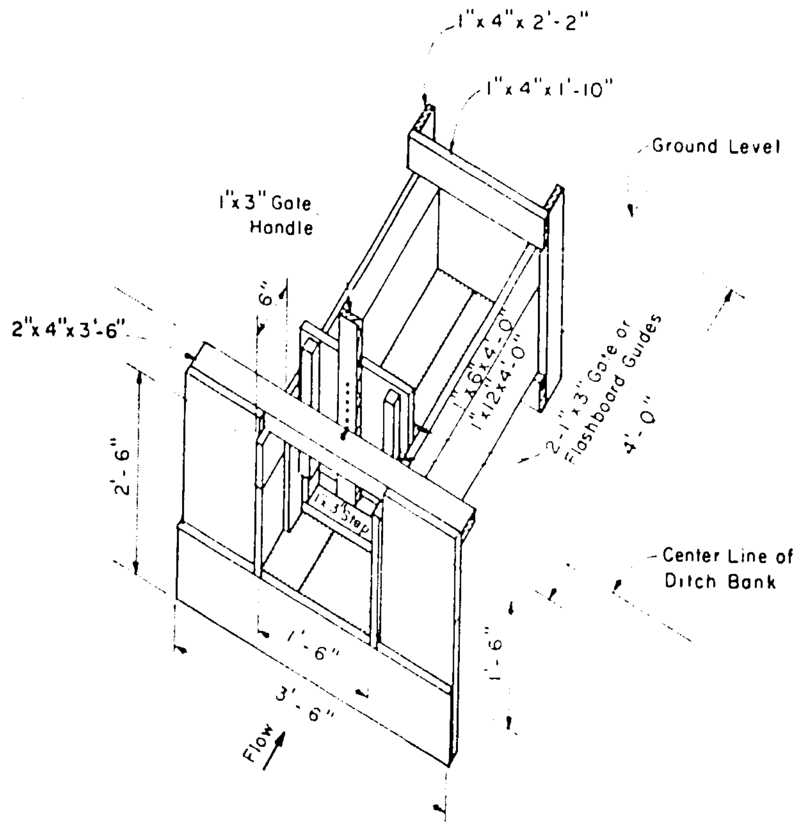


Figure 44. Wooden turnout for basin and border irrigation (34).

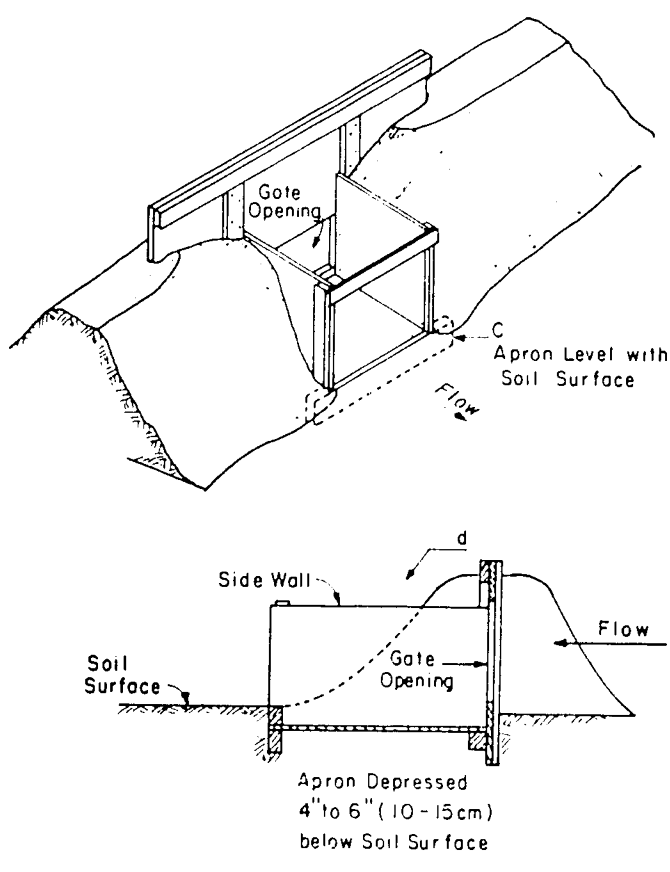


Figure 45. Two designs of wooden turnouts--apron (1) level with soil surface and (2) depressed below soil surface (19).

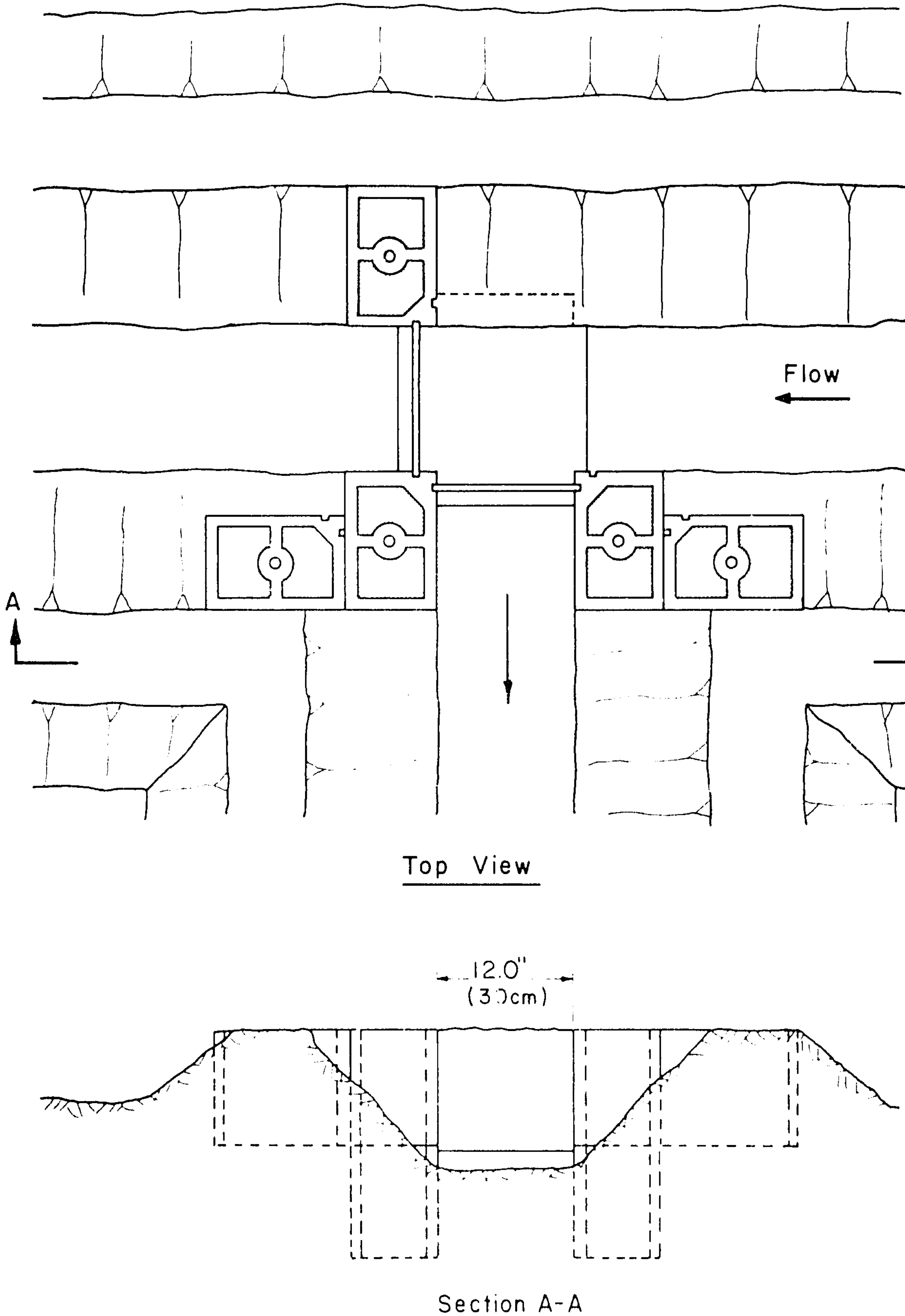


Figure 46. Turnout-check structure using extruded concrete sections (9).

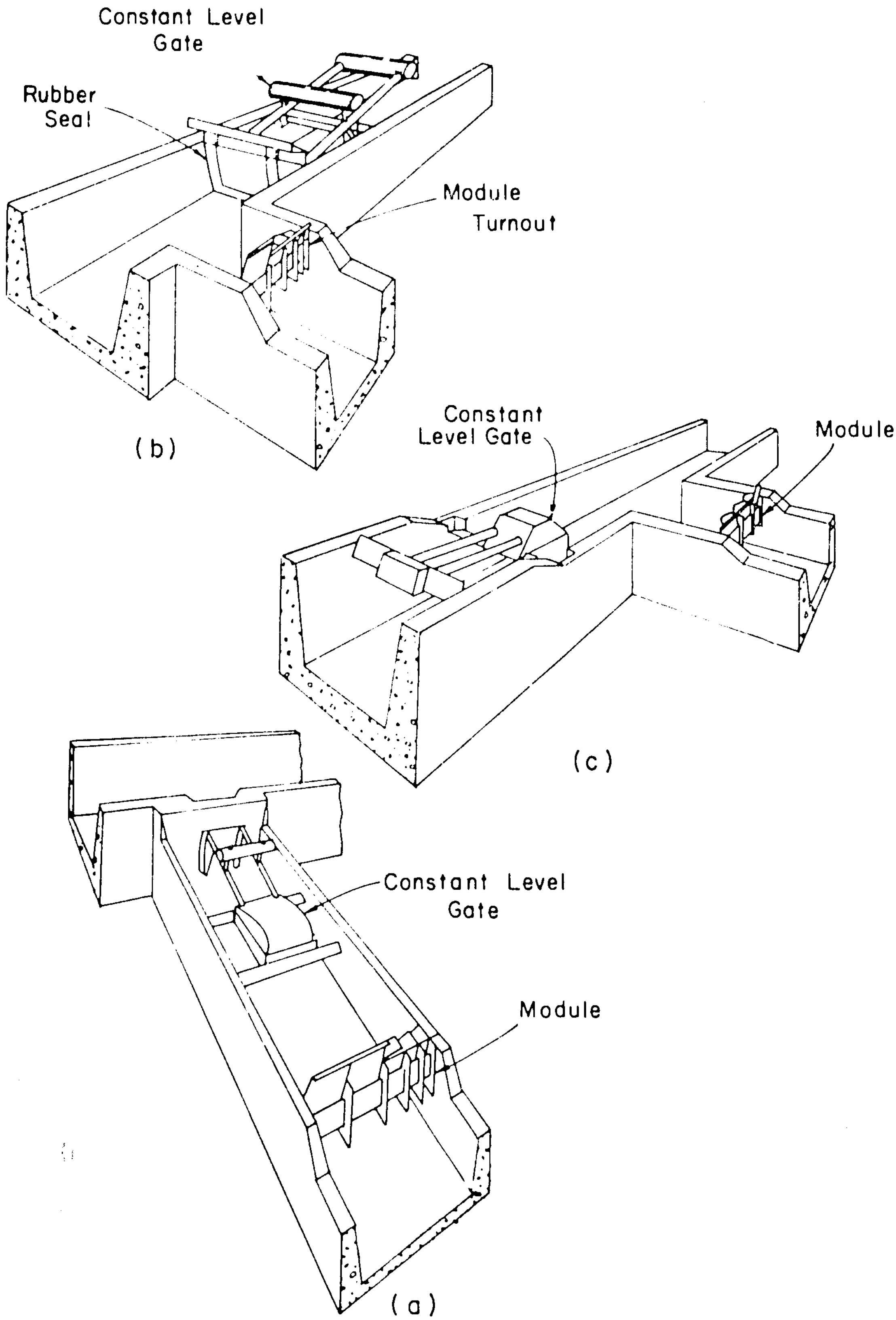


Figure 47. Neyrpic gates and modular turnouts (6,21).

Trapezoidal panel outlets as used in Spain and Pakistan (31) are shown in Figure 48. They are precast from concrete and easily inserted in the ditch by the farmer. Forms are used for constructing the panel and lid so that the two fit together closely for a minimum of leakage. Outlets of variable heights can be constructed. The outlets do not have an adjustment for variable openings and must be operated either fully open or fully closed. This feature is not desirable for many systems.

Circular concrete turnouts have been recently developed and are widely used in Pakistan (Figures 49, 50 and 51) (31). The outlet gates are operated fully open or closed, which is particularly adapted to the system. Irrigation water is rotated to each farmer on Pakistan watercourses on a weekly basis. Consequently, outlets from tertiary to quaternary ditches are used frequently. The water is seldom divided, so gates do not need to regulate flows--only to direct it. Field sizes are small, and thus the conveyance system is extensive, requiring numerous outlets along the tertiary channels. The turnouts are generally used in pairs--one to check the flow in the ditch and the other to divert the flow. The circular concrete turnouts also serve as drops when opened in series along a tertiary canal.

Because cement, sand and bricks are readily available and local craftsmen are talented in constructing masonry structures, the outlet structures are particularly adapted to fit local needs. The circular panel is precast using steel molds constructed locally. The lid is cast in the panel and then ground smooth by turning the lid in its panel to assure a tight, relatively leakproof fit. Most of the supporting structures are brick masonry and are constructed in place as shown in Figure 50. Another turnout design using precast concrete sections is shown in Figure 51. The head losses for a range of discharges and sizes of circular panel turnouts are given in Figure 52 (31).

To install, the soil placed around the structures needs to be carefully compacted to prevent washouts. Soils should be moist when placed and compacted in layers. A hand tamper should be used. For some installations a cutoff wall as shown in Figure 50 may be needed, but for cohesive soils, the wall is probably not needed. However, since water leakage around structures is a major problem, care should be exercised to prevent leaks and possible loss of the structure.

Spiles are used to release irrigation water from the head ditch into the field for furrow or corrugation irrigation. Spiles are usually short pieces of pipe inserted through the ditch bank. Normally, the spiles are installed each season although they can be left in place for longer periods. Figure 53 gives the general plan for using spiles and Table 4 gives the discharge for sizes ranging from 1.0 cm (0.5 in.) to 10 cm (4.0 in.) in diameter.

Siphon tubes are widely used for furrow and flood irrigation from lined and unlined head ditches, as shown in Figure 54. Aluminum and plastic tubes are quite common and are set manually over the ditch bank with each irrigation. Very large siphons can be used for turnouts as shown in Figure 55. The flow is started using a hand pump. The large siphons are useful for irrigating large, level borders. The discharge for individual siphon tubes is given in Figure 56 and Table 4. Head is the difference in elevation of the water level in the ditch and the discharge from the pipe if flowing free, or the water level in the ditch and water level in the furrow or field if the end is submerged.

For conventional siphon tubes, as shown in Figure 54, there is need to reset or reprime after an interruption of flow in the supply canal. Siphon tubes are available that hold their "prime" after flow interruption, so that flow continues after the supply canal is refilled. However, these tubes are quite expensive compared to the conventional ones.

Be sure to set the di arge end of spiles and siphon tubes as low as possible to minimize erosion. If there is erosion, cloth, sacking, plastic sheeting or vegetative material should be place under the discharge.

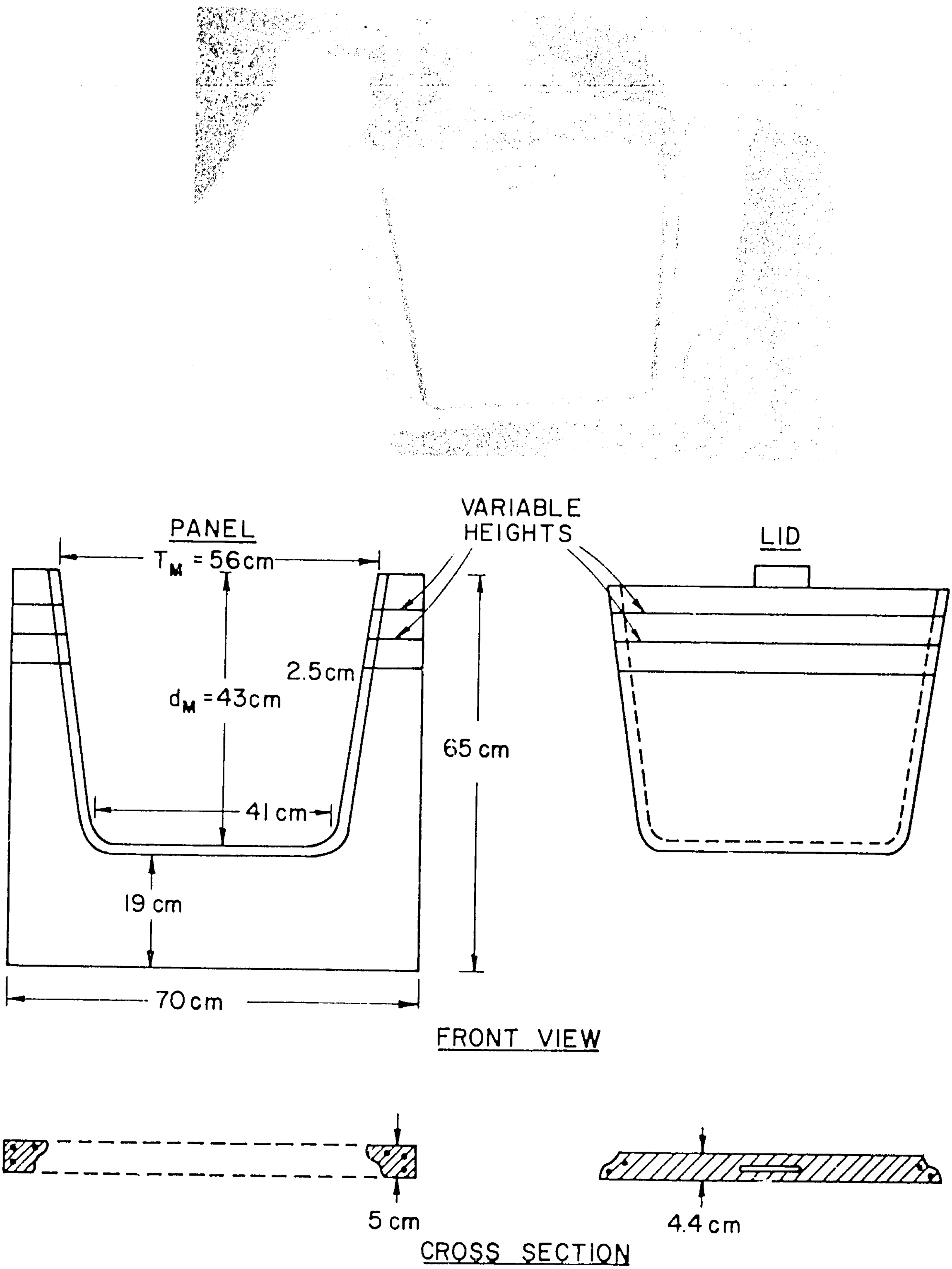


Figure 48. Trapezoidal panel outlet (31).

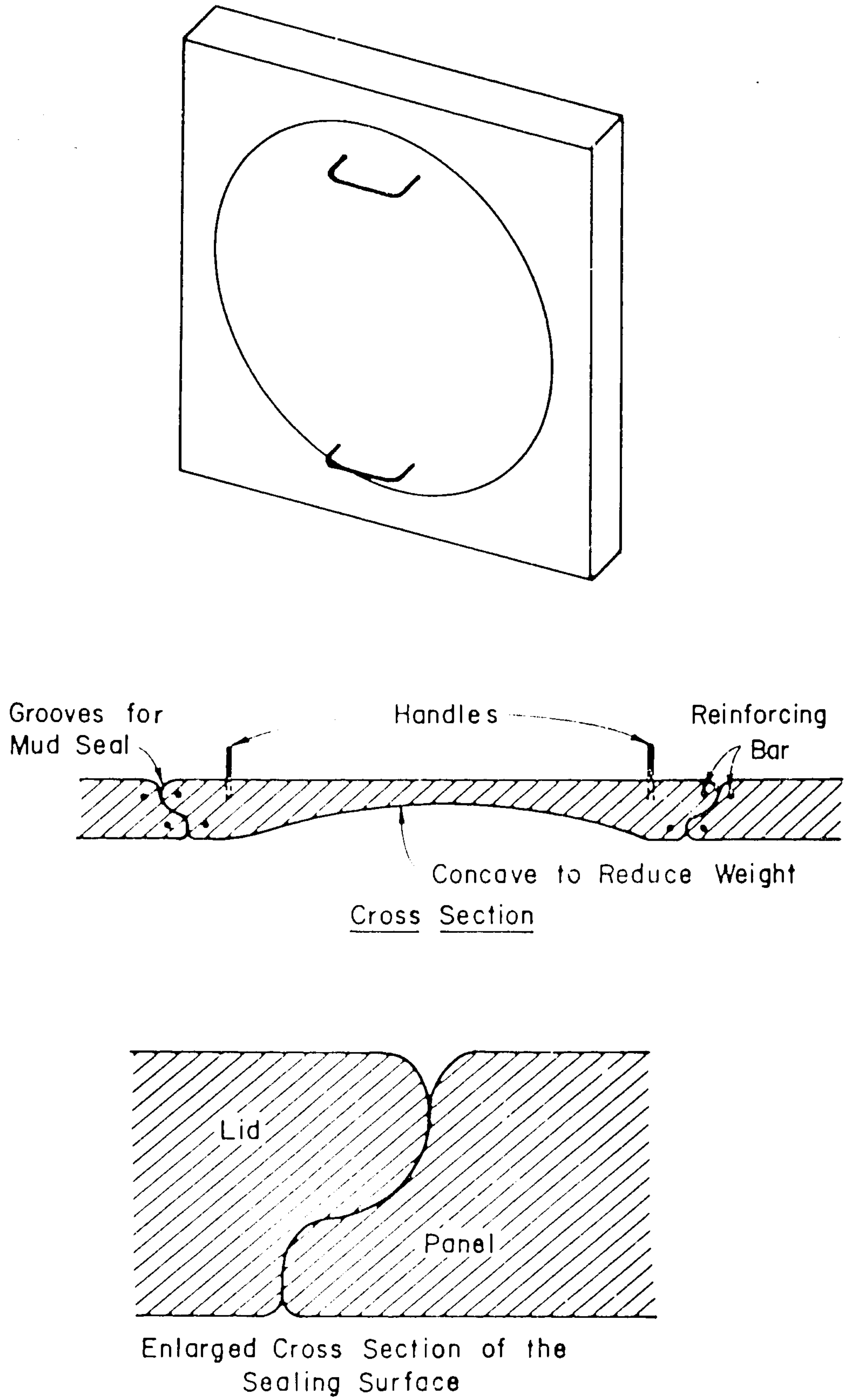


Figure 49. Concrete orifice panel outlet (31).

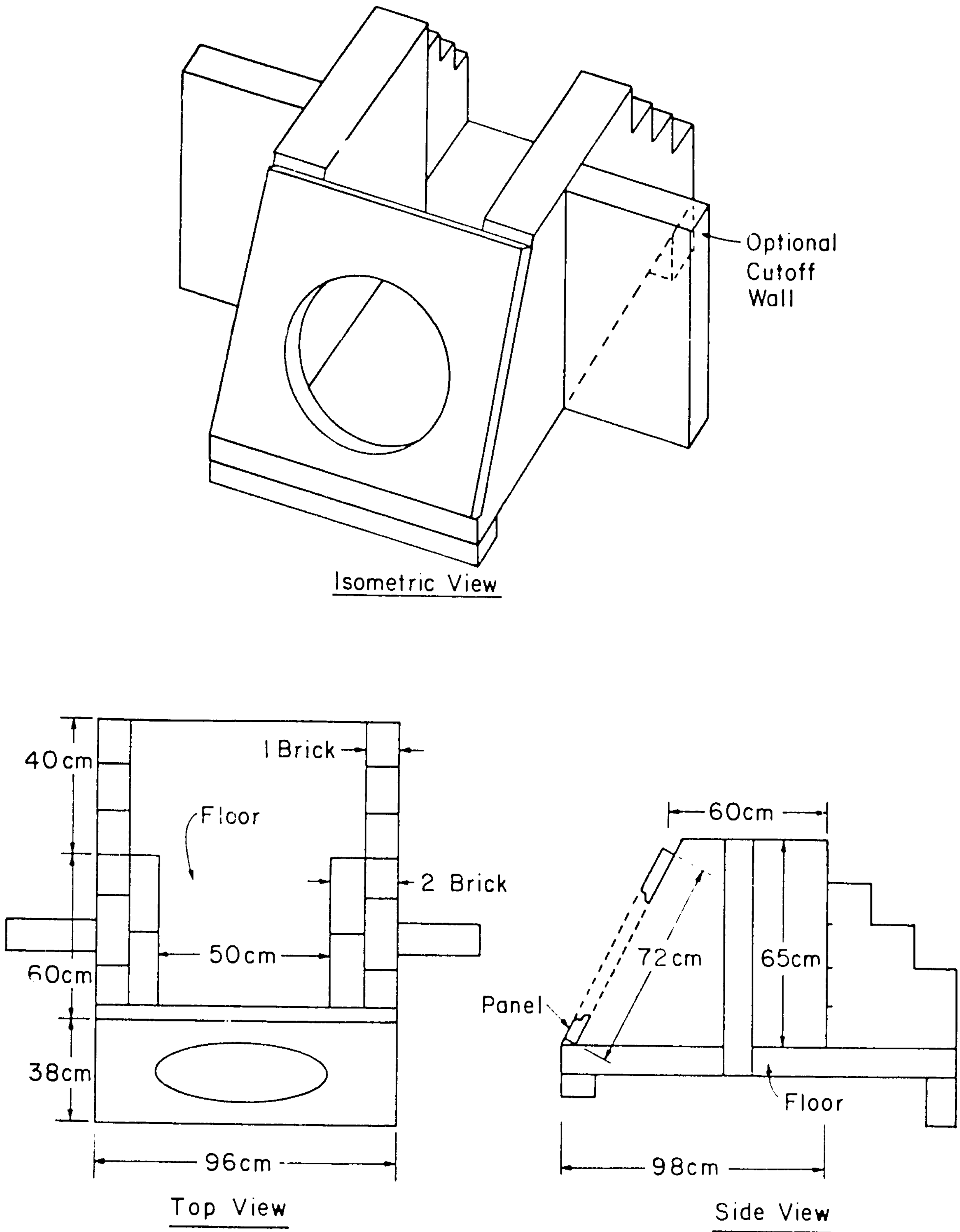


Figure 50. Brick masonry installation for panel outlet (31).

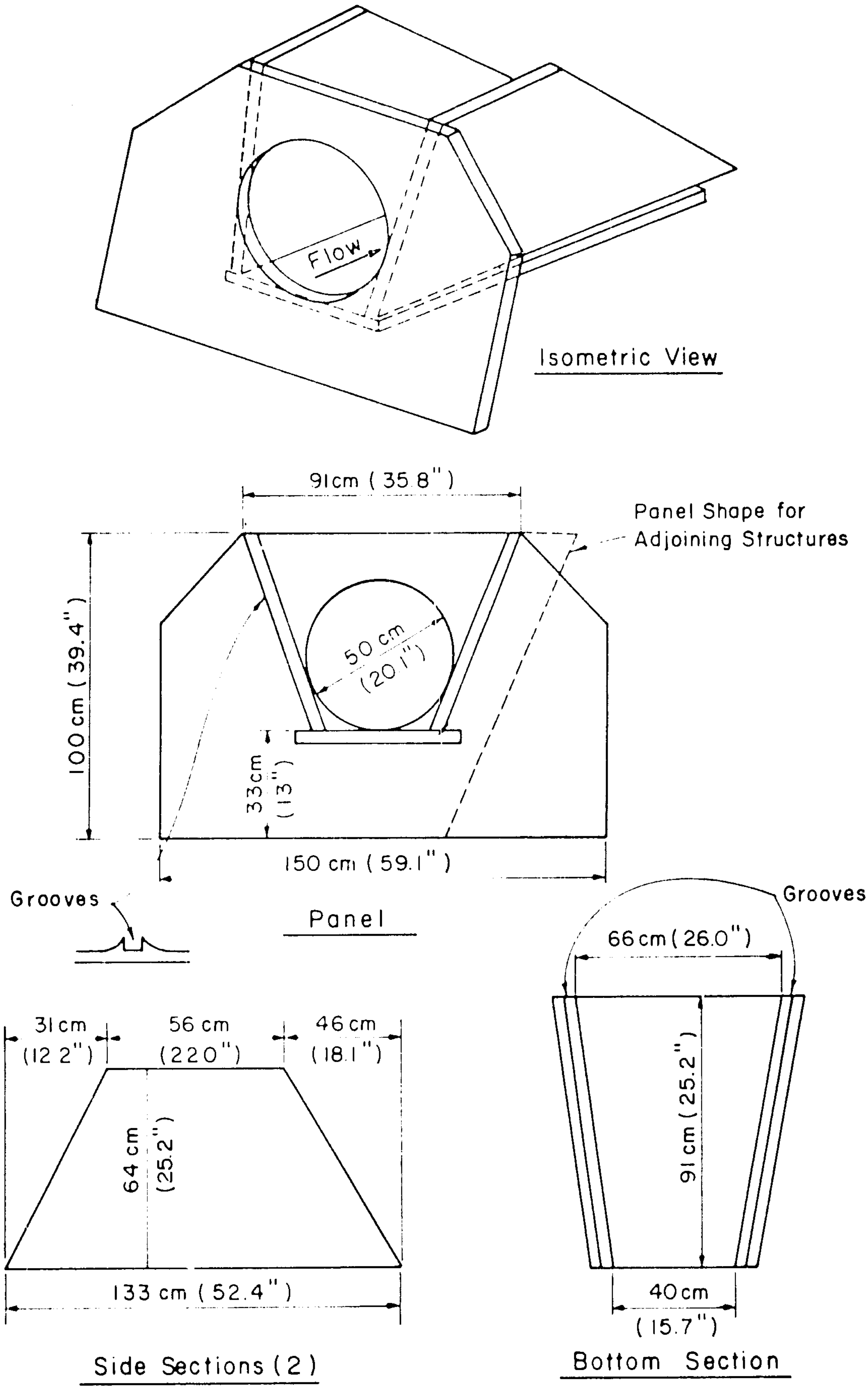


Figure 51. Precast concrete slab installation for panel outlet. (31).

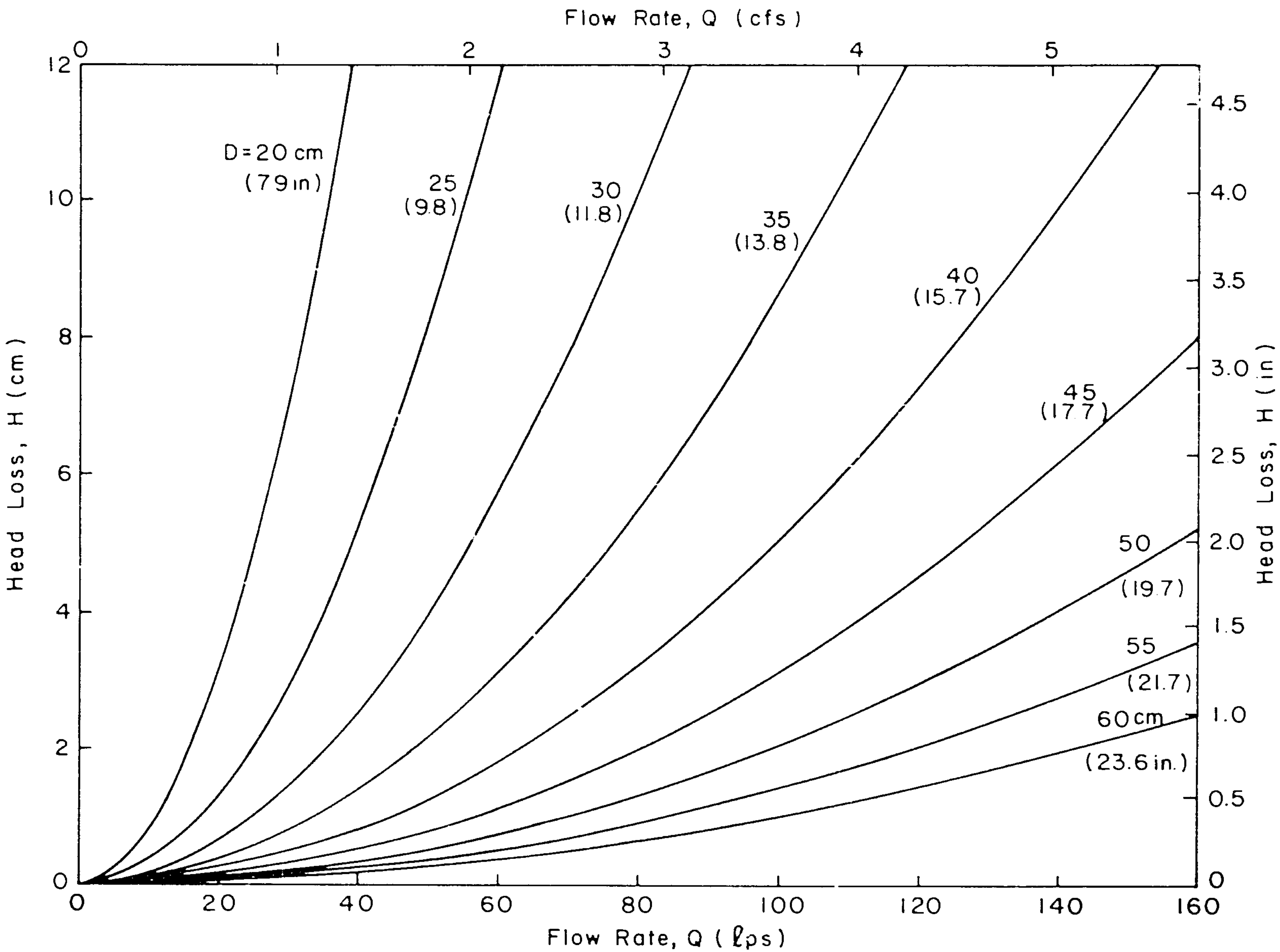


Figure 52. Head loss through circular concrete turnouts of various diameters, *D*, flowing full, assuming a submerged orifice coefficient of 0.8 (31).

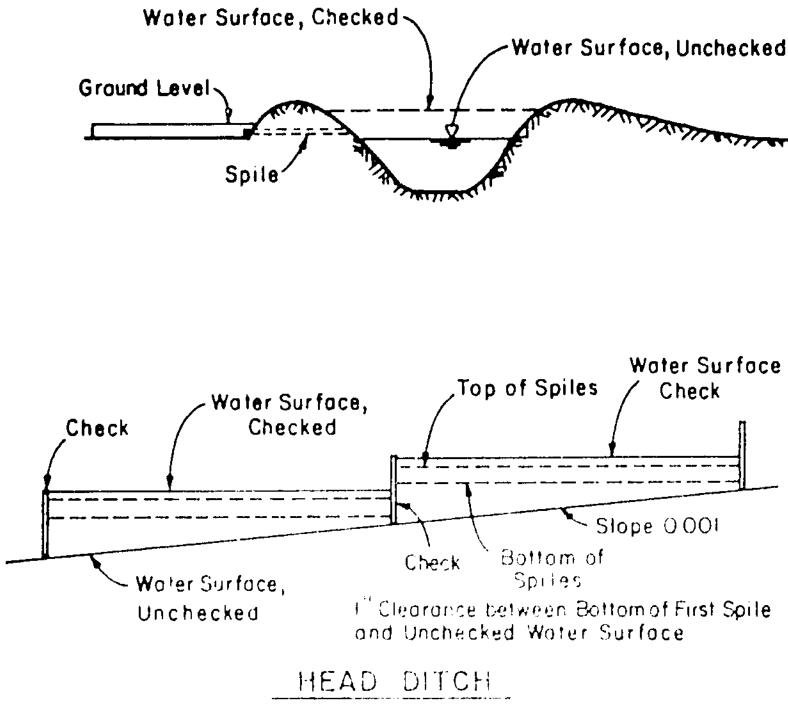


Figure 53. Spiles used for furrow or corrugation irrigation (34).

Figure 54. Siphon tubes for furrow irrigation.

Figure 55. Large siphon with priming pump for turnout (SCS Photo).

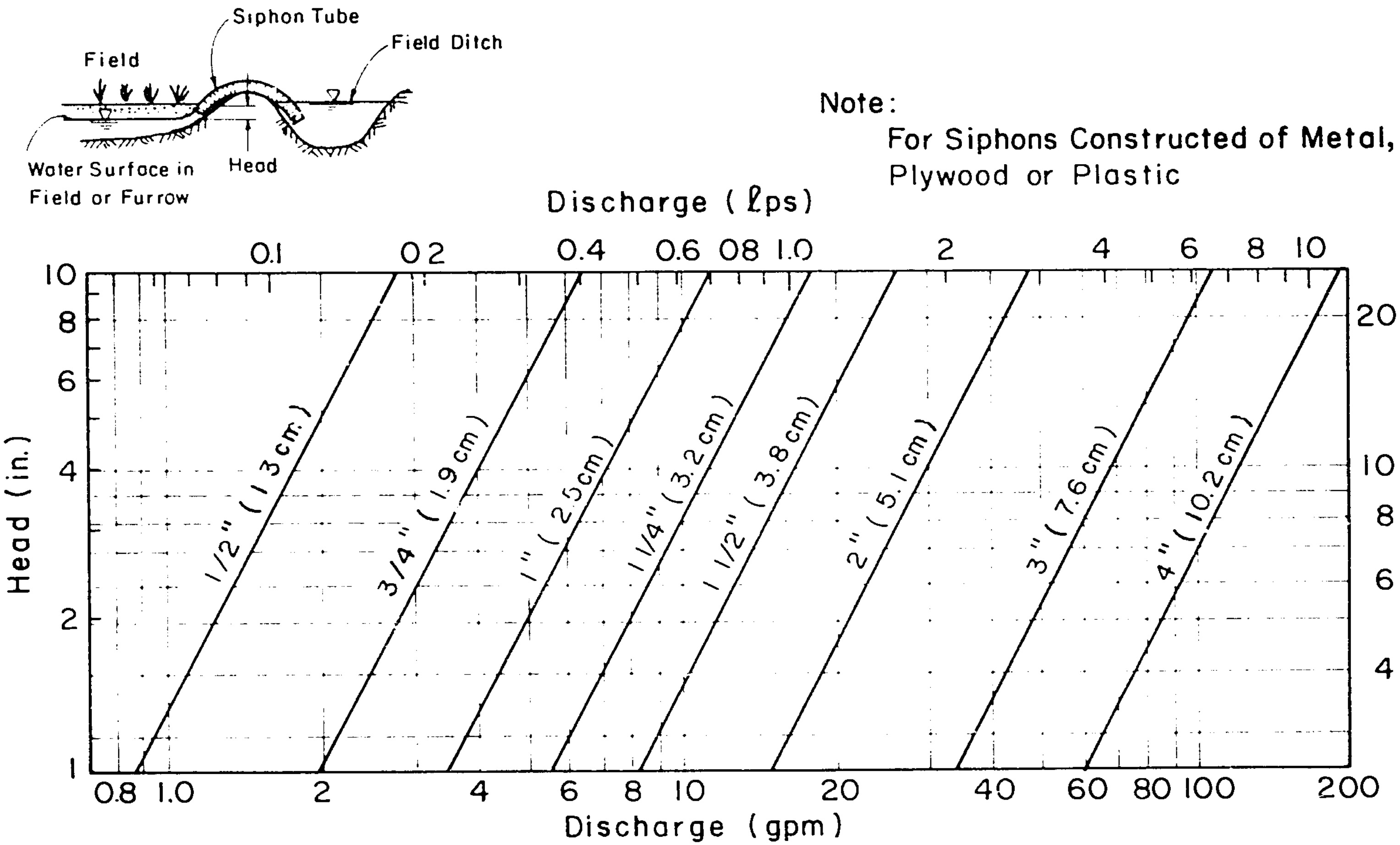


Figure 56. Discharge of siphon tubes (34).

Table 4. Flow through small spiles and siphons (5).

Diameter of Spile or Siphon	Pressure head (centimeters)							
	2.5	5	7.5	10	12.5	15	17.5	20
cm.	Liters per second							
1	0.03	0.05	0.06	0.07	0.07	0.08	0.09	0.09
2	0.13	0.19	0.23	0.26	0.30	0.32	0.35	0.73
3	0.30	0.42	0.51	0.59	0.66	0.73	0.79	0.84
4	0.53	0.75	0.91	1.06	1.18	1.29	1.40	1.49
5	0.83	1.17	1.43	1.65	1.85	2.02	2.18	2.33
6	1.19	1.68	2.06	2.38	2.66	2.91	3.14	3.36
7	1.62	2.29	2.80	3.24	3.62	3.96	4.28	4.58
8	2.11	2.99	3.66	4.23	4.72	5.18	5.59	5.98
9	2.67	3.78	4.63	5.35	5.98	6.55	7.07	7.56
10	3.30	4.67	5.72	6.60	7.38	8.09	8.73	9.34

Bank or ditch *cuts* (notches) are the simplest method for irrigating from a head ditch. Figure 57 is a concrete ditch with notches and small gates spaced frequently along the ditch. The least desirable method is cuts made in the earth canal bank with a shovel. This method is probably the least costly but results in variable flow rates and the channel bank cuts erode. Also, the refilled cuts frequently leak or wash out.

To avoid numerous cuts in the canal bank in furrow irrigation, use small, temporary *set ditches*. These are small ditches running parallel to the quaternary canals. The water is diverted from the quaternary canal into the set ditch by one bank cut, or preferably, by some type of turnout. The water is then distributed by cuts in the set ditches to 6-12 furrows, depending on conditions. This method is better than having a cut in the quaternary canals for each furrow, although siphon tubes are the most desirable method.

3. Water Measuring Structures

On-farm water measurement has generally been ignored in many areas of the world, but is very important for good irrigation water management and use. Just as it is important for the farmer to know how much seed he plants, how much fertilizer he applies, and how much crop he harvests, he should know how much water was applied to each crop, each field, over the entire season. Water measurements can be made at the field and farm level by several means (4, 6, 7, 21, 25, 27, 35). A summary of the characteristics and limitations of several methods is given by Boz (6).

a. Weirs.

One method of measurement is to use an existing drop, check or turnout structure as a rectangular weir. Structures like Figures 24, 25, 26, 32, 34, 35 and 36 can be adapted for measurement by determining the width of the opening and mounting a staff gage upstream to obtain the depth of flow over the base of the opening. For an accurate measurement, the downstream water surface must be below the base of the opening and the nappe of the jet must be aerated underneath. The rectangular weir equation is

$$Q = CLh^{3/2} \qquad (2)$$

where Q = discharge - m^3/s (cfs)
C = coefficient = 1.83 (metric)
3.33 (English)
L = width of opening, m (ft)
and
h = head over the weir, m (ft).

Actually, Equation 2 is for a sharp edge weir with a deep pool upstream, but using it as suggested will not result in a significant error in measurement provided that *h* and *L* are determined with care.

The *standard rectangular weir* is easy to construct and simple to use. It can be made from concrete, masonry, steel or wood but should have a metal blade (4, 6, 35), Figure 58. In the U.S.A., small rectangular weirs are usually made in standard widths of 30.5, 45.7, 61.0 cm (1, 1.5 and 2 ft) for which rating tables are available. A prescribed distance must be maintained between the tip of the weir blade and the bottom of the ditch and between the sides of the opening and the ditch banks. Only the upstream measure of water depth over the weir crest (Figure 58) and a rating table are needed to obtain the discharge. One major drawback is the large head loss requirement to obtain an accurate measurement. The downstream depth must be at some point below the weir blade elevation for correct measurement.

The *90-degree V-notch weir* (Figure 59) has some advantages in irrigation water measurement (35). The water depth over the bottom of the V is determined as with the rectangular weir and the installation requirements are similar. The V-notch has the advantage of being able to accurately measure a large range of flows, particularly low flows. However, the head requirement is even greater than the rectangular weir, which can be a distinct disadvantage.

b. Flumes.

There are several measuring flumes that should be considered when measuring irrigation flows less than 142 l/s (5 cfs). The *Parshall measuring flume* has received wide acceptance and is used in many areas (Figure 60) (6, 21, 35). The 15.2 and 22.9 cm (6 and 9 in.) flumes have the desired flow range and rating tables have been prepared for them. The Parshall flume, as with other flumes, can be used while submerged, i.e., when the depth of flow downstream relative to that upstream is greater than approximately 70 percent. This means that the loss of hydraulic head can be considerably less than when using weirs. This is a distinct advantage. Requirements, rating tables and curves for discharge with various degrees of submergence are available (35). The Parshall flume can be constructed as a permanent installation with concrete, masonry, metal and wood, or as a portable device using metal, wood or fiberglass.

The *Cutthroat measuring flume* is an acceptable flume for irrigation uses. It can be constructed from the above materials and can be permanent or portable (27). The construction is much simpler than the Parshall flume since there is a flat bottom throughout and the parallel-walled throat section is eliminated (Figure 61). The flume will operate under free-flow or submerged conditions, and there are tables and charts available for determining the flow. The 4 in. by 3 ft and 8 in. by 3 ft flumes (27) cover the desired range of discharges for small gravity irrigation systems.

The *trapezoidal flume* is used to measure flow in small systems (26). This flume, which has sloping sidewalls, was initally designed to be an integral part of a concrete-lined irrigation ditch (Figure 62). Flume E-1 (26) has the desired flow range up to 5 cfs (142 l/s). It has a flat bottom throughout and 1:1 sloping sidewalls. The flume can be constructed with concrete, masonry, metal, wood and fiberglass. The advantages are: 1) the shape fits the common ditch shape, 2) there is less head loss, and 3) it accommodates a large range of flows. The disadvantages are: 1) it is more difficult to construct, and 2) the cost is greater than other flumes.

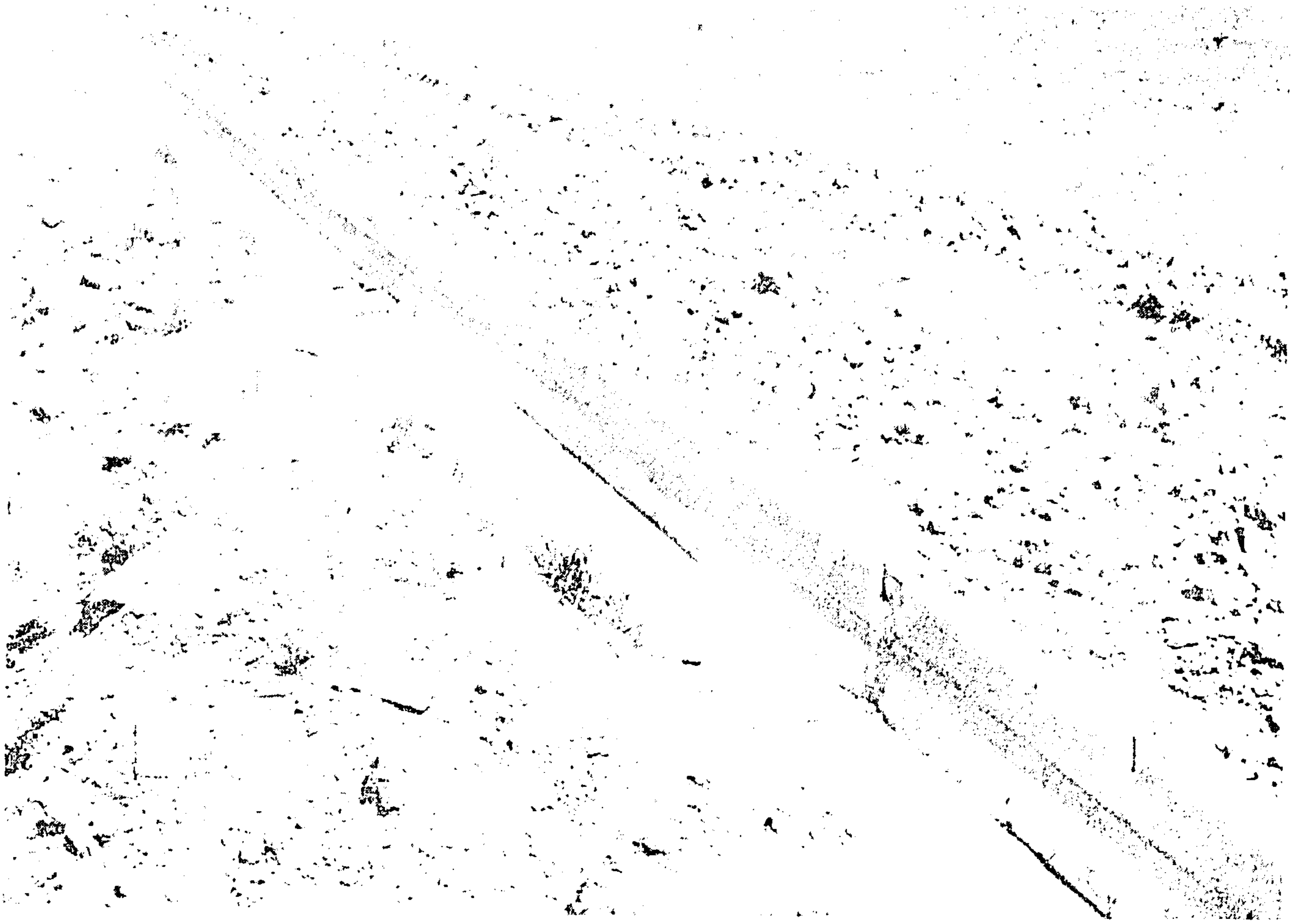


Figure 57. Concrete lined ditch with bank cuts for irrigating.

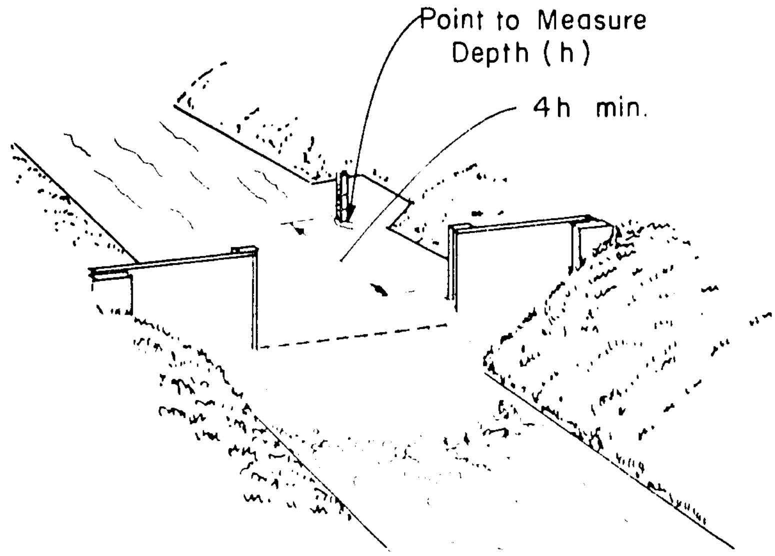


Figure 58. Rectangular weir used as a combination measuring device and drop structure (35).

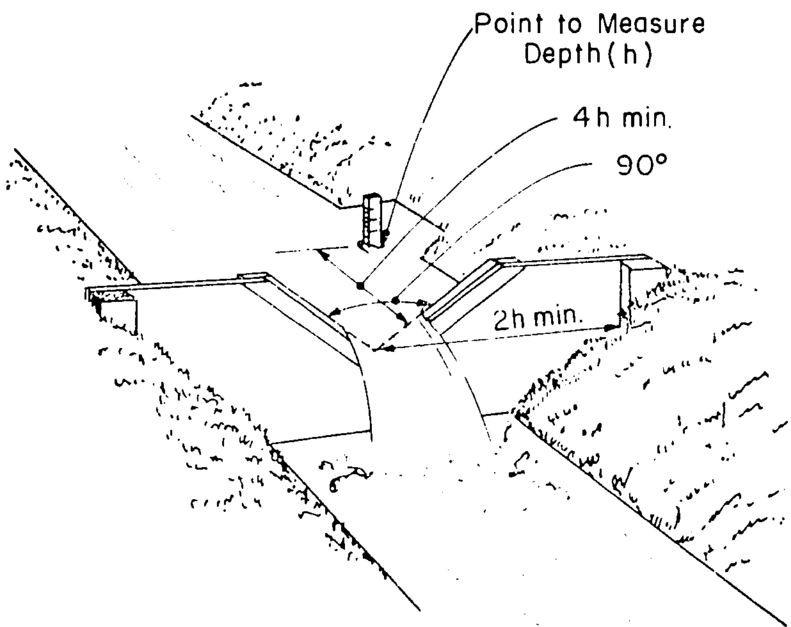


Figure 59. Ninety-degree V-notch weir (35).

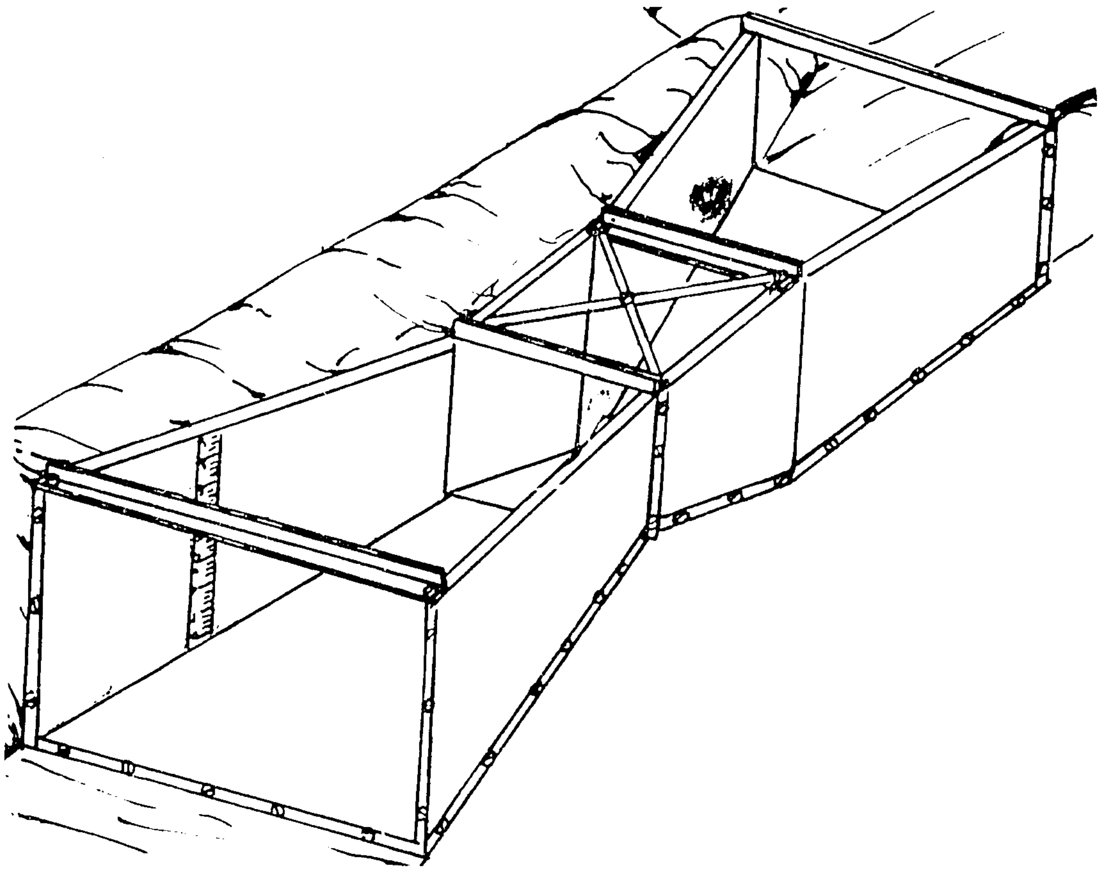


Figure 60. Parshall measuring flumes. (35).

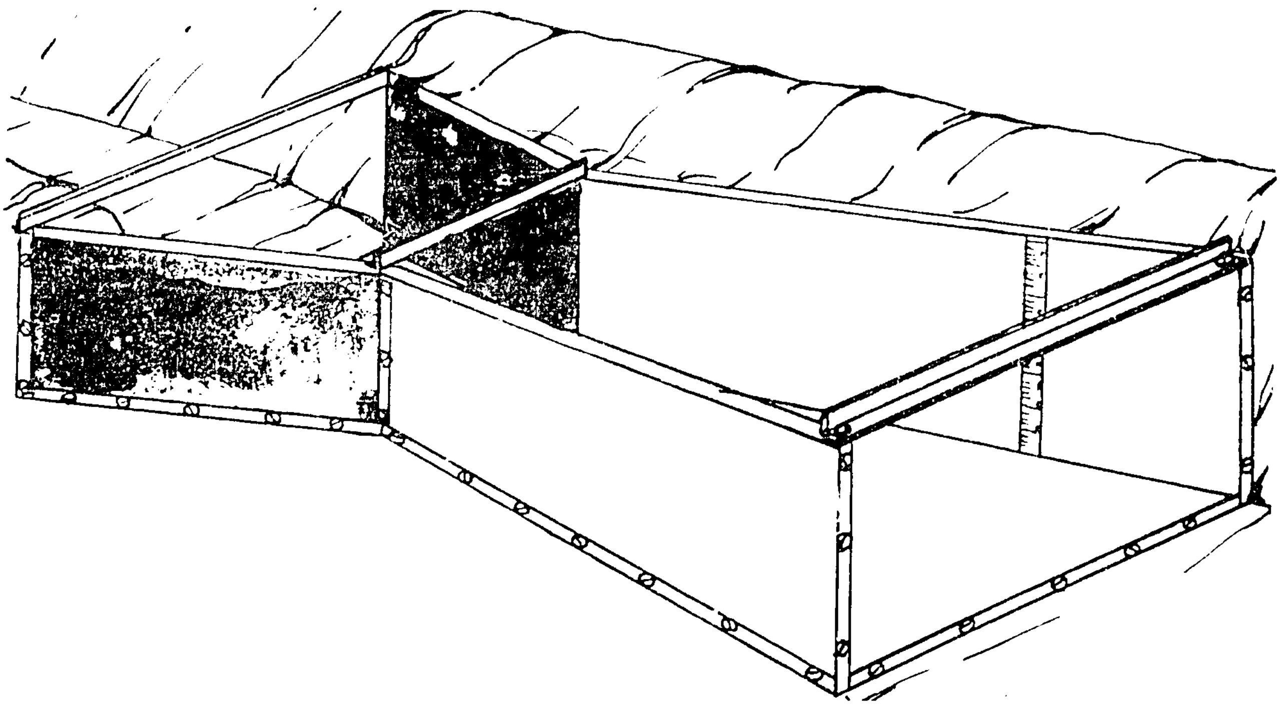


Figure 61. Cutthroat measuring flume (27).

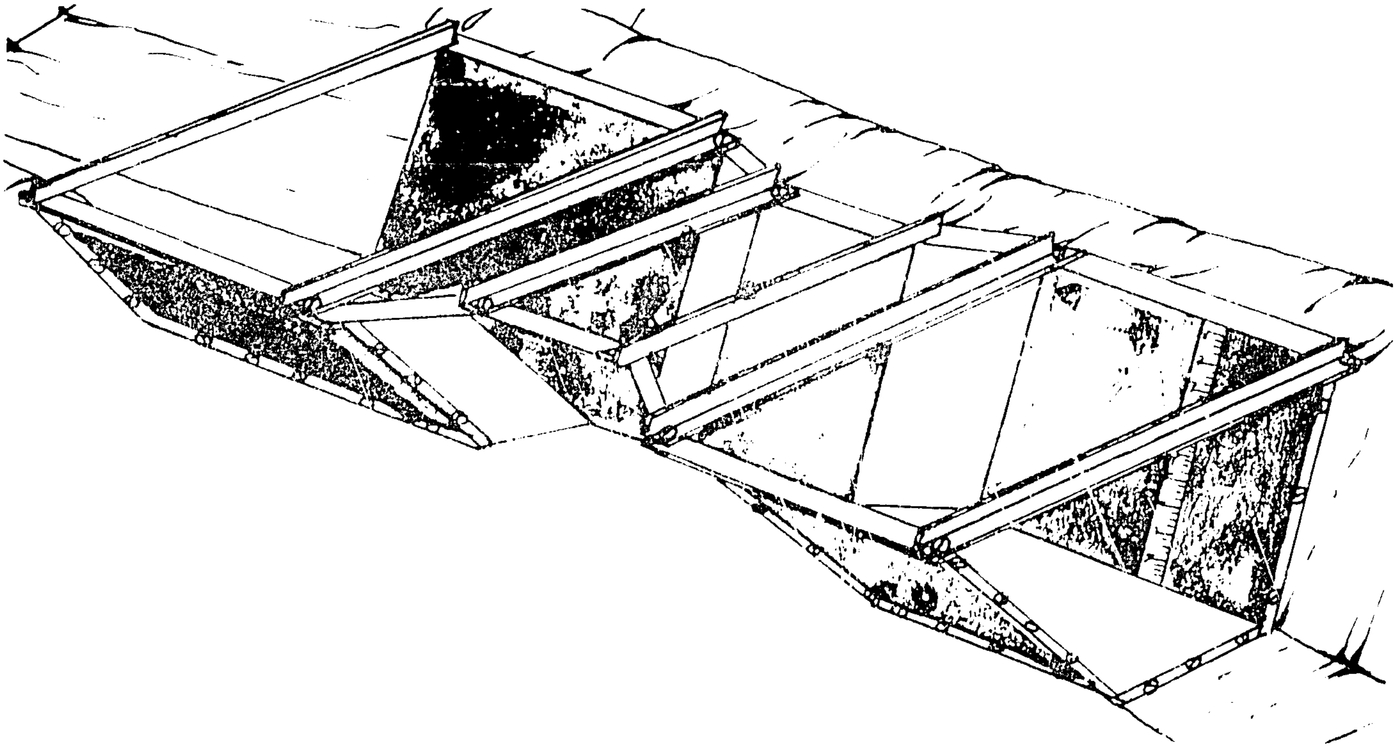


Figure 62. Trapezoidal measuring flume (26).

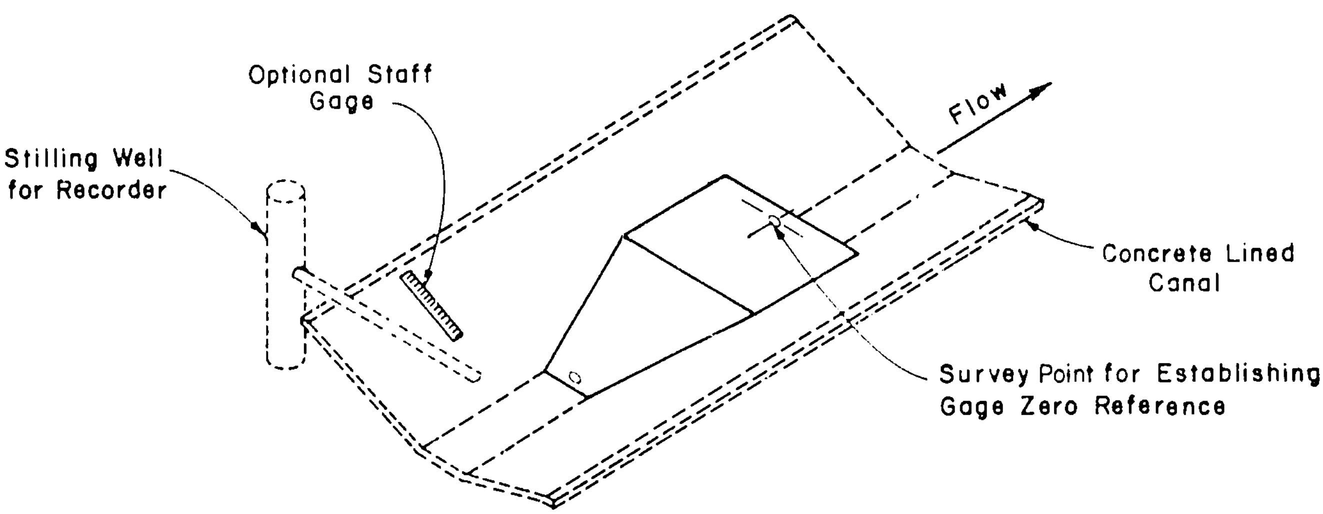


Figure 63. Broad crested weir (b-c-w) measuring flume (7).

A recent development has been the *broad crested weir flume* (b-c-w) (7), which is particularly adapted for placement in an existing lined ditch. Details of the flume are shown in Figure 63. The construction is simply a concrete block with an approach ramp placed with defined incremental heights; the correct height being determined from a design discharge and corresponding normal depth of flow. The design discharge is usually the maximum sustained flow in the channel. Depth of flow through the b-c-w flume is determined at a definitive point upstream using a staff gage or water stage recorder. The advantages are: 1) simple construction, 2) adequate accuracy of measurement, and 3) low head loss at design discharge. The disadvantages may be: 1) high head losses at flows less than design discharge, 2) canal blockage, and 3) sediment deposition.

c. Orifices and Other Devices.

Several of the control structures can be modified to serve as orifice type measuring devices also. The canal gate shown in Figure 41 serves the purpose very well with the addition of wells to measure the water depths upstream and downstream from the gate and an elbow or obstruction on the downstream end so that the pipe flows full. The gate has been calibrated for a range of standard sizes and flows (35). Other orifice type structures such as Figures 13, 33, 34, 39 and 50 can be adapted for measurement by determining the area of the opening and the flow depth on the upstream and downstream sides of the opening. Figure 52 can be used to determine the flow rate for the circular panel turnout (Figures 50 and 51).

For small flows, spiles and siphon tubes (Figures 53 and 54) can be used for flow measurement using Table 4 and Figure 56. An accurate measurement of the pipe inside diameter and the hydraulic head is necessary.

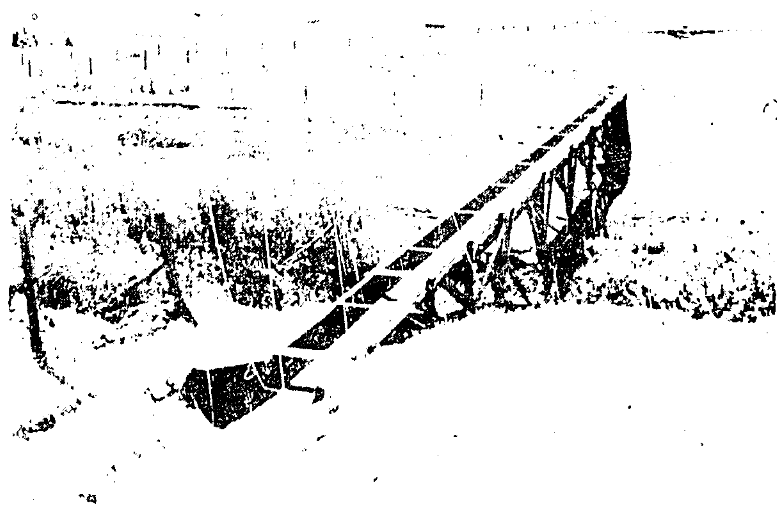


Figure 64. Flume for carrying irrigation water across a depression (SCS Photo).

4. Miscellaneous Structures

a. Culverts, Bridges, Flumes, Crossings, Siphons.

Open irrigation ditches must have crossing structures so that people and equipment can cross. Concrete and metal pipe is most commonly used for culverts over irrigation ditches. Bridges made from concrete, lumber and metal sections are used in most areas. Bridges are sometimes combined with check and drop structures.

Flumes constructed of wood, metal or concrete carry water across depressions. These can be constructed with sub- or super-structure to support a channel or pipe as shown in Figure 64. The structure must have ample strength to support the flume when it is flowing at maximum capacity.

Inverted siphons made from concrete or steel pipe are useful for carrying the flow across depressions, channels and underneath roads. They differ from culverts since the pipe is lower than the water surface in the irrigation ditch. The siphons may be more expensive than flumes to build, but are more durable. Figure 65 shows an inverted siphon made from standard concrete pipe with concrete entrance and exit structures. The earth fill over the pipe should be a minimum of 1.0 m (3 ft) because of vehicular loads. Designs for siphon crossing inlet and exit structures are given in Figure 66. The structures are made from extruded concrete sections (9). Designs of concrete inlet and outlet structures for siphons of different diameters are given on Plate 20, Appendix 2.

b. Drainage Structures and Wasteways.

Irrigation water that leaves the canal, farm, or areas of application must be conducted to a drainage way. This water is normally called waste water and may result from several causes.

In some areas, waste water can come from excess rainfall and leakage and spillage from canals and structures are possible sources. Generally, though, waste water is from over-irrigation. One major source of waste is irrigation water furnished on a 24-hour basis, which is only tended by the farmer during the daylight hours. However, for surface irrigation it is normal for some water to pass into drainage ways or "tail" ditches in order that all areas in the field, including the ends, receive sufficient water.

Some of the same structures used to convey irrigation water are used for wasteways and drainage ways. In particular, drops and chutes are required and pipe crossings are commonly used. Of particular concern is the section where the small open drain enters a larger main drain. Erosion will occur unless some structure is used to safely convey the flow from the higher level to the drain level. Many times a cantilevered pipe outlet is used for this situation. Because of the pipe, large scour holes and excess bank erosion usually occur. Simple outlet structures and rock riprap can alleviate this situation.

Whenever possible, waste water should be diverted to a ditch or canal at a lower elevation so it can be reused on another field.

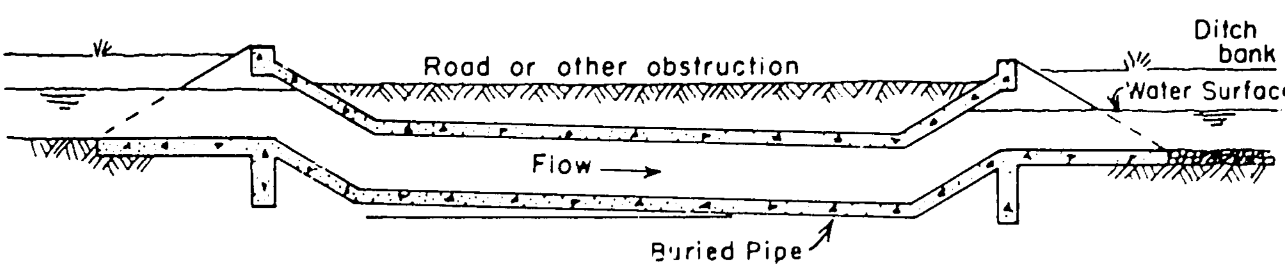


Figure 65. Inverted siphon made from concrete pipe (25).

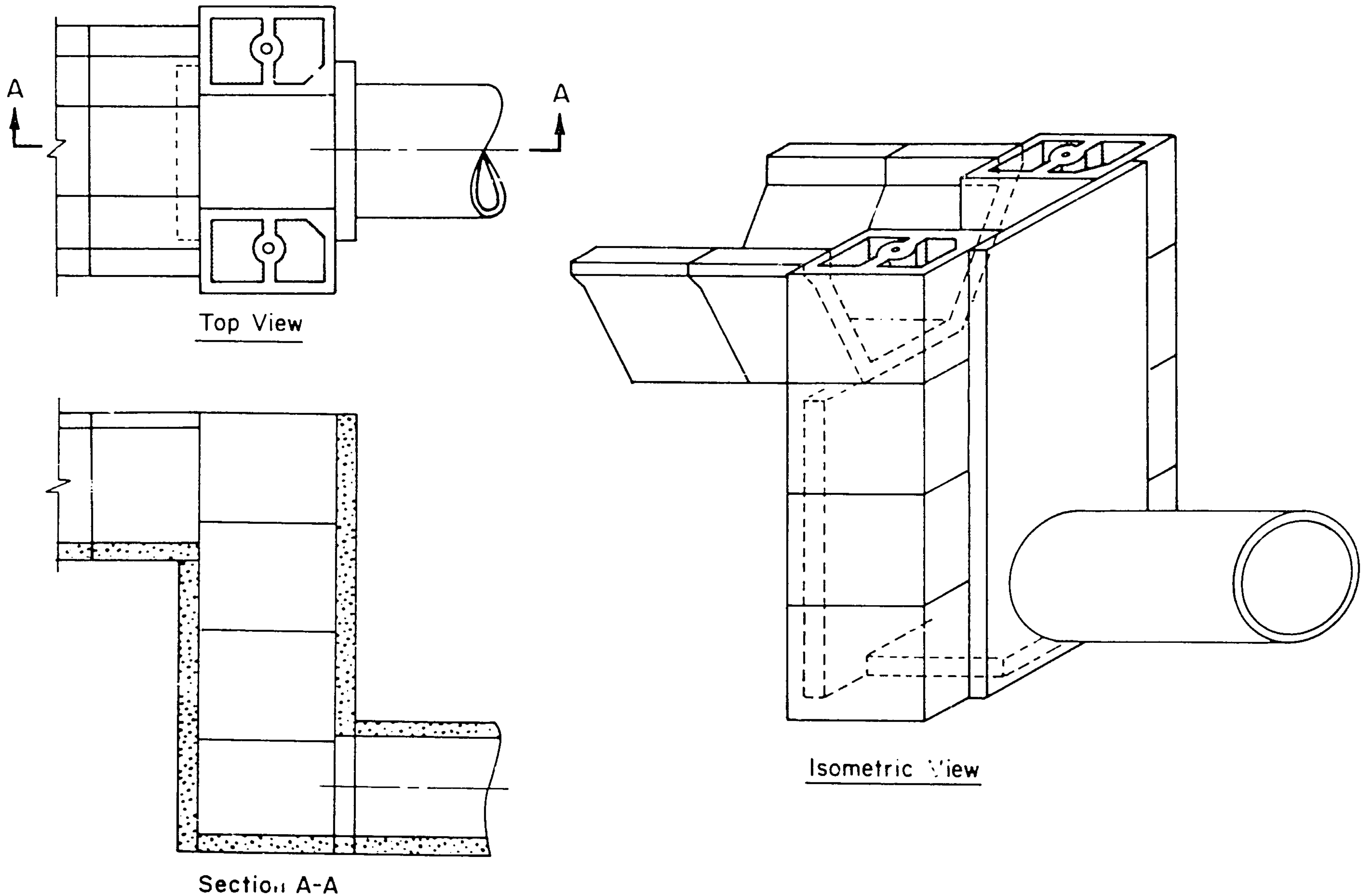


Figure 66. End details for siphon crossing (9).

c. Automated Structures.

Automatic control devices have been developed for underground pipe systems and, to a limited extent, for surface systems (3). Many of the automatic devices utilize pneumatic, radio and electronic controls. In addition to turning water on and off, structures and devices have been developed to automatically reduce the flow after an initial high flow for "cut-back" irrigation. Self-propelled, traveling siphons have been used successfully where a large discharge is needed for border irrigation.

Most of the equipment and devices for automation are quite complicated and require a great amount of skill and attention for operation. There are a limited number of irrigation gates and checks operated by manual timers, hydraulic pressure (Figure 47), sinking floats and water clock principles that may be of limited use in developing countries.

d. Other Structures.

In some cases, sand traps and trash racks (screens) can be used in areas where there is a large amount of sediment and trash (3). These structures are quite specialized and require a great amount of observation and maintenance for successful operation. They are rarely used for individual surface farm irrigation systems. Usually the farmers will manually remove trash and deposited sediment from channels and structures.

Refer to the section on pipe line structures for details on trash screens and desilting boxes that can be adapted to the needs of surface systems.

III. LOW PRESSURE PIPE SYSTEMS

There are three types of on-farm low pressure pipe systems (3). The first is a buried pipe system where water enters by gravity from an irrigation ditch or pumpstand. Water is released directly to the field from risers on the buried pipeline. Buried pipelines with risers are usually made of concrete, asbestos - concrete, or more recently, rigid plastic.

The second pipe system is a combination of buried pipe and portable surface pipe attached to the risers. The surface pipe is usually gated so that water is delivered at many points along the pipe.

The third type is completely portable surface pipe where the water is delivered directly by pump or from an open ditch turnout. The surface pipe is used to deliver water from the open end or from gated sections along the pipe. Surface pipeline is usually rigid aluminum or plastic pipe but can also be flexible plastic and rubber-like materials.

Pipe distribution systems offer many advantages and are finding increasing use in developing countries (3, 4, 20, 24, 25, 33, 36). The advantages are: 1) minimal seepage and evaporation losses, 2) no loss of land to ditches, 3) better weed control through elimination of ditch banks, 4) ease of water distribution on uneven land, 5) reduced maintenance, and 6) good control of irrigation water. The disadvantages are: 1) high initial cost and 2) damage or loss from vandalism.

1. Pipe Design

a. Underground.

Nonreinforced concrete pipe has been used extensively for low pressure systems with asbestos-cement, plastic, and very recently, fiberglass pipe, also used. Most low pressure systems operate at a head less than 500 cm (16 ft). If pressure heads exceed 650 cm (21 ft), reinforced concrete, steel, asbestos-cement, plastic and other pressure pipe is used. Low pressure concrete pipe commonly has mortared tongue and groove joints, although rubber-gasket, flexible-joint pipe is also used. The pipe should be located with care to serve the area and should be positioned away from heavy traffic. Experience has shown that a minimum depth of 0.6 m (2 ft) over the top of the pipe is safe in areas using animal-drawn vehicles and implements and 1.0-1.5 m (3-5 ft) in areas using mechanical cultivation practices and heavy vehicular traffic. Pipe capacity must be large enough for the maximum water requirements. Appendix 3 gives ASAE S261.5 for design and installation of nonreinforced concrete irrigation pipe systems and Appendix 4, ASAE S376 for design, installation and performance of underground thermoplastic irrigation pipelines.

b. Surface.

Surface pipe is used instead of an open ditch and usually has a multitude of small gates (gated pipe) to distribute the flow. The pipe is usually aluminum. Common sizes include 127, 152, 203, and 254 mm (5, 6, 8, and 10 in) diameter in 6-9 m (20-30 ft) lengths. In some areas, plastic pipe (PVC) is used. Quick couplings are available which are relatively leakproof.

c. Pipeline Capacity.

The pipeline diameter which will deliver the desired amount of irrigation water must be determined. The capacity depends on the size and roughness of the pipe, hydraulic losses of the entrance, exit, bends, joints, valves, etc., and the difference in elevation (head) at the entrance and exit.

There are a number of relationships (equations) that have been developed for determining the friction losses (head losses) in pipe. However, for low head, on-farm irrigation systems, the Manning equation adapted for pipe flow is recommended for simplicity. Because, for pipe, the hydraulic radius, R, equals $D/4$, and the area, A, equals $\pi D^2/4$,

$$Q = C D^{8/3} S^{1/2}/n \qquad (3)$$

For Equation 3, C is 0.31 for metric and 0.46 for English units.

Table 5 gives the head loss for concrete pipe with gasket joints (n = 0.011). Head losses for other types of pipe (such as plastic) of similar diameter can be determined by increasing or decreasing the values in Table 5 for the different roughness, n, values. As an example, if the n value is 0.008 (plastic pipe), then the head losses in Table 5 for the same diameter and discharge should be decreased by the factor 0.008/0.011 = 0.73. The actual head loss for pipe friction (H_f) is found by dividing the length of pipe system by 100 and multiplying by the value found from Table 5 (n = 0.011).

The total head loss in a pipeline (H_T) is the sum of the pipe friction losses and other head losses caused by entrances, valves, elbows, tees and structures. These losses are accumulated depending on the friction and head losses, thus

$$H_T = (K_1 + K_2 + K_3 + \ldots K_n)\frac{Q^2}{2A^2 g} + H_f \qquad (4)$$

where g = acceleration of gravity, 9.8 m/sec² (32.2 ft/sec²). Values of the resistance coefficient K for different features are given in Table 6. For the gravity system, the pipe discharge and size is determined by the following steps.

1. Determine H_T as the difference in elevation of the water surface at the entrance and at the point of lowest discharge. The point of discharge for the low head pipe system will probably be one or more alfalfa type valves discharging through hydrants.

2. From a selection of pipe diameter and estimated discharge, determine H_f from Table 5.

3. Determine and sum the resistance coefficients $K_1 \ldots K_n$ for the different pipeline features, such as line valves, elbows, tees, risers and delivery valves.

4. Solve Equation 4 for the discharge Q.

5. If the determined discharge Q is different from the estimated discharge used in step (2), repeat the computations from step (2) using the computed discharge to obtain H_f. The final computation after these trials will give the correct discharge.

To actually design the system, it is necessary to first have a topographic profile of the system showing the ground surface and pipeline elevations. Upon knowing the location of each pipeline feature, such as stands, risers, valves, etc., and the water delivery schedule, the next step is to determine the hydraulic grade line for each possible schedule. The hydraulic gradient at the point of delivery and the valve characteristic determines the discharge there.

The capacity of a gravity pipe system is basically determined by the pipe diameter and the difference in the elevation between the upper water surface and that where the lower end discharges. If the system is supplied by a pump, the head can be increased. The usual practice is to design the underground pipe system so that the hydraulic gradient is 30 cm (1 ft) above the ground surface at the discharging alfalfa valve. This will insure adequate flow without excessive erosion for direct irrigation. If rigid or flexible gated pipe is used with a hydrant, the hydraulic gradient will need to be 61-91 cm (2-3 ft) at the alfalfa valve level.

Table 5. Head loss in concrete pipe with concentric gasket joints* (3).

Flow rate (Q)		Pipe Diameter 203 mm (8in.)	254 mm (10 in.)	305 mm (12 in.)	356 mm (14 in.)	381 mm (15 in.)	406 mm (16 in.)	457 mm (18 in.)	533 mm (21 in.)
L/s	ft^3/s	Head loss (m/100 m or ft/100 ft)							
15.0	0.53	0.14							
20.0	0.71	0.25	0.07						
30.0	1.06	0.55	0.17	0.06					
40.0	1.41	0.98	0.30	0.11					
50.0	1.77	1.54	0.46	0.18	0.08				
60.0	2.12	2.21	0.67	0.25	0.11	0.08			
70.0	2.47	3.01	0.91	0.34	0.15	0.10	0.07		
80.0	2.82	3.93	1.19	0.45	0.20	0.14	0.10	0.05	
90.0	3.1	4.97	1.51	0.57	0.25	0.17	0.12	0.07	
100.0	3.53		1.86	0.70	0.31	0.21	0.15	0.08	
150.0	5.30		4.18	1.58	0.69	0.48	0.34	0.18	0.08

*Computed from Manning Formula, $n = 0.011$

Table 6. Resistance coefficient *K* for use in formula $H = KQ^2/2A^2g$ for fittings and valves (33).

Fitting or valve	150 mm (6 in.)	175 mm (7 in.)	200 mm (8 in.)	250 mm (10 in.)	300 mm (12 in.)
	Nominal diameter				
			Standard pipe		
Elbows:					
Regular flanged 90 degrees	0.28	0.27	0.26	0.25	0.24
Long radius flanged 90 degrees	0.18	0.17	0.15	0.14	0.12
Tees:					
Flanged line flow	0.12	0.11	0.10	0.09	0.08
Flanged branch flow	0.60	0.58	0.56	0.52	0.48
Valves:					
Globe flanged	5.80	5.70	5.60	5.50	5.40
Gate flanged	0.11	0.09	0.075	0.06	0.045
Swing check flanged	2.00	2.00	2.00	2.00	2.00
Foot	0.80	0.80	0.80	0.80	0.80
Alfalfa (20)*			2.04	2.04	2.04
Orchard (20)*			2.78	2.78	2.78

Other

Inlets or entrances:		
Inward projecting	0.78	All diameters
Sharp cornered	0.50	All diameters
Slightly rounded	0.23	All diameters
Bell-mouth	0.04	All diameters

Sudden enlargements $K = \left(1 - \frac{d_1^2}{d_2^2}\right)^2$ where d_1 = diameter of smaller pipe and d_2 = diameter of large pipe

Sudden contractions $K = 0.7\left(1 - \frac{d_1^2}{d_2^2}\right)^2$ where d_1 = diameter of smaller pipe and d_2 = diameter of large pipe

*Where A is the nominal port area, ponding over valve is 15 cm (0.5 ft) and hydraulic gradient is 30 cm (1.0 ft) above valve.

2. Structures

Control structures are needed for the low pressure underground pipe systems to deliver the correct flow at the desired locations (3, 4, 10, 33). Appendix 2 gives design information on structures for the systems. Although concrete pipe is shown, the design can be adapted to other pipe materials.

a. Inlet Structures.

Water may enter a pipeline by gravity from an irrigation ditch or may be pumped from a well or canal. Inlet structures are used to direct the water into the pipe system and also may serve as a debris and sediment trap.

1. Gravity Inlet. When water enters a pipeline from an open ditch, a structure like that shown in Figure 67 is used. The inlet stands are constructed of poured concrete, concrete blocks, brick masonry or large diameter concrete pipe. They are equipped with a trash rack or screen and a removable cover. Plates 21, 22 and 23, Appendix 2 give designs for gravity inlets.

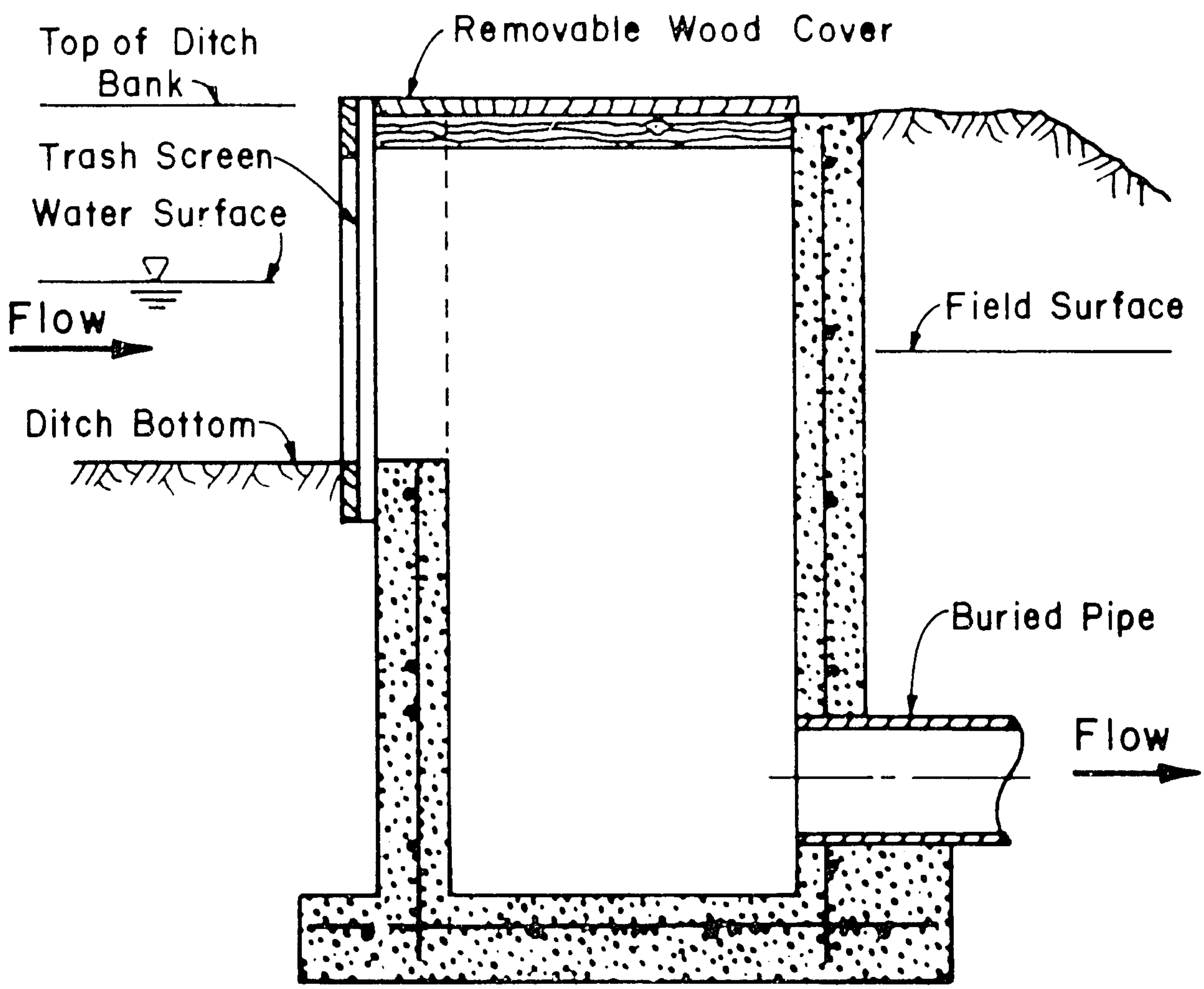


Figure 67. An inlet stand for taking water by gravity from a ditch into an underground pipeline (4, 32).

2. *Pump Stands.* Stands are installed to receive water from a pump and convey it into a pipeline. They are open at the top and the stand diameter is larger than the pipeline, usually 3 to 4 times larger. This allows the stand to act as a surge chamber and allows entrapped air to escape. They are built high enough to develop the head needed and are given some freeboard. A typical stand is shown in Figure 68. If an unusually high stand is needed, the stand is capped and a smaller diameter steel pipe is extended to the necessary height. A flexible coupling is put in the pump line to protect the stand from pump vibrations. Plates 24 and 25, Appendix 2, give designs for pump stands. Plate 26, Appendix 2, shows an inlet stand combined with a sand trap.

b. Pressure and Flow Control Stands.

Control structures are needed to maintain delivery water levels, regulate the flow into branching lines, limit pipe pressures, and provide for the removal of entrained air.

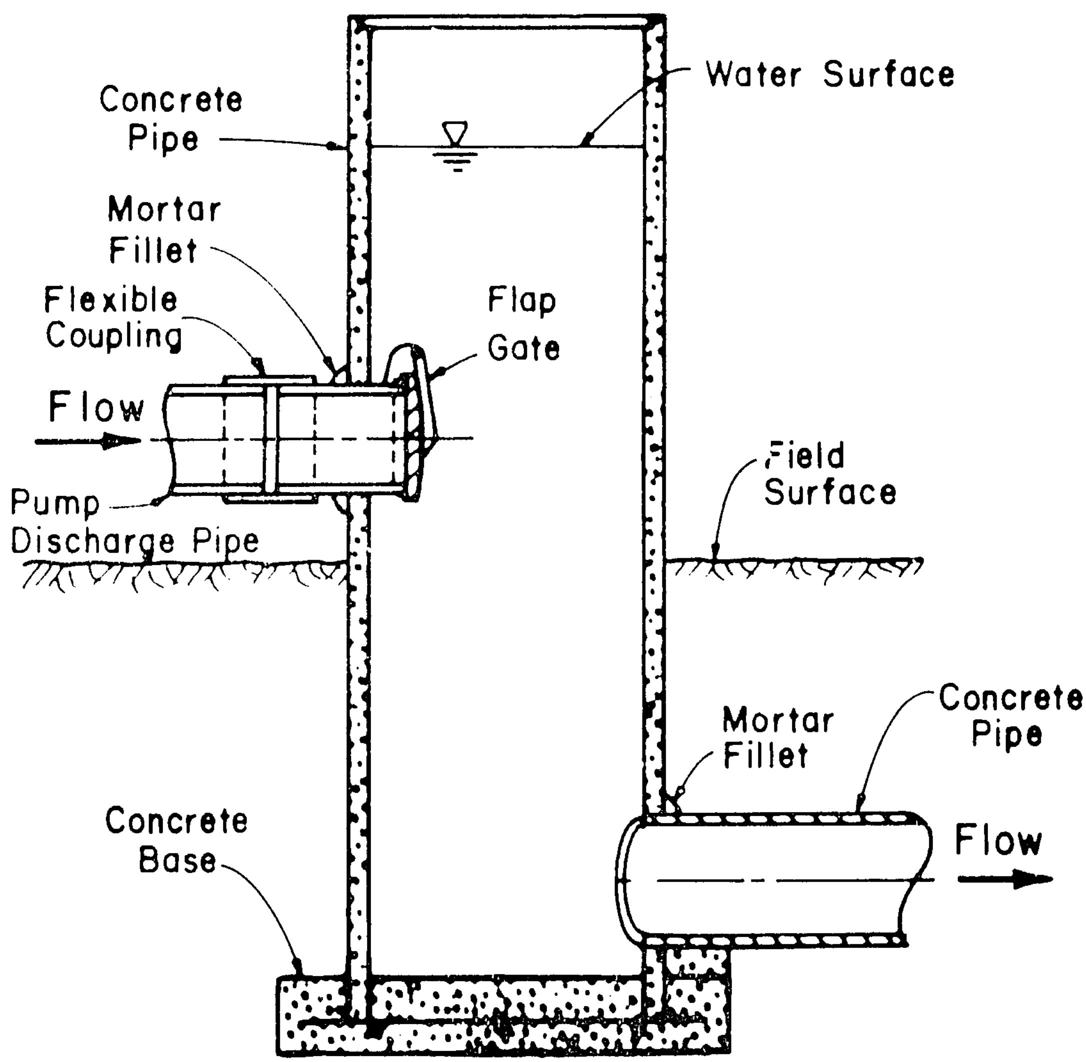


Figure 68. Typical concrete pump stand: The flexible coupling is needed to absorb vibrations from the pump; the flap gate prevents back flow to the pump (4, 32).

1. Gate Stands. Gate stands are diversion structures that control the flow into laterals. They are also used to increase the pressure upstream, to prevent high pressures, and to act as air vents and surge chambers. The gates are often used to control pressures as required by upstream outlets. A single structure is often built to function as a gate stand and as an overflow stand as shown in Figure 69. Plate 27, Appendix 2, gives the design of a gate stand combined with an overflow stand.

2. Overflow Stands. These serve as check and drop structures in addition to the other functions of a stand. As a check, the stand regulates upstream pressures to maintain uniform flow from outlets or into laterals. As a drop, it limits the excess head developed by the natural slope. It may be used with or without the side turnout shown in Figure 69. It has the disadvantage that air is often entrained in the water as it spills from the overflow baffle. To minimize this, a gate which is normally open is installed between the two chambers in the stands. When pressure is needed for upstream diversion, the gate is closed sufficiently to bring the water level to the crest with only a small overflow. An overflow stand usually is not needed in areas of flat or very slight slopes (see Plate 27, Appendix 2).

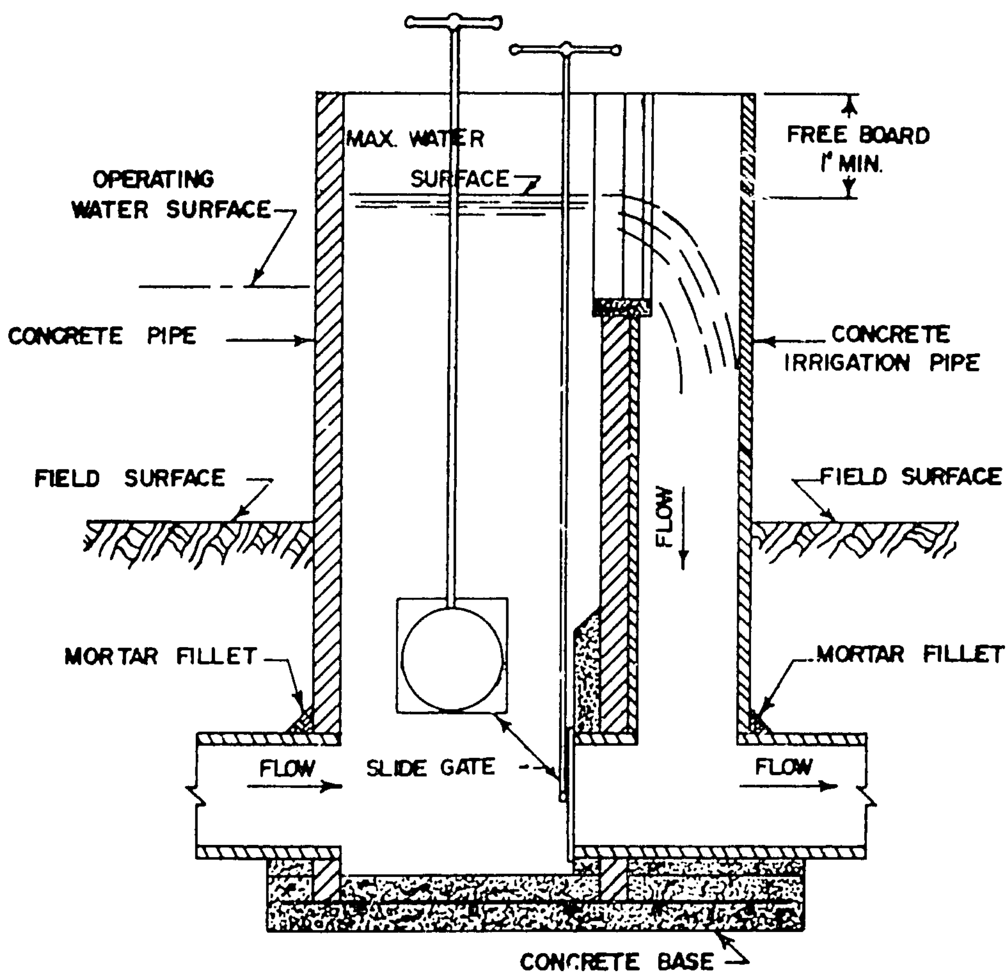


Figure 69. Combination gate and overflow stand used for regulating upstream pressures and diverting water into other pipeline laterals or for direct irrigation (4, 32).

3. *Float Valve Stands.* On steep slopes, it is advantageous to install a semiclosed system with float valve stands as shown in Figure 70. Design of the stand is given on Plate 28, Appendix 2. The float valves open when the downstream pressure falls to a predetermined level, and admit into the line only as much flow as can be released by the hydrants that are open. Thus, each valve automatically controls pressure in the reach of pipe downstream from it. When a pipeline is served directly from storage, float valves provide full control of the water from the lower end of the line. High overflow stands on steep slopes may be eliminated by using float valves. A semiclosed system is efficient, since surplus water is not wasted at the end of the line as is sometimes done when overflow stands are used. Tables giving head loss for various size and types of valves at different openings are useful in selecting the proper valve (24).

4. *Line Gate Valves.* Line gates in each lateral are sometimes substituted for gate stands. These valves are regular gate valves with special hubs that are mortared directly into the line. They permit operation from the ground rather than from the stand top. The present trend is toward increased use of line gate valves with adjacent, small diameter, capped, vent stands instead of large gate stands. Friction losses in wide open gate valves are low and are often expressed as equivalent lengths of straight pipe (24).

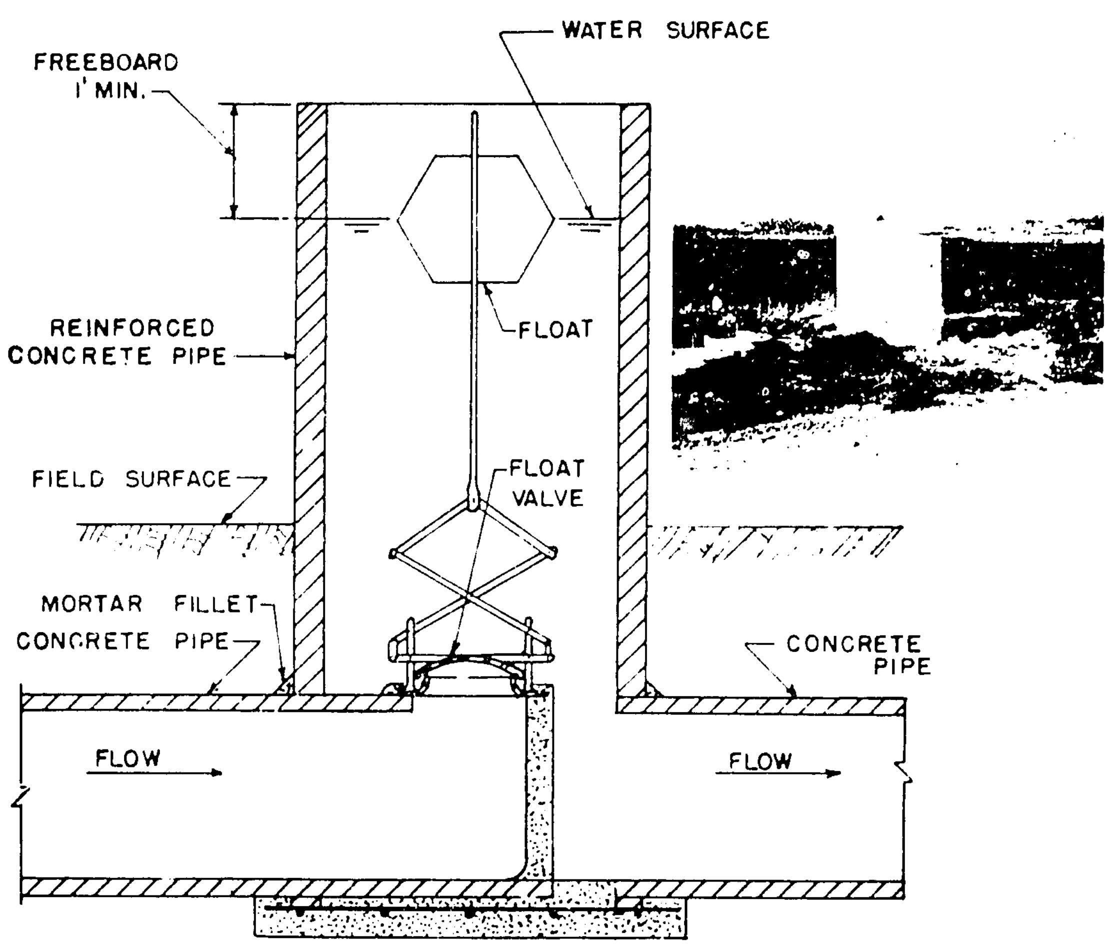


Figure 70. Float valve stand; pressure and flow in the line are automatically regulated by the float valve (4, 32).

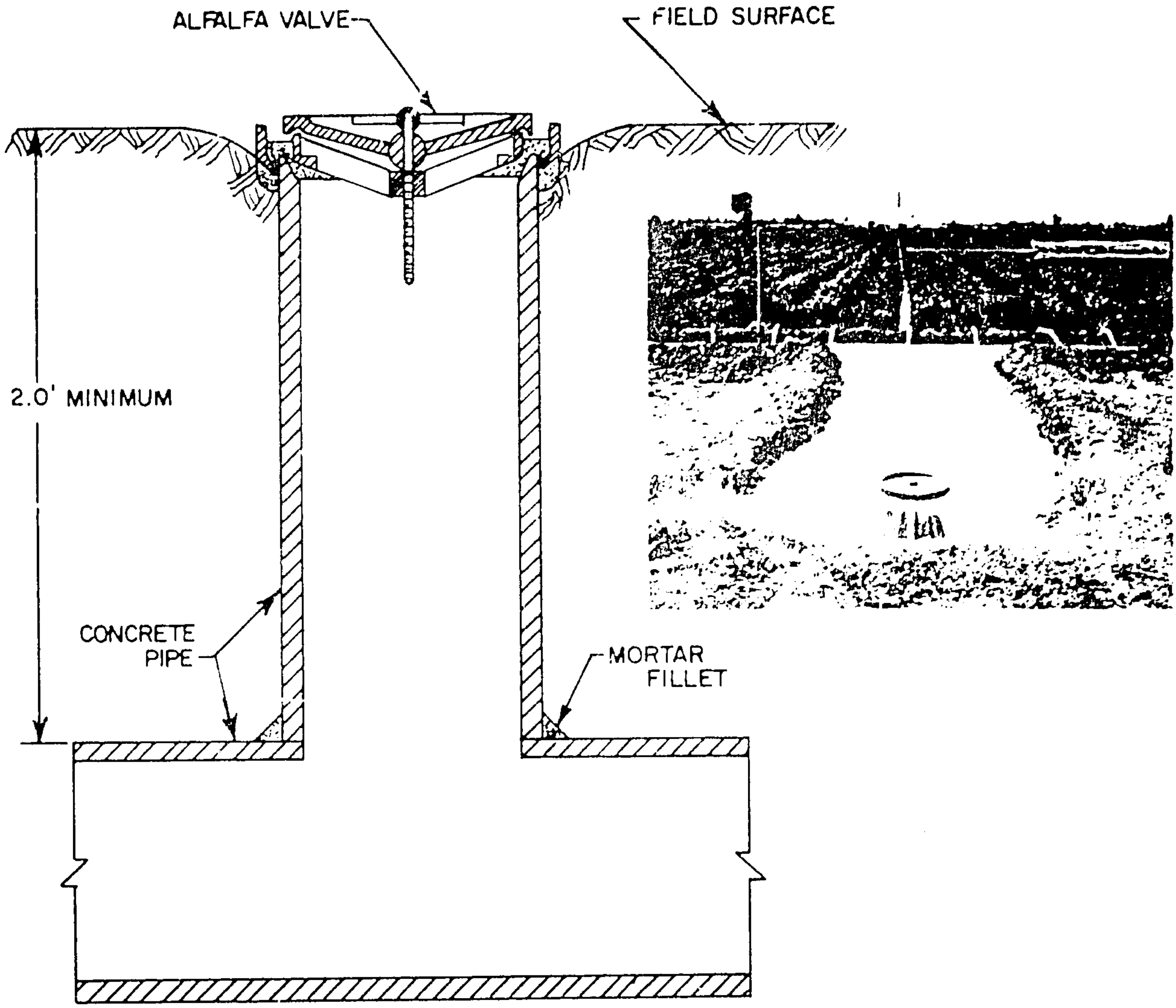


Figure 71. Typical alfalfa valve hydrant; the riser and valve assembly are sometimes cast to a short section of main pipeline to simplify installation (4, 32).

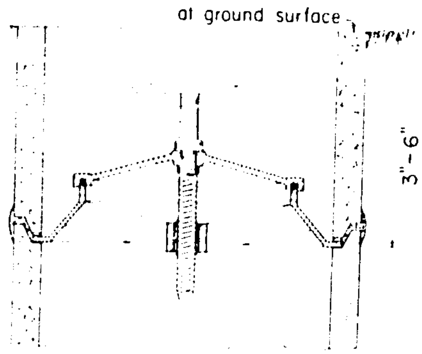


Figure 72. Orchard valve hydrant showing the recommended installation of the valve in the riser (4).

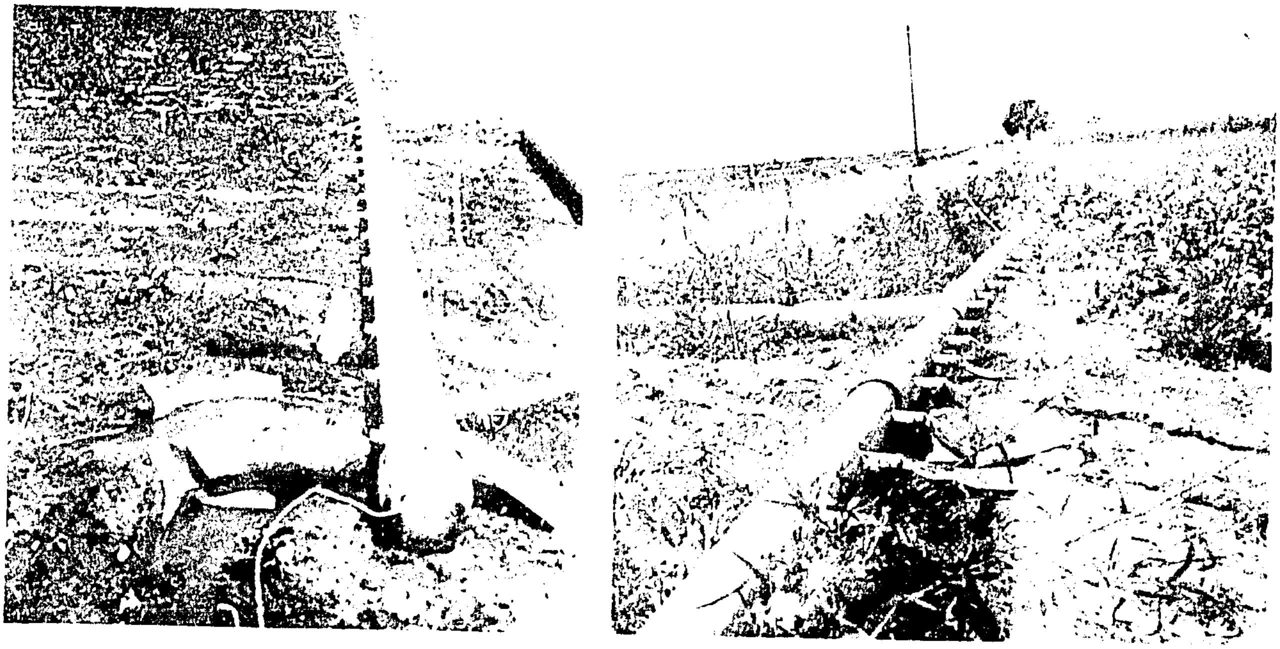


Figure 73. Gated surface pipe and tubing attached to portable hydrants fitted over alfalfa or orchard valves. The flow to the furrows is adjusted from individual outlets in the pipe or tube (4).

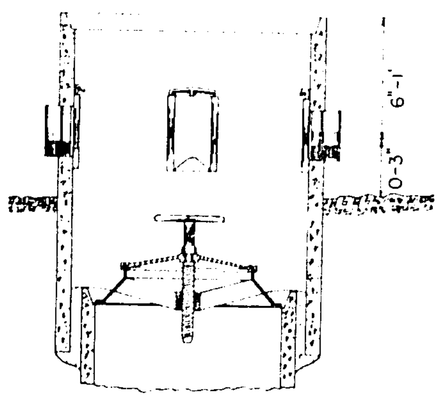


Figure 74. Open-pot hydrant with orchard valve and slide gate control (4).

c. Discharge Control Structures.

Outlets are necessary to deliver water from the pipeline to the land surface or into some distributing device. They consist of risers built of vertical sections of pipe into which outlet valves or gates are installed to control discharge.

1. Outlet Valves. Outlet valves are used to distribute water directly into border strips, basins or ditches where relatively large flows are needed. Two general types are used in the United States of America, *alfalfa valves* and *orchard valves*. Alfalfa valves are normally grouted to the top of a pipe riser as shown in Figure 71 (Plate 29, Appendix 2). This is referred to as an alfalfa valve hydrant. Orchard valves are smaller than alfalfa valves and are used where smaller flows are acceptable. They are usually installed inside the riser as shown in Figure 72 (Plate 30, Appendix 2). Since water usually flows from an orchard valve with lower velocities, they are commonly used in place of alfalfa valves where erosion is a problem or where the pressure in the riser is extra high.

Portable hydrants and sheet metal stands can fit over the valves for water delivery into surface pipe or ditches. The hydrants are constructed so that the valves can be regulated with the hydrant in place. Gated surface pipe or tubing can be attached to the hydrant to distribute water to furrows or corrugations (Figure 73). Sheet metal stands are sometimes fitted with multiple-connections so that one stand may serve several surface pipes individually or simultaneously.

2. Pot Hydrants. There are several types of distributing hydrants, two of which are the alfalfa and orchard type where the water flows from the top of the riser. Another, used for furrow irrigation, consists of a riser pipe extending to the ground level with a larger pipe, called a pot, fitted over it as shown in Figure 74. The pot has openings fitted with slide gates through which water is distributed to the furrows. The slide gates are placed on the inside of the open pot to minimize erosion. The water level in the hydrant is regulated with an orchard valve. When line pressures are low enough, the valve in the riser may be omitted with control at the slide gates.

In installations where the hydraulic gradient is not more than 30 to 60 cm (1 to 2 ft) above the ground, the pot may be capped and the orchard valve eliminated. In this case, the slide gates are installed and operated from the outside of the riser. The flow is controlled by adjustment of line pressures and the gate.

Capped pot outlets have the advantage of not allowing leaves or debris to enter the riser to clog the slide gates. However, they provide less control of the flow, and erosion from the water jet is more severe. The slide gates are often replaced by special screw-type valves which allow less erosion. The use of capped pot outlets is usually limited to orchards and permanent crops where small flows are distributed into furrows.

With low line pressures, the pot is sometimes omitted and the slide gate put in the sides of the riser which may be left open or capped. Flow ratings and maximum recommended design capacities for slide gates have been determined (24).

3. Surface Pipe Hydrant. Several different types of hydrants are used to connect the pipelines to surface pipe or tubing. These are essentially variations of those mentioned previously in which the slide gates are replaced by nipples or connections for attachment of the surface pipe. Unless excess pressure is in the pipeline, the riser must extend high enough to produce the required pressure in the surface pipe. If the pressure in the pipeline is more than required, the riser may be equipped with an orchard valve to prevent it from overflowing.

The height of an open hydrant should equal or exceed the head loss in the gated pipe or tubing plus freeboard. References are available which are helpful in determining head loss in surface pipe and tubing (24). Discharge into the furrows is controlled by individual outlets along the pipe or tube.

d. Miscellaneous Structures.

1. Sand Traps. Sand traps are usually built into the pipe inlet structures. Most of the suspended material may be removed by making the stand extra large in diameter to insure low water velocity and to provide a settling basin. The bottom of the stand is set some distance below the invert of the outlet pipes to provide space for sediment deposition (see Plate 26, Appendix 2).

Sediment collecting in the pipeline is minimized if a minimum velocity of 60 cm/sec (2 ft/sec), and preferably 90 cm/sec (3 ft/sec), is maintained. Sediment deposits in the pipeline reduce the capacity and eventually may plug the line. It is particularly important that sediment be removed from the water when surface pipe and tubes are used. Sediment deposition in this equipment makes the pipe very difficult to move.

2. Debris and Weed Screens. Debris and weed screens should be provided at every gravity inlet. Much of the difficulty caused by this material will be eliminated if provision is made to remove it from the water before entering the pipe. Designs are given in Plates 31 and 32, Appendix 2.

3. Air Vents. Vents are required on every pipeline to release air and to prevent high surge pressures. Vents are needed at all high points of a line, where the pipe slope increases sharply down grade, at sharp turns in the line, at the end of the line, and directly below any structure that entrains air in the flowing water. In addition to releasing air, open vents serve to release pressure surges and prevent damage to the line when gates or valves are opened or closed. They also prevent pipe collapse from vacuum when the line is drained.

The cross-sectional area of the vent riser should be at least one-half the area of the pipeline. A typical installation is shown in Figure 75 with design information given on Plate 33, Appendix 2. It is often recommended that the small vent pipe extend part way down into the riser. Air trapped in the space between the end of the pipe and the concrete cap absorbs pressure waves and the riser thus acts as a surge chamber. The area of the smaller vent pipe should not be less than one-sixtieth of the main line area and in no case less than 5 cm (2 in.) in diameter.

All vents should extend at least 120 cm (4 ft) above the ground or as high as necessary to prevent overflow during normal operation. The 120 cm (4 ft) height is for visibility to prevent damage during field operation. If an excessively high vent stand is required, it may be advisable to install an air-relief valve to reduce the height as indicated in Figure 75. Air-relief valves permit air to escape or enter but do not allow water to pass. They should not be located where it may be necessary to relieve momentary high pressure surges.

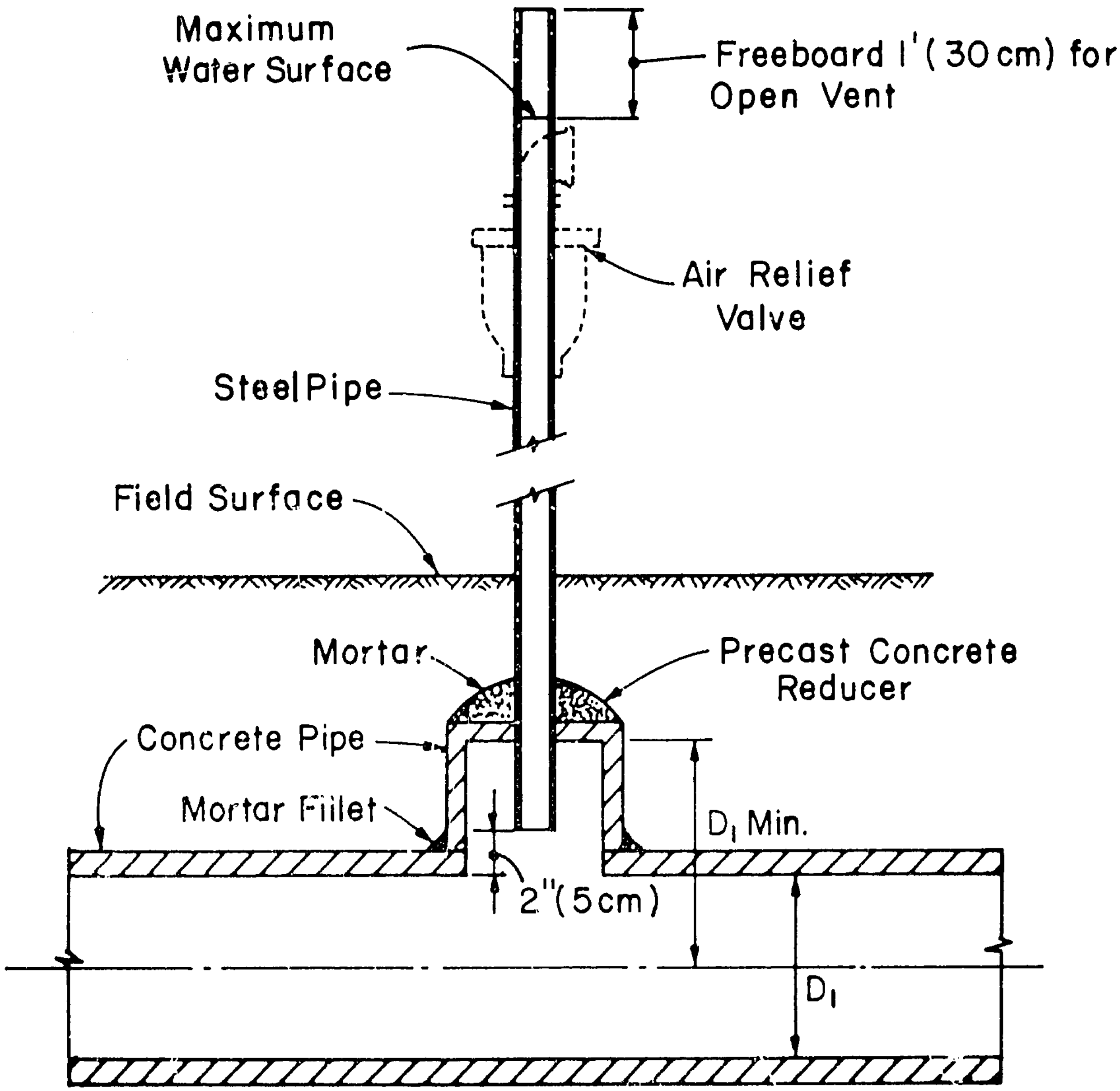


Figure 75. Air vent for underground pipelines. The vent pipe is sometimes allowed to project into the riser to form an air pocket and surge chamber in the top of the riser (4, 32).

IV. CONSTRUCTION AND INSTALLATION

There is an apparent lack of concern and understanding of the construction and operation requirements of small gravity irrigation systems and water control structures. This partly results from past emphasis on the design of large structures and from governmental responsibility for irrigation systems ending at the point where the use of small structures starts. The small structures and systems require an emphasis on economy in construction and installation while insuring reliable, simple operation for the farmer-operator of the system. There is some similarity between the small structures and ditch linings and the larger structures and canals in some construction phases — like foundations, concrete mix, backfilling, erosion prevention, etc.

Two types of construction for irrigation structures predominate in developing countries. The most common type is brick-masonry, which may be covered with concrete mortar. Concrete footings and slabs are used under the brick construction. Precast concrete blocks, sections and complete structures are next in common use. The sections may be flat, part circular, or parabolic for forming masonry linings or structures. Metal structures have only limited uses and wood, hardly at all, because of shortage, susceptibility to fire, and need for using the wood for other uses. Other than canal lining, poured-in-place concrete has only limited use in small irrigation control structures. The need for forms, and on-site mixing and pouring of concrete makes these structures costly and labor intensive.

1. Ditch Construction

Small earth irrigation ditches can be formed by hand or machine in either cut or fill sections. The fill sections should be well compacted in layers 10 to 15 cm (4-6 in.) deep. If the ditch section is higher than the field, an earth pad should be constructed and the ditch formed as shown in Figure 5. The side slopes of an earth ditch should not be steeper than 1:1 with recommended side slopes given in Table 2. Permanent ditch banks or berms should be at least 30 to 76 cm (12-30 in.) wide at the top as given in Table 3. The banks must be high enough to give a freeboard over the maximum water level of 15 cm (6 in.).

A discussion of lined ditch construction has been given in Section II, 1c. For the small ditches, masonry lining using brick, concrete lining and precast concrete lining are the most common. Other types of lining such as asphaltic and plastic are almost never used in developing countries but would be useful as liners below precast linings.

The ditch sections need to be overexcavated to accommodate the thickness of the lining material. The soil should be compacted using a hand tamper if a roller or other mechanical method is not available. Prior to placing the lining, the soil should be sprinkled with water if it has dried.

The concrete and mortar used for ditch linings should follow the specifications given in Appendix 1. It should be thoroughly mixed, preferably by machine and kept free of soil and debris. The local concrete and/or brick mason should be relied upon to lay the brick or place the concrete in an acceptable manner. For the small concrete ditches, it is not necessary to use reinforcing steel. The thickness of the lining should range between 5 and 10 cm (2-4 in.) depending on the ditch size and flow velocity (13). There should be contraction joints cut transversely in the wet concrete to about one-third its depth, about 3 m (10 ft) apart. If the slope of the ditch lining is over 0.02 (2 percent), collars extending 30 cm (1 ft) below and laterally from the sides of the lining are required at each joint. The joint should be over the collar. After a period of time the joint should be filled with expansive material like bitumen. Construction joints abutting structures should contain a suitable expansion joint material, such as bitumen or rubber.

Precast concrete liners should be placed on the earth sections that have been compacted and shaped to the outside shape of the liner. It is important that the liner is in complete contact with the soil or the plastic sheet underlying the liner. The joints of the precast sections must be perfectly aligned when mortared. Misalignment will reduce the flow capacity of the ditch and also promote cracking at the joint.

Brick lined channels should be covered with a 1 cm (0.4 in.) layer of concrete mortar, particularly when the brick is not of good quality. It is very important that the soil be well compacted, moistened, and shaped for placing the brick lining.

2. Ditch Structures

Take care to prepare the foundation for structures that are to be poured from concrete or built from brick. A concrete pad, slab or footing 10 cm (4 in.) thick should be sufficient. This is placed on the excavated soil that has been leveled and compacted (also wetted). It is important that the soil be wetted before pouring the concrete since a dry soil will remove water from the fresh concrete, reducing its final strength.

If forms are required, they must be tied or braced so that they cannot move. Brick structures must be made using the best local construction practices with the water side covered with a thin layer of mortar. Gate slots should be made with metal angles or channels and fastened to the structure rather than forming the slots in the concrete or mortar.

The new concrete should be allowed to cure for at least 5 days by covering it with cloth, canvas, burlap or sand, which should remain wet for the 5 day period. In some areas, concrete curing compounds are available. Directions for their use are printed on the containers.

After the concrete or brick mortar has had time to properly cure, the structures should be carefully backfilled. This phase is very important since the most common structural failure is improper or insufficient backfilling. Backfill soil should be moist and compacted in 10-15 cm (4-6 in.) layers. When complete, the backfill should extend above the sidewalls of the structure.

3. Pipe Systems

The construction, installation and testing of low pressure pipe systems should generally follow the specifications given by the American Society of Agricultural Engineers for nonreinforced concrete irrigation pipe systems (2) (Appendix 3), and for underground plastic irrigation pipelines (1) (Appendix 4). These standards relate the best practices for construction and installation. For specifications and instructions on making concrete and mortar for the pipeline structures, refer to Appendix 1.

Pipe trenches should be excavated deeply enough so that 0.75 to 1.20 m (30-48 in.) of cover is placed over plastic pipe and a minimum of 0.6 m (24 in.) cover over concrete pipe (3). The pipe should be uniformly supported over its entire length on firm, stable material in the trench. When trenches are excavated in soils containing rock or in soils subject to appreciable swelling or shrinkage, the trenches should be overexcavated and backfilled with stable materials to provide a firm, uniform base. Trench widths just adequate to allow room for pipe installation provide maximum support for the finished pipeline.

Before backfilling, fill plastic pipe with water and check for leaks. Keep the pipe full of water during backfilling to prevent collapse of the pipe. The trench is partially backfilled and water is added until the fill is thoroughly saturated. Allow to dry and then complete the backfilling.

For concrete pipe with mortar joints, partially backfill the trench while the mortar is still plastic. Complete backfilling after the mortar joints have set for at least 30 hours. The pipeline should not be filled with water before backfilling is completed. Pipelines should be tested for leaks by observing the trench after two weeks of continuous water in the lines.

Connect concrete pipelines to structures using mortar. Structures are constructed in a manner similar to those previously described for concrete ditches. For pump stands, the line from the pump must have a flexible joint so that vibrations are not transmitted to the stands.

V. OPERATION AND MAINTENANCE

1. Operation.

The operation of farm irrigation systems varies widely and is somewhat dependent on the operation and water delivery schedule of the secondary delivery canals (distributary canals, Figure 1). Usually the farmers, either by organization or individually, operate the balance of the system (tertiary and quaternary canals, Figure 1). The water delivery methods or schedules for the secondary canals can be broadly classified as demand, rotation or continuous flow systems (3). Another method classifies the system water delivery as either rigid (predetermined) or flexible (modified) schedule. In developing countries, water delivery from the government operated system usually follows a rigid schedule and gives a varied amount on a fixed frequency. The rigid schedule and varied amount often supply excess water during periods of low crop demand, resulting in water waste and drainage problems. Conversely, during periods of greater crop water demand, not enough water is available resulting in farmer conflicts. The above factors indicate that the flows in a particular system vary widely and this is usually the case.

The proper operation of an irrigation system depends on an organizational structure that will insure equitable delivery to the water users. To have equitable delivery, there must be water measurement, good conveyance systems and positive control, which will result from properly designed on-farm irrigation systems. A farmer-run organization is necessary to obtain correct water delivery based on the right and land holding of each individual. One individual employed or designated by the farmer organization should be responsible for equitable water delivery and also for recommending maintenance of the system.

2. Maintenance.

Good maintenance of irrigation systems and structures is necessary for efficient delivery and use of water. The maintenance of small on-farm irrigation canals and structures should be the responsibility of the cultivator and is a continuing task.

1. Seep areas in ditches or around structures should be immediately repaired.
2. Remove sediment and vegetation from ditches, structures, and repair features that have been damaged or deteriorated.
3. Design structures in unlined ditches so they will not interfere with ditch cleaning when mechanical equipment is used.
4. Ditches need to be cleaned at least once a year (more often where weed growth is very rapid), and
5. the ditch should be reshaped at the same time.

Clean, reshaped canals have a lower roughness value (Table 1) and will therefore allow the water to flow faster with less ponding than poorly maintained canals. Clean channels can conserve head in areas where gravity irrigation systems operate with limited available head. Since the discharge is inversely proportional to the roughness coefficient (Equation 1), a channel may carry as much as four times the flow when clean as when containing dense weeds (Table 1).

Low, short growing grass on the ditch banks is recommended for stabilization, but should not interfere with the flow capacity of the ditch. Weeds along the ditch banks should be eliminated. Rodents and burrowing animals are a major cause of ditch and structure failures and should be controlled. If rodents are known to be a major problem in an area, rodent activity around an irrigation structure can be reduced by mixing coarse sand and gravel with the backfill material when the structure is installed.

Ditch erosion, bank scouring, weak or low spots in the ditch bank, structure cracking and deterioration, and erosion around or below structures are all maintenance items that must be corrected. If scouring or erosion is occurring, changes and/or additions to the ditch structures should be made. Additional grade control or energy dissipating structures may be necessary. In areas where it is available, placing crushed rock or coarse gravel on the ditch bank and bed downstream from a structure may assist in reducing scour and erosion. Broken concrete or brick is useful for this purpose.

Cracks in concrete, masonry and brick structures should be repaired by cement mortar or by other means. Many cracks are caused by temperature and moisture changes. However, when cracking is caused by foundation settlement and/or backfill movement and pressure, the structure may need to be removed and rebuilt. The structures should be installed so that water does not pond in the ditches when the irrigation flow is off. Ponding enhances seepage, bank failure, mosquito breeding and contributes to structure failure. However, in some areas ponding is desired for domestic purposes such as livestock watering.

Metal structures and metal parts of other structures should be protected by painting and/or rustproofing. Metal in contact with the soil may need special treatment. Metal parts should be kept to a minimum, but if used, must be firmly attached and protected against vandalism or removal for other uses.

A good maintenance program can prolong the life of canals and structures several times over. A routine, thorough program should be maintained.

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VII. DEFINITION OF TERMS (17)

1. CHANNELS AND STRUCTURES

a. Delivery Channels. [Watercourse, farm lateral or field lateral.]

Secondary canals. [Distributary, secondary canal.] Canals which take off from main canals or branches and supply water to minors, outlets, or turnouts.

Tertiary canals. [Farm laterals, distributary minors.] Canals taking off from secondary distributaries and supplying water to sub-minors, outlets or turnouts - laterals (USA), *meska* (Egypt), minors (India and Pakistan).

Quaternary canals. [Field laterals, sub-minor distributary.] Canals taking off from minors and supplying water to outlets and or turnouts. Head-ditch (USA), watercourse (India and Pakistan), and *marwa* (Egypt).

b. Division Structures.

Distributor, divisor, divider. Structure built on a canal to distribute the flow either between two or more offtakes at the end of a canal or between itself and one or more offtakes.

Division box, proportional distributor. A farm structure to divide the water supply between two or more ditches.

c. Drops, Chutes.

Drops

Drop structure, fall structure. A structure designed to lower the water surface in a channel in a short distance and with safe dissipation of energy.

Ditch drop. A farm structure built to absorb the excess grade when the slope of the ditch is greater than the grade which should be used in the ditch. Erosive velocities are reduced upstream.

Chutes. An inclined drop or fall in which the lowering of the water surface is achieved over a relatively short length of channel.

d. Checks.

Check, check structure. Structure built or placed across a channel at suitable points to control water levels and regulate water supply. Stop logs and check panels are the movable sections placed in slots to control depths.

Check drop. Check structure combined with a drop, for a dual purpose structure.

Check stop. A permanent or temporary structure inserted in a ditch to partly or wholly check the flow. Used to raise the water level to permit irrigation through bank cuts, spiles or siphon tubes. May be portable.

e. Turnouts.

Offtake regulator. A structure built at the head of an offtaking distributary channel to control and regulate water from the parent canal.

Headgate. Structure at the head of a watercourse, farm lateral or field lateral which connects with the distributing channel. The turnout may be placed through the banks of tertiary and quaternary canals for water delivery to the fields.

Turnout. Structure that releases the water from a head ditch. Can be used to allow the water to pass through the banks of the head ditch onto a field, thus acting as a check gate and a head gate at the same time.

f. Miscellaneous Structures and Devices

Measuring structures. Weirs, measuring flumes, structures used to find depth-discharge relationship.

Pipeline structures. Structures used with underground, low pressure irrigation pipeline systems.

Drainage structures, tail escape structures. Structures for conveying water away from irrigated areas into a drainage system.

Ditch structures. Inlets, outfalls, overflow sections, checks, drops, crossings, outlets.

Drainage tile and pipe structures. Inlets, outlets.

Desilting boxes, sand traps. Structures to reduce the flow velocities so that sand and silt settles and can be removed.

Other devices.
- •Siphon tubes — Pipes over ditch banks.
- •Spiles — Pipes through ditch banks.
- •Flexible pipe and tubing — To convey flow from pipes or spiles.

2. HYDRAULICS

a. Aerated Nappe. Undersurface of water flowing over a weir-type structure that has an ample supply of air

b. Critical Depth. Depth of flow in an open channel where the energy of flow is at a minimum value, i.e., where $H + V^2/2g$ is at a minimum.

c. Friction Losses. Loss in hydraulic head due to friction caused by the roughness of the surfaces that are in contact with the flowing water.

d. Velocity Head. The average velocity (V) multiplied by itself (V^2) divided by twice (2) the gravity acceleration (g) 9.81 m/s^2 (32.2 ft/s^2). This results in the velocity head in meters (or feet).

e. Available Head. Difference in elevation of an upper water surface and a lower surface such as a field or water surface.

f. Hydraulic Head. Depth of water referenced to a lower elevation. Height that water will stand in a tube. Energy available.

g. Head Loss. Energy lost as a result of friction, impact or turbulence. Simply, the difference in two water surfaces connected by pipes or channels.

h. Hydraulic Grade Line. In an open channel, the water surface is the hydraulic grade line. In a closed pipe, a line joining the elevations to which water would stand in open gage tubes.

i. Hydraulic Gradient. Slope of hydraulic grade line.

j. Hydraulic Radius. Area of the flowing water divided by the wetted perimeter. For pipes flowing full, this is equal to the diameter divided by four (4).

k. Submerged Flow. A condition of flow through or over a structure where such flow is affected by the depth of water on the downstream side.

l. Supercritical Flow. Velocity of flow is greater than critical flow (see **2*b***). High velocity flow.

CONCRETE INFORMATION

PORTLAND CEMENT ASSOCIATION

Concrete for Small Jobs

Concrete is a widely used building material around the farm and home. Foundations, walls, sidewalks, patios, steps, floors, and driveways are built by homeowners everywhere. Concrete has several desirable properties that make it a versatile and popular building material. Freshly mixed concrete can be formed into practically any shape. Hardened concrete is strong and durable.

The layman often confuses the words "cement" and "concrete."

Cement is the fine powder sold by building materials dealers in bags. It should be called by its more exact name, "portland cement," to differentiate it from other kinds of cements used for other purposes, such as, for example, masonry cement.

Concrete is the mixture of two components, paste and aggregates. Paste is composed of portland cement, water, and air. Aggregates are inert minerals such as sand, gravel, and crushed stone.

During mixing, the cement and water form a paste that coats the surface of every piece of aggregate. Usually within two to three hours after mixing, a chemical reaction starts between the cement and the water. As this chemical reaction progresses, the cement paste hardens gradually and the concrete is said to set. Finally, the cement-water paste will harden much like glue and bind the aggregates together to form the solid mass that is concrete.

Although ready mixed concrete is widely used for large construction jobs, it is not always practical to use ready mixed concrete on small jobs. In some cases, the amount of concrete you require may be less than 1 cu.yd., which is less than most ready mix producers will supply. And in some areas there is no ready mix plant.

If you are faced with one of these circumstances, making your own concrete may be the only practical solution. This is hard work but it has the advantage of low cost, and the amount of concrete mixed can be adjusted to suit your own work pace.

Quality concrete costs no more to make than poor concrete, but is far more economical in the long run because of its greater durability. The rules for making good concrete are simple:

1. Use proper ingredients.
2. Proportion the ingredients correctly.
3. Measure the ingredients accurately.
4. Mix the ingredients thoroughly.

CHOOSING THE INGREDIENTS

Portland cement is not a brand of cement but a type. Most portland cement is grey in color. However, white portland cement is manufactured from special raw materials that produce a pure white color. It can be used instead of the normal grey portland cement, but it is higher in price, which may restrict its use to decorative work and other special jobs.

You can buy portland cement in bags at your local building materials dealer. In the United States, a bag weighs 94 lb. and holds 1 cu.ft.; in Canada, a bag weighs 80 lb. and holds about [illegible] cu.ft.

Cement in bags should be stored in a dry location, preferably on raised wooden platforms. Sometimes when bags have been stored for a long time, the cement in the lower part of a pile develops warehouse pack, that is, the cement appears to be hardened around the edges of the bags. You can usually correct this by rolling the bag on the floor. To avoid warehouse pack, bags should not be stacked more than seven high.

Cement suitable for use in concrete should be free-flowing. The presence of lumps that cannot be pulverized readily between your thumb and finger indicates that the cement has absorbed moisture. Such cement should never be used for important work, but when the lumps have been screened out through an ordinary house screen, it can be used for certain minor jobs such as setting fence posts.

Water for making concrete can be almost any natural water that is drinkable and has no pronounced taste or odor. Although some waters that are not suitable for drinking will make satisfactory concrete, to be on the safe side, use only water fit to drink.

Air is also an important ingredient for making good concrete. In the late 1930's, it was discovered that air in the form of microscopic bubbles evenly dispersed throughout the concrete improved its durability and virtually eliminated scaling due to freeze-thaw and de-icer salt action. Concrete containing such air bubbles is called air-entrained concrete.

Hardened concrete usually contains some water. When this water freezes, it expands, causing pressure that can rupture (scale) the concrete surface. The tiny air bubbles act as reservoirs or relief valves for the expanding water, thus relieving pressure and preventing damage to the concrete.

Air entrainment is most important for concrete exposed to alternate cycles of freezing and thawing or use of deicers. In cold climates, and even in mild climates that have several cycles of freezing and thawing each year, it should be used for all exterior concrete work, including driveways, sidewalks, patios, and steps.

Air entrainment also has other advantages. For example, the tiny air bubbles act like ball bearings in the mix, increasing its workability, with the result that less mixing water is required.

To create the tiny air bubbles in air-entrained concrete, chemicals specially made for this purpose, called air-entraining agents, are added to the mixing water. Building materials suppliers sometimes carry air-entraining agents. Ready mix plants stock them for their own use and would probably sell you a small quantity. The amount to be added to the mix depends on the brand of air-entraining agent. This information can be obtained from the building materials supplier or the ready mix producer.

There is another method of obtaining air-entrained concrete. To save you the trouble of buying and measuring an air-entraining agent (and eliminating a possible error in dosage), many cement manufacturers market portland cements that contain an interground air-entraining agent. These cements are identified on the bag as "air-entraining" and are available from the same suppliers that sell regular portland cements.

Aggregates are minerals such as sand, gravel, and crushed stone that make up 60 to 80 percent of the volume of concrete. They act as an inert filler material to reduce the amount of cement required in concrete. Without aggregates, concrete would be very expensive. Furthermore, without aggregates, concrete would shrink a great deal upon drying and this would lead to excessive cracking. Aggregates restrain the shrinkage that occurs when concrete hardens.

Aggregates are divided into two sizes, fine and coarse. Fine aggregate is always sand, and coarse aggregate is usually gravel or crushed stone.

Natural sand is the most commonly used fine aggregate; however, manufactured sand, made by crushing gravel or stone, is also available in some areas. Sand should have particles ranging in size from ¼ in. down to dust-size particles small enough to pass through a No. 100 mesh sieve (10,000 openings to the square inch). *Mortar sand should not be used for making concrete since it contains only small particles.*

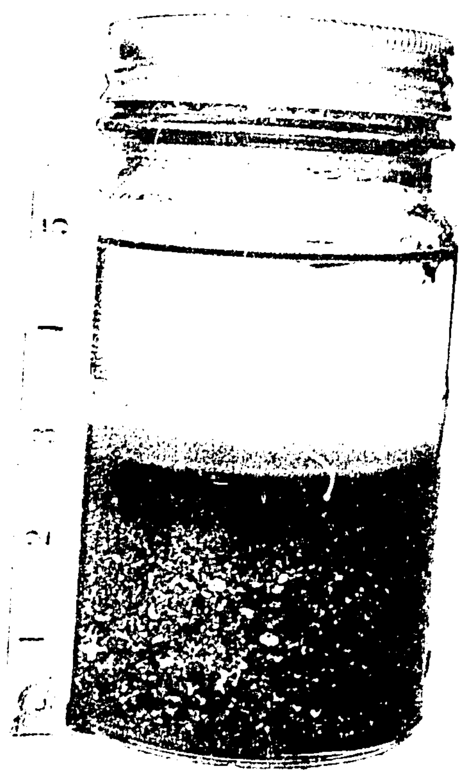
Gravel or crushed stone are the most commonly used coarse aggregates. They should consist of particles that are sound, hard, and durable, not soft or flaky, with a minimum of long, sliver-like pieces. Particles should range in size from ¼ in. up to the maximum size used for the job. The common maximum sizes are ⅜, ½, ¾, 1, or 1½ in. Generally, the most economical mix is obtained by using the largest-size coarse aggregate that is practical or available. Coarse aggregate up to 1½ in. in size, for example, may be used in a thick foundation wall or heavy footing. In walls, the largest pieces should never be more than one-fifth the thickness of the finished wall section. For slabs, the maximum size should not exceed one-third the thickness of the slab. If concrete is to be placed around reinforcing bars or pipes, the maximum size of the aggregate should not be more than three-fourths of the clear space between the bars or pipes, or between the form and the pipe or bar closest to it.

All sizes of aggregates may not be available locally, but within the above limitations, try to use the largest-size aggregate readily available.

Both fine and coarse aggregates for making concrete must be clean and free of excessive dirt, clay, silt, coal or other organic matter such as leaves, roots, etc. These foreign materials will prevent the cement from properly binding the aggregate particles together, resulting in porous concrete with low strength and durability.

If you suspect that the sand contains too much extremely fine material, such as clay and silt, check its suitability for use in making concrete by the so-called silt test (Fig. 1). Fill an ordinary quart canning jar or milk bottle to

Fig. 1. Silt test being made in a quart canning jar.



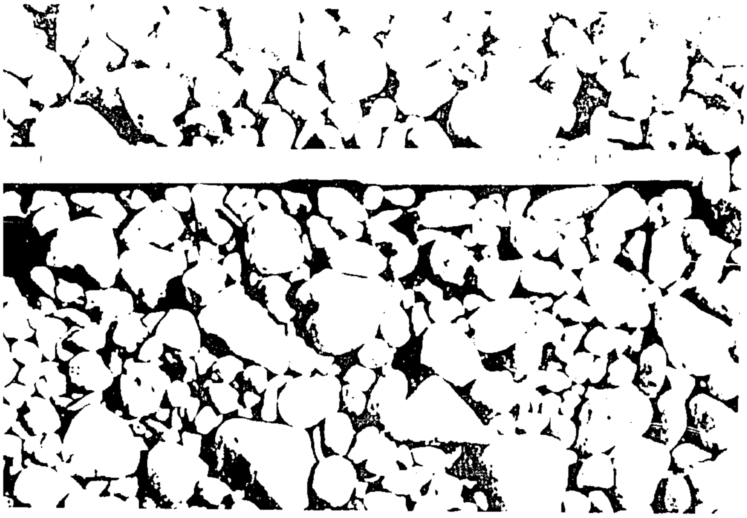


Fig. 2. Well-graded aggregates have particles of various sizes. Shown here is 1½-in. maximum-size coarse aggregate. Pieces vary in size from ¼ to 1½ in.

a depth of 2 in. with a representative sample of the sand in question. The sample should be taken from at least five different locations in the sand pile and thoroughly mixed together. Add clean water to the sand in the jar or bottle until it is about three-quarters full. Shake the container vigorously for about a minute. Use the last few shakes to level off the sand. Allow the container to stand for an hour. Any clay and silt present will settle out in a layer above the sand. If this layer is more than ⅛ in. thick, the sand is not satisfactory unless the clay and silt are removed by washing.

Good fine and coarse concrete aggregates have a full range of sizes from the smallest to the largest, but no excess amount of any one size. The big particles fill out the bulk of a concrete mix and the smaller ones fill in the spaces between the larger ones. Aggregates with an even distribution of particle sizes are said to be well graded (Fig. 2). Such aggregates produce the most economical and workable concrete. Mixtures of fine and coarse aggregates taken directly from gravel banks or stone crushers usually contain an excess of sand in proportion to coarse material. Before using this material in concrete, it should be screened and recombined into properly graded fine and coarse aggregates.

Buy fine and coarse aggregates separately from a reputable building materials supplier. If there is a ready mix producer in your area, it is preferable to purchase aggregates from him. He will make sure that the aggregates you buy have the correct sizes and are suitable for making concrete.

Store aggregates on a clean, hard surface, if possible, and not directly on the ground. Apart from wastage of material, ground storage may cause contamination with mud and dirt. It is good practice to cover aggregate piles to prevent them from becoming wet in case of rain. Do not use the bottom layer of an uncovered aggregate pile, as this part is usually saturated with water and may contain an accumulation of dirt washed through from higher layers.

PROPORTIONING THE INGREDIENTS

In concrete, the cement and water form a paste that surrounds every piece of aggregate. Within a few hours, the concrete starts to harden due to hydration, a chemical reaction between cement and water. As hydration occurs, the paste binds the aggregate together into a strong, durable, solid mass.

The quality of the concrete is directly related to the quality of the cement paste. The quality of the paste, in turn, is directly related to the amount of water mixed with the cement. If too much mixing water is used, the paste will be thin and diluted, making the concrete weak and porous. As the amount of water is reduced, the strength of the paste increases, making the concrete stronger and more durable.

To find the correct amount of mixing water, use the proportions given in Tables 1 or 2 as a starting point. These proportions may not always give a workable mix with your aggregates. However, simple adjustments in the mix can be made, as will be explained in Adjusting the Trial Batch on page 7.

The values in Table 1 should be used if you are measuring your materials by weight. Measuring by weight rather than by volume is recommended for reasons that will be explained in Measuring the Ingredients, page 4.

The proportions in Table 1 are based on use of coarse aggregates consisting of gravel. Gravels are more or less smooth and rounded, while crushed stone aggregates are rougher and more angular. Angular particles produce mixtures that are a little more difficult to work than mixtures with rounded particles, hence a little less crushed material must be used in each cubic foot of concrete to obtain the same workability. Accordingly, when using crushed stone, reduce the value for coarse aggregate in Table 1 by 3 lb. and increase the value for sand by 3 lb.

The weights of material given in Table 1 will make a 1-cu.ft. batch. This is about the right amount for hand mixing. For machine mixing, multiply the values in Table 1 by the capacity of the mixer. For example, if your mixer capacity is 3 cu.ft. and you are making air-entrained concrete with ¾-in. maximum-size gravel aggregate, you would weigh out 25 × 3 = 75 lb. of air-entraining cement, 42 × 3 = 126 lb. of sand, 65 × 3 = 195 lb. of gravel, and 10 × 3 = 30 lb. of water. If you are using crushed stone of the same size, weigh out 3 lb. per cubic foot more sand, or 45 × 3 = 135 lb. of sand instead of 126, and 3 lb. per cubic foot less coarse aggregate, or 62 × 3 = 186 lb. of crushed stone instead of 195 lb. of gravel.

The proportions given in Table 2 are by volume or parts and can be measured in pails, cans, or any other sturdy container. An ordinary galvanized steel water pail makes a convenient batching container.

Estimating quantities needed. Before getting down to the job of measuring and mixing, you will need to know just how much cement, sand, and coarse aggregate to buy for your project. To do this you will first have to estimate the amount of concrete your project will require. Use the following simple formula, which works for any square or other rectangular-shaped area.

$$\frac{\text{Width (ft.)} \times \text{Length (ft.)} \times \text{Thickness (in.)}}{12} = \text{Cubic feet}$$

Table 1. Proportions by Weight to Make 1 Cu.Ft. of Concrete

Maximum-size coarse aggregate, in.	Air-entrained concrete: Cement, lb.	Sand, lb.	Coarse aggregate, lb.*	Water, lb.	Concrete without air: Cement, lb.	Sand, lb.	Coarse aggregate, lb.*	Water, lb.
3/8	29	53	46	10	29	59	46	11
1/2	27	46	55	10	27	53	55	11
3/4	25	42	65	10	25	47	65	10
1	24	39	70	9	24	45	70	10
1 1/2	23	38	75	9	23	43	75	9

*If crushed stone is used, decrease coarse aggregate by 3 lb. and increase sand by 3 lb.

Table 2. Proportions by Volume

Maximum size coarse aggregate, in.	Air-entrained concrete: Cement	Sand	Coarse aggregate	Water	Concrete without air: Cement	Sand	Coarse aggregate	Water
3/8	1	2 1/4	1 1/2	1/2	1	2 1/2	1 1/2	1/2
1/2	1	2 1/4	2	1/2	1	2 1/2	2	1/2
3/4	1	2 1/4	2 1/2	1/2	1	2 1/2	2 1/2	1/2
1	1	2 1/4	2 3/4	1/2	1	2 1/2	2 3/4	1/2
1 1/2	1	2 1/4	3	1/2	1	2 1/2	3	1/2

For example, a 4-in.-thick patio slab, 12 ft. wide and 15 ft. long, would require: $\frac{12 \times 15 \times 4}{12} = 60$ cu.ft. of concrete. A wall 3 ft. high, 10 ft. long, and 8 in. thick would require: $\frac{3 \times 10 \times 8}{12} = 20$ cu.ft.

The amount of concrete determined by the above formula does not allow for losses due to uneven subgrade, spillage, etc., so add 5 to 10 percent for such contingencies. In the case of the wall, the total amount of concrete required would be $20 + (0.10 \times 20) = 22$ cu.ft.

The quantities of material to buy can be calculated by multiplying the number of cubic feet of concrete (22 in this example) by the weights of materials needed for 1 cu.ft., given in Table 1. Assuming the wall will require air-entrained concrete and the maximum size of available aggregate to be 3/4 in., the quantities of material needed would be as follows:

$22 \times 25 = 550$ lb. of cement
$22 \times 42 = 924$ lb. of sand
$22 \times 65 = 1{,}430$ lb. of gravel

Since it is generally impossible to recover all of the material, a 10 percent allowance should be made to cover normal wastage. It is preferable to have some material left over than to run the risk of being short of material near the end of the job.

The quantities of material needed should therefore be increased to:

$550 + (0.10 \times 550) = 605$ lb. of cement
$924 + (0.10 \times 924) = 1{,}016$ lb. of sand
$1{,}430 + (0.10 \times 1{,}430) = 1{,}573$ lb. of gravel

Since a U.S. bag of cement weighs 94 lb., you will need to buy $\frac{605}{94} = 6.4$ or 7 bags. A Canadian bag weighs 80 lb., so you will need $\frac{605}{80} = 7.5$ or 8 bags if you buy your cement in Canada. If air-entraining cement is not available, you will also need to obtain an air-entraining agent.

Aggregates are sold by the ton (2,000 lb.) or by the cubic yard (27 cu.ft.). Quantities of aggregates can be converted from pounds to cubic yards, or vice versa, by assuming a value of 90 lb. per cubic foot for the weight of sand and 100 lb. per cubic foot for the weight of coarse aggregate. Accordingly, 1,016 lb. of sand contains $\frac{1{,}016}{90} = 11.3$ cu.ft. or $\frac{11.3}{27} = 0.42$ cu.yd., and 1,573 lb. of gravel contains $\frac{1{,}573}{100} = 15.7$ cu.ft. or $\frac{15.7}{27} = 0.58$ cu.yd.

MEASURING THE INGREDIENTS

Ingredients must be measured accurately to ensure production of uniform batches of quality concrete. Ingredients may be measured by weight or by volume.

Measurement by weight is recommended because it is more accurate and hence produces greater uniformity from batch to batch. Also, it is easier to make adjustments in mix proportions when measuring by weight. A common bathroom scale is accurate enough for weighing the materials. Each ingredient should be weighed in a separate container. Three- to five-gallon galvanized steel pails or buckets are suitable. Remember to "zero" the scale with the empty container on it. After weighing each ingredient once, mark the level of the material inside the container. Subsequent batches may be measured by using this mark. The scale will no longer be required except to check the marks against the weight of material once or twice a day, or when the moisture content of the sand has changed.

Although less accurate, measurements may be made by volume if no scale is available. For example, a 1:2¼:3 concrete mix from Table 2 would be batched by measuring out 1 pail of cement, 2¼ pails of sand, 3 pails of coarse aggregate, and ½ pail of water. Take care when batching by volume not to overload the mixer. This will reduce mixing efficiency.

Adjusting for water in the sand. Dry sand is rarely available for concrete work. Sand used on most jobs contains some moisture which must be accounted for as part of the mixing water.

The proportions given in Table 1 are based on *wet sand* (Fig. 3), which is the condition of sand usually available. When squeezed in the hand, wet sand forms a ball and leaves no noticeable moisture on the palm. *Damp sand* (Fig. 4) falls apart when squeezed in the hand. *Very wet sand* (Fig. 5) forms a ball when squeezed in the hand and leaves noticeable moisture on the palm. This is the condition of sand exposed to recent rain.

If you are using damp sand, decrease the quantity of sand given in Table 1 by 1 lb. and increase the quantity of water by 1 lb. If your sand is very wet, increase the quantity of sand by 1 lb. and decrease the quantity of water by 1 lb.

The proportions given in Table 2 also are based on wet sand, but measurement by volume involves too many inaccuracies to justify making corrections for the moisture in damp or very wet sand. For example, moisture in sand causes an increase in volume known as bulking. The extent of bulking depends on the amount of moisture in the sand and its fineness. Dry sand can bulk to 1¼ times its volume when wetted. Accordingly, if you are measuring by volume, try to use wet sand.

MIXING THE INGREDIENTS

Proper mixing is an essential step in making good concrete. It is not sufficient merely to intermingle the ingredients. They must be thoroughly mixed so that cement paste coats every particle of fine and coarse aggregate in the mix. Concrete may be machine mixed or hand mixed.

Machine mixing. The best way to mix concrete is with a concrete mixer. It ensures thorough mixing of the ingredients and is the only way to produce air-entrained concrete.

Small mixers from ½- to 6-cu.ft. capacity can be rented or purchased. For extensive work around the home, it might pay to purchase a mixer. For the occasional small job, however, it is preferable to rent a mixer from your local rental service store or yard.

Mixers are powered by gasoline or electricity. The gasoline-powered mixer is more versatile in that it can be operated anywhere. The electric-powered mixer is quieter and simpler to operate, but requires an electrical outlet.

Mixer sizes are designated according to the maximum concrete batch in cubic feet that can be mixed efficiently. This is usually 60 percent of the total volume of the mixer drum. The maximum batch size is usually shown on an identification plate attached to the mixer. For proper mixing, never load a mixer beyond its maximum batch capacity. The choice of mixer size will depend on the extent of your project and the amount of concrete that you want to handle in any one batch. Keep in mind that to mix a 1-

Fig. 3 *(left). Wet sand*, **which describes most sands, forms a ball when squeezed in your hand, but leaves no noticeable moisture on the palm. Fig. 4** *(center). Damp sand* **falls apart when you try to squeeze it into a ball in your hand. Fig. 5** *(right). Very wet sand*, **which describes sand exposed to a recent rain, forms a ball if squeezed in your hand and leaves moisture on the palm.**

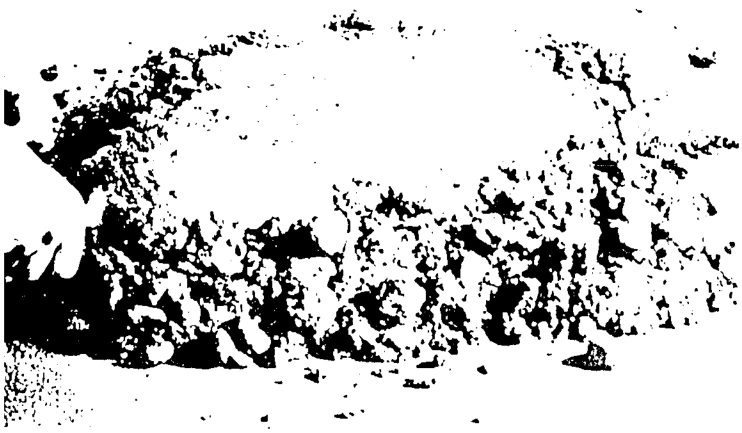


Fig. 6. A *workable mix* contains the correct amount of cement paste, sand, and coarse aggregate. With light troweling, all spaces between coarse aggregate particles are filled with sand and cement paste.

cu.ft. batch of concrete you will have to handle 140 to 150 lb. of materials.

For best results, load the ingredients into the mixer in the following sequence.

1. With the mixer stopped, add all the coarse aggregate and half of the mixing water. If an air-entraining agent is used, mix it with this part of the mixing water.
2. Start the mixer, then add the sand, cement, and remaining water with the mixer running.

After all ingredients are in the mixer, continue mixing for at least three minutes, or until all materials are thoroughly mixed and the concrete has a uniform color.

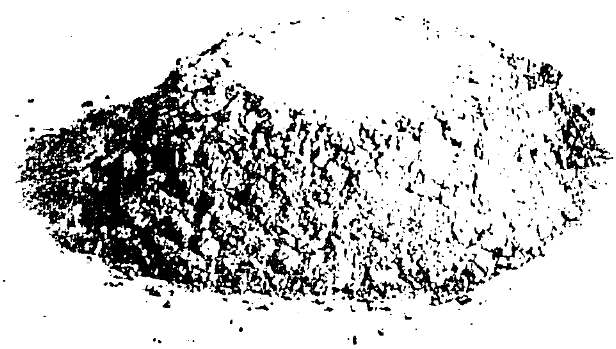
Concrete should be placed in the forms as soon as possible after mixing. If the concrete is not placed within 1½ hours and shows signs of stiffening, remixing for about two minutes may restore its workability. Discard the concrete if after remixing it is still too stiff to be workable. *Never add water to concrete that has stiffened to the point where remixing will not restore its workability.*

Mixing the trial batch. The proportions of sand and coarse aggregate from Table 1 are based on typical gravel aggregates. If these proportions do not give a workable mix with your aggregates, an adjustment will be necessary. The so-called trial batch will enable you to make these adjustments.

First make a batch of concrete using the proportions from Table 1. Discharge a sample of concrete from the mixer into a wheelbarrow or onto a slab and examine it for stiffness and workability. If this sample is a smooth, plastic, workable mass that will place and finish well, proportions used are correct and need no adjustment. The suitability of the sample can be judged by working the concrete with a shovel and smoothing it with a float or trowel. A good, workable mix should look like the sample shown in Fig. 6. The concrete should be just wet enough to stick together without crumbling. It should slide down, not run off, a shovel. In a workable mix there is sufficient cement paste to bind the pieces of aggregate together so that they will not separate when the concrete is transported and placed in the forms. There should be sufficient sand-cement paste to give clean, smooth surfaces free from rough spots (called honeycomb) when forms are stripped. In other words, there should be just enough cement paste to completely fill the spaces between the particles of aggregate and to ensure a plastic mix that finishes easily.

Fig. 7. This mix is *too wet* because it contains too little sand and coarse aggregate for the amount of cement paste. Such a mix would not be economical or durable and would have a strong tendency to crack.

Fig. 8. This mix is *too stiff* because it contains too much sand and coarse aggregate. It would be difficult to place and finish properly.



Adjusting the trial batch. If the trial batch is too wet, too stiff, too sandy, or too stony, it will be necessary to adjust the proportions of aggregates used in the mix.

If the mix is too wet (Fig. 7), it contains too little aggregate for the amount of cement paste. Return the sample to the trial batch, then weigh out about 5 to 10 percent more sand and coarse aggregate, depending on how wet the mix is. Add them to the trial batch in the mixer and mix for at least one minute. If the mix is still too wet, add some more sand and coarse aggregate until the desired workability is obtained. Record the total weight of added sand and coarse aggregate. In subsequent batches, use the original quantities of aggregate, but reduce the amount of water by 1 lb. for every 10 lb. of aggregate added to the trial batch.

If the mix is too stiff (Fig. 8), it contains too much aggregate. Reduce the amounts of sand and coarse aggregate in subsequent batches until the desired workability is obtained. Record the new weights of sand and coarse aggregate, and correct the weight marks in the batch cans according to the adjusted weights. To save a trial batch that is much too stiff to place, cement and water may be added in the proportion of 1 lb. water to 2 lb. cement. This will increase the amount of cement paste and make the concrete more workable. *Never add water alone to a mix that is too stiff.*

If the mix is too sandy (Fig. 9), decrease the amount of sand by 2 lb. and add 2 lb. of coarse aggregate. If it is still too sandy, leave out some more sand and add an equal weight of coarse aggregate in the next batch. Record the new weights of sand and coarse aggregate, and correct the weight marks in the batch cans according to the adjusted weights.

If the mix is too stony (Fig. 10), decrease the amount of coarse aggregate by 2 lb. and add 2 lb. of sand. If it is still too stony, leave out some more coarse aggregate and add an equal weight of sand in the next batch. Record the new weights of sand and coarse aggregate, and correct the weight marks in the batch cans according to the adjusted weights.

Your adjusted trial batch proportions are your final mix proportions and need not be changed again for future batches as long as your sand and coarse aggregate remain the same. If the moisture content of your sand changes, due to rain, for example, adjust the quantities of sand and water as explained under Measuring the Ingredients, page 4.

Hand mixing. For very small jobs, where the volume of concrete required is less than a few cubic feet, it is sometimes more convenient, though less efficient, to mix by hand.

Hand mixing is not vigorous enough to make air-entrained concrete, regardless of whether air-entraining cement or an air-entraining agent is used. Hand mixing, therefore, should not be used for concrete that will be exposed to freezing-thawing or de-icers.

Hand mixing should be carried out on a clean, hard surface or in a mortar box to prevent contamination by mud and dirt. A concrete slab makes a good working surface. The measured quantity of sand is spread out evenly on the slab. Then the required amount of cement is dumped on the sand and evenly distributed. Mix the cement and sand thoroughly by turning with a short-handled, square-end shovel until you have a uniform color, free from streaks of brown and grey. (Streaks indicate that the sand and cement have not been thoroughly mixed.) Next, spread this mixture out evenly over the slab and dump the required quan-

Fig. 9. This mix is *too sandy* because it contains too much sand and not enough coarse aggregate. It would place and finish easily, but would not be economical, and would be very likely to crack.

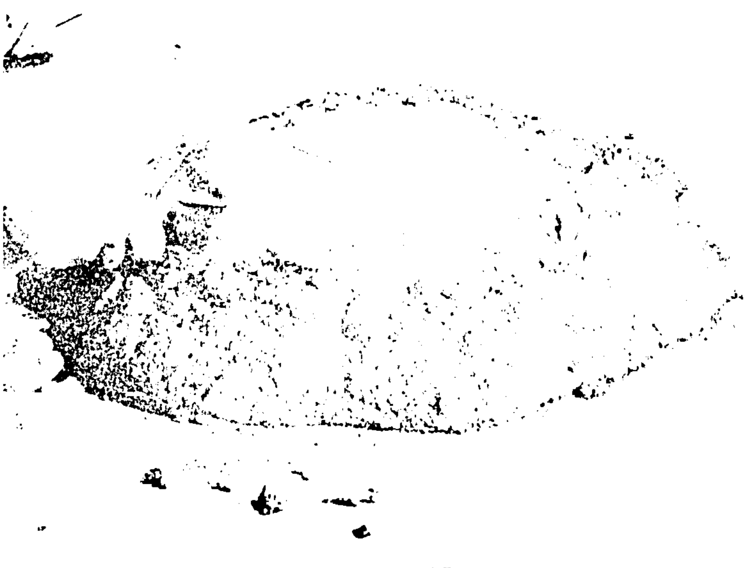


Fig. 10. This mix is *too stony* because it contains too much coarse aggregate and not enough sand. It would be difficult to place and finish properly and would result in honeycomb and porous concrete.

tity of coarse aggregate in a layer on top. The materials are again turned by shovel until the coarse aggregate has been uniformly blended with the mixture of sand and cement. After at least three turnings, form a depression or hollow in the center of the pile and slowly add the proper amount of water. Finally, turn all the materials in toward the center and continue mixing until the water, cement, sand, and coarse aggregate have all been thoroughly combined.

Prepackaged mixes. Jobs small enough for hand mixing can usually be done with convenient prepackaged concrete mixes. Building materials suppliers, hardware stores, and even some supermarkets sell prepackaged concrete mixes. All the necessary ingredients—portland cement, dry sand, and dry coarse aggregate—are combined in the bag in the correct proportions. Packages are available in different weights, but the most common sizes are 45 and 90 lb. A 90-lb. package makes ⅔ cu.ft. of concrete. All you do is add water and mix. Directions for mixing and the correct amount of water to add are given on the bag.

To ensure that you get good quality from prepackaged concrete mixes, the American Society for Testing and Materials has adopted Specifications for Packaged, Dry, Combined Materials for Mortar and Concrete (ASTM C387). This specification covers the quality of the ingredients, the strength of concrete obtained with the ingredients, and the type of bag in which the ingredients are packaged. ASTM C387 also requires that prepackaged concrete mixes meeting this specification be so identified on the bag. To obtain a quality product, make sure the prepackaged mix you buy contains a statement on the bag that it meets ASTM C387.

If the concrete will be exposed to freeze-thaw or de-icers, prepackaged mixes must be machine mixed and must be made with air-entraining cement, or an air-entraining agent must be added to the mixing water.

As pointed out above, prepackaged mixes are most convenient for the very small job requiring only a few cubic feet of concrete. However, for larger jobs up to 1 cu.yd. (27 cu.ft.), you would be wise to compare the cost of using prepackaged mixes with the cost of buying the separate ingredients.

Cleaning the mixer. Soon after you finish using the mixer (before the concrete can harden), it should be thoroughly cleaned. To clean the inside of the mixer drum, add water and a few shovels of coarse aggregate while the drum is turning. Follow this by hosing with water. The thin cement film that builds up on the exterior parts of the mixer may be removed with vinegar. Concrete that builds up inside the mixer drum requires scraping and wire brushing for removal. Heavy hammers or chisels that might tear up the drum and blades should not be used. Remove stubborn buildup with a solution of 1 part hydrochloric acid (muriatic acid) in 3 parts of water. Allow 30 minutes for penetration, then scrape or wire brush and rinse with clear water.

Hydrochloric acid is hazardous and toxic and requires adequate safety precautions. Skin contact and breathing of fumes should be avoided. As a general precautionary rule, rubber or plastic gloves and chemical safety goggles should be worn. If the acid is used indoors, adequate ventilation should be provided. Follow the storage and handling precautions stated on the label of the acid container.

Dry the mixer drum thoroughly to prevent rusting and store the mixer with the opening of the drum pointing down. Do not apply oil to the inside of the drum unless the mixer is to be stored for an extended period of time. Thoroughly wipe off the oil before using the mixer again, as it may adversely affect the quality of the concrete.

Caution: Avoid prolonged contact between unhardened (wet) cement or concrete mixtures and skin surfaces. To prevent such contact, it is advisable to wear protective clothing. Skin areas that have been exposed to wet cement or concrete, either directly or through saturated clothing, should be thoroughly washed with water.

PORTLAND CEMENT ASSOCIATION

An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through scientific research, engineering field work, and market development.

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APPENDIX 2

STANDARD DESIGNS OF FARM IRRIGATION STRUCTURES

U.S. Department of Agriculture
Soil Conservation Service

Index - Appendix 2

Plate

1 Steel division box.
2 Concrete trapezoidal division box.
3 Combination pump outlet and division box.
4 Trapezoidal chute drop, 0.30 m (1 '-0") drop.
5 Trapezoidal chute drop, 0.61 m (2 '-0") drop.
6 Trapezoidal chute drop, 0.91 m (3 '-0") drop.
7 Concrete vertical drop, 0.15 m (0 '-6") drop.
8 Concrete vertical drop, 0.30 m (1 '-0") drop.
9 Concrete vertical drop, 0.46 m (1 '-6") drop.
10 Concrete vertical drop, 0.61 m (2 '-0") drop.
11 Concrete block drive-through irrigation drop, 0.30 m (1 '-0") drop.
12 Concrete drive-through irrigation drop, 0.30 m (1 '-0") drop.
13 Vertical wood drop, 0.30 - 0.61 m (1 '-0" to 2 '-0") drop.
14 Pipe drop, 0.31 m (12") water depth.
15 Pipe drop with check inlet, 0.31 m (12") water depth.
16 Pipe drop, 0.38 m (15") water depth.
17 Concrete check.
18 Concrete turnout.
19 Pipe turnout.
20 Siphon inlet or outlet.
21 Gravity inlet for buried pipelines.
22 Gravity inlet for underground pipe.
23 Inlet structure for irrigation pipelines.
24 High head pump stand for concrete pipe.
25 Low head pump stand for concrete pipe.
26 Pipe sand trap for concrete pipeline.
27 Overflow gate stand for pipelines.
28 Float valve stands for pipelines.
29 Alfalfa valve outlet for pipelines.
30 Orchard valve outlet for pipelines.
31 Irrigation water desilting box and trash screen.
32 Irrigation water trash screen.
33 Vent for concrete pipelines.

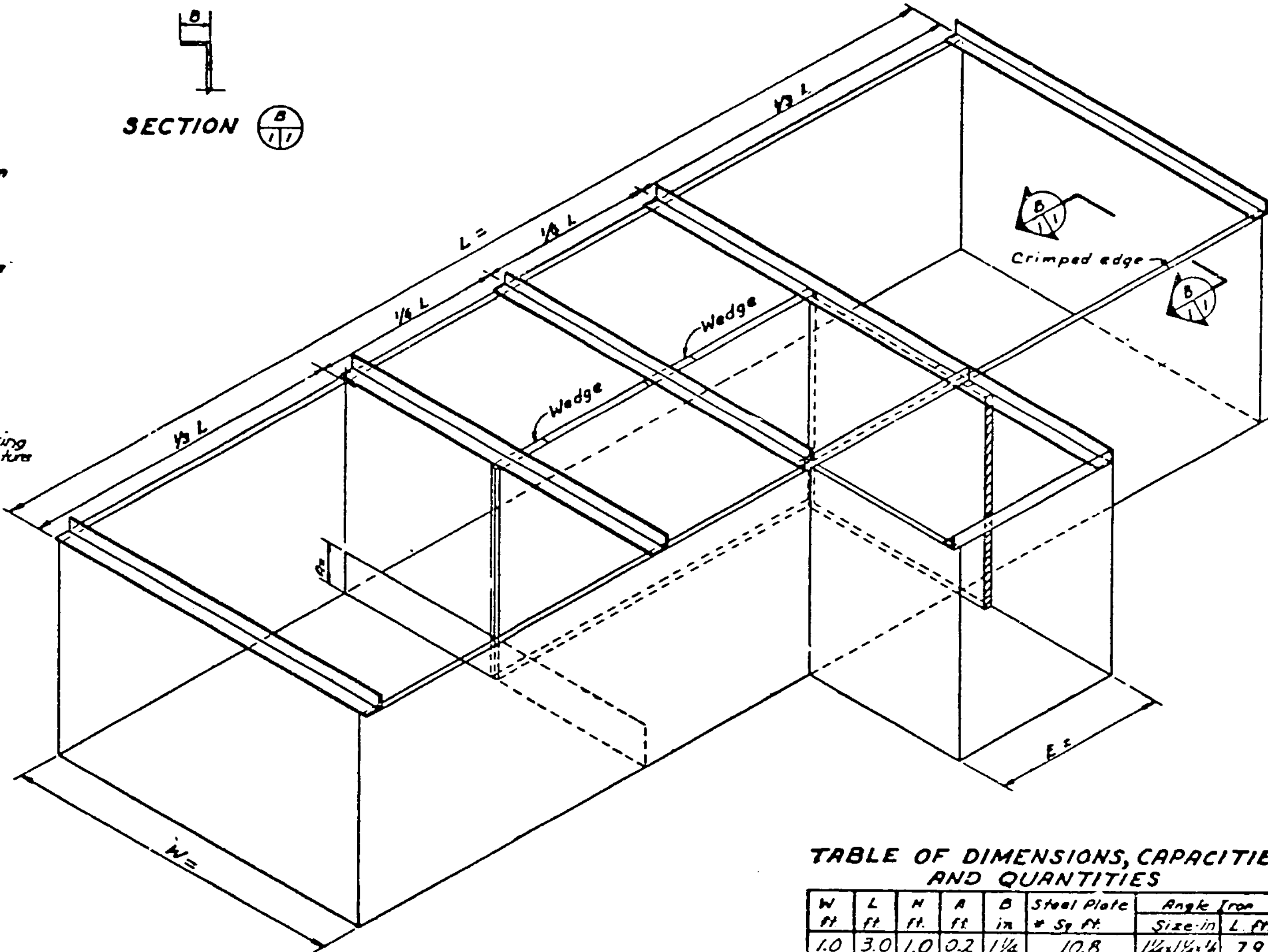
<u>*Dimensional Conversions*</u>

____meters (m) = 0.30 times (×) ____ feet (ft ') = 0.025 × ____inches (in")
____centimeters (cm) = 30.48 × ____ft = 2.54 × in.
____m^2 = 0.093 × ____ft^2
____m^3 = 0.028 × ____ft^3
____m^3 = 0.76 × ____yd^3 (cubic yard) - 1 yard = 3 ft = 36 in.
____m^3/s = 0.000063 × ____GPM (gallons per minute)
____m^3/s = 0.028 × ____cfs (cubic feet per second)

SECTION B/1

NOTES:

1. Wingwalls and cut-off may be used on each end and side outlet.
2. The box can be constructed to divide left, right or both by adding another wooden dividing board.
3. For boxes 9.0' or 12.0' long, place angle iron on the sides and bottom of both ends and in the middle in addition to that shown on the top.
4. Steel shall conform to the requirements of Construction Specifications 81, Metal Fabrication and Installation.
5. <u>STANDARDIZED DESIGNS</u> - <u>Must Be Adapted To The Specific Site</u>.
6. The structure shall conform to Engineering Standard and Specifications 587, Structure for Water Control.



ISOMETRIC VIEW
No Scale

TABLE OF DIMENSIONS, CAPACITIES AND QUANTITIES

W ft	L ft	H ft.	A ft	B in	Steel Plate * Sq ft.	Angle Iron Size-in	Angle Iron L. ft.
1.0	3.0	1.0	0.2	1¼	10.8	1¼×1¼×⅛	7.9
2.0	6.0	1.5	0.3	1¼	33.2	1¼×1¼×¼	13.4
3.0	9.0	2.0	0.4	2	71.5	2×2×¼	47.2
4.0	12.0	2.5	0.5	2	121.2	2×2×¼	60.7

* Minimum steel plate thickness = 3/16"

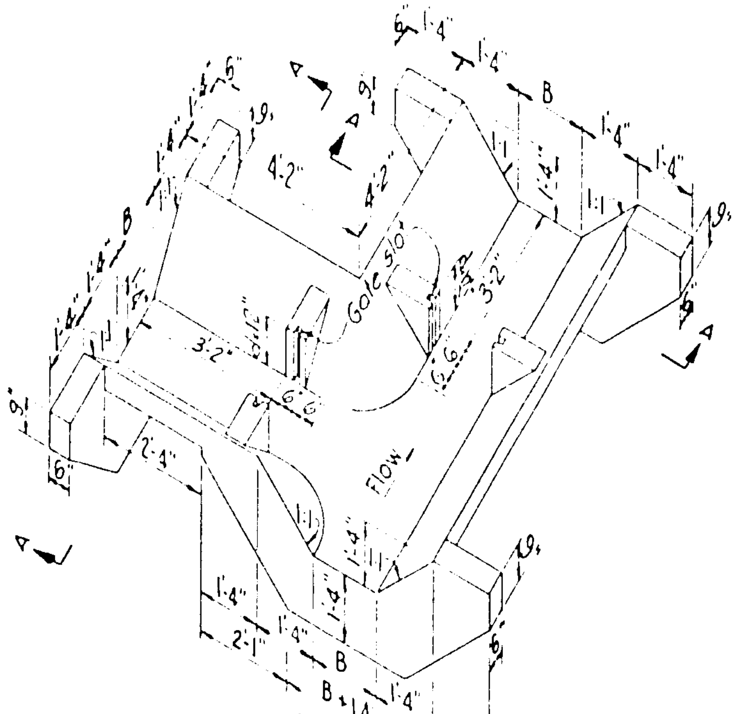
STEEL DIVISION BOX

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

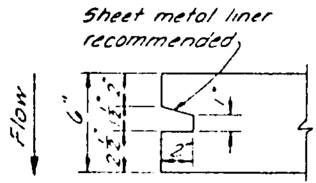
CO-35P-21

Rev. 5-80

Plate 1



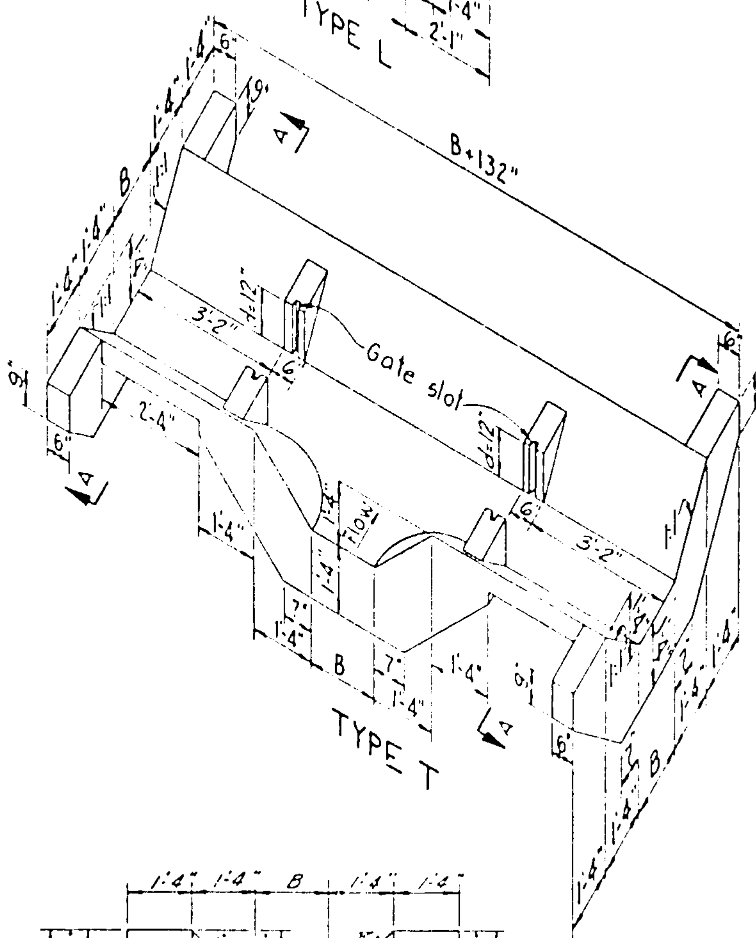
TYPE L



DETAIL OF GATE SLOT

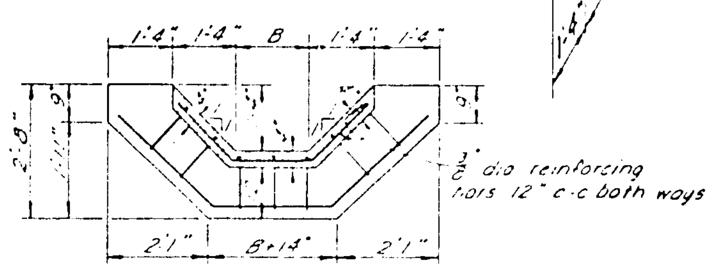
TABLE OF QUANTITES
FOR DIVISION BOXES

B In	TYPE L STRUCTURE Concrete Cu Yd	TYPE L STRUCTURE Steel Lin Ft	TYPE T STRUCTURE Concrete Cu Yd	TYPE T STRUCTURE Steel Lin Ft
12	1.30	174	1.29	168
18	1.44	205	1.42	198
24	1.57	216	1.56	209



TYPE T

Notes
1 Reinforcement to be placed in center of slabs and walls.
2 6"×6" No 10 wire mesh may be used in place of ⅜" dia. reinforcing bars
3 Nomenclature
d = depth of water in ditch
B = bottom width of structure channel



SECTIONAL ELEVATION A-A

CONCRETE TRAPEZOIDAL DIVISION BOX
d=12", B=12", 1'-6", & 2'-0"

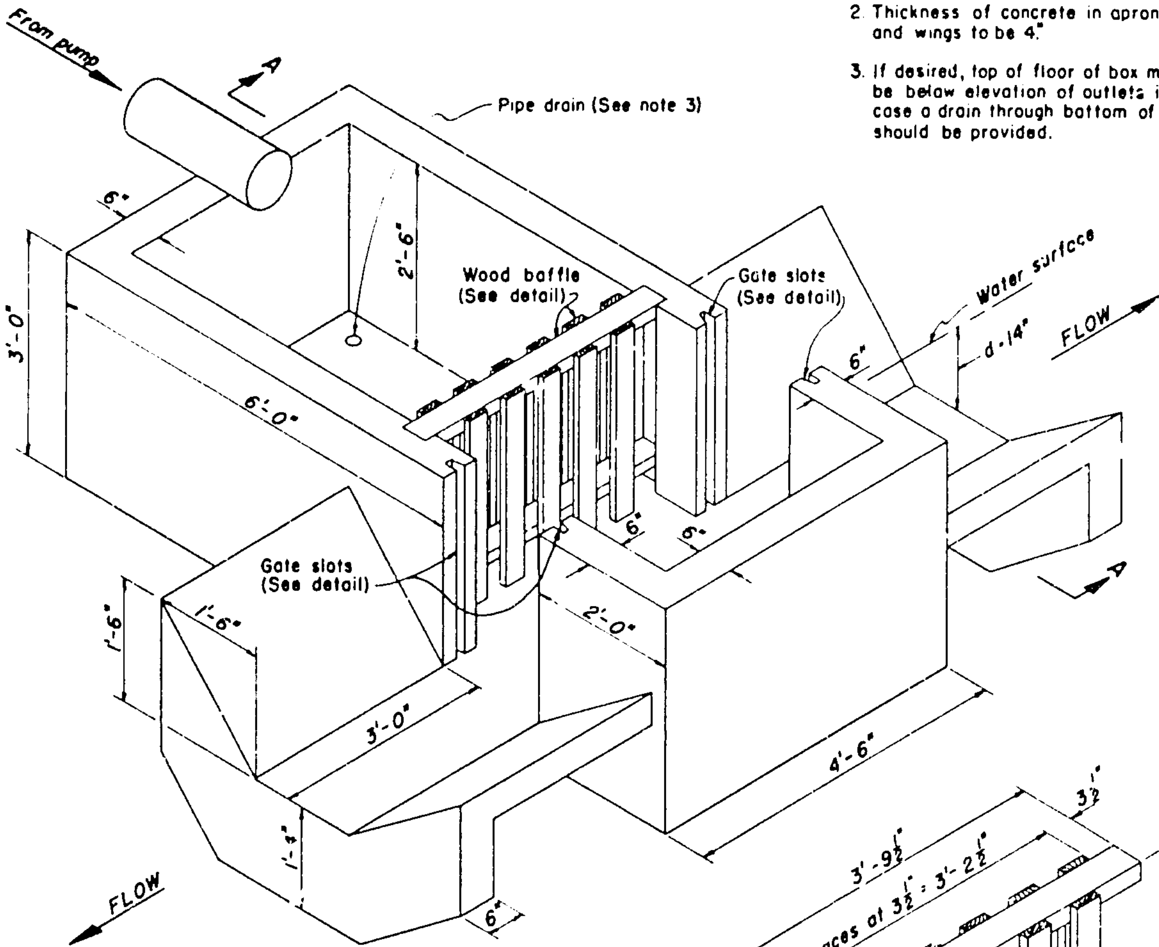
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Compiled	Checked	Date	Drawing No.
		1-64	5,0-19,000 12-1

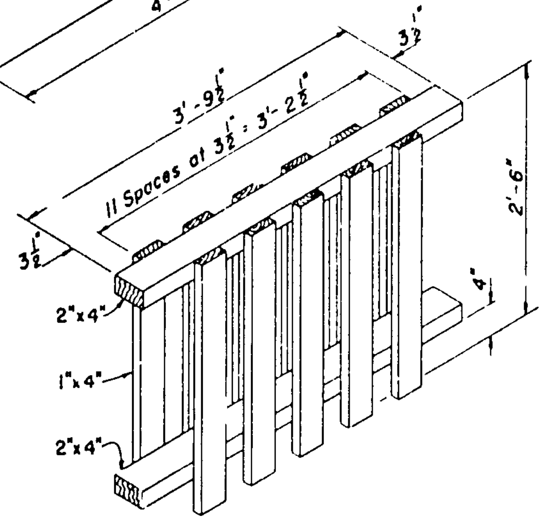
Note: 1. Place 6"x6" No. 10 wire mesh at center of all sections. Lap mesh 6" at all joints.

2. Thickness of concrete in apron floor and wings to be 4".

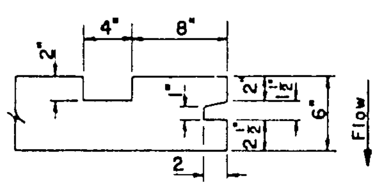
3. If desired, top of floor of box may be below elevation of outlets in which case a drain through bottom of floor should be provided.



ISOMETRIC



DETAIL OF WOOD BAFFLE



DETAIL OF GATE SLOT AND BAFFLE SLOT

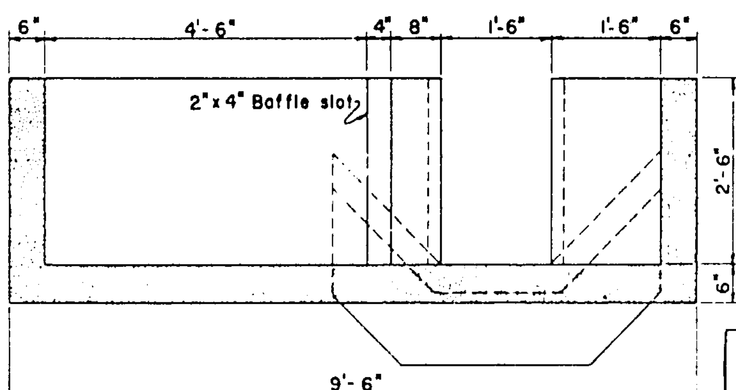
TABLE OF QUANTITIES

LUMBER L. FT.		WIRE MESH	CONCRETE
1"x 4"	2"x 4"	Sq. Ft.	Cu. Yds.
27.5'	7.6'	163	2.5
		* Without aprons	
		117	1.9

* In cohesive soil, when outlets are on opposite sides, aprons may be omitted.

d = Depth of flow in ditch
f = Freeboard from water surface to top of ditch bank

d = 14"
Capacity 1050 - 1600 G. P. M.

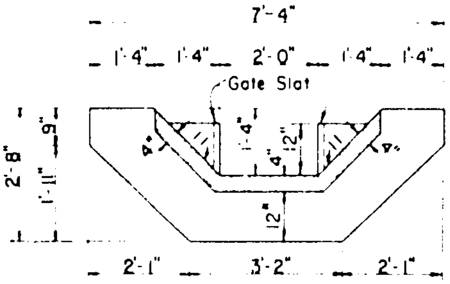


SECTION A-A

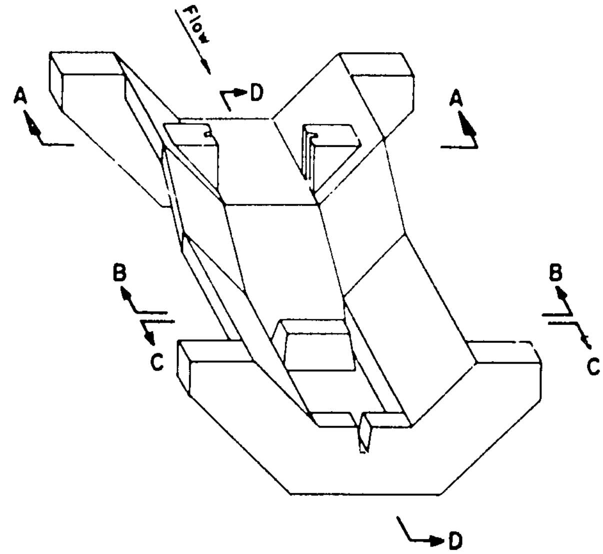
COMBINATION PUMP OUTLET AND DIVISION BOX

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

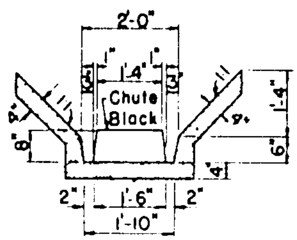
COMPILED	CHECKED	DATE	DRAWING NO.
		6-8-59	7-L-36-23



SECTIONAL ELEVATION A-A

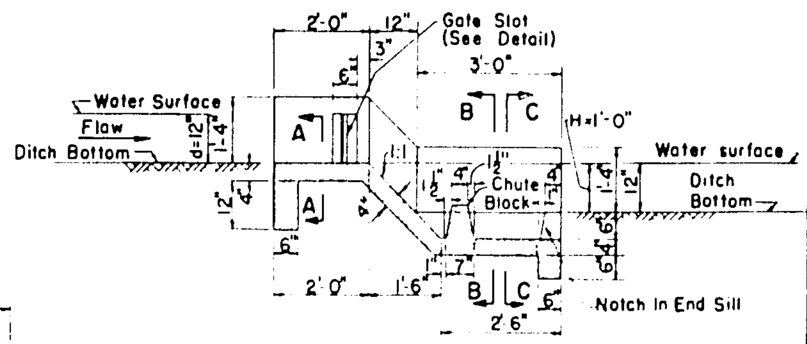


OBLIQUE VIEW

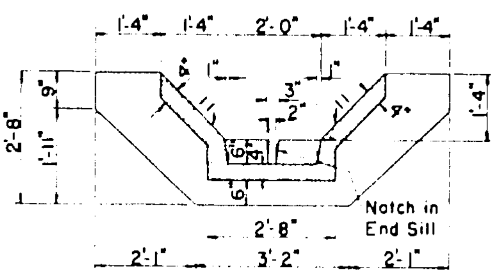


SECTION B-B

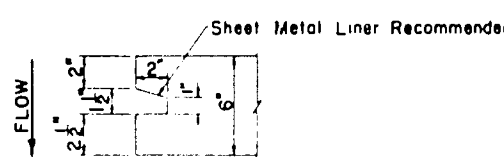
(DETAIL OF CHUTE BLOCK)



SECTIONAL ELEVATION D-D



SECTIONAL ELEVATION C-C



PLAN SHOWING GATE SLOT DETAIL

TABLE OF QUANTITIES

ITEM	AMOUNT
CONCRETE	0.86 CU.YDS.

NOMENCLATURE

d - DEPTH OF WATER IN DITCH

H - HEIGHT OF FALL IN WATER SURFACE

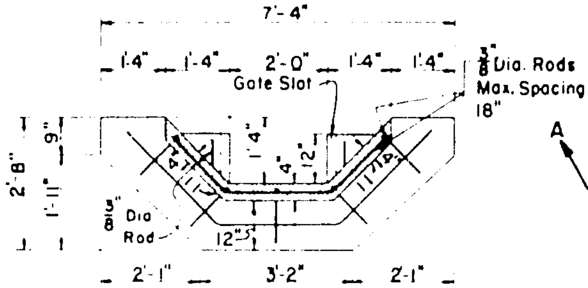
Q = 6.0 c.f.s.

TRAPEZOIDAL CHUTE DROP

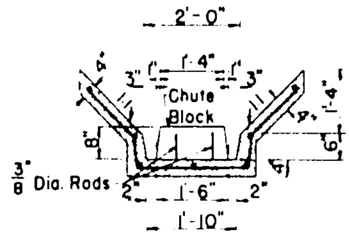
d=12" H=1'-0"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPILED	CHECKED	DATE	DRAWING NO.
		3-60	7-L-36-30

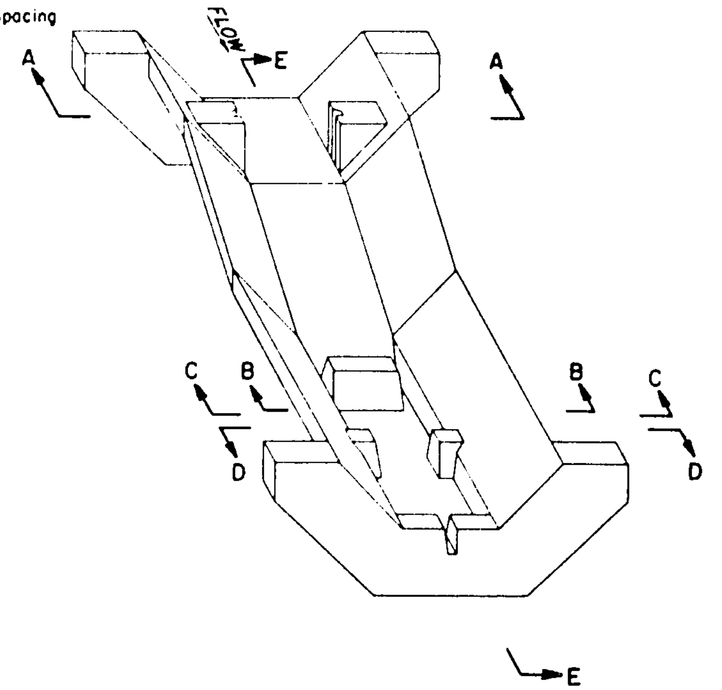


SECTIONAL ELEVATION A-A

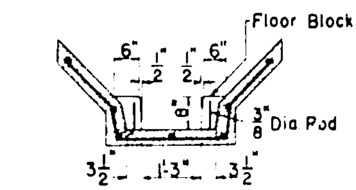


SECTION B-B

(DETAIL OF CHUTE BLOCK)

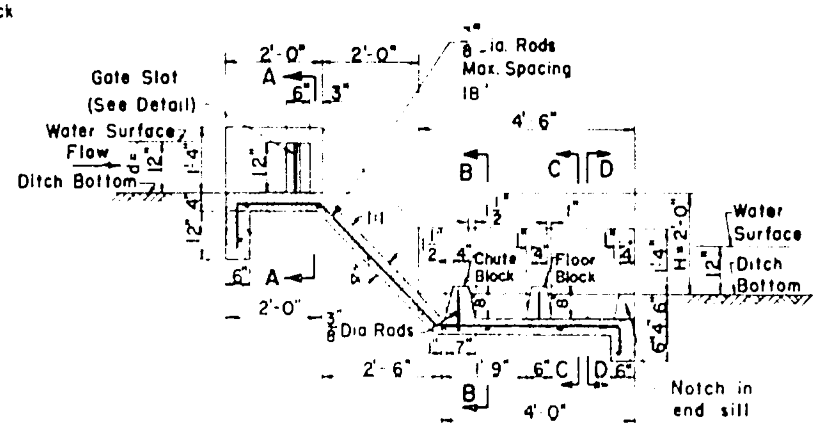


OBLIQUE VIEW

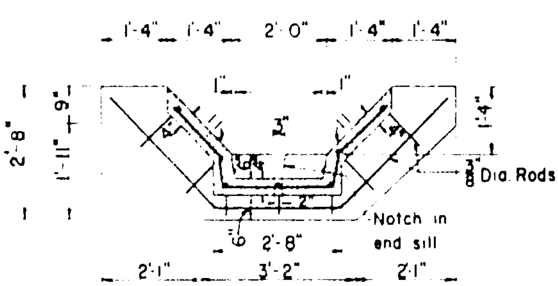


SECTION C-C

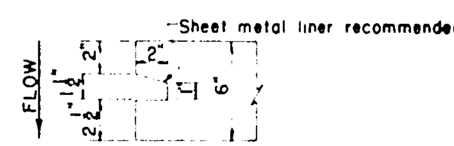
(DETAIL OF FLOOR BLOCKS)



SECTIONAL ELEVATION E-E



SECTIONAL ELEVATION D-D



PLAN SHOWING GATE SLOT DETAIL

NOTE: 6" X 6" NO. 10 WIRE MESH MAY BE USED IN PLACE OF 3/8" DIAMETER RODS.

TABLE OF QUANTITIES

ITEM	DESCRIPTION	AMOUNT
CONCRETE		[illegible] CU. YDS.
REINFORCING STEEL	3/8" DIAMETER RODS	120 LIN. FT.

NOMENCLATURE

d - DEPTH OF WATER IN DITCH

H - HEIGHT OF FALL IN WATER SURFACE

Q = 6.0 c.f.s

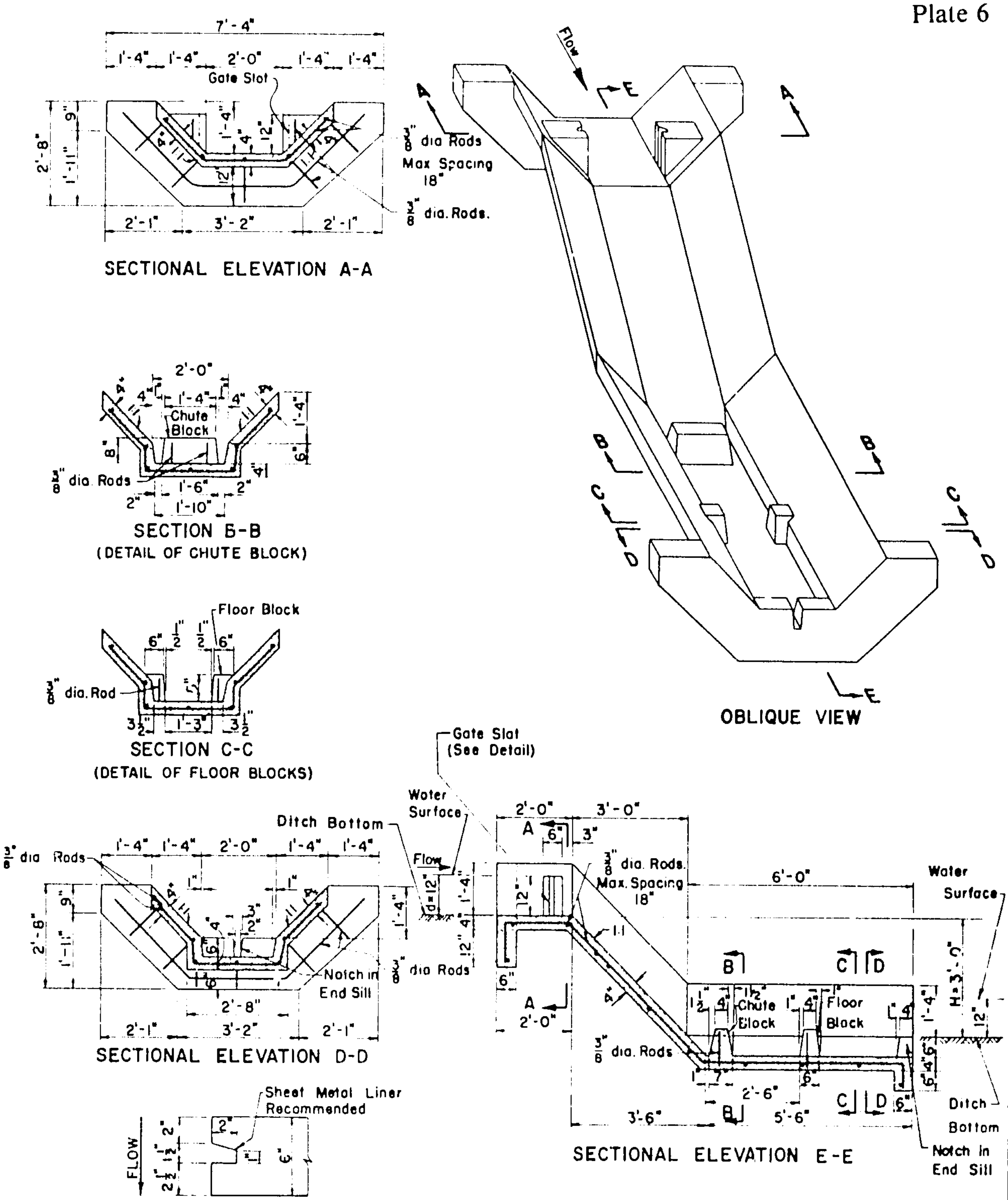
TRAPEZOIDAL CHUTE DROP

d = 12" H = 2'-0"

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

DRAWN	CHECKED	DATE	DRAWING NO.
		2-60	7-L-36-32



NOMENCLATURE

d - DEPTH OF WATER IN DITCH

H - HEIGHT OF FALL IN WATER SURFACE

NOTE: 6" X 6" NO. 10 WIRE MESH MAY BE USED IN PLACE OF 3/8" DIAMETER RODS.

TABLE OF QUANTITIES

ITEM	DESCRIPTION	AMOUNT
CONCRETE		1.33 CU. YDS.
REINFORCING STEEL	3/8" DIAMETER RODS	157 LIN. FT.

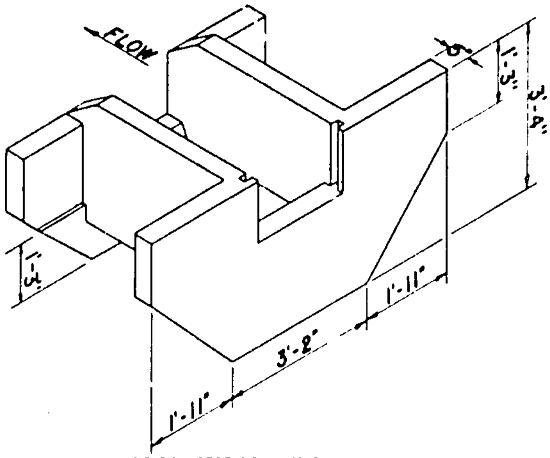
Q = 6.0 cfs.

TRAPEZOIDAL CHUTE DROP

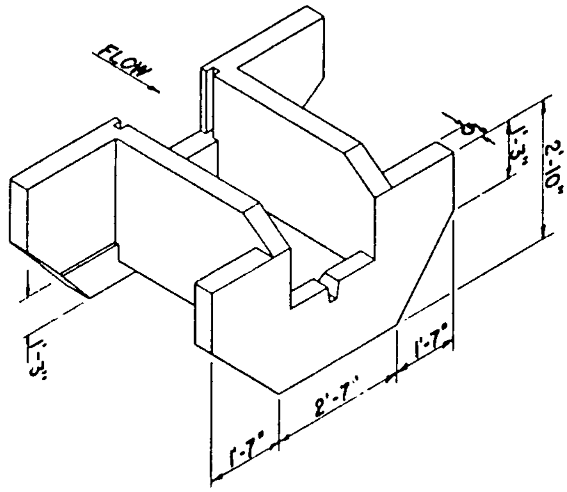
d = 12" H = 3'-0"

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

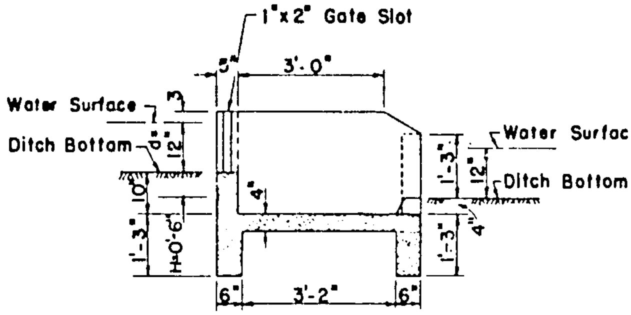
COMPILED	CHECKED	DATE	DRAWING NO.
		3-60	7-L-36-34



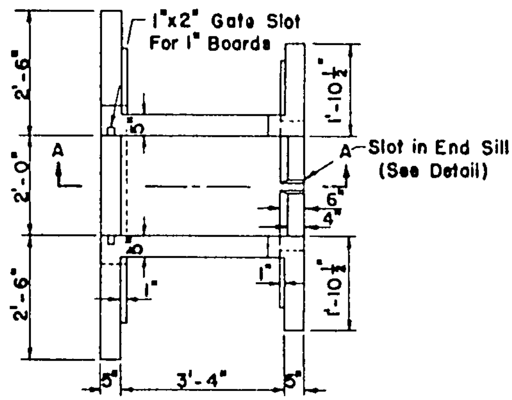
ISOMETRIC VIEW
(LOOKING DOWNSTREAM)



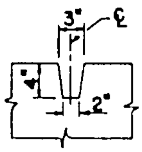
ISOMETRIC VIEW
(LOOKING UPSTREAM)



SECTIONAL ELEVATION A-A



PLAN



ELEVATION
(DETAIL OF SLOT IN END SILL)

NOTES

THE CONCRETE FOOTINGS FOR UPSTREAM WALL AND DOWNSTREAM WALL SHALL BE POURED AGAINST CONSOLIDATED MATERIAL. THE THICKNESS OF THE FOOTINGS SHALL NOT BE LESS THAN SIX INCHES.

THE THICKNESS OF THE CONCRETE IN THE FORMED WALLS SHALL NOT BE LESS THAN FIVE INCHES.

THE THICKNESS OF THE CONCRETE IN THE FLOOR SLAB SHALL NOT BE LESS THAN FOUR INCHES.

TABLE OF QUANTITIES

ITEM	AMOUNT
CONCRETE	0.78 CU.YDS.

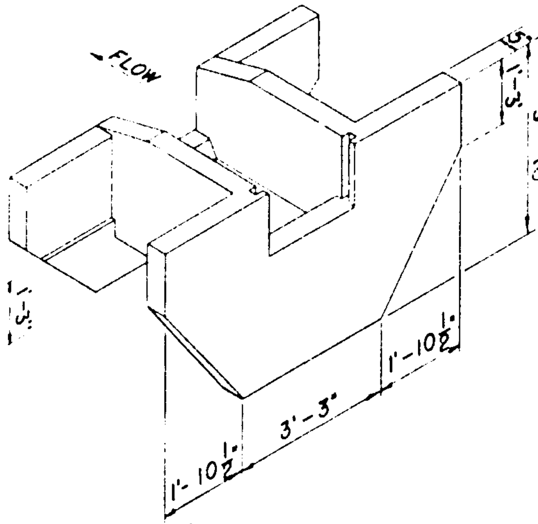
Q = 6.0 cfs

CONCRETE VERTICAL DROP
FOR NONCOHESIVE SOILS

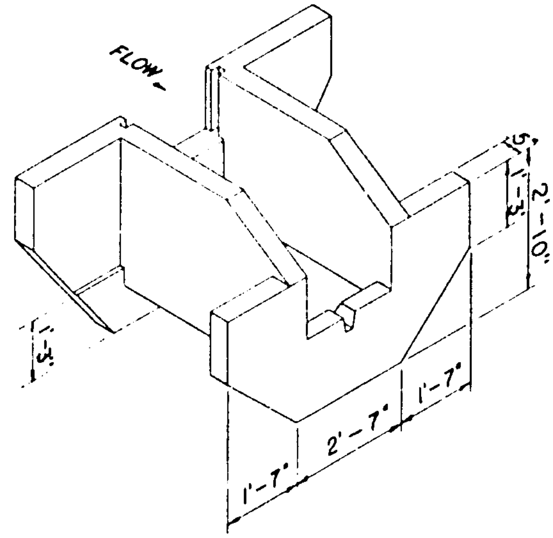
d = 12" H = 0'-6"

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

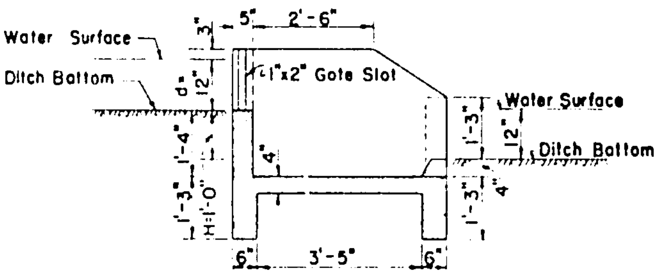
DRAWN	CHECKED	DATE	DRAWING NO.
		12-59	7-L-36-35



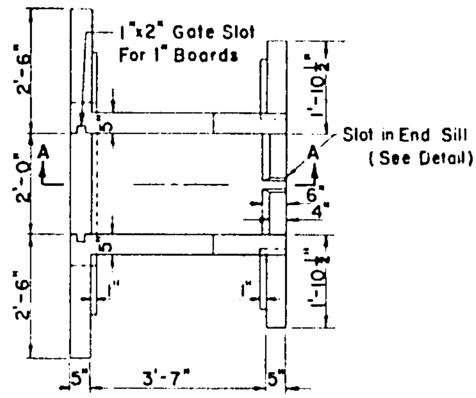
ISOMETRIC VIEW
(LOOKING DOWNSTREAM)



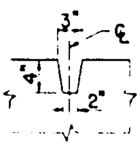
ISOMETRIC VIEW
(LOOKING UPSTREAM)



SECTIONAL ELEVATION A-A



PLAN



ELEVATION
(DETAIL OF SLOT IN END SILL)

NOTES

THE CONCRETE FOOTINGS FOR UPSTREAM WALL AND DOWNSTREAM WALL SHALL BE POURED AGAINST CONSOLIDATED MATERIAL. THE THICKNESS OF THE FOOTINGS SHALL NOT BE LESS THAN SIX INCHES.

THE THICKNESS OF THE CONCRETE IN THE FORMED WALLS SHALL NOT BE LESS THAN FIVE INCHES.

THE THICKNESS OF THE CONCRETE IN THE FLOOR SLAB SHALL NOT BE LESS THAN FOUR INCHES.

TABLE OF QUANTITIES

ITEM	AMOUNT
CONCRETE	0.60 CU.YDS.

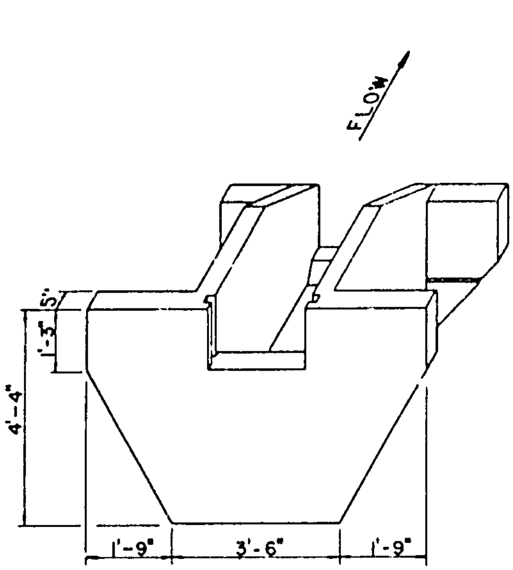
Q = 60 cfs

CONCRETE VERTICAL DROP
FOR NONCOHESIVE SOILS

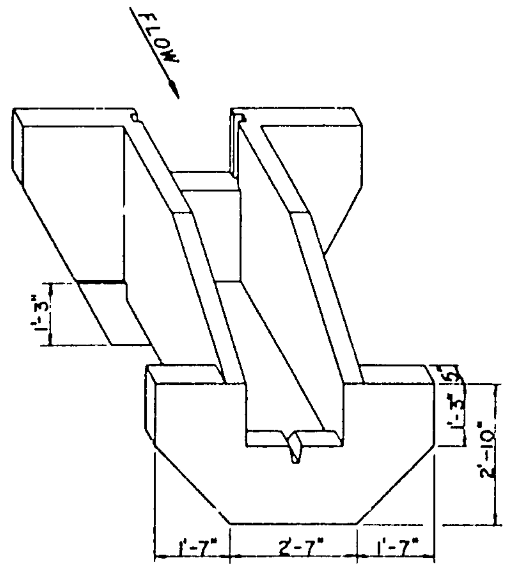
d = 12" H = 1'-0"

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

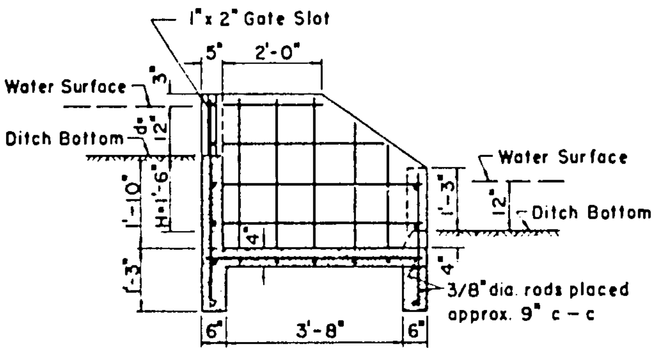
12-59 7-L-36-36



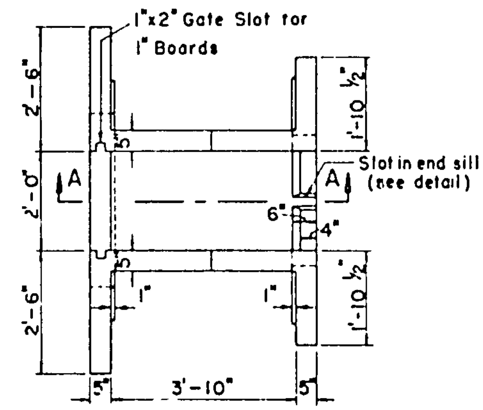
OBLIQUE VIEW
(LOOKING DOWNSTREAM)



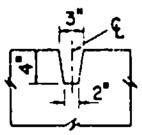
OBLIQUE VIEW
(LOOKING UPSTREAM)



SECTIONAL ELEVATION A-A



PLAN



ELEVATION
(DETAIL OF SLOT IN END SILL)

NOTES

THE CONCRETE FOOTINGS FOR UPSTREAM WALL AND DOWNSTREAM WALL SHALL BE POURED AGAINST CONSOLIDATED MATERIAL. THE THICKNESS OF THE FOOTINGS SHALL NOT BE LESS THAN SIX INCHES.

THE THICKNESS OF THE CONCRETE IN THE FORMED WALLS SHALL NOT BE LESS THAN FIVE INCHES.

THE THICKNESS OF THE CONCRETE IN THE FLOOR SLAB SHALL NOT BE LESS THAN FOUR INCHES.

REINFORCEMENT STEEL IN FLOOR, UPSTREAM FOOTING, AND DOWNSTREAM FOOTING SHALL BE 3/8" DIAMETER RODS PLACED AT CENTER OF SLAB AND SPACED APPROX. 9" CENTER TO CENTER BOTH WAYS.

REINFORCEMENT STEEL IN FORMED WALLS SHALL BE 3/8" DIAMETER RODS PLACED AT CENTER OF WALL AND SPACED APPROX. 9" CENTER TO CENTER BOTH WAYS. ALL VERTICAL RODS IN THE FORMED WALLS SHALL EXTEND FROM THE GROUND UP. THESE RODS ARE TO BE PLACED ABOUT 2" FROM THE DIRT SIDE OF THE WALL AND THREE INCHES FROM THE AIR OR WATER SIDE. HORIZONTAL RODS IN FORMED WALLS SHALL BE PLACED ABOUT 3" FROM BOTTOM OF FOOTING AND SPACED APPROX. 9" CENTER TO CENTER UPWARD FROM BOTTOM RODS, AS SHOWN IN THE ELEVATION SECTION A-A.

TABLE OF QUANTITIES

ITEM	DESCRIPTION	AMOUNT
CONCRETE		1.02 CU.YD.
REINFORCING STEEL	3/8" DIAMETER RODS	164.5 LIN.FT.

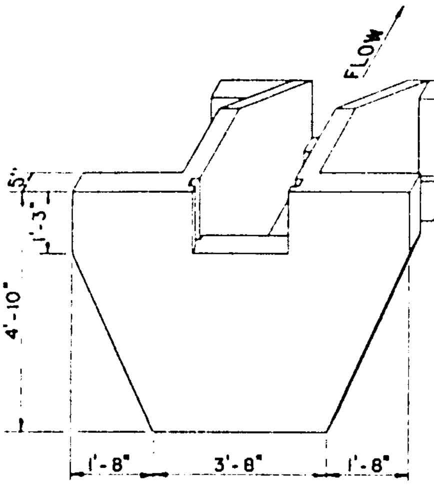
Q = 6.0 cfs

CONCRETE VERTICAL DROP
FOR NONCOHESIVE SOILS

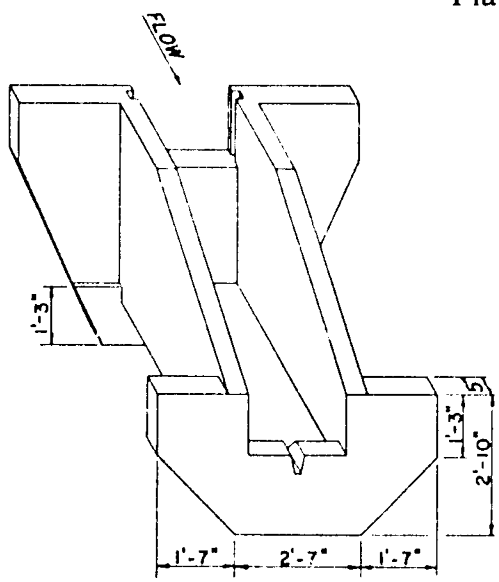
d = 12" H = 1'-6"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

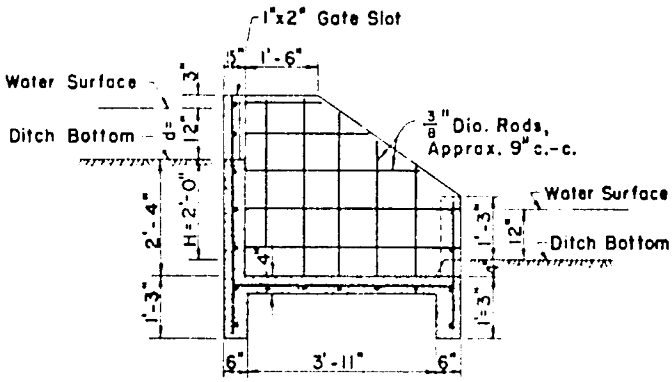
COMPILED	CHECKED	DATE	DRAWING NO.
		12-59	7-L-36-37



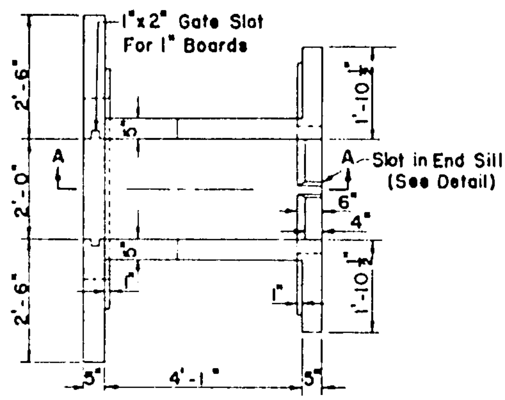
OBLIQUE VIEW
(LOOKING DOWNSTREAM)



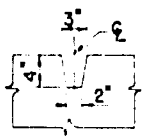
OBLIQUE VIEW
(LOOKING UPSTREAM)



SECTIONAL ELEVATION A-A



PLAN



ELEVATION
(DETAIL OF SLOT IN END SILL)

NOTES

THE CONCRETE FOOTINGS FOR UPSTREAM WALL AND DOWNSTREAM WALL SHALL BE POURED AGAINST CONSOLIDATED MATERIAL. THE THICKNESS OF THE FOOTINGS SHALL NOT BE LESS THAN SIX INCHES.

THE THICKNESS OF THE CONCRETE IN THE FORMED WALLS SHALL NOT BE LESS THAN FIVE INCHES.

THE THICKNESS OF THE CONCRETE IN THE FLOOR SLAB SHALL NOT BE LESS THAN FOUR INCHES.

REINFORCEMENT STEEL IN FLOOR, UPSTREAM FOOTING, AND DOWNSTREAM FOOTING SHALL BE 3/8" DIAMETER RODS PLACED AT CENTER OF SLAB AND SPACED APPROX. 9" CENTER TO CENTER BOTH WAYS.

REINFORCEMENT STEEL IN FORMED WALLS SHALL BE 3/8" DIAMETER RODS PLACED AT CENTER OF WALL AND SPACED APPROX. 9" CENTER TO CENTER BOTH WAYS. ALL VERTICAL RODS IN THE FORMED WALLS SHALL EXTEND FROM THE GROUND UP. THESE RODS ARE TO BE PLACED ABOUT 2" FROM THE DIRT SIDE OF THE WALL AND THREE INCHES FROM THE AIR OR WATER SIDE. HORIZONTAL RODS IN FORMED WALLS SHALL BE PLACED ABOUT 3" FROM BOTTOM OF FOOTING AND SPACED APPROX. 9" CENTER TO CENTER UPWARD FROM BOTTOM RODS, AS SHOWN IN THE ELEVATION SECTION A-A.

TABLE OF QUANTITIES

ITEM	DESCRIPTION	AMOUNT
CONCRETE		1.13 CU. YDS.
REINFORCING STEEL	3/8" DIAMETER RODS	180.0 LIN. FT.

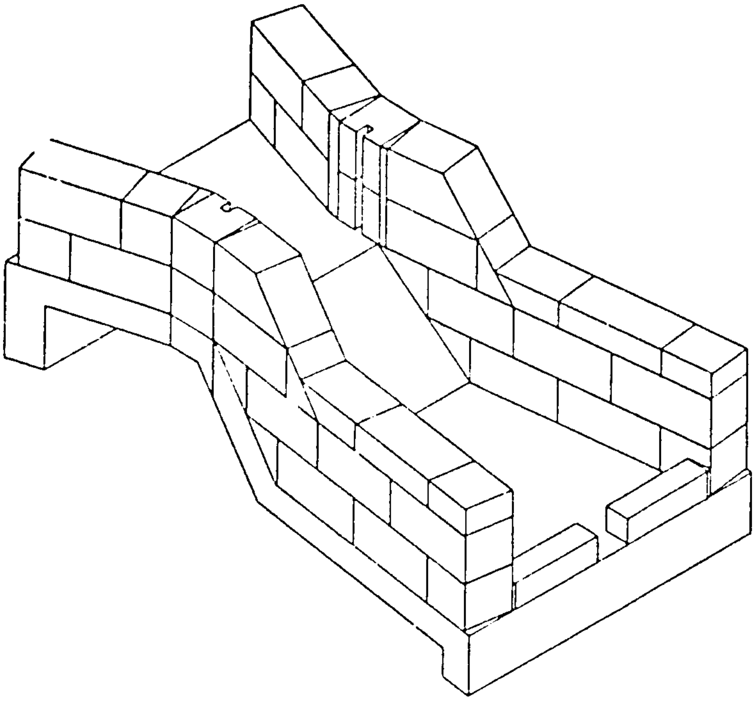
Q = 6.0 c.f.s.

CONCRETE VERTICAL DROP
FOR NONCOHESIVE SOILS

d = 12" H = 2'-0"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

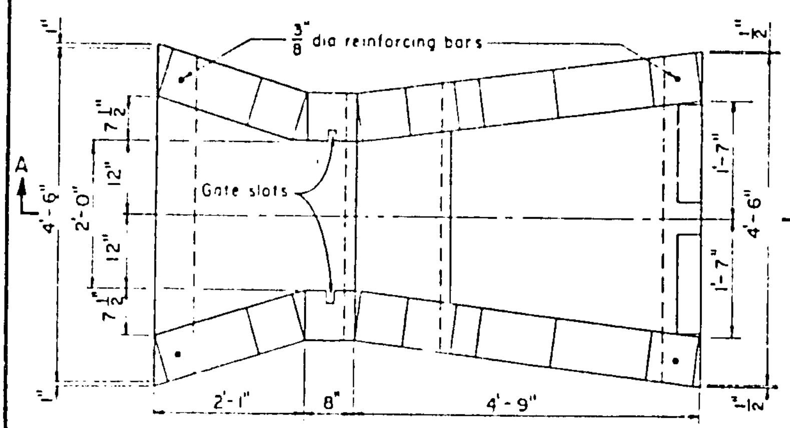
COMPUTED	CHECKED	DATE	DRAWING NO.
		12-59	7-L-36-38



ISOMETRIC VIEW

BILL OF MATERIAL

MARK	DESCRIPTION	UNIT	QUAN.
1	8"x 8"x 16", corner block.	Ea.	4
2	8"x 8"x 16", stretcher block	Ea.	12
3	8"x 3"x 8", corner block.	Ea.	6
4	8"x 8"x 8", corner block with gate slot.	Ea.	4
5	Not used		
6	4"x 8"x 16", stretcher block	Ea.	2
7	8"x 8"x 16", stretcher block with a 45° end cut	Ea.	2
8	1/4 of an 8"x 8"x 16", stretcher block with a 45° end cut.	Ea.	6
9	5/8 of a 4"x 8"x 16", stretcher block with a 45° end cut.	Ea.	2
10	4"x 4"x 16", corner block may replace usual concrete end sill.	Ea.	2
11	4"x 8"x 8", corner block.	Ea.	2
	CONCRETE		
	with concrete end sill	Cu.yd.	0.62
	with mark 10 blocks	Cu.yd.	0.60
	MORTAR	Cu.yd.	0.05
	GROUT	Cu.yd.	0.26
	BLOCK WEB MESH	Lin.ft.	32
	REINFORCEMENT STEEL 3/8" dia bars.	Lin.ft.	109

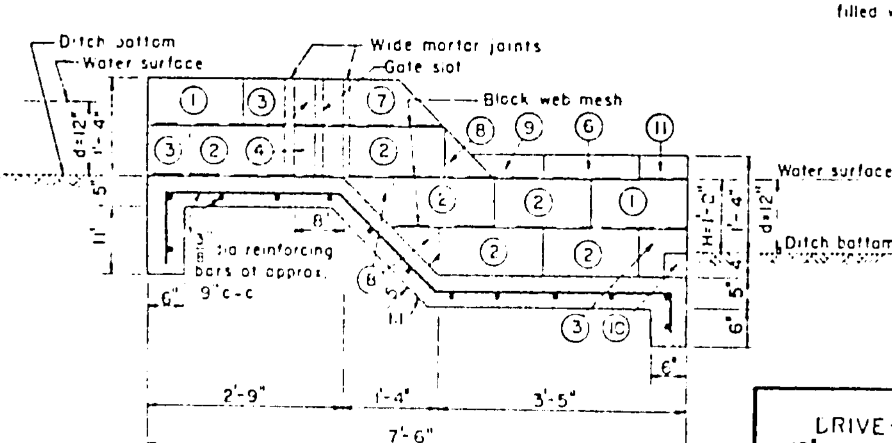


PLAN

NOTES

Concrete block walls to be reinforced by placing high tension steel wire mesh, No. 9 wire, similar to Carter-Waters Blok-Mesh in horizontal block joints as shown in Sectional Elevation A-A.

The joint thickness between concrete blocks shall be about 1/4 in. The concrete blocks shall be laid with staggered vertical joints as shown on the plans. The openings in the blocks shall be aligned vertically and filled with concrete grout.



SECTIONAL ELEVATION A-A

Q = 6.0 c.f.s.

CONCRETE BLOCK
DRIVE-THRU IRRIGATION DROP

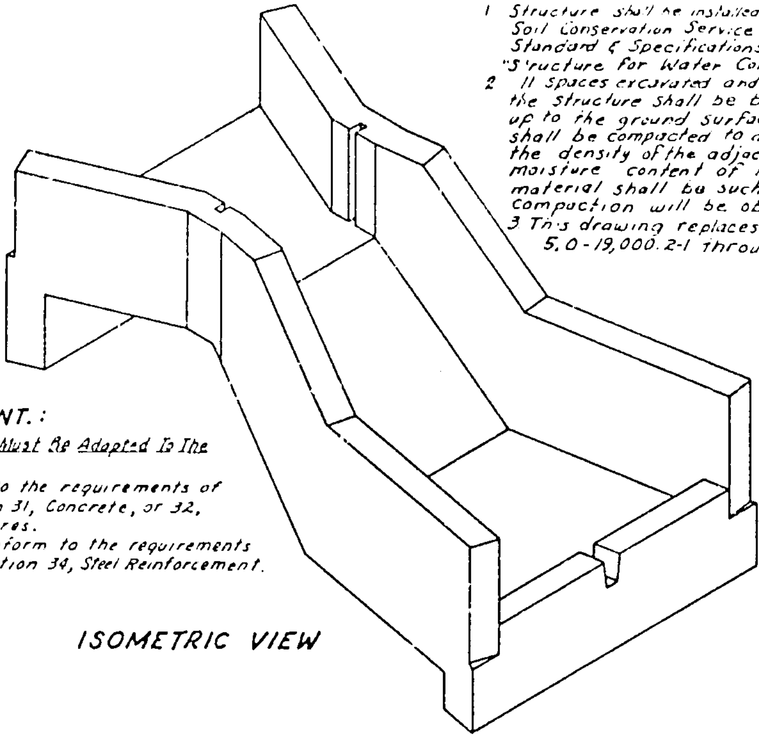
d = 12" H = 1'-0"

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

1-64 | 5-0-19,000 3-1

NOTES:

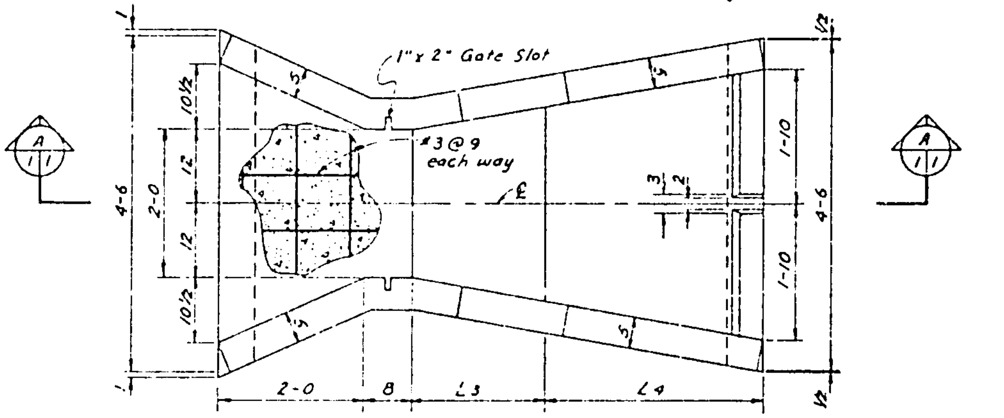
1. Structure shall be installed in accordance with Soil Conservation Service Engineering Standard & Specifications No. Colo-387 "Structure for Water Control."
2. All spaces excavated and not occupied by the structure shall be backfilled with earth up to the ground surface. The backfill shall be compacted to a density equal to the density of the adjacent earth. The moisture content of the backfill material shall be such that the required compaction will be obtained.
3. This drawing replaces Drawings No. 5,0-19,000.2-1 through 5,0-19,000.2-4.



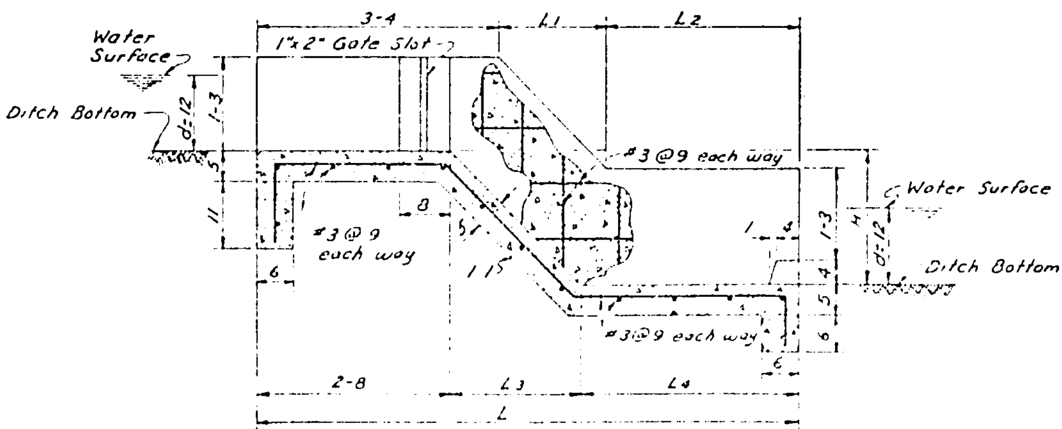
NOTES CONT.:

4. STANDARDIZED DESIGNS - Must Be Adapted To The Specific Site.
5. Concrete shall conform to the requirements of Construction Specification 31, Concrete, or 32, Concrete for Minor Structures.
6. Reinforcing steel shall conform to the requirements of Construction Specification 34, Steel Reinforcement.

ISOMETRIC VIEW



PLAN VIEW



SECTIONAL ELEVATION (A/1 1)

TABLE OF DIMENSIONS, QUANTITIES & CAPACITIES

H ft-in	L ft-in	L1 ft-in	L2 ft-in	L3 ft-in	L4 ft-in	Concrete (Cu yds)	Reinforcing Steel (lbs)	Q cfs
0-6	5-6	0-6	1-8	0-10	2-0	0.72	51	5.0
1-0	6-6	1-0	2-2	1-4	2-6	0.84	56	6.0
1-6	7-6	1-6	2-8	1-10	3-0	0.96	62	6.0
2-0	9-0	2-0	3-8	2-4	4-0	1.11	74	6.0

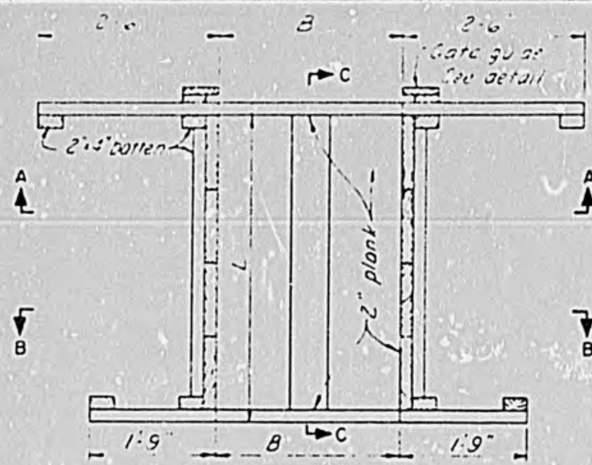
Revised 8-73

CONCRETE DRIVE-THRU IRRIGATION DROP

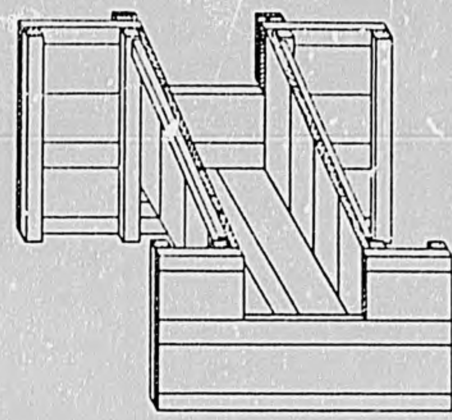
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Lincoln, Nebr 1-64 — State Cons Engr.
RDM 4-69
ARM 4-69
CRP 4-69

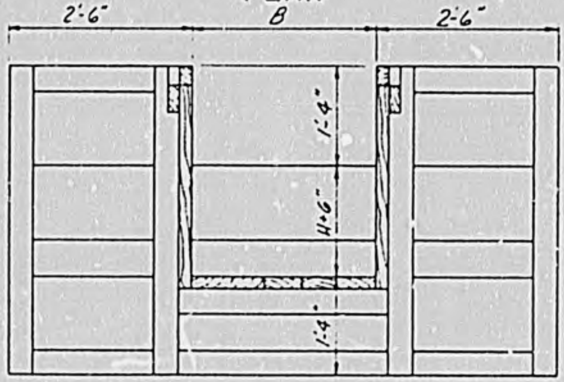
CO-SSP-13



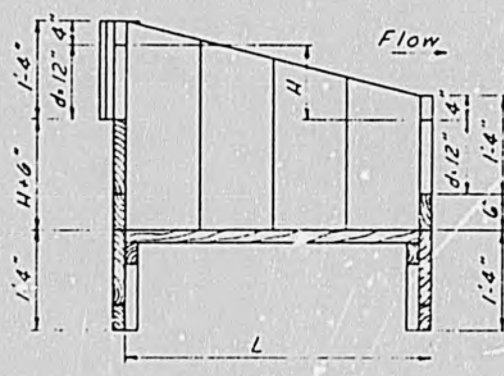
PLAN



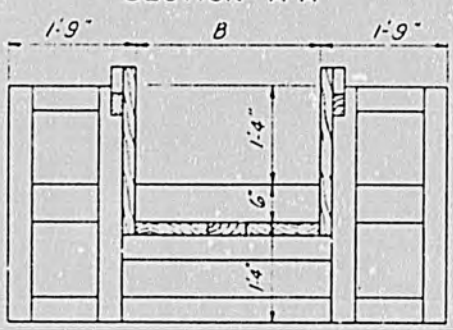
OBLIQUE VIEW



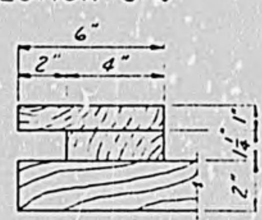
SECTION A-A



SECTION C-C



SECTION B-B



DETAIL OF GATE GUIDE
(OPTIONAL)

Notes

1. All lumber to be pressure treated and secured with cement coated nails
2. B = width of opening
   d = depth of water in ditch
   H = height of fall in water surface
   L = length of apron
   Q = capacity in c.f.s

Dimensions and Capacity				Bill of Material Gate guide not included												
Drop H	Apron Length L	Width B	Capacity Q	2"x12"x16'	2"x12"x14'	2"x12"x12'	2"x6"x14'	2"x6"x12'	2"x6"x10'	2"x6"x8'	2"x6"x6'	2"x4"x16'	2"x4"x14'	2"x4"x12'	2"x4"x10'	Board feet per structure
1'-0"	4'-2"	2'-0"	6.94	1	3		1					1	1	3		174
1'-0"	4'-2"	2'-6"	8.58	2	2				1	1		1	2	2		184
1'-6"	4'-6"	2'-0"	6.94		4	1					1	1	1	3	1	193
1'-6"	4'-6"	2'-6"	8.58	1	4			1				1	2	2	1	208
2'-0"	4'-8"	2'-0"	6.94	2	2	1		1		1		1		3	2	212
2'-0"	4'-8"	2'-6"	8.58	3	2		1	1				1	1	2	2	228

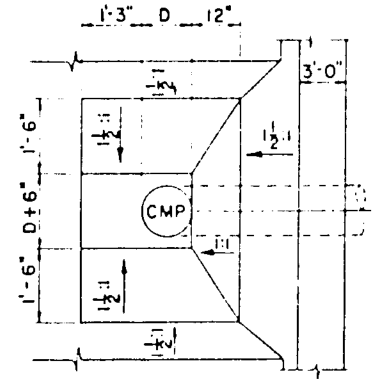
VERTICAL WOOD DROP

d=12" H=1'-0" to 2'-0"

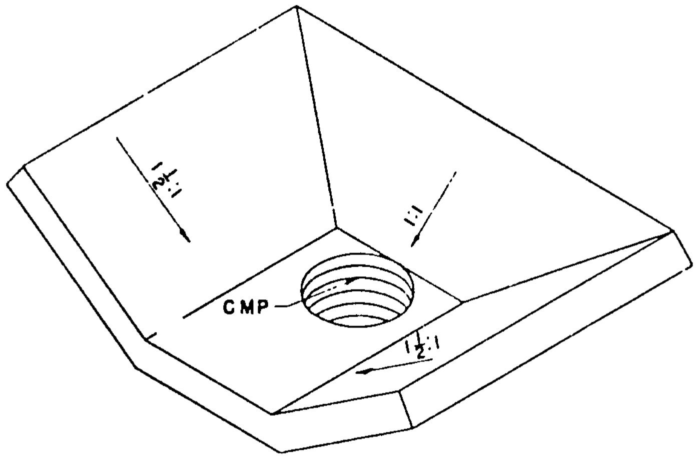
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Compiled	Checked	Date	Drawing No.
		1-64	5,0-19,000.10-2

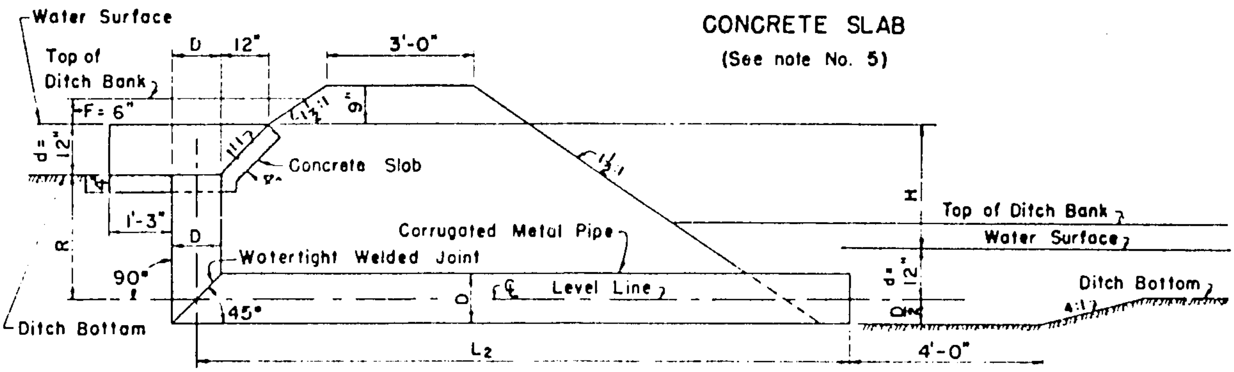
120



PLAN



ISOMETRIC VIEW OF
CONCRETE SLAB
(See note No. 5)



SECTIONAL ELEVATION ON CENTER LINE

CAPACITY AND LENGTHS REQUIRED

PIPE SIZE	D = 10"				D = 12"				D = 15"			
RECOMMENDED DESIGN CAPACITY	1.6 C.F.S.				2.4 C.F.S.				3.7 C.F.S.			
H = R	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$
1'-0"	4.2	2.3	11'-0"	12'-0"	4.3	3.4	11'-0"	12'-0"	4.5	5.5	11'-0"	12'-0"
1'-6"	5.1	2.8	12'-6"	14'-0"	5.2	4.1	12'-6"	14'-0"	5.4	6.6	12'-6"	14'-0"
2'-0"	5.9	3.2	14'-0"	16'-0"	6.0	4.7	14'-0"	16'-0"	6.1	7.5	14'-0"	16'-0"
2'-6"	6.4	3.5	13'-6"	16'-0"	6.6	5.2	13'-6"	16'-0"	6.6	8.1	13'-6"	16'-0"
3'-0"	6.8	3.7	15'-0"	18'-0"	6.6	5.2	15'-0"	18'-0"	6.5	8.1	15'-0"	18'-0"

NOTES

1. SELECT A PIPE SIZE THAT WILL PROVIDE A GREATER CAPACITY THAN IS REQUIRED TO DISCHARGE THE NORMAL STREAM USED WHEN IRRIGATING. TRY TO KEEP THE VELOCITY IN THE PIPE BELOW 3 FPS BASED ON NORMAL IRRIGATING STREAM.
2. WHEN THE CORRUGATED METAL PIPE DROP IS USED AT A DITCH CROSSING, INCREASE WIDTH OF TOP OF [illegible] AND DIMENSION L_2 BY 5'-0".
3. THE DROP (H) FOR ANY SPECIFIC STRUCTURE CAN BE INCREASED 3 INCHES BY PLACING THE TOP OF THE RISER PIPE 3 INCHES BELOW THE TOP OF THE CONCRETE FLOOR OF THE INLET. THE THICKNESS OF THE FLOOR SLAB ADJACENT TO THE PIPE SHOULD BE INCREASED 3 INCHES TO MAKE A WATERTIGHT CONNECTION WITH THE PIPE. THE INLET TO THE PIPE SHOULD BE ROUNDED TO A 3 INCH RADIUS TO SAVE FORMING AND IMPROVE THE EFFICIENCY OF THE INLET.
4. THE DROP STRUCTURE IS FORMED BY CUTTING A STANDARD LENGTH OF CORRUGATED METAL PIPE, WHICH IS MANUFACTURED IN MULTIPLES OF 2 FT. IN LENGTH, ON A 45° ANGLE AND WELDING THE CUT JOINTS TOGETHER TO FORM A 90° BEND. PIPE TO BE 16 GA. CORRUGATED METAL. JOINT BETWEEN HORIZONTAL AND VERTICAL PIECES OF PIPE TO BE BUTT WELDED AND WATERTIGHT.
5. SIX INCH HAND PLACED RIP-RAP MAY BE SUBSTITUTED FOR CONCRETE SLAB.

NOMENCLATURE

d - DEPTH OF WATER IN DITCH
F - FREEBOARD IN DITCH
D - DIAMETER OF PIPE
R - LENGTH OF VERTICAL PIPE ALONG CENTER LINE
L_2 - LENGTH OF HORIZONTAL PIPE ALONG CENTER LINE
V - VELOCITY OF PIPE - FPS.
Q - DISCHARGE THROUGH PIPE - C.F.S.
H = DROP OF WATER SURFACE

TABLE OF
CONCRETE QUANTITIES

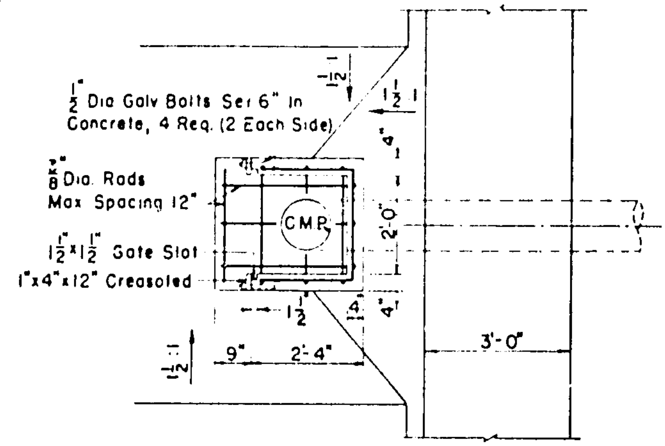
D = 10"	0.25 CU.YDS.
D = 12"	0.26 CU.YDS.
D = 15"	0.29 CU.YDS.

CORRUGATED METAL PIPE DROP

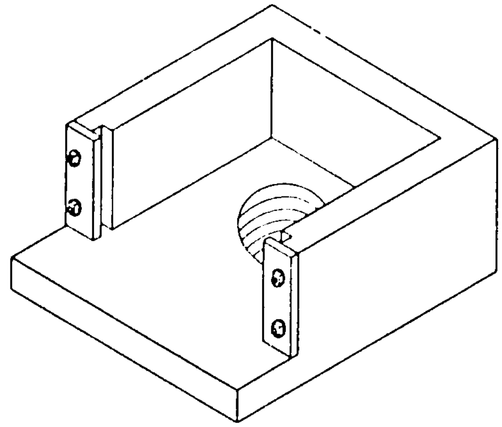
d = 12"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

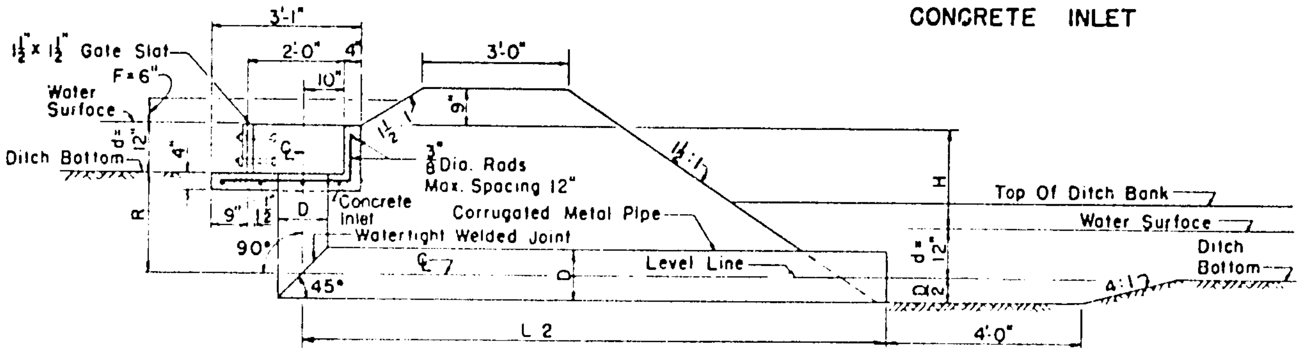
DATE	DRAWING NO.
2-60	7-L-36-24



PLAN



ISOMETRIC VIEW OF CONCRETE INLET



SECTIONAL ELEVATION ON CENTERLINE

CAPACITY AND LENGTHS REQUIRED

PIPE SIZE	D = 10"				D = 12"				D = 15"			
RECOMMENDED DESIGN CAPACITY	1.6 C.F.S.				2.4 C.F.S.				3.7 C.F.S.			
H = R	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$
1'-0"	4.2	2.3	11'-0"	12'-0"	4.3	3.4	11'-0"	12'-0"	4.5	5.5	11'-0"	12'-0"
1'-5"	5.1	2.8	12'-6"	14'-0"	5.2	4.1	12'-6"	14'-0"	5.4	6.6	12'-6"	14'-0"
2'-0"	5.2	3.2	12'-0"	14'-0"	6.0	4.7	12'-0"	14'-0"	6.1	7.5	12'-0"	14'-0"
2'-6"	6.4	3.5	13'-6"	16'-0"	6.6	5.2	13'-6"	16'-0"	6.6	8.1	13'-6"	16'-0"
3'-0"	6.8	3.7	15'-0"	15'-0"	6.6	5.2	15'-0"	18'-0"	6.6	8.1	15'-0"	18'-0"

NOTES

1. SELECT A PIPE SIZE THAT WILL PROVIDE A GREATER CAPACITY THAN IS REQUIRED TO DISCHARGE THE NORMAL STREAM USED WHEN IRRIGATING. TRY TO KEEP THE VELOCITY IN THE PIPE BELOW 3 FPS BASED ON NORMAL IRRIGATION STREAM.
2. WHEN THE CORRUGATED METAL PIPE DROP IS USED AT A DITCH CROSSING, INCREASE WIDTH OF TOP OF DAM AND DIMENSION L_2 BY 8'-0".
3. THE DROP (H) FOR ANY SPECIFIC STRUCTURE CAN BE INCREASED 3 INCHES BY PLACING THE TOP OF THE RISER PIPE 3 INCHES BELOW THE TOP OF THE CONCRETE FLOOR OF THE INLET. THE THICKNESS OF THE FLOOR SLAB ADJACENT TO THE PIPE SHOULD BE INCREASED 3 INCHES TO MAKE A WATERTIGHT CONNECTION WITH THE PIPE. THE INLET TO THE PIPE SHOULD BE ROUNDED TO A 3 INCH RADIUS TO SAVE FORMING AND IMPROVE THE EFFICIENCY OF THE INLET.
4. THE DROP STRUCTURE IS FORMED BY CUTTING A STANDARD LENGTH OF CORRUGATED METAL PIPE, WHICH IS MANUFACTURED IN MULTIPLES OF 2 FT. IN LENGTH, ON A 45° ANGLE AND WELDING THE CUT JOINTS TOGETHER TO FORM A 90° BEND. PIPE TO BE 16 GA. CORRUGATED METAL. JOINT BETWEEN HORIZONTAL AND VERTICAL PIECES OF PIPE TO BE BUTT WELDED AND WATERTIGHT.

TABLE OF QUANTITIES

ITEM	DESCRIPTION	AMOUNT		
		D = 10"	D = 12"	D = 15"
CONCRETE		0.16 CU.YDS.	0.17 CU.YDS.	0.17 CU.YDS.
REINFORCING STEEL	3/8" DIA. RODS	35 LIN.FT.	35 LIN.FT.	35 LIN.FT.
1" X 4" X 12"	CREOSOTED BOARDS	2	2	2
GALV. BOLTS	1/2" DIA. 8" LONG	4	4	4
GALV. WASHERS	1/2" DIA.	4	4	4

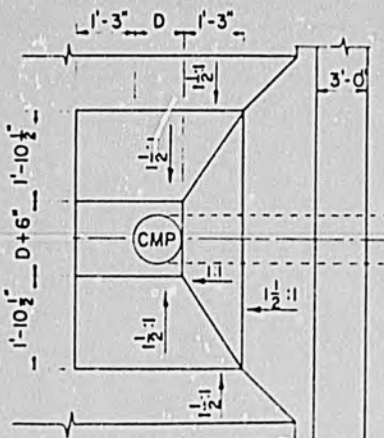
NOMENCLATURE

d - DEPTH OF WATER IN DITCH
F - FREEBOARD IN DITCH
D - DIAMETER OF PIPE
R - LENGTH OF VERTICAL PIPE ALONG CENTER LINE
L_2 - LENGTH OF HORIZONTAL PIPE ALONG CENTER LINE
V - VELOCITY IN PIPE - FPS.
Q - DISCHARGE THROUGH PIPE - C.F.S.
H - DROP OF WATER SURFACE

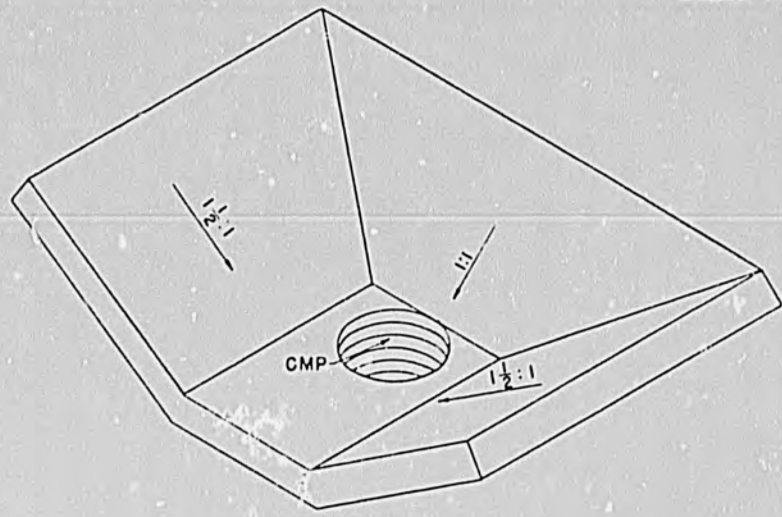
CORRUGATED METAL PIPE DROP WITH CHECK INLET
d=12"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

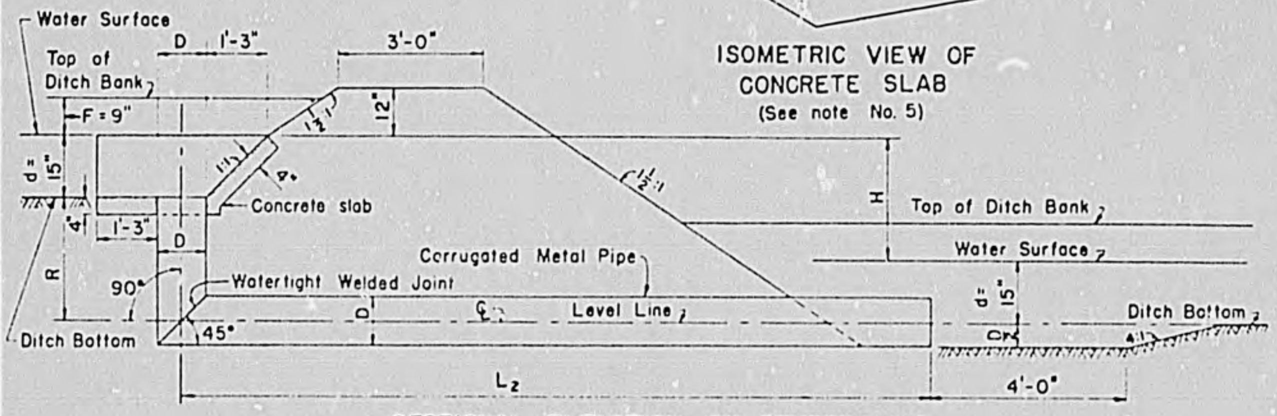
		2-60	7-L-36-25



PLAN



ISOMETRIC VIEW OF CONCRETE SLAB
(See note No. 5)



SECTIONAL ELEVATION ON CENTER LINE

CAPACITY AND PIPE LENGTHS REQUIRED

PIPE SIZE	D = 10"				D = 12"				D = 15"			
RECOMMENDED DESIGN CAPACITY	1.6 C.F.S.				2.4 C.F.S.				3.7 C.F.S.			
H = R	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$	V	Q MAX.	L_2	$R+L_2$
1'-0"	4.2	2.3	13'-0"	14'-0"	4.3	3.4	13'-0"	14'-0"	4.5	5.5	13'-0"	14'-0"
1'-6"	5.1	2.8	14'-6"	16'-0"	5.2	4.1	14'-6"	16'-0"	5.4	6.6	14'-6"	16'-0"
2'-0"	5.9	3.2	14'-0"	16'-0"	6.0	4.7	14'-0"	16'-0"	6.1	7.5	14'-0"	16'-0"
2'-6"	6.4	3.5	15'-6"	18'-0"	6.6	5.2	15'-6"	18'-0"	6.8	8.4	15'-6"	18'-0"
3'-0"	6.8	3.7	15'-0"	18'-0"	7.1	5.6	15'-0"	18'-0"	7.4	9.1	17'-0"	20'-0"

NOTES

1. SELECT A PIPE SIZE THAT WILL PROVIDE A GREATER CAPACITY THAN IS REQUIRED TO DISCHARGE THE NORMAL STREAM USED WHEN IRRIGATING. TRY TO KEEP THE VELOCITY IN THE PIPE BELOW 3 FPS BASED ON NORMAL IRRIGATING STREAM.
2. WHEN THE CORRUGATED METAL PIPE DROP IS USED AT A DITCH CROSSING, INCREASE WIDTH OF TOP OF DAM AND DIMENSION L_2 BY 8'-0".
3. THE DROP (H) FOR ANY SPECIFIC STRUCTURE CAN BE INCREASED 3 INCHES BY PLACING THE TOP OF THE RISER PIPE 3 INCHES BELOW THE TOP OF THE CONCRETE FLOOR OF THE INLET. THE THICKNESS OF THE FLOOR SLAB ADJACENT TO THE PIPE SHOULD BE INCREASED 3 INCHES TO MAKE A WATERTIGHT CONNECTION WITH THE PIPE. THE INLET TO THE PIPE SHOULD BE ROUNDED TO A 3 INCH RADIUS TO SAVE FORMING AND IMPROVE THE EFFICIENCY OF THE INLET.
4. THE DROP STRUCTURE IS FORMED BY CUTTING A STANDARD LENGTH OF CORRUGATED METAL PIPE, WHICH IS MANUFACTURED IN MULTIPLES OF 2 FT. IN LENGTH, ON A 45° ANGLE AND WELDING THE CUT JOINTS TOGETHER TO FORM A 90° BEND. PIPE TO BE 16 GAUGE CORRUGATED METAL. JOINT BETWEEN HORIZONTAL AND VERTICAL PIECES OF PIPE TO BE BUTT WELDED AND WATERTIGHT.
5. SIX INCH HAND PLACED RIP-RAP MAY BE SUBSTITUTED FOR CONCRETE SLAB.

NOMENCLATURE

d - DEPTH OF WATER IN DITCH
F - FREEBOARD IN DITCH
D - DIAMETER OF PIPE
R - LENGTH OF VERTICAL PIPE ALONG CENTER LINE
L_2 - LENGTH OF HORIZONTAL PIPE ALONG CENTER LINE
V - VELOCITY IN PIPE - FPS
Q - DISCHARGE THROUGH PIPE - C.F.S.
H - DROP OF WATER SURFACE

TABLE OF CONCRETE QUANTITIES

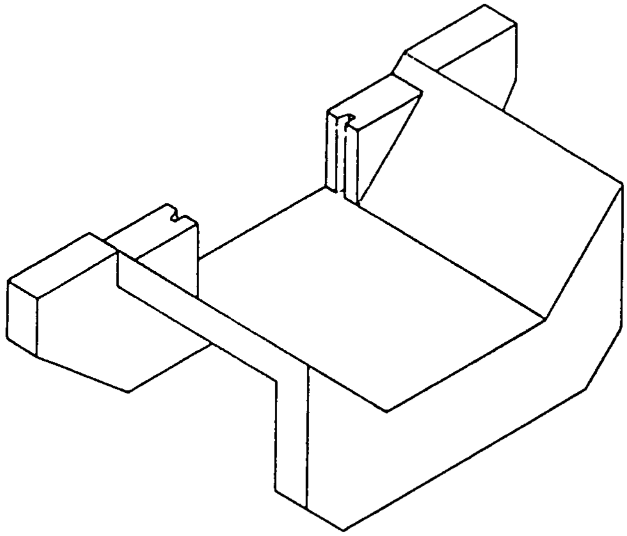
D 10"	0.33 CU.YDS.
D 12"	0.35 CU.YDS.
D 15"	0.39 CU.YDS.

CORRUGATED METAL PIPE DROP

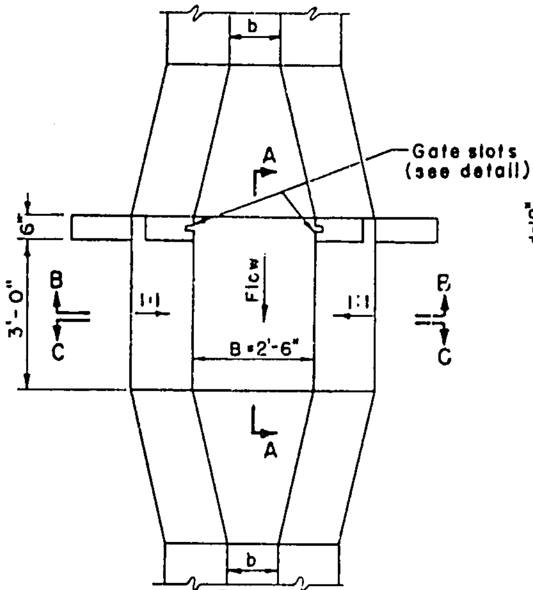
d = 15"

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

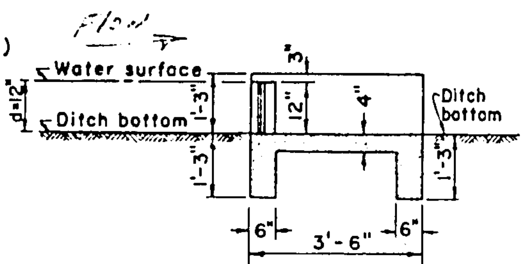
2-60 | 7-L-36-26



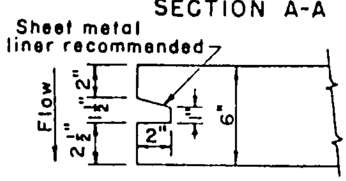
ISOMETRIC VIEW



PLAN

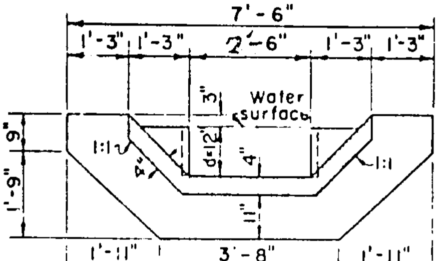


SECTION A-A



GATE SLOT DETAIL

Concrete quantity = 0.55 cu. yd.



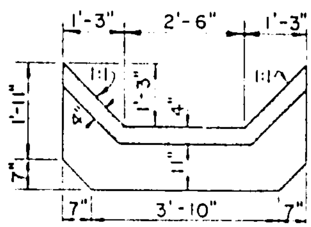
SECTIONAL ELEVATION B-B

NOMENCLATURE

B = Bottom width of structure

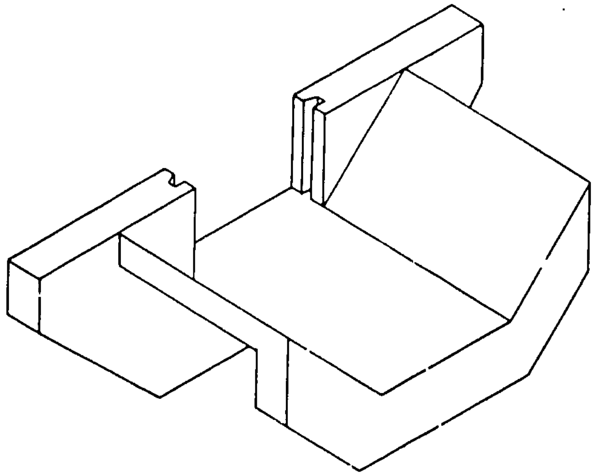
b = Bottom width of ditch

d = Depth of water in ditch

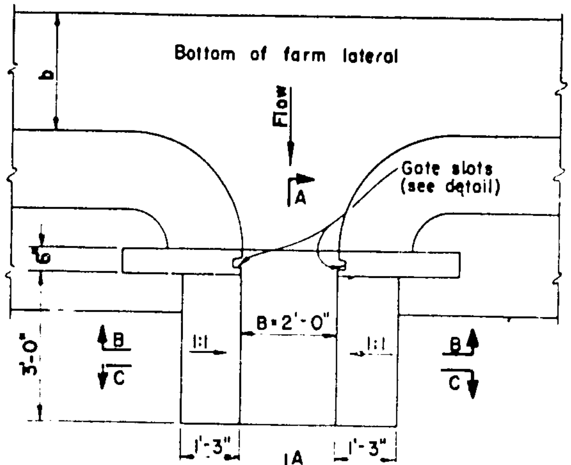


SECTIONAL ELEVATION C-C

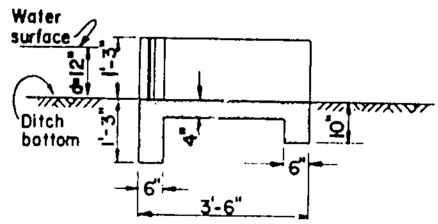
CONCRETE CHECK			
d = 12"		B = 2'-6"	
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE			
COMPILED	CHECKED	DATE 4-60	DRAWING NO. 7-L-36-44



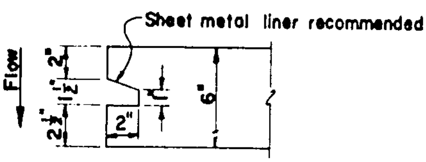
ISOMETRIC VIEW



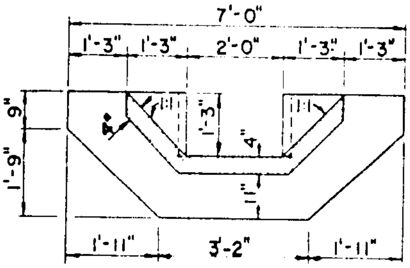
PLAN



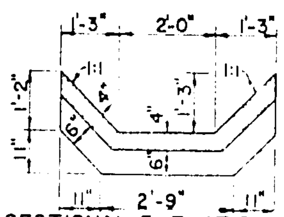
SECTION A-A



DETAIL OF GATE SLOT



SECTIONAL ELEVATION B-B



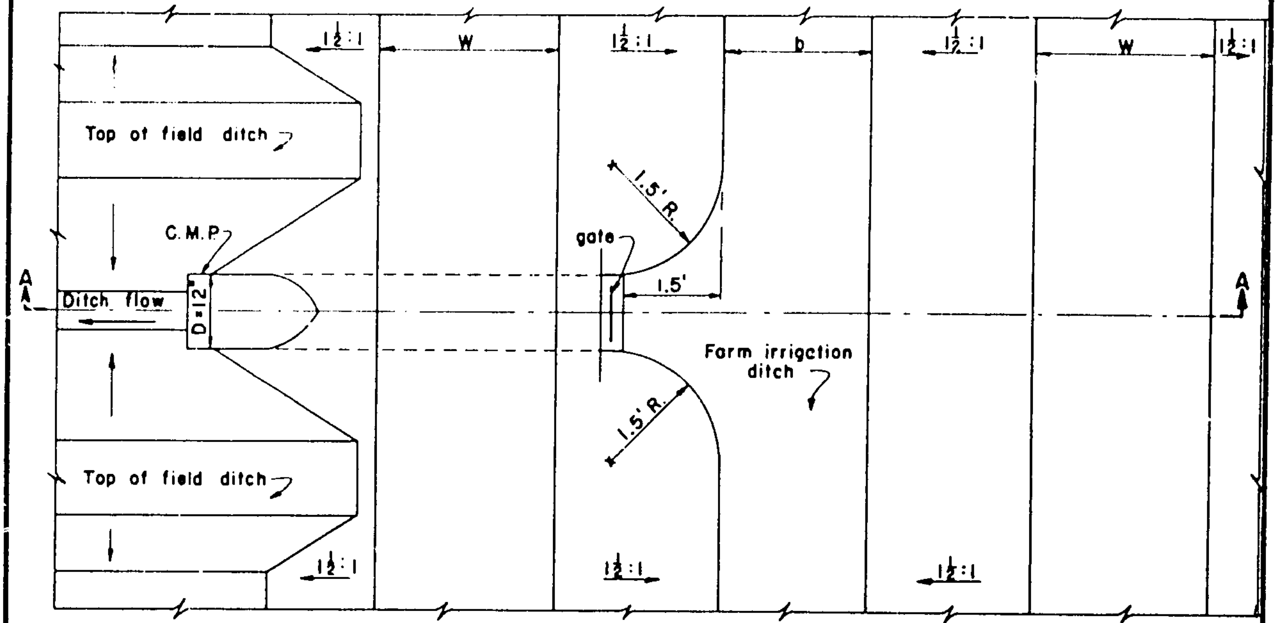
SECTIONAL ELEVATION C-C

Concrete quantity = 0.47 cu. yd.

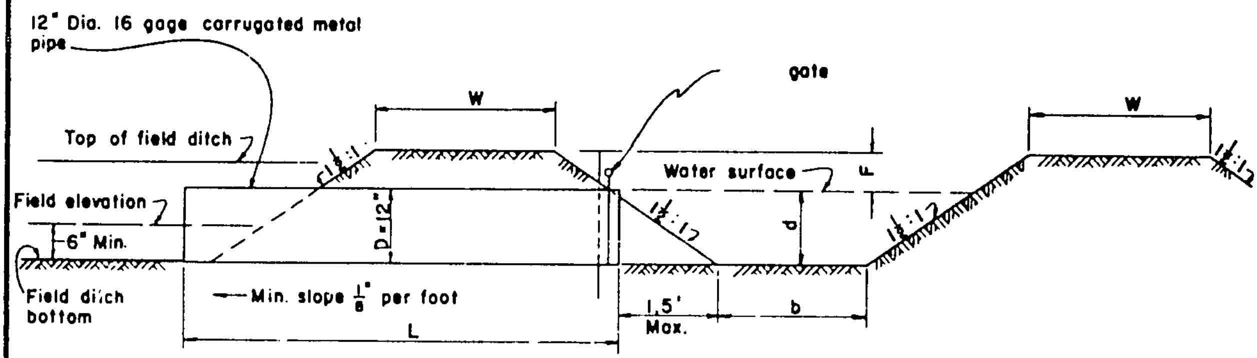
NOMENCLATURE

B = Bottom width of structure
b = Bottom width of ditch
d = Depth of water in ditch

CONCRETE TURNOUT			
d = 12"			B = 2'-0"
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE			
COMPILED	CHECKED	DATE 4-60	DRAWING NO. 7-L-36-45



PLAN



SECTIONAL ELEVATION A-A

Note: Top of pipe inlet not to be above the water surface.

b	d	F	W	L
feet	feet	feet	feet	feet
0.5	1.0	0.5	1.00	4.0
1.0	1.0	0.5	1.50	6.0
1.5	1.0	0.5	2.00	6.0
2.0	1.0	0.5	2.50	6.0
1.0	1.2	0.7	1.25	6.0
1.5	1.2	0.7	1.50	6.0
2.0	1.2	0.7	2.00	8.0
1.5	1.33	0.77	1.75	8.0
2.0	1.33	0.77	2.00	8.0

Pipe capacity with water surface at inlet same elevation as top of pipe and outlet unsubmerged.

Pipe diameter "D" in inches	Turnout Capacity c.f.s.
12	2.3

NOMENCLATURE

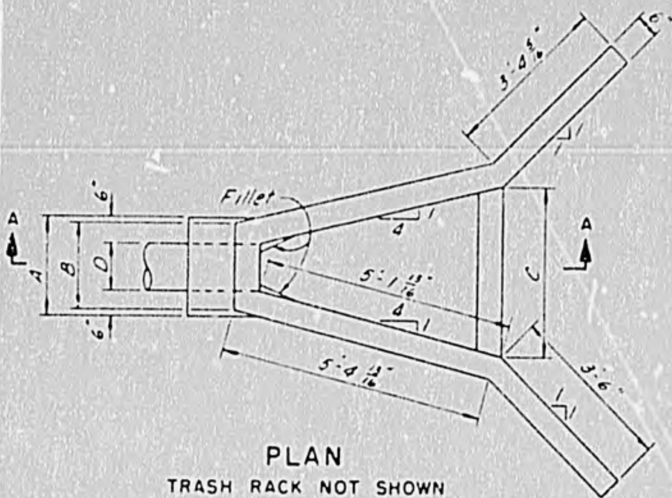
b = Bottom width of farm irrigation ditch
d = Depth of water in farm irrigation ditch
W = Top width
F = Free board
L = Length of pipe
D = Diameter of pipe

12" Diameter Gate

CORRUGATED METAL PIPE TURNOUT

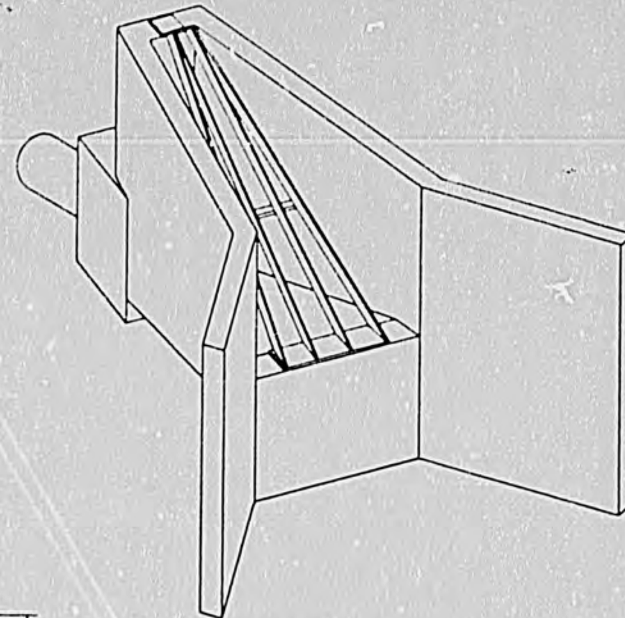
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPILED	CHECKED	DATE	DRAWING NO.
		3-60	7-L-36-40

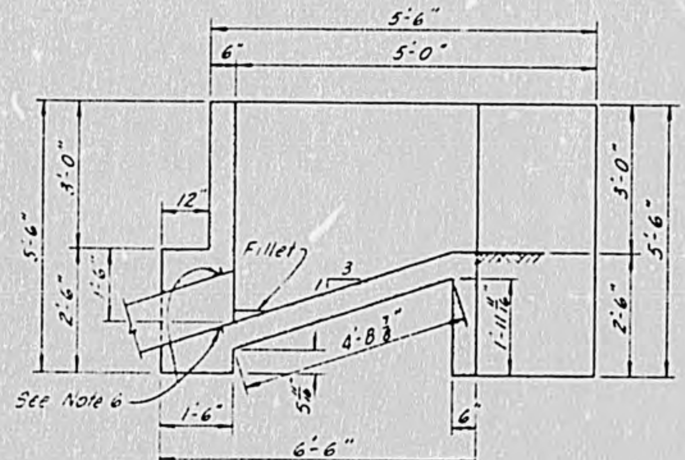


PLAN

TRASH RACK NOT SHOWN

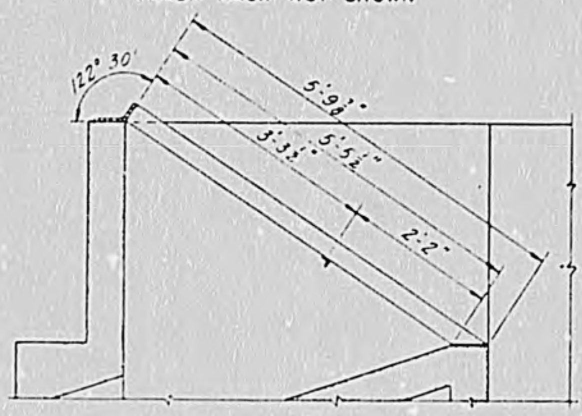


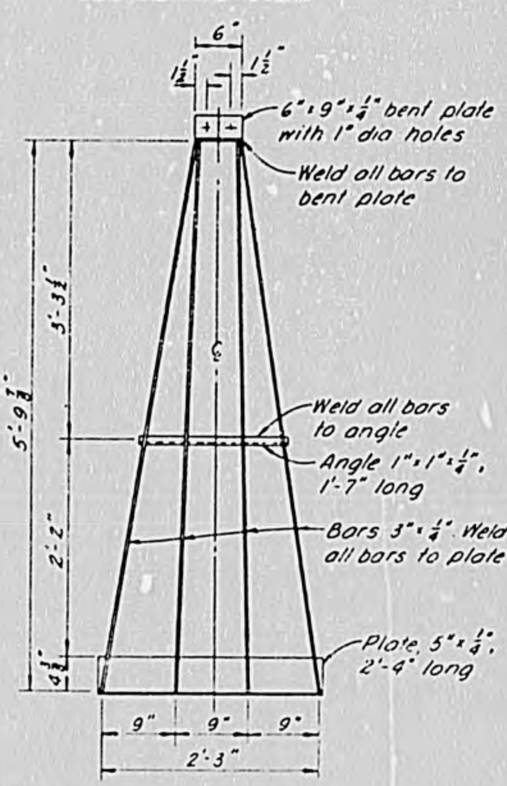
ISOMETRIC VIEW



SECTIONAL ELEVATION A-A

TRASH RACK NOT SHOWN





TRASH RACK DETAIL

TABLE OF DIMENSIONS AND QUANTITIES

PIPE DIA	DIMENSIONS A	DIMENSIONS B	DIMENSIONS C	DIMENSIONS D	REINFORCING STEEL Lbs	CONCRETE Cu. Yds
8"	1'-8"	1'-5"	3'-2"	0'-8"	175	2.25
10"	2'-0"	1'-9"	3'-6"	1'-0"	178	2.35
12"	2'-0"	1'-9"	3'-6"	1'-0"	178	2.34
15"	2'-3"	2'-0"	3'-9"	1'-3"	181	2.40

Total structural steel = 76 lbs

Notes

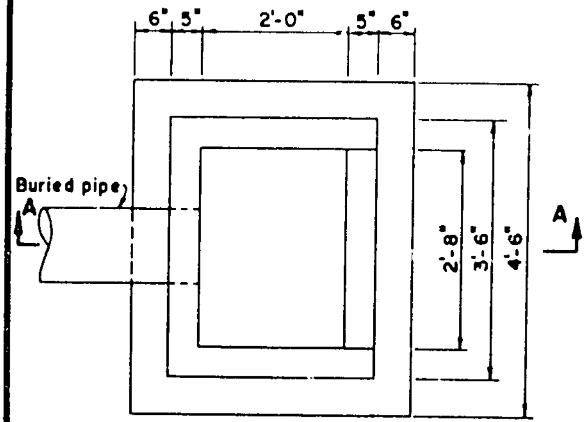
1. All concrete is Class B.
2. All reinforcing steel to be 1/2" dia placed 12" center to center in center of slabs
3. Round corners at entrance to pipe and place fillets in corners after forms are removed
4. Pipe ends can be mitered to avoid additional forming
5. Clear distance of reinforcing steel to outside face (dirt side) to be 2"
6. Weld 2" x 2" x 1/4" angle irons, 3" long at quarter points, 4" from end on smooth metal pipe to provide bond
7. Minimum heigth of fill over pipe to be 3'.

CONCRETE SIPHON INLET AND OUTLET FOR 8" TO 15" DIA. PIPE

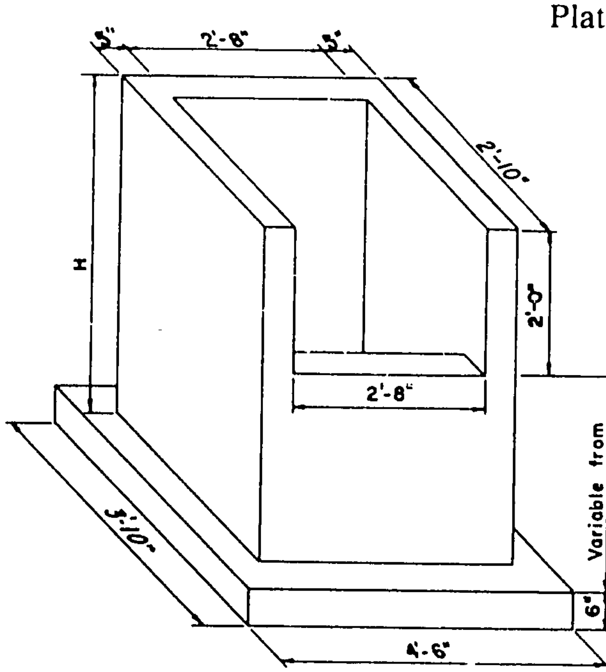
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

DRAWN	CHECKED	DATE	DRAWING NO.
		1-64	5,0-19,000 35-1

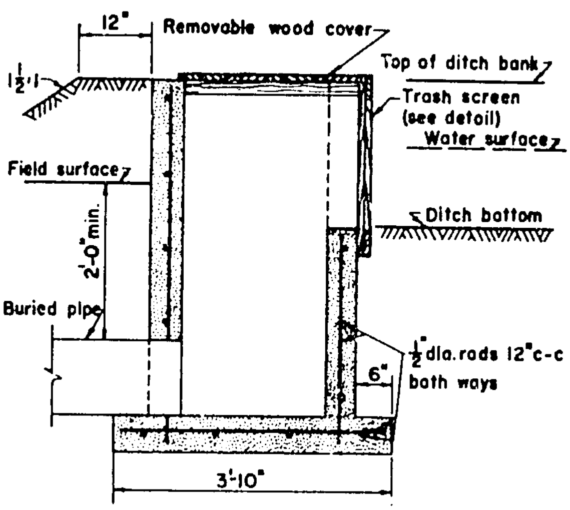
127



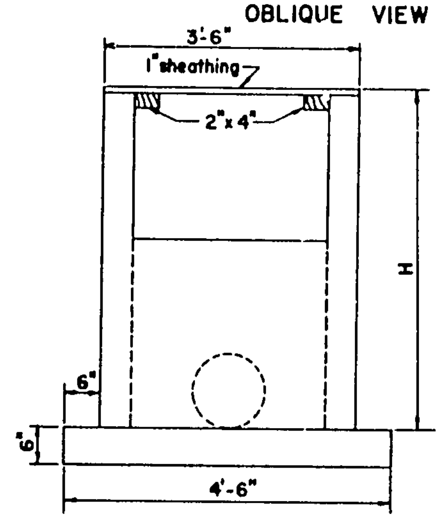
PLAN



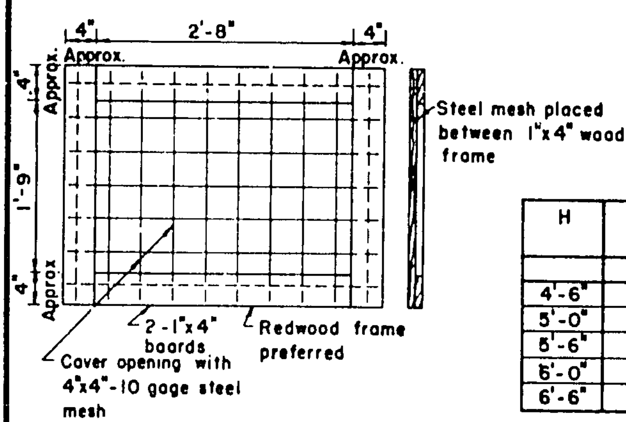
OBLIQUE VIEW



SECTIONAL ELEVATION A-A



ELEVATION



DETAIL OF TRASH SCREEN

TABLE OF QUANTITIES

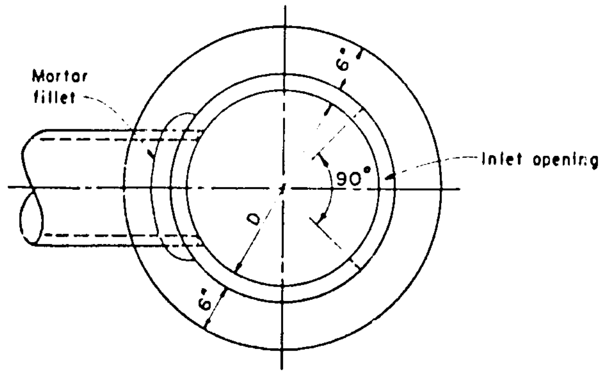
H	CONCRETE	REINFORCING STEEL ½" dia. rods	STEEL MESH 4"x4"-10 gage	WOOD
	cu. yds.	lin. ft.	sq. ft.	2"x4"-4.83 lin. ft.
4'-6"	1.00	128.5	8	1" sheathing
5'-0"	1.10	134.5	8	10 bd. ft.
5'-6"	1.20	152.5	8	1"x4"-21 lin. ft.
6'-0"	1.30	158.5	8	(redwood
6'-6"	1.40	175.6	8	preferred)

NOTES: 1. Maximum Q = 5.3 c.f.s. = 2390 g.p.m.
2. May also be used as Terminal Outlet for pipe line with trash screen omitted.

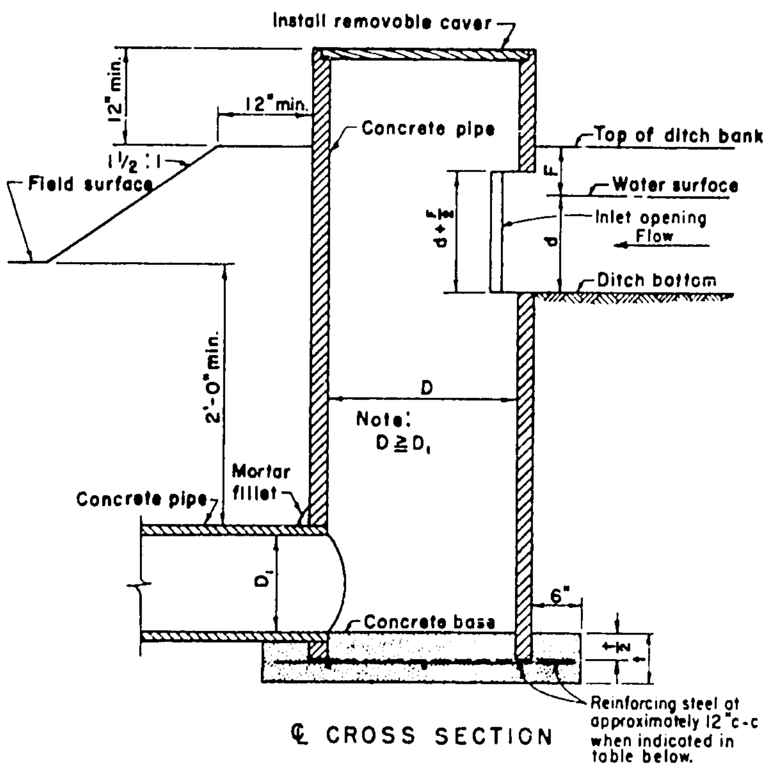
CONCRETE GRAVITY INLET FOR BURIED PIPE LINES

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

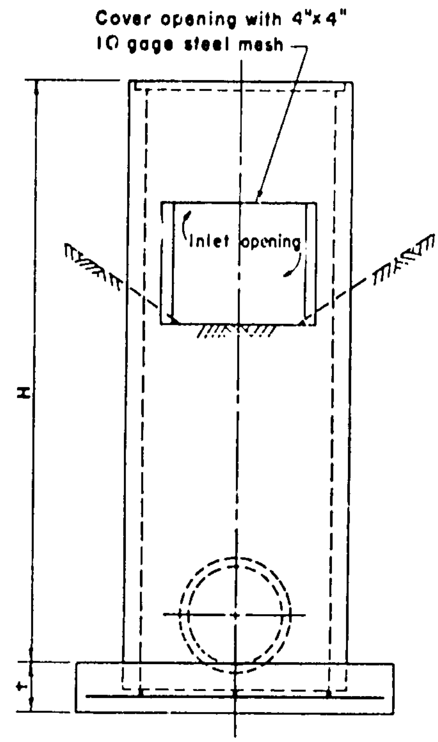
COMPILED	CHECKED	DATE	DRAWING NO.
		3-60	7-L-36-1



PLAN



℄ CROSS SECTION



ELEVATION

NOMENCLATURE

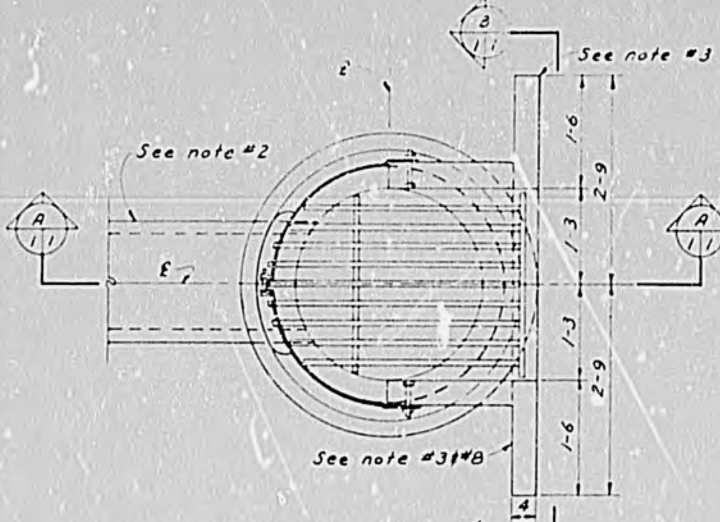
d = Depth of water in ditch
F = Freeboard in ditch
D = Diameter of vertical pipe
D_1 = Diameter of underground pipe
t = Thickness of concrete base
H = Height of vertical pipe above top of concrete base
Q = Discharge through structure in c.f.s. and g.p.m.

Max. Q		D	A.S.T.M. Spec		Concrete Base					
					H=10' or less		H=more than 10'		Reinforcing steel	
c. f. s.	g.p.m.	Inches	No.	Type	t	Cu.yd.	t	Cu.yd.	Size	Length
0.79	355	12	C-118	Concrete Irrigation Pipe	4"	0.05	6"	0.07	—	—
1.07	480	14			4"	0.05	6"	0.08	—	—
1.23	550	15			4"	0.06	6"	0.09	—	—
1.40	630	16			4"	0.06	6"	0.10	—	—
1.77	795	18			4"	0.07	6"	0.11	—	—
2.18	980	20			6"	0.13	8"	0.17	—	—
2.41	1080	21			5"	0.14	8"	0.18	—	—
3.14	1410	24			6"	0.16	8"	0.22	—	—
3.98	1785	27	C-76	Class II Reinforced Concrete Pipe	6"	0.20	8"	0.26	3/8"	19'
4.91	2205	30			6"	0.23	8"	0.30	3/8"	21'
5.94	2665	33			8"	0.35	8"	0.35	3/8"	22'
7.07	3175	36			8"	0.39	8"	0.39	3/8"	23'
9.62	4320	42			8"	0.50	3"	0.50	3/8"	38'
12.57	5640	48			8"	0.62	6"	0.62	1/2"	46'

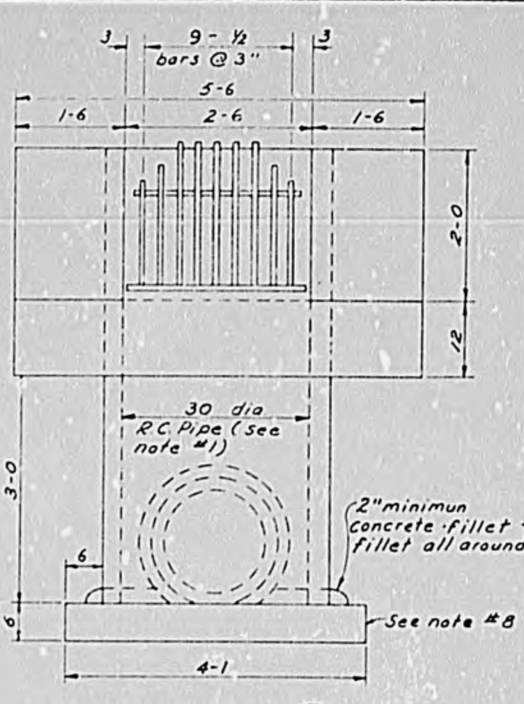
GRAVITY INLET
FOR CONCRETE PIPE

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

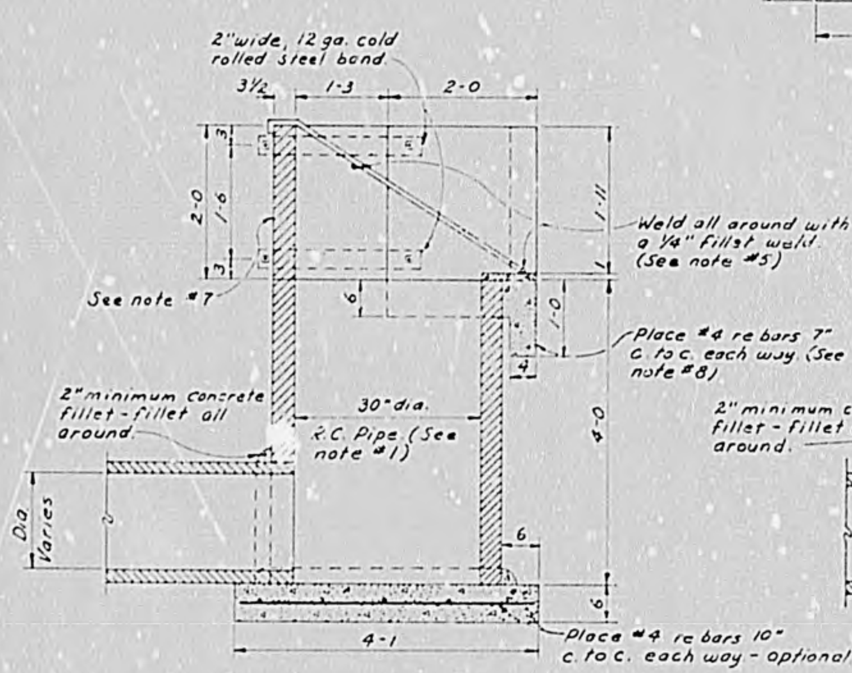
12-59 | 7-L-36-2



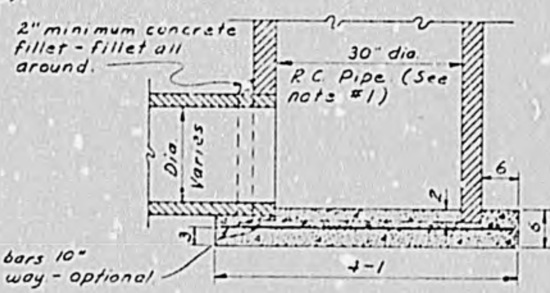
PLAN VIEW



SECTION B



SECTION A



ALTERNATE SECTION A

NOTES:

1. Pipe shall be class II (1500-D-Ultimate) pipe as specified in the latest revision of ASTM Designation C-76.
2. All non-reinforced concrete pipe lines laid with Mortar Joints shall conform to or exceed the requirements of the latest revision of ASTM C-118. All non-reinforced concrete pipe lines laid with rubber gasket joints shall conform to or exceed the requirements of the latest revision of ASTM C-505. All non-reinforced cast-in-place concrete pipe shall conform to or exceed the requirements of the latest revision of ASTM C-477.
3. Precast wingwalls. Use #4 rebars spaced at 7½".
4. All spaces excavated and not occupied by the structure shall be backfilled with earth up to the ground surface. The backfill shall be compacted to a density equal to the density of the adjacent earth. The moisture content of the backfill material shall be such that the required compaction will be obtained.
5. Bars shall be #4 reinforcing steel.
6. This structure may be used as an inlet or an outlet. The trash rack is not needed for an outlet. Provide drain for pipe line when used as an outlet.
7. This section will conform to note #1 but will be a half section of pipe.
8. Structure shall be installed in accordance with Soil Conservation Service Engineering Standard & Specifications No. Colo.-587 "Structure for Water Control."
9. STANDARDIZED DESIGNS - Must Be Adapted To The Specific Site.

SCALE IN FEET: 1 0 1 2

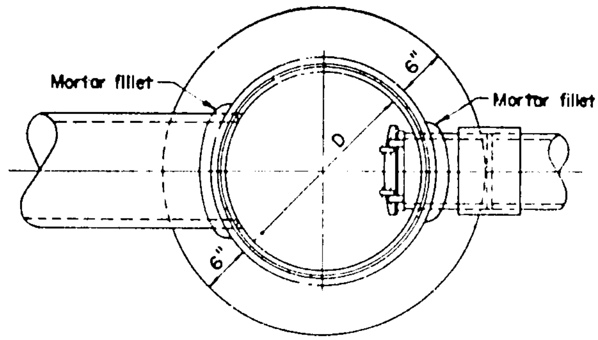
30" DIA. R.C. PIPE INLET STRUCTURE FOR IRRIGATION PIPE LINES

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

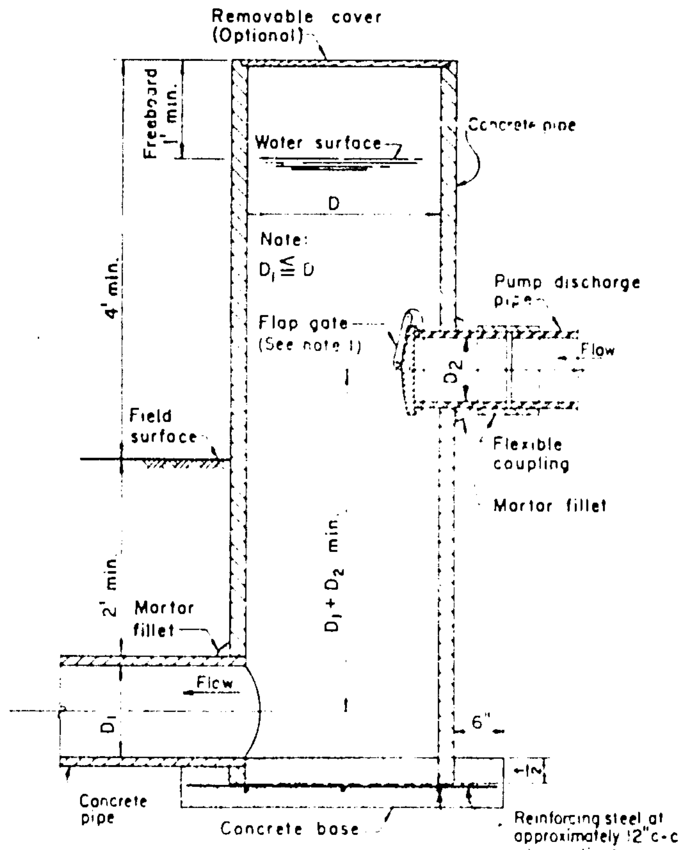
ARM 2-69 State Cons. Engr.

CO-SSP-6

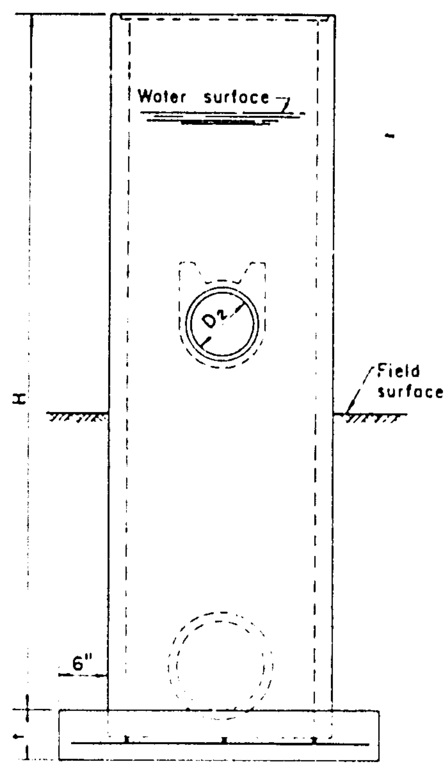
Revised 8-73



PLAN



℄ CROSS SECTION



ELEVATION

Notes:

1. When D ≤ 27" or when D_2 is greater than ½ D eliminate flap gate and use a check valve in pump discharge pipe

NOMENCLATURE

D - Diameter of vertical pipe

D_1 - Diameter of underground pipe

D_2 - Diameter of pump discharge pipe

t - Thickness of concrete base

H - Height of vertical pipe above top of concrete base

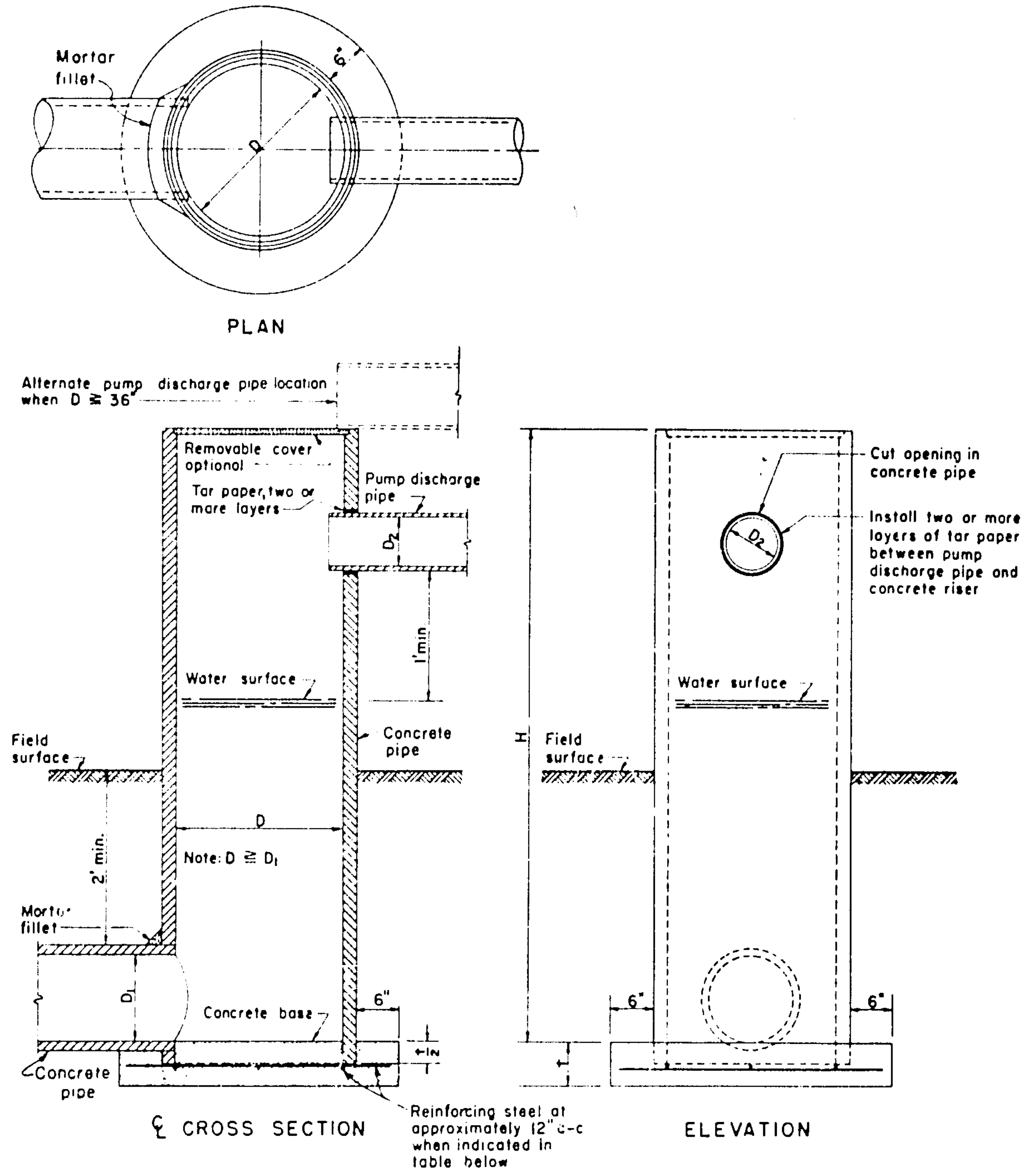
Q - Discharge through structure in cfs. and g.p.m.

Max Q		D	A.S.T.M. Spec.		Concrete Base					
					H=10' or less		H=more than 10'		Reinforcing steel	
c.f.s.	g.p.m.	Inches	No.	Type	t	Cu yd	t	Cu yd	Size	Length
0.79	355	12	C-118	Concrete Irrigation Pipe	4"	0.05	6"	0.07	—	—
1.07	480	14			4"	0.05	6"	0.08	—	—
1.23	550	15			4"	0.06	6"	0.09	—	—
1.40	630	16			4"	0.06	6"	0.10	—	—
1.77	795	18			4"	0.07	6"	0.11	—	—
2.18	980	20			6"	0.13	8"	0.17	—	—
2.41	1080	21			6"	0.14	8"	0.18	—	—
3.14	1410	24			6"	0.16	8"	0.22	—	—
3.98	1785	27	C-76	Class II Reinforced Concrete Pipe	6"	0.20	8"	0.26	3/8"	19'
4.91	2205	30			6"	0.23	8"	0.30	3/8"	21'
5.94	2665	33			8"	0.35	8"	0.35	3/8"	22'
7.07	3175	36			8"	0.39	8"	0.39	3/8"	23'
9.62	4320	42			8"	0.50	8"	0.50	3/8"	38'
12.57	5640	48			8"	0.62	8"	0.62	1/2"	46'

HIGH HEAD NON-TAPERED PUMP STAND FOR CONCRETE PIPE

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

12-59 | 7-L-36-7



Max. Q		D	A S T M Spec		Concrete Base					
					H=10' or less		H=more than 10'		Reinforcing steel	
c.f.s.	g.p.m.	inches	No	Type	t	Cu. yd.	t	Cu. yd.	Size	Length
0.79	355	12	C-118	Concrete Irrigation Pipe	4"	0.05	6"	0.07	—	—
1.07	480	14			4"	0.05	6"	0.08	—	—
1.23	550	15			4"	0.06	6"	0.09	—	—
1.40	630	16			4"	0.06	6"	0.10	—	—
1.77	795	18			4"	0.07	6"	0.11	—	—
2.18	980	20			6"	0.13	8"	0.17	—	—
2.41	1080	21			6"	0.14	8"	0.18	—	—
3.14	1410	24			6"	0.16	8"	0.22	—	—
3.98	1785	27	C-76	Class II Reinforced Concrete Pipe	6"	0.20	8"	0.26	3/8"	19'
4.91	2205	30			6"	0.23	8"	0.30	3/8"	21'
5.94	2665	33			8"	0.35	8"	0.35	3/8"	22'
7.07	3175	36			8"	0.39	8"	0.39	3/8"	23'
9.62	4320	42			8"	0.50	8"	0.50	3/8"	38'
12.57	5640	48			8"	0.62	8"	0.62	1/2"	46'

NOMENCLATURE

D – Diameter of vertical pipe

D_1 – Diameter of underground pipe

D_2 – Diameter of pump discharge pipe

t – Thickness of concrete base

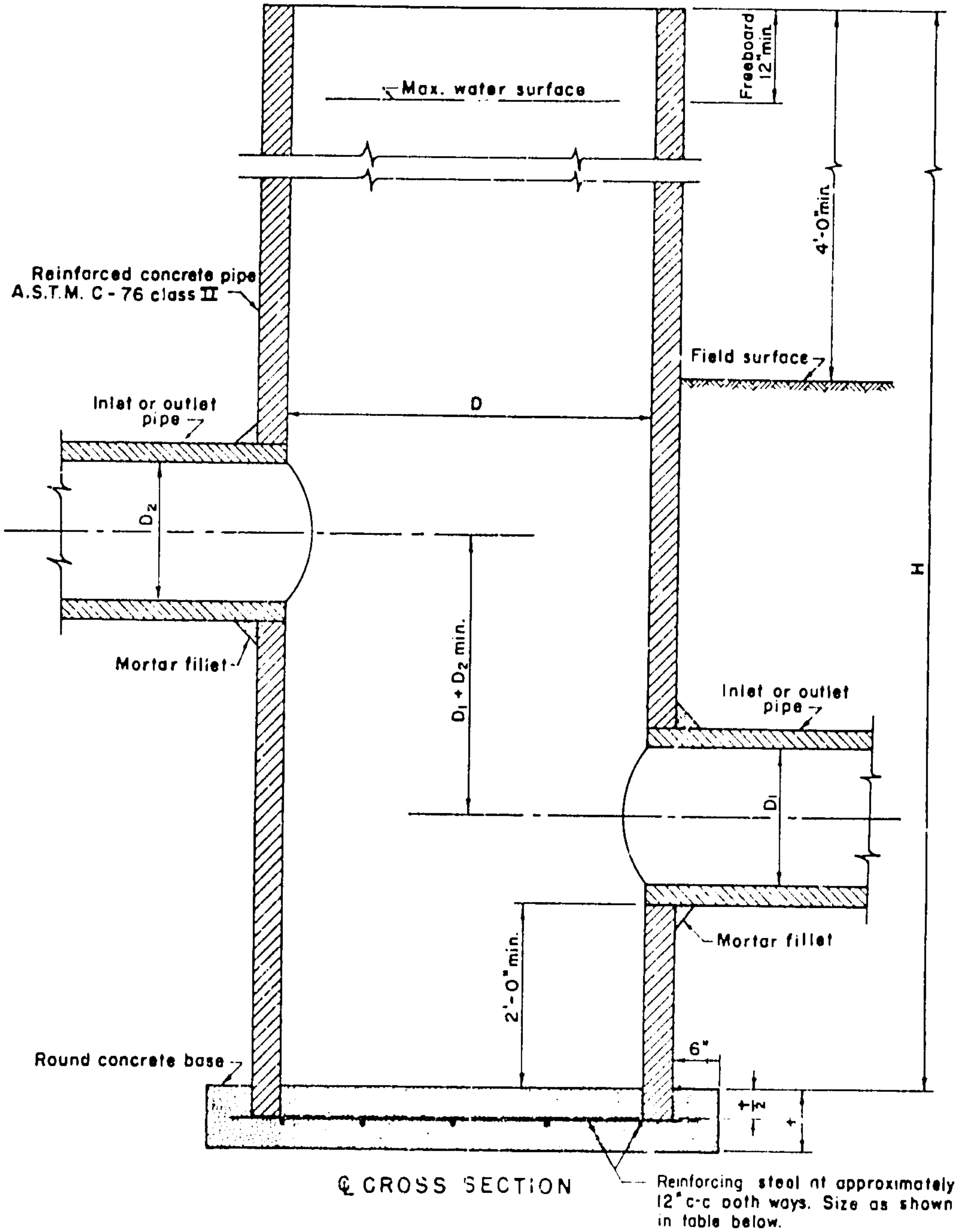
H – Height of vertical pipe above top of concrete base

Q – Discharge through structure in cfs. and g.p.m.

LOW HEAD PUMP STAND FOR CONCRETE PIPE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

12-59 | 7-L-36-43



℄ CROSS SECTION

NOMENCLATURE

D = Diameter of vertical concrete pipe

D_1 = Diameter of inlet or outlet pipe

D_2 = Diameter of inlet or outlet pipe

t = Thickness of concrete base

H = Height of vertical concrete pipe above top of concrete base

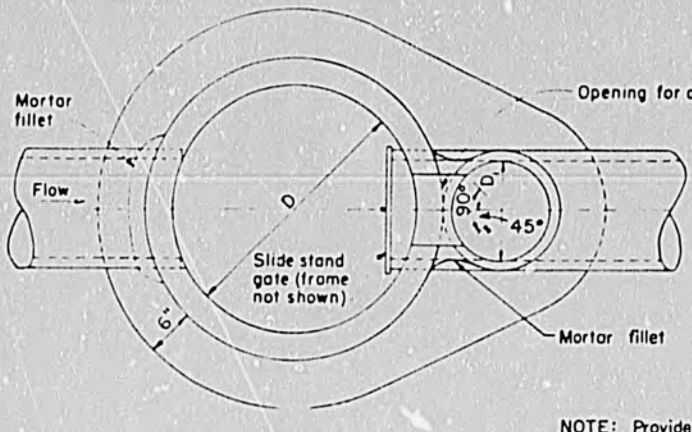
Q = Discharge through structure in c.f.s. and g.p.m.

Max. Q		D	Concrete Base					
			H=10' or less		H=more than 10'		Reinforcing steel	
c.f.s.	g.p.m.	Inches	t	cu yd	t	cu yd	Size	Length
1.22	550	30	6"	.23	8"	.30	3/8"	21'
1.49	670	33	8"	.35	8"	.35	3/8"	22'
1.77	795	36	8"	.39	8"	.39	3/8"	23'
2.40	1075	42	8"	.50	8"	.50	3/8"	38'
3.14	1410	48	8"	.62	8"	.62	1/2"	46'
3.98	1785	54	8"	.76	8"	.76	1/2"	53'
4.90	2200	60	8"	.91	8"	.91	1/2"	71'

CONCRETE PIPE SAND TRAP
FOR CONCRETE PIPE LINE

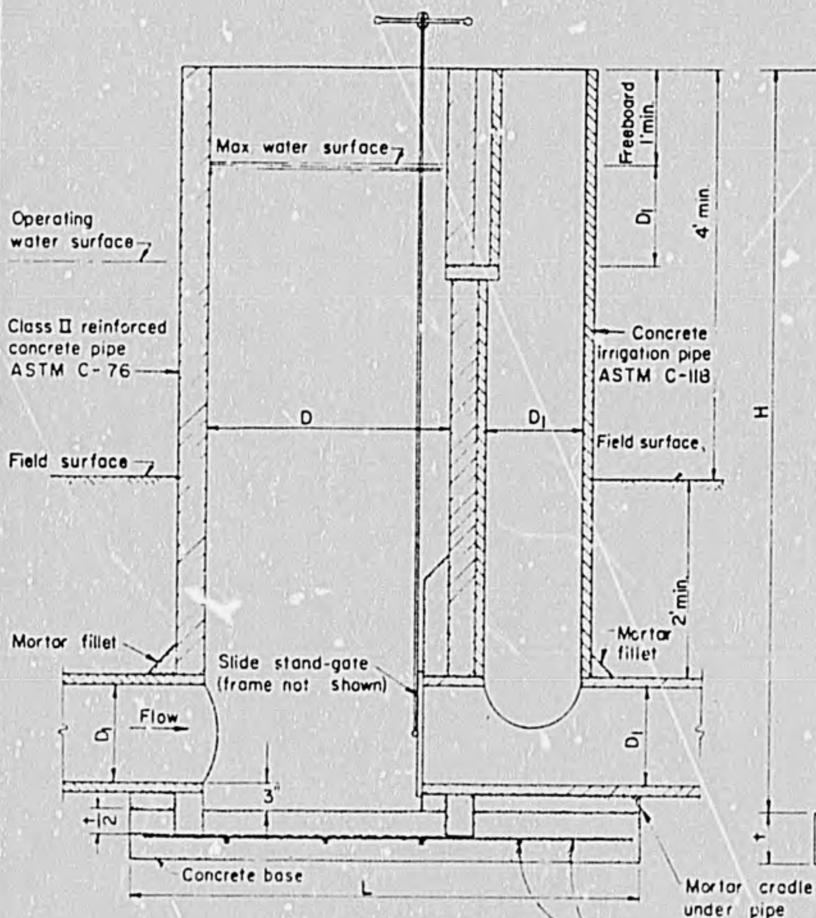
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Date: 12-59 | Drawing No.: 7-L-36-8

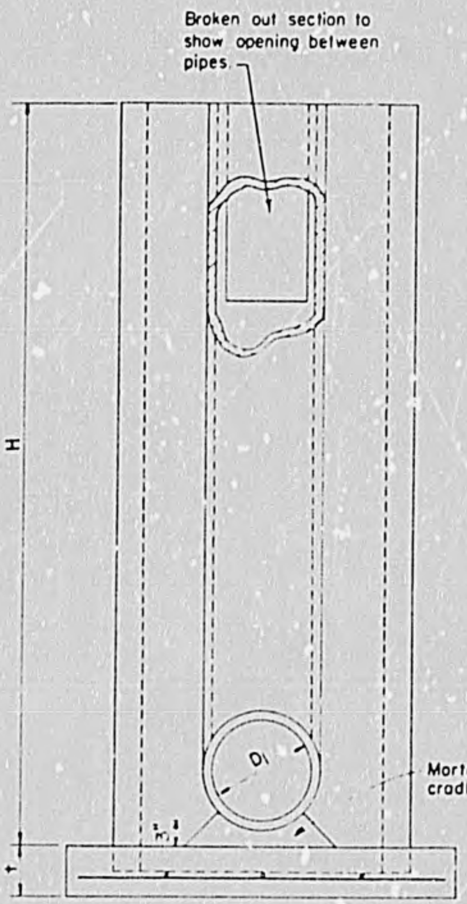


PLAN

NOTE: Provide permanent ladder fastened to stand to provide access to gate wheel.



℄ CROSS SECTION



ELEVATION
(Gate not shown)

TABLE OF DIMENSIONS AND QUANTITIES

D_1	D	L	Concrete Base					
			H=10' or less		H=more than 10'		Reinforcing steel	
			t	volume	t	volume		
Inches	Inches	Feet - Inches	Inches	Cu yd.	Inches	Cu yd.	Size	Lgth ft.
8	30	4' - 10 1/2"	6	0.31	8	0.41	3/8"	24
10	30	5' - 0 1/2"	6	0.32	8	0.42	3/8"	25
12	30	5' - 2 3/4"	6	0.34	8	0.44	3/8"	26
14	30	5' - 5"	6	0.36	8	0.46	3/8"	27
15	30	5' - 6"	6	0.37	8	0.47	3/8"	28
16	30	5' - 7 1/4"	6	0.38	8	0.48	3/8"	29
18	30	5' - 9 1/2"	6	0.39	8	0.50	3/8"	31
20	42	7' - 1 3/4"	8	0.75	8	0.75	3/8"	53
21	42	7' - 3"	8	0.77	8	0.77	3/8"	53
24	48	8' - 1 3/4"	8	0.97	8	0.97	1/2"	64

NOMENCLATURE

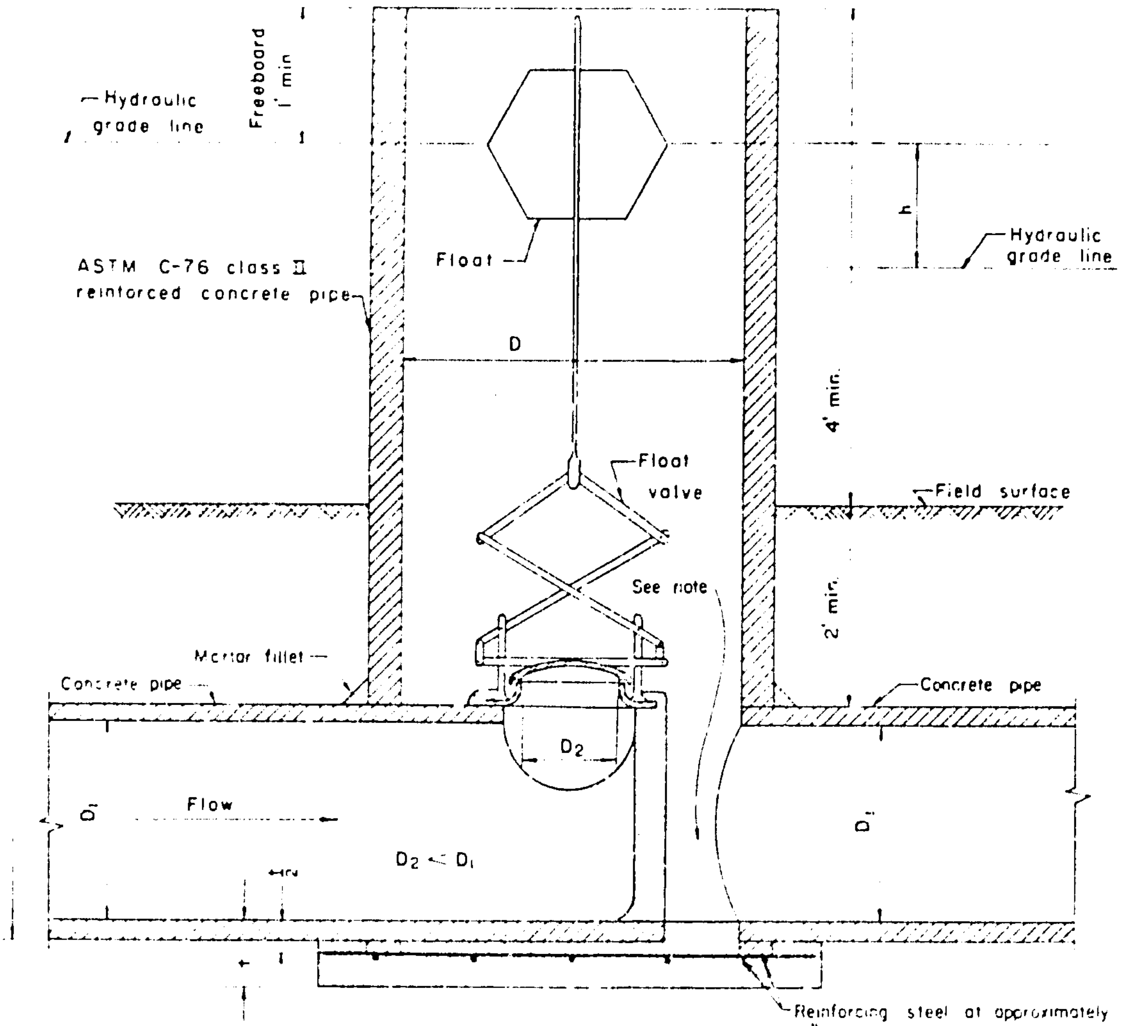
- D - Diameter of concrete stand pipe
- D_1 - Diameter of underground concrete pipe and concrete overflow pipe
- H - Height of structure
- t - Thickness of concrete base

OVERFLOW GATE STAND
for
CONCRETE PIPE LINES

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Compiled	Checked	Date	Drawing No.
		1-64	5,0-19,000 29-1

134



℄ CROSS SECTION

Note 1 Provide an outlet area equal to or greater than the area of the valve (whose diameter is D_2)

D_2 Nominal inches	Design Flow Capacity and Stand Diameter											
	h=0.5'			h=1.0'			h=2.0'			h=5.0'		
	Capacity		D	Capacity		D	Capacity		D	Capacity		D
	c.f.s.	g.p.m.	in	c.f.s.	g.p.m.	in	c.f.s.	g.p.m.	in	c.f.s.	g.p.m.	in
4	0.32	145	30	0.45	200	30	0.64	285	30	1.01	455	30
5	[illegible]	225	30	0.71	320	30	1.00	450	30	1.58	710	30
8	[illegible]	575	30	1.8	810	30	2.56	1150	30	4.05	1820	30
12	2.87	1290	30	4.07	1825	30	5.75	2580	33	9.10	4085	42
16	5.12	2300	33	7.24	3250	42	10.23	4590	48	16.17	7260	60

NOMENCLATURE

D - Diameter of concrete stand pipe

D_1 - Diameter of underground concrete pipe

D_2 - Nominal diameter of float valve

H - Height of stand pipe

t - Thickness of concrete base

h - Difference in head (hydraulic grade lines) between inlet and outlet pipe

TABLE OF QUANTITIES

D Inches	Concrete base				Rein. steel	
	H=10' or less		H=more than 10'			
	t	cu. yd	t	cu. yd	size	length
30	6"	0.23	8"	0.30	[illegible]	21'
33	8"	0.35	8"	0.35	[illegible]	22'
42	8"	0.50	6"	0.50	[illegible]	38'
48	8"	0.62	8"	0.62	[illegible]	46'
60	8"	0.91	8"	0.91	[illegible]	71'

NON-BALANCED FLOAT VALVE STANDS
for
CONCRETE PIPE LINES

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

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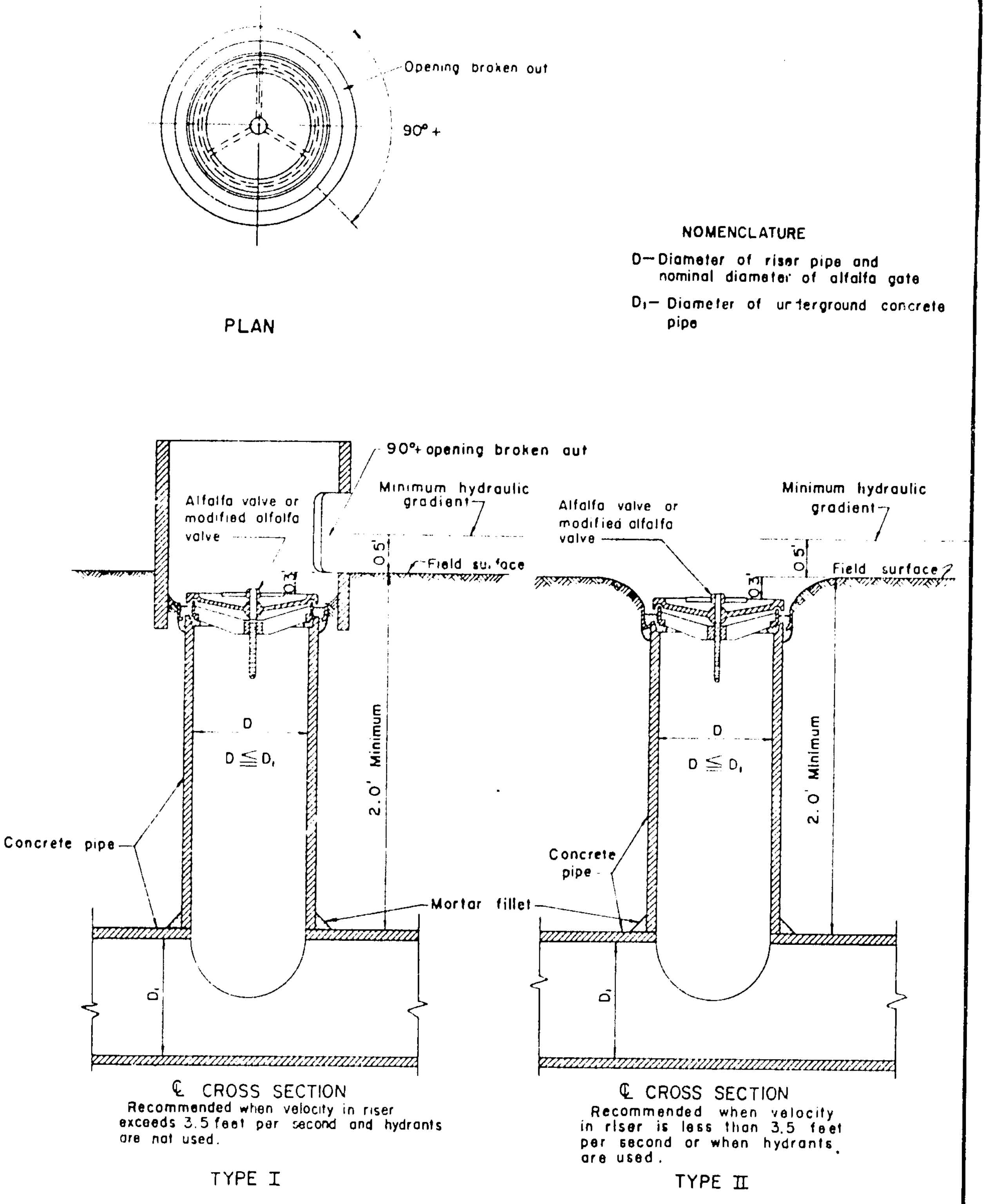
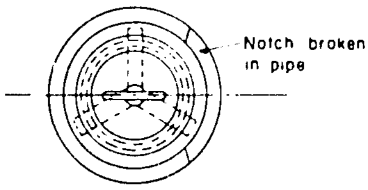


ALFALFA VALVE or MODIFIED ALFALFA VALVE OUTLET for CONCRETE PIPE LINES

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

12-59

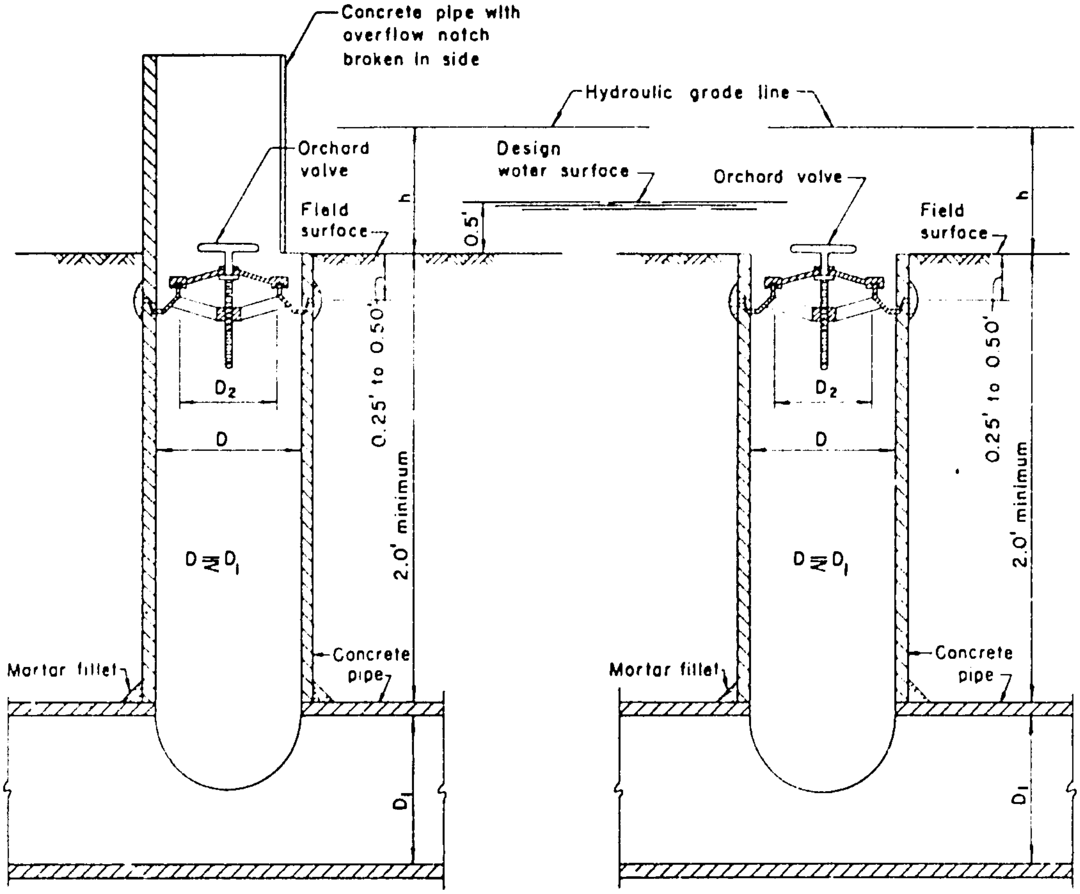
7-L-36-3



PLAN

NOMENCLATURE

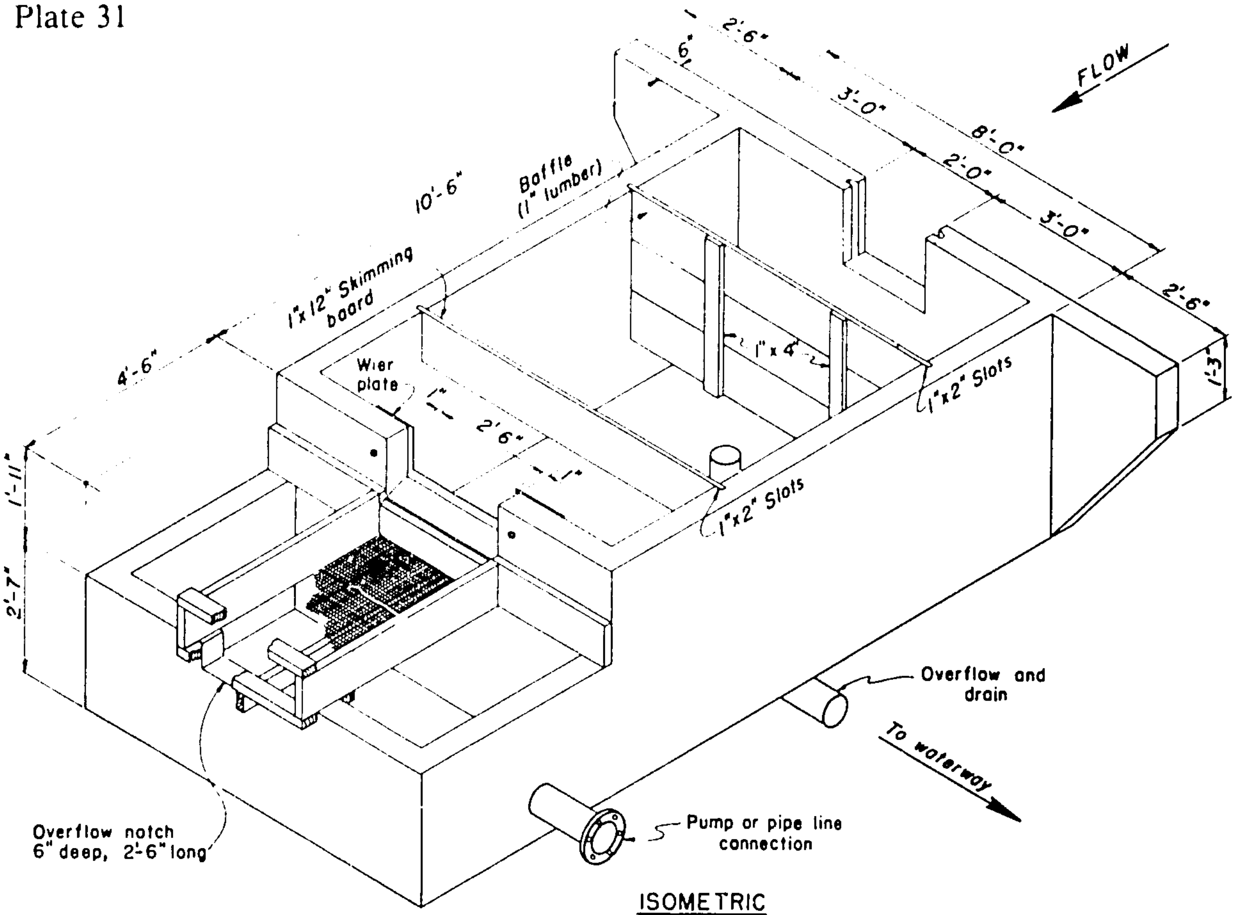
- D – Diameter of concrete riser pipe
- D_1 – Diameter of underground concrete pipe
- D_2 – Diameter of valve outlet
- h – Height of hydraulic grade line above field surface



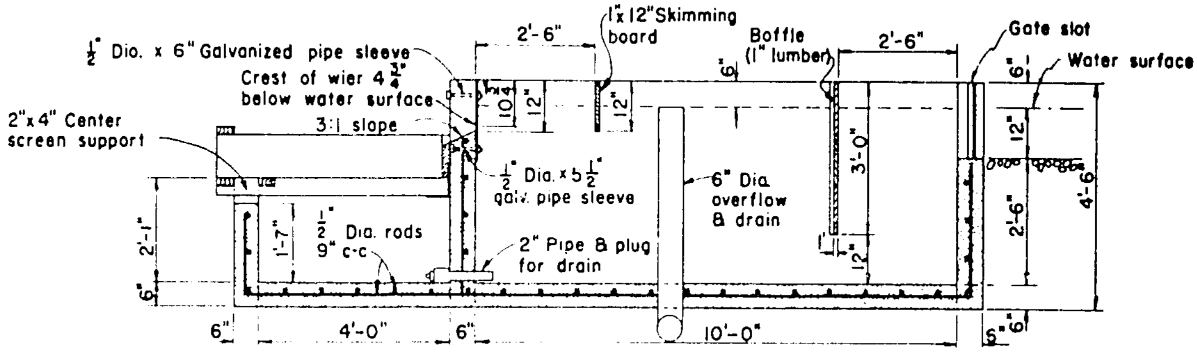
TYPE I — ℄ CROSS SECTION

TYPE II — ℄ CROSS SECTION

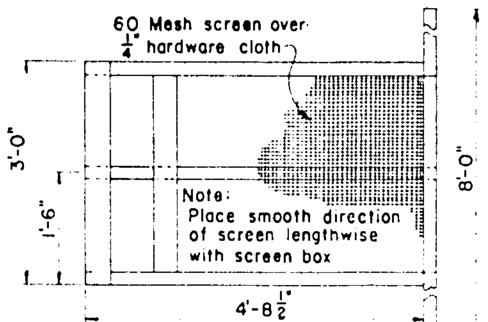
ORCHARD VALVE OUTLET FOR CONCRETE PIPE LINES			
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE			
Compiled	Checked	Date 12-59	Drawing No. 7-L-36-4



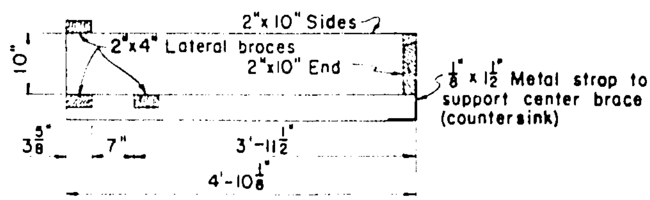
ISOMETRIC



CENTERLINE OF CROSS SECTION



PLAN



SIDE VIEW

DETAIL OF TRASH SCREEN

TABLE OF QUANTITIES

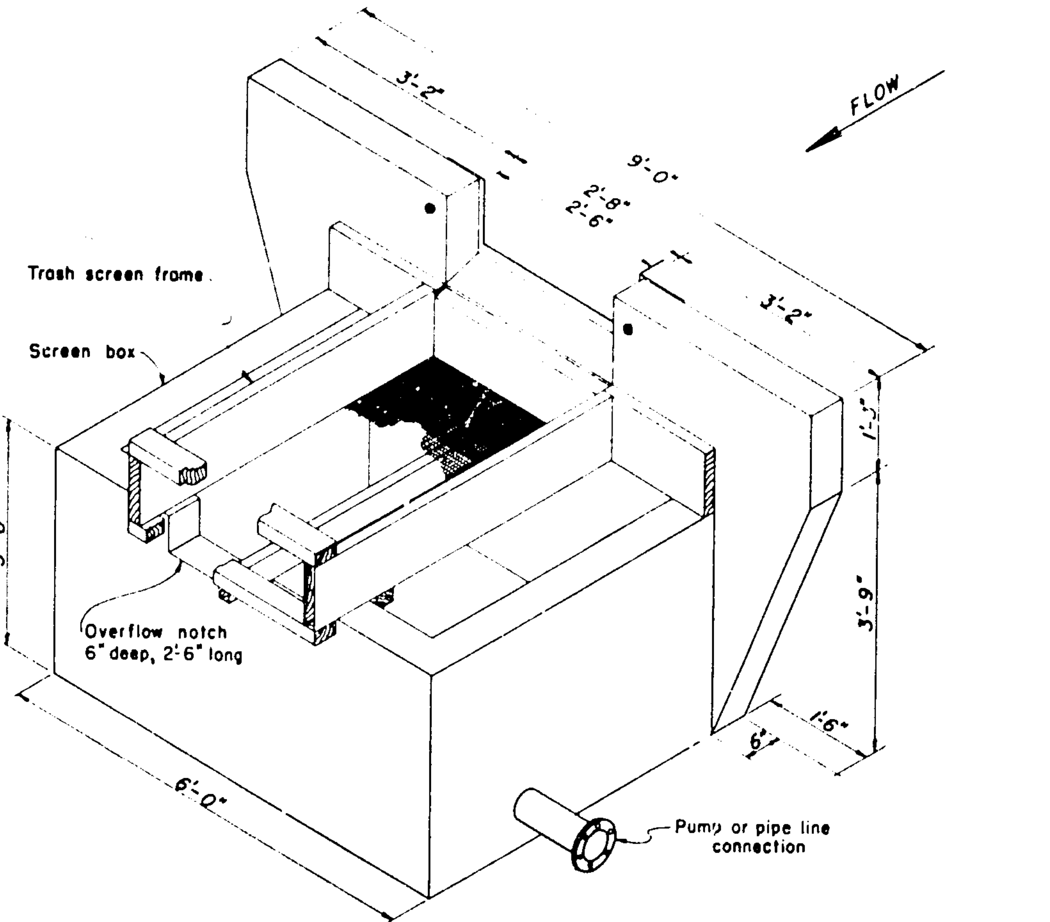
ITEM	UNIT	QUANTITY
CONCRETE	CU. YDS.	5.7
REINFORCING STEEL	LIN. FT.	883
60 MESH COPPER SCREEN	SQ. FT.	14
1/4" HARDWARE CLOTH	SQ. FT.	14
LUMBER	BD. FT.	71
1/2" GALVANIZED PIPE SLEEVES, 6" LONG	EACH	2
1/2" GALVANIZED PIPE SLEEVES, 5 1/2" LONG	EACH	3
3/8" DIA. GALVANIZED BOLTS, 6 1/2" LONG	EACH	2
3/8" DIA. GALVANIZED BOLTS, 6" LONG	EACH	3
2" DIA. PIPE, 8" LONG	EACH	1
2" PIPE COUPLING	EACH	1
2" PIPE PLUG	EACH	1
6" DIA. PIPE, 3'-6" LONG	EACH	2
6" PIPE ELBOW	EACH	1
1/8" WEIR PLATE	EACH	1
1/8" X 1 1/2" X 12" METAL STRAP	EACH	1

Capacity up to 900 G.P.M.

IRRIGATION WATER DESILTING BOX AND TRASH SCREEN

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

COMPILED	CHECKED	DATE	DRAWING NO.
		3-31-59	7-L-36-20



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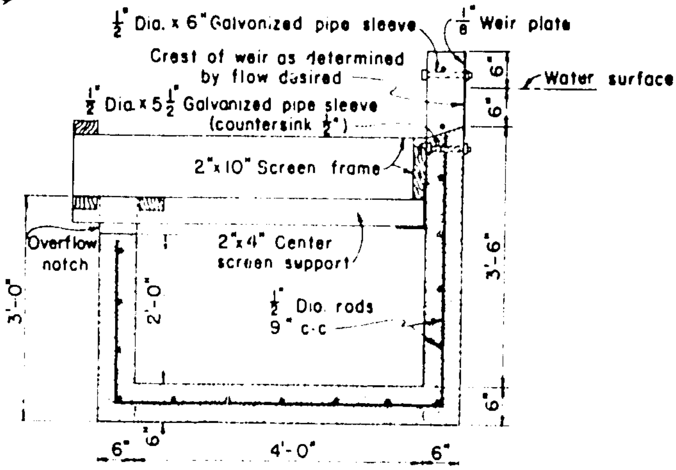
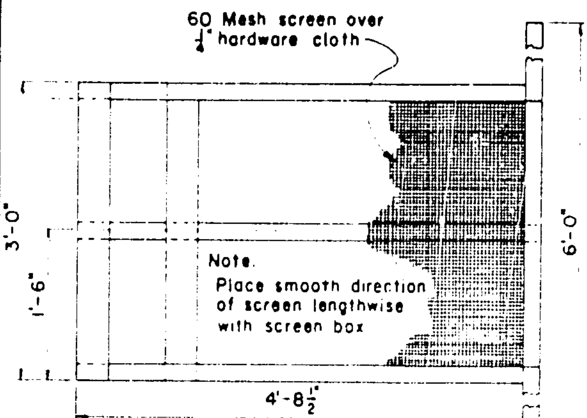
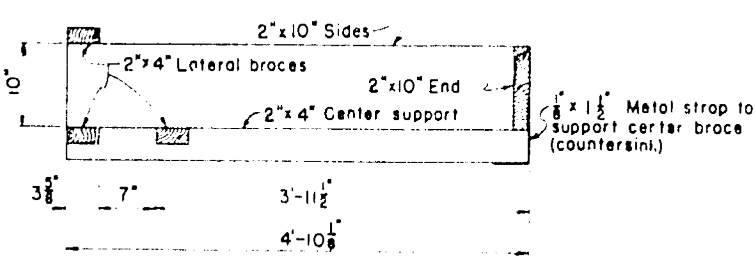


CENTERLINE OF CROSS SECTION



PLAN



SIDE VIEW

DETAIL OF TRASH SCREEN

TABLE OF QUANTITIES

ITEM	UNIT	QUANTITY
CONCRETE	CU. YDS.	1.85
REINFORCING STEEL	LIN. FT.	264
60 MESH COPPER SCREEN	SQ. FT.	14
1/4" HARDWARE CLOTH	SQ. FT.	14
LUMBER	BD. FT.	35
1/2" GALVANIZED PIPE SLEEVE 6" LONG	EACH	2
1/2" GALVANIZED PIPE SLEEVE, 5 1/2" LONG	EACH	1
3/8" GALVANIZED BOLTS, 6" LONG	EACH	3
3/8" GALVANIZED BOLTS, 6 1/2" LONG	EACH	2
1/8" WIER PLATE	EACH	1
1/8" X 1/2" X 12" METAL STRAP	EACH	1

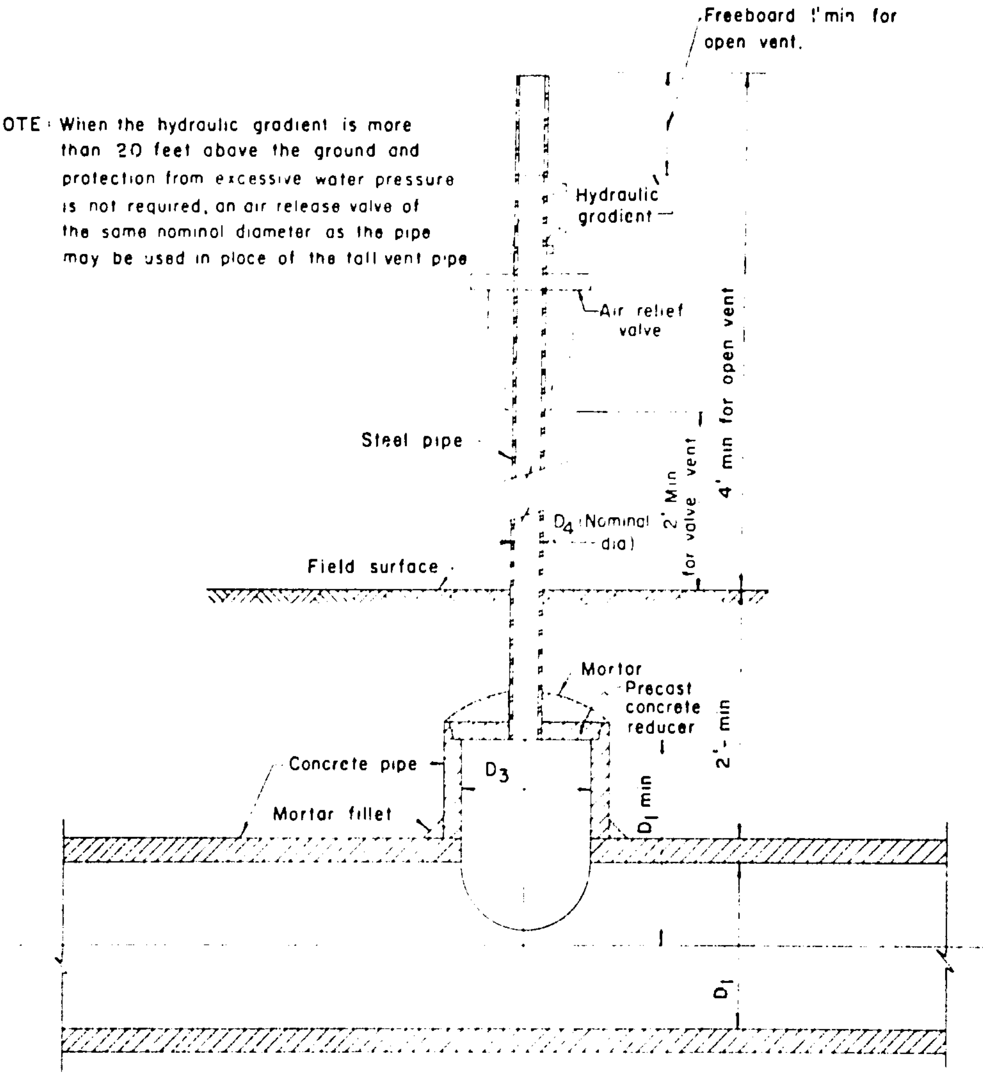
Capacity up to 900 G.P.M.

IRRIGATION WATER TRASH SCREEN

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

DRAWN	CHECKED	DATE	DRAWING NO.
		3-31-59	7-L-36-21

NOTE: When the hydraulic gradient is more than 20 feet above the ground and protection from excessive water pressure is not required, an air release valve of the same nominal diameter as the pipe may be used in place of the tall vent pipe



℄ CROSS SECTION

D_1 Inches	D_3 Min. Inches	D_4 Min Nominal Dia Inches
8	6	2
10	8	2
12	10	2
14	10	2
15	12	2
16	12	2 1/2
18	14	2 1/2
20	16	3
21	16	3
24	18	3 1/2

NOMENCLATURE

D_1 - Diameter of underground concrete pipe

D_3 - Diameter of concrete vent pipe

D_4 - Diameter (nominal) of steel vent pipe

VENT FOR CONCRETE PIPE LINES

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

12-59 | 7-L-36-9

APPENDIX 3

DESIGN AND INSTALLATION OF NONREINFORCED CONCRETE IRRIGATION PIPE SYSTEMS

American Society of Agricultural Engineers
ASAE Standard: ASAE S261.5

DESIGN AND INSTALLATION OF NONREINFORCED CONCRETE IRRIGATION PIPE SYSTEMS

Developed by Concrete Irrigation Pipe System Committee; approved by Soil and Water Division Standards Committee (SW-03); adopted by ASAE 1957; revised 1960, 1961, 1962, 1963, June 1968, May 1974, December 1978; reconfirmed for one year December 1979, March 1981, March 1982.

SECTION 1—PURPOSE AND SCOPE

1.1 This Standard is intended as a guide to engineers in the design and installation of low or intermediate pressure nonreinforced concrete irrigation pipelines and for the preparation of detailed specifications for a particular installation. It is restricted to pipelines with vents or stands open to the atmosphere. It is not intended to serve as a complete set of design criteria and construction specifications.

1.2 The systems designed and or installed under this Standard shall utilize pipe conforming to one or more of the following type of nonreinforced concrete irrigation pipe.

1.2.1 Pipelines with mortar joints. The pipe shall conform to the requirements of American Society for Testing and Materials Standard C118, Specifications for Concrete Pipe for Irrigation or Drainage.

1.2.2 Pipelines with rubber gasket joints. The pipe and gaskets shall conform to ASTM C505 Specifications for Nonreinforced Concrete Irrigation Pipe with Rubber Gasket Joints.

1.2.3 Cast-in-place pipelines. The pipe shall conform to American Concrete Institute Standard 346-70, Specifications for Cast-in-Place Nonreinforced Concrete Pipe.

SECTION 2—DESIGN CRITERIA

2.1 Pipeline

2.1.1 Safety factors

2.1.1.1 External load limit. Although loads are generally light on this type of installation, where there are excessively high fills over the pipe, a safety factor of at least 1.25 shall be applied to the 3-edge-bearing test in computing allowable heights of fill over precast pipe. The loads shall be determined by the methods outlined in ASAE Engineering Practice ASAE EP260, Design and Construction of Subsurface Drains in Humid Areas.

2.1.1.2 Pressure. Maximum working head for cast-in-place pipelines shall be 15 ft (4.6 m) above the centerline of the pipe. Maximum working heads for precast pipe shall not exceed 1/4 the certified hydrostatic test pressure as prescribed in ASTM C118 for mortar-jointed pipelines or 1/3 the certified hydrostatic test pressure as prescribed in ASTM C505 for pipelines with rubber gasket joints.

2.1.1.3 Soil conditions

2.1.1.3.1 Concrete pipelines shall not be installed on sites where the sulfate salt concentration exceeds 1.0 percent as water soluble sulfate in soil samples, or 4000 parts per million sulfate in groundwater samples. Concrete pipe made with Type V cement or cement whose tricalcium aluminate content does not exceed 5 percent shall be used on sites where the water soluble sulfate content in soil samples is 0.20 to 1.0 percent, or where the sulfate content of groundwater samples ranges from 1000 to 4000 parts per million. Concrete pipe made with Type II cement or cement with a tricalcium aluminate content of not more than 8 percent shall be used on sites where the water soluble sulfate concentration in soil samples ranges from 0.10 to less than 0.20 percent, or the sulfate content in groundwater samples is from 150 to 1000 parts per million. There are no restrictions as to the type of cement used in concrete pipe for sites where the sulfate content is less than 0.10 percent in soil, or 150 parts per million in groundwater.

2.1.1.3.2 Cast-in-place pipe shall be used only in stable soils or soils that have been stabilized as in Section 3.3 of the referenced ACI specifications, where the trench form conforms to the requirements for the trench as prescribed in Chapter 3 of ACI 346-70.

2.1.2 Friction loss. In computing friction loss for motar-jointed or cast-in-place pipelines, Scobey's concrete pipe equation with a coefficient of retardance K_s = 0.310 or Manning's equation with roughness coefficient n = 0.013 shall be used. Similar coefficients should be used for pipe with rubber gasket joints, except that for the smoothest makes of such pipe produced, the Scobey coefficient of retardance may range up to K_s = 0.370 and the Manning's roughness coefficient down to n = 0.011. Minor losses can be computed in accordance with current recommendations.

2.2 Stand requirements

2.2.1 Stands shall be placed at each inlet to a concrete irrigation pipe system and at such other points as required. All stands shall be supported on a base adequate to support the stand and prevent undue movement or stress on the pipeline. All stands shall serve as vents in addition to their other functions as follows:

2.2.1.1 They shall avoid entrainment of air.

2.2.1.2 They shall allow 1 to 5 ft (0.3 to 1.5 m) of freeboard.

2.2.1.3 If constructed of concrete pipe, they shall be constructed of Class II Reinforced Concrete Pipe as specified in ASTM C76, Specifications for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe, if of greater diameter than 24 in. (610 mm).

2.2.1.4 If cast in place, they shall contain steel reinforcing on not more than 1 ft centers to provide steel areas equal to or greater than the least values specified for Class II Reinforced Concrete Pipe in ASTM C76.

2.2.1.5 The tops of all stands shall be at least 4 ft (1.2 m) above the ground surface. If visibility is not a factor, stands may be lower if covered or equipped with trash guards.

2.2.2 Pump stands. Pump stands shall be:

2.2.2.1 Concrete box stands with vertical sides suitably reinforced to withstand handling and installation stresses.

2.2.2.2 Nontapered stands of concrete pipe suitably reinforced to withstand handling and installation stresses.

2.2.2.3 Nontapered concrete pipe stands, capped and having a vent pipe to the height of the hydraulic gradeline plus freeboard.

2.2.2.4 Steel cylinder stands mortared to a single concrete pipe riser.

2.2.3 The centerline of the pump discharge pipe shall have a minimum vertical offset above the centerline of the outlet pipe equal to the sum of the diameters of the inlet and outlet pipes.

2.2.4 Check valves shall be used in the pump discharge line wherever the potential backflow from the pipeline would be sufficient to drain the pipeline or damage the pump.

2.2.5 Construction shall be such as to insure that the vibration from the pump discharge pipe is not carried to the stand.

2.2.6 Velocities in stands

2.2.6.1 Downward water velocities shall not exceed 2 ft/sec (0.6 m/sec). In no case shall such velocities exceed the average pipeline velocity.

2.2.6.2 If the size of the stand is decreased above the pump discharge pipe, the top vent portion shall be of such inside cross-sectional area that, if the entire flow of the pump were discharging through it, the average velocity would not exceed 10 ft/sec (3.0 m/sec).

2.2.7 **Sand traps.** Pump stands serving as sand traps shall have a minimum inside diameter of 30 in. (762 mm) and shall be constructed so that the bottom is at least 24 in. (610 mm) below the invert of the outlet pipeline. Suitable provisions for cleaning sand traps shall be provided.

2.2.8 **Gate stands**

2.2.8.1 Gate stands shall be constructed of concrete pipe or cast in place. Reinforcing requirements under paragraphs 2.2.1.3 and 2.2.1.4 apply.

2.2.8.2 Dimensions of gate stands shall be sufficient to accommodate the gate or gates required.

2.2.8.3 Gate stands shall serve as vents.

2.2.8.4 Gate stands shall be of such dimensions that gates are accessible for repair.

2.2.9 **Float valve stands.** Float valve stands shall be of sufficient diameter to provide accessibility for maintenance and to dampen surge. (The wide-open friction loss for the valve approximates 2.4 velocity heads for the single disk type and 1.9 velocity heads for the double disk type.)

2.3 **Vent requirements**

2.3.1 **Locations.** Vents shall be placed:

2.3.1.1 At the downstream end of each lateral.

2.3.1.2 At a design point downstream from where there is opprotunity for air entrainment and inadequate opportunity for escape of that air.

2.3.1.3 At high points wherever there are changes in grade downward in direction of flow of more than 10 deg.

2.3.1.4 At all turns of 90 deg or more with the exception of lines not more than 50 ft (15 m) in length.

2.3.2 The design point in 2.3.1.2 shall be determined by the equation

$$L = 1.76\,VD$$

where

L = distance downstream from the air entraining stand in ft
V = maximum design velocity in ft per sec
D = inside diameter of the pipe in ft.

2.3.3 Any stand shall substitute for a vent.

2.3.4 There shall be considered opportunity for air entrainment at all gravity inlets and at pump stands where the pump might possibly pump air. When pumping from wells, if there is a downdraft of air into the well casing while the pump is in operation, the well shall be considered to pump air. In such case a vent shall be placed immediately downstream from the pump stand if the average downward velocity in the stand from the pump discharge to the pipeline exceeds 1 ft/sec (0.3 m/sec).

2.3.5 **Size**

2.3.5.1 The cross-sectional area of the vent shall be at least half the cross-sectional area of the pipeline (both inside measurements) for a distance of at least 1 pipeline diameter up from the centerline of the pipeline. Above this the vent may be reduced to 1/60 of the cross-sectional area of the pipeline, but not less than 2 in. (50 mm) diameter pipe shall be used.

2.3.5.2 Vents shall have a minimum freeboard of 1 ft (0.3 m) above the hydraulic gradeline. The maximum height shall not exceed the maximum working head of the pipe.

2.3.6 **Air release valves.** An air-vacuum release valve may be used in lieu of an open vent. The valve outlet shall have a 2 in. (50 mm) nominal minimum diameter. Two inch (50 mm) outlets shall be used for pipelines of 6 in. (152 mm) diameter or less, 3 in. (75 mm) outlets for pipelines of 7 to 10 in. (178 to 254 mm) diameter and 4 in. (102 mm) outlets for pipelines of 12 in. (305 mm) and larger diameter.

2.4 **Anchors**

2.4.1 Abrupt changes in pipeline grade or alignment require a stand of diameter greater than the pipeline or an anchor to absorb any axial thrust of the pipeline. An abrupt change shall be considered to be:

2.4.1.1 An angle of 45 deg or greater when the maximum working head is under 10 ft (3.0 m).

2.4.1.2 An angle of 30 deg or greater when the maximum working head is between 10 and 20 ft (3.0 and 6.1 m).

2.4.1.3 An angle of 15 deg or greater when the maximum working head is 20 ft (6.1 m) or more.

2.4.2 Anchors shall be constructed of concrete poured to fill the space between the pipe and the undisturbed earth at the side of the trench on the outside of bends or of plastic soil cement with at least 1 part of cement to 10 parts of soil of sandy loam or coarser texture, similarly placed.

2.4.3 The anchors shall be to the full height of the outside diameter of the pipe and shall have a minimum thicknes of 6 in. (152 mm). The length in feet normal to the direction of thrust is determined by the equation:

$$L = 98\,\frac{HD}{B}\,\sin\frac{a}{2}$$

where

H = maximum working head in ft
D = inside diameter of the pipe in ft
B = allowable passive pressure of the soil in lb per sq ft
a = deflection angle of the pipe bend
L = length of anchor in ft.

2.4.4 The pipe shall be clean and wet when placing the anchor, to provide a good bond between anchor and pipe. Where adequate soil tests ae now available, the allowable passive soil pressure shall be considered to be 500 lb/sq ft (23.9 kPa).

SECTION 3—INSTALLATION

3.1. **Size and location.** The pipe and appurtenances shall conform to the standards specified and shall be located and constructed as shown on the engineer's plans and in the construction specifications.

3.2 **Joints and connections**

3.2.1 Joints shall be motar or rubber gasket, as specified and where required. All joints shall be constructed to leave the inside of the pipeline and appurtenances free of any obstruction which would reduce capacity below design standards.

3.2.2 Joints in stands and connections to appurtenances shall conform to the requirements of ASTM C118 or ASTM C505.

3.2.3 Stoppage and horizontal joints for cast-in-place pipelines shall conform to the requirements of ACI 346-70. Connection joints shall be prepared by cleaning and freeing of loose or defective concrete, coatings, and foreign material. The contact faces of the pipe and fittings shall be wetted and the fitting mortared into place using bonding mortar as specified in ACI 346-70.

3.3 **Placement**

3.3.1 The pipelines shall be placed deep enough below the land surface to permit covering the pipe a minimum of 2 ft (0.6 m) unless shallower covering is specified for rocky areas or other local conditions. If shallower covering is specified, there shall be provision to protect the line from damage by vehicular traffic. Greater depths of cover shall be specified when local conditions indicate a need.

3.3.2 Where trenches are excavated in soils containing rock or other hard materials, or in soils subject to appreciable swelling and shrinking on wetting or drying, or where the trench bottom is unstable, the trenches shall be overexcavated and backfilled with selected materials to sufficient depth to provide a suitable base. if water is in the trench, it shall be drained away or controlled in a manner to prevent damage to the joint mortar and to maintain a suitable base.

3.3.3 Provisions in paragraphs 3.3.1 and 3.3.2 apply to mortar-jointed and rubber gasket pipe. Placement for cast-in-place pipe shall be as specified in ACI 346-70.

3.3.4 Rubber gasket pipe shall not be placed with the joints rammed together tight enough so that longitudinal compression develops from wetting expansion of the pipe. If there is any question concerning this with any particular joint design, the end of the spigot shall be pulled back from the shoulder of the bell a slight distance but not more than 0.06 in. (1.5 mm).

3.4 **Curing and backfilling**

3.4.1 Paragraphs 3.4.2, 3.4.3 and 3.4.4 apply to mortar-jointed pipelines only, and 3.4.5 applies to mortar-jointed and rubber gasket pipelines.

3.4.2 There shall be an initial backfill of soil around the pipe and covering the pipe to a depth of at least 6 in. (152 mm) for the full width of the trench and not more than 7 sections of pipe behind the laying. If laying ceases for 2 hrs or more, the initial backfill shall be brounght up to and cover the last completed joint. Nothing in this section shall prohibit the complete backfilling while mortar bands are still plastic. If complete backfilling is not done at this time, the completion shall be delayed at least 20 hrs, but must be completed to the minimum specified cover or 2 ft (0.6 m), whichever is less, before water is put into the line.

3.4.3 Mortar joints are to be protected from drying out. If the soil in the initial backfill is not thoroughly moist, a suitable membrane over the mortar shall be used. Membranes consisting of one layer of kraft paper, or paper cut from cement sacks, or membranes conforming to ASTM C171, Specifications for Sheet Materials for Curing Concrete, or ASTM C309, Specifications for Liquid Membrane-Forming Compounds for Curing Concrete, shall be considered suitable.

3.4.4 In areas where rips (longitudinal cracks or ruptures) have been known to occur, it is important that the soil used in the initial backfill be thoroughly moist and of a texture coarser than clay. (Soil will not ribbon in fingers when moist, or shall not have more than 30 percent by weight of material finer than 2 microns.) This generally may be accomplished by scheduling laying no more than 1 day following trenching and by being sure the soil is moist when trenching is undertaken. This may indicate the need for deep irrigation along the trench line 3 days to a week before actual trenching.

3.4.5 All openings into a pipeline shall be covered or closed to prevent air circulation, except while work is actually in progress, and shall be kept closed until the pipeline is completed and is to be filled with water.

3.4.6 For cast-in-place pipelines, curing and backfilling shall be in accord with the provisions of ACI 346-70.

3.5 Testing

3.5.1 It shall be demonstrated that all pipelines function properly at agreed design capacity. At or below agreed design capacity there shall be no objectionable surge or water hammer. To be objectionable there shall be either:

3.5.1.1 Continuing, unsteady delivery of water.

3.5.1.2 Damage to the system.

3.5.1.3 Detrimental overflow from vents or strands.

3.5.2 Pipelines shall be tested for leaks by observing normal operation any time after a period of 2 wks of continuous wetting. All visible leaks shall be repaired. Losses shall not exceed 0.10, 0.05 nor 0.02 ft³/ft² (3.0, 1.5, nor 0.5 cm³/cm²) of inside surface per 24 hrs for cast-in-place, mortar-jointed, and rubber gasket pipelines, respectively. Water less than 50 deg F (10 deg C) shall not be used for the testing of mortar-jointed or cast-in-place pipelines.

3.6 Guarantee. Unless otherwise specifically agreed to, the installer guarantees the system for 1 yr against faulty materials, faulty workmanship and failure to meet the requirements of the contract. Such guarantee shall not apply to earthquake or land settlement damage (except from improper trench bottom preparation or inadequate foundations for structures), external damage not caused by the seller's men or equipment, or leakage of mortar-jointed pipe caused by water of temperature less than 50 deg F (10 deg C).

SECTION 4—DEFINITION OF TERMS

4.1 Appurtenances

4.1.1 Gate. A device used to control the flow of water in the pipeline. It may be opened and closed by screw action or by slide action. The latter is used only where pressures and velocities in the line are so low that sudden closure will not cause excessive water hammer. Types of gates, indicative of the place they will be used, are:

4.1.1.1 Gate, line. A hub-end gate which is mortared into the pipeline. It is a screw type gate.

4.1.1.2 Gate, stand. A gate in a stand covering an inlet into a pipeline, and controlling the flow into such a pipeline. It is of either the screw or the slide type. The slide type has a device to lock it in any desired position.

4.1.1.3 Gate, stand, pressure. A gate in a stand covering an outlet from a pipeline. It is of the screw type.

4.1.2 Inlet. An appurtenance to deliver water to a pipeline system.

4.1.2.1 Inlet, gravity. A structure to control the flow of water from an open conduit into a pipeline. It may be combined with a baffle, gate, screen and/or a sand trap.

4.1.2.2 Stand, pump. A structure where water enters a system from a pump or pressure system.

4.1.3 Outlet. An appurtenance to deliver water from a pipe system to the land or to any surface pipe system. An outlet may involve the use of a valve, riser pipe, and/or an outlet gate. Several types of outlets are defined as follows:

4.1.3.1 Distributor, swivel-arm. This type of outlet has a valve and two arms of gated pipe attached to and swiveling from the top of the riser (usually a steel pipe riser). Chained to a center post, they are out of the way of cultivation, and when dropped, the gates allow the water to be distributed into furrows.

4.1.3.2 Gate, outlet. Usually a slide gate, or other type of valve, used to control the flow out of the outlet.

4.1.3.3 Hydrant, portable. An outlet used for connecting surface pipe to an alfalfa valve outlet.

4.1.3.4 Outlet, pipe, surface. An outlet used for attachment of surface pipe without a portable hydrant.

4.1.3.5 Pot, open. A simple orchard valve with a vertical piece of pipe (larger than the riser) placed over and mortared to the top of the riser. There are two or more slide gates on the sides of the pot.

4.1.3.6 Riser, capped or pot. A riser extending above ground surface with a water tight cap over the top and with outer gates on the sides slightly above the ground surface (capped riser). To accommodate more outlet gates, sometimes a pot is used on top, larger than the riser (capped pot). Sometims die-cast screw type valves are used on these capped pots in place of the outlet gates. Outlet gates, placed on the outside of the riser or pots, as they must be for the closed type, produce an erosive jet of water, which the die-cast valves eliminate.

4.1.3.7 Valve, alfalfa. An outlet which has a disk on top to control flow, and has an opening equal in diameter to the inside diameter of the riser. A ring around the base outside of the disk provides a seat and seal for a portable hydrant. Some alfalfa valves have a small air release valve on the disk which provides for drainage of puddles following irrigation as a mosquito abatement measure, and for supplemental air release from a pipeline during filling.

4.1.3.8 Valve, alfalfa, modified. This modified alfalfa valve is the same as the alfalfa valve, except that the ring is omitted. The only portable hydrants that can be used, therefore, are those types which fit over the riser pipe.

4.1.3.9 Valve, orchard. The orchard valve is inserted inside the riser pipe. Like the alfalfa valve, the orchard valve provides flow control by screwing a horizontal disk up and down from a seat below. However, the opening is smaller than the inside diameter of the riser pipe, and, therefore, the flow capacity is much less. The top of the riser may be cut off at or slightly below ground surface, may rise 6 to 12 in. (152 to 305 mm) above ground surface with a notch cut in one side, or may similarly rise above the ground and have two or more outlet gates inserted in the riser a few inches off the ground.

4.1.4 Stand. A structure formed from vertical sections of pipe, or sometimes from concrete cast-in-place (box stands). It may serve as a pump stand, gate stand, or float valve stand. In addition it may also function as a vent and/or sand trap. Sometimes, when gates are not required in the stands, they are capped with a smaller vent to above the hydraulic gradeline. Float valve stands are used on the steeper slopes where the rate of supply can be varied and automatic control offers advantages.

4.1.5 Valve, float. A valve, actuated by a float in a stand, which controls the flow of water into the stand.

4.1.6 Vent. An appurtenance to the pipeline which permits the passage of air to or from the pipeline.

4.2 Hydraulic terms. Hydraulic terms shall be as defined in the American Society of Civil Engineers Manual of Engineering Practice No. 11, except as noted below:

4.2.1 Freeboard. The vertical distance to tops of vents, or stands above the elevation of the hydraulic gradeline at working head.

4.2.2 Head, working. The vertical distance that water will rise in a vent or stand above the centerline of the pipeline at design flow at any point in the system. On profiles of the pipelines, maximum working head shall be, at any point, the vertical distance from the centerline of the pipeline to a straight line drawn between the tops of consecutive vents and/or stands. It is, thus, the working head plus freeboard.

4.2.3 Surge. That phenomenon wherein a rocking or oscillating motion of the water is set up, causing flow to be unsteady.

4.2.4 Water hammer. That phenomenon, resulting from checking of flow, etc., wherein pressure waves pass through the water at the speed of sound. Water hammer can produce excessive momentary pressure. It is not to be confused with surge, although under certain conditions both may be activated simultaneously.

Cited Standards:

ASAE EP260, Design and Construction of Subsurface Drains in Humid Areas
ACI 346-70, Specifications for Cast-in-Place Nonreinforced Concrete Pipe
ASTM C76, Specifications for Reinforced Culvert, Storm Drain, and Sewer Pipe
ASTM C118, Specifications for Concrete Pipe for Irrigation or Drainage
ASTM C171, Specifications for Sheet Materials for Curing Concrete
ASTM C309, Specifications for Liquid Membrane-Forming Compounds for Curing Concrete
ASTM C505, Specifications for Nonreinforced Concrete Irrigation Pipe with Rubber Gasket Joints

APPENDIX 4

DESIGN, INSTALLATION AND PERFORMANCE OF UNDERGROUND THERMOPLASTIC IRRIGATION PIPELINES

American Society of Agricultural Engineers
ASAE Standard: ASAE S376.1

DESIGN, INSTALLATION AND PERFORMANCE OF UNDERGROUND, THERMOPLASTIC IRRIGATION PIPELINES

Developed by the ASAE Irrigation Water Supply and Conveyance Committee; approved by the Soil and Water Division Standards Committee; adopted by ASAE April 1975; reconfirmed for one year December 1979, February 1981; revised April 1982.

SECTION 1—PURPOSE AND SCOPE

1.1 Purpose. Thermoplastic pipe is manufactured in several size classifications from different materials of various grades, types and formulations involving many different specifications. It is used for applications other than irrigation where certain requirements often apply to pipe used for a specific purpose. This Standard pertains to thermoplastic pipe used underground for irrigation and is intended to:

1.1.1 Provide minimum guidelines for engineers and others in planning, designing and specifying thermoplastic pipe commonly used for irrigation. It is not intended as a complete specification nor to replace the judgment of personnel familiar with site conditions or other controlling factors.

1.1.2 Consolidate applicable reference information and technical data in readily available form.

1.1.3 Establish uniform standards for materials used in the manufacture of thermoplastic irrigation pipe and to promote uniformity in classifying, pressure rating, testing and marking the pipe.

1.1.4 Establish minimum requirements for the design, installation and testing of pipelines which are necessary for the satisfactory performance and safe operation of the irrigation system and to prevent damage to the system.

1.2 Scope. This Standard applies to underground, thermoplastic pipelines used in the conveyance of irrigation water to the point of distribution and may or may not apply to potable water systems.

1.2.1 **High pressure pipelines.** This term applies to underground pipelines constructed of thermoplastic pipe from 21 to 710 mm (1/2 to 27 in.) nominal diameter that are closed to the atmosphere, and subject to internal pressures, including surge pressures, from 550 to 2170 kPa (80 to 315 psi).

NOTE: Nominal pipe size in millimeters is the actual outside pipe diameter to the nearest millimeter for OD controlled pipe and the actual inside diameter to the nearest millimeter for ID controlled pipe.

1.2.2 **Low pressure pipelines.** This term applies to underground thermoplastic pipelines 114 to 630 mm (4 to 24 in.) nominal diameter that are used in systems subject to pressures of 545 kPa (79 psi) or less.

SECTION 2—DEFINITIONS

2.1 Design area: The specific land area in which pipelines are planned and located to serve as integral parts of an irrigation water distribution or conveyance system, designed to facilitate conservation, use and management of water and soil resources, and which the supplier or designer and purchaser mutually understand to be irrigated.

2.2 Irrigation system: All equipment required to apply water to the design area.

2.3 Irrigation pipelines: Includes the underground, thermoplastic pipelines and appurtenances installed in an irrigation system.

2.4 Outlets: Appurtenances required to deliver water from the pipeline to an individual sprinkler or to a lateral of sprinklers, to surface pipe located on the ground, to distribution pipe or laterals containing surface or subsurface emitters or tricklers, to surface valves, or to open ditches.

2.5 Hydrostatic design stress: The estimated maximum tensile stress in the wall of the pipe in the circumferential orientation, due to internal hydrostatic water pressure, that can be applied continuously with a high degree of certainty that failure of the pipe will not occur.

2.6 Pressure rating (PR): The estimated maximum pressure that water in the pipe can exert continuously with a high degree of certainty that failure of the pipe will not occur.

2.7 Dimension ratio (DR): The ratio of pipe diameter to wall thickness.

2.7.1 For outside diameter (OD) based pipe, which includes polyvinyl chloride (PVC), acrylonitrile-butadiene-styrene (ABS) pipe and some polyethylene (PE) pipe, the ratio is calculated by dividing the pipe's average outside diameter by the pipe's minimum wall thickness. The minimum wall thickness shall not be less than 1.52 mm (0.060 in.). Certain DR values have been selected as standard and given the designation Standard Dimension Ratio (SDR). The SDR and DR values for PVC and ABS are rounded to the nearest 0.5.

2.7.2 For inside diameter (ID) based pipe, which includes some PE pipe, the ratio is calculated by dividing the average inside diameter of the pipe by the pipe's minimum wall thickness. The minimum wall thickness shall not be less than 1.52 mm (0.060 in.). The SDR values shall be rounded to the nearest 0.1.

2.8 Relation between standard dimension ratio, hydrostatic design stress and pressure rating: The following expression, commonly known as the ISO equation (from International Organization for Standardization Standard ISO 161/1-1978, Thermoplastic Pipes for the Transport of Fluids—Nominal Outside Diameters and Nominal Pressures—Part 1: Metric Series), is used to relate standard dimension ratio, hydrostatic design stress, and pressure rating:

2.8.1 For OD based pipe:

$$2\,S/P = R - 1$$

or

$$2\,S/P = (D_o/t) - 1$$

where

S = hydrostatic design stress, kPa (psi)
P = pressure rating, kPa (psi)
D_o = average outside diameter, mm (in.)
t = minimum wall thickness, mm (in.)
R = dimension ratio, DR (equals D_o/t for PVC, ABS, and other OD based pipe)

2.8.2 For ID based pipe:

$$2\,S/P = R + 1$$

or

$$2\,S/P = (D_i/t) + 1$$

where

R = dimension ratio, DR (equals D_i/t for ID based pipe such as some PE pipe
D_i = average inside diameter, mm (in.)

SECTION 3—DESIGN CRITERIA

3.1 Working pressure

3.1.1 **General.** The pipeline shall have a pressure class rating (see Table 1) greater than the static or working pressure plus surge at any point in the system. Surge pressures should not exceed 28 percent of the pipe's pressure class rating; therefore, if surge is not

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TABLE 1—PRESSURE RATINGS (PR) FOR NONTHREADED THERMOPLASTIC PIPE*†

SDR‡ OD based pipe	SDR‡ ID based pipe	PVC materials (all pipes OD based): PVC 1120, PVC 1220, PVC 2120 psi	kPa§	PVC 2116 psi	kPa	PVC 2112 psi	kPa	PVC 2110 psi	kPa	PE materials (pipes made to both OD & ID basis): PE 3408 psi	kPa	PE 3406, PE 3306, PE 2306 psi	kPa	PE 2305 psi	kPa	ABS materials (all pipes OD based): ABS 1316 psi	kPa	ABS 2112 psi	kPa	ABS 1210 psi	kPa
	5.3									250	1725	200	1380	160	1105						
	7.0									200	1380	160	1105	125	860						
11.0	9.0									160	1105	125	860	100	690						
13.5	11.5	315	2170	250	1725	200	1380	160	1105							250	1725	200	1380	160	1105
17.0	15.0	250	1725	200	1380	160	1105	125	860	100	690	80	550	63	435	200	1380	160	1105	125	860
21.0		200	1380	160	1105	125	860	100	690	80	550	64	440			160	1105	125	860	100	690
26.0		160	1105	125	860	100	690	80	550	64	440	50	345			125	860	100	690	80	550
32.5		125	860	100	690	80	550	63	435	50	345	40	275			100	690	80	550	64	440
41.0		100	690	80	550	63	435	50	345	40	275	31	215			80	550	64	440	50	345
51.0		80	550	63	435	50	345	40	275							64	440	50	345	40	275
64.0		63	435	50	345	40	275	30	205												
81.0		50	345	40	275	30	205	25	170							40	275	30	205	25	170
93.5#		43	295																		
50 ft head		22	150																		

*For water at 23 °C (73.4 °F).
†Pressure ratings are determined by the ISO equation as shown in paragraph 2.8 using the hydrostatic design stress values shown in Table 5.
‡SDR = Standard Dimension Ratio determined as shown in paragraph 2.7.
§kPa = kilopascals, kN/m²
#The dimension ratio 93.5 is non standard and is referred to as DR (Dimension Ratio)

TABLE 2—MAXIMUM ALLOWABLE PRESSURE FOR NONTHREADED THERMOPLASTIC PIPES WHEN SURGE PRESSURES ARE NOT KNOWN*†

SDR OD based pipe	SDR ID based pipe	PVC materials (all pipes OD based): PVC 1120, PVC 1220, PVC 2120 psi	kPa	PVC 2116 psi	kPa	PVC 2112 psi	kPa	PVC 2110 psi	kPa	PE materials (pipes made to both OD & ID basis): PE 3408 psi	kPa	PE 3406, PE 3306, PE 2306 psi	kPa	PE 2305 psi	kPa	ABS materials (all pipes OD based): ABS 1316 psi	kPa	ABS 1212 psi	kPa	ABS 1210 psi	kPa
	5.3									180	1240	144	995	115	795						
	7.0									144	995	115	795	90	620						
11.0	9.0									115	795	90	620	72	495						
13.5	11.5	227	1565	180	1240	144	995	115	795							180	1240	144	995	115	795
17.0	15.0	180	1240	144	995	115	795	90	620	72	495	58	400	45	310	144	995	115	795	90	620
21.0		144	995	115	795	90	620	72	495	58	400	45	310			115	795	90	620	72	495
26.0		115	795	90	620	72	495	58	400	45	310	36	250			90	620	72	495	58	400
32.5		90	620	72	495	58	400	45	310	36	250	29	200			72	495	58	400	46	315
41.0		72	495	58	400	45	310	36	250	29	200	22	150			58	400	46	315	36	250
51.0		58	400	45	310	36	250	29	200							46	315	36	250	29	200
64.0		45	310	36	250	29	200	22	150												
81.0		36	250	29	200	22	150	18	125							29	200	22	150	18	125
93.5		31	215																		
50 ft head		21	145																		

*Maximum allowable working pressure = pressure rating (PR) × 0.72 for SDR and DR pipe.
†For water at 23°C (73.4°F).

TABLE 3—MAXIMUM, OR CRITICAL, SURGE PRESSURE FOR THERMOPLASTIC PIPE

Pipe SDR (or DR) OD based	Pipe SDR (or DR) ID based	Surge pressure* per ft/s (0.3 m/s) of sudden change in flow velocity: For pipe material of 400,000 psi (2800 MPa) modulus (includes most PVC) psi	kPa	For pipe material of 300,000 psi (2100 MPa) modulus (includes most ABS) psi	kPa	For pipe material of 100,000 psi (700 MPa) modulus (includes most PE) psi	kPa
	5.3	28.1	195	24.3	170	14.0	95
	7.0	25.1	175	21.7	150	12.5	85
11.0	9.0	22.5	155	19.5	135	11.2	75
13.5	11.5	20.3	140	17.6	120	10.2	70
17.0	15.0	18.0	125	15.6	110	9.0	60
21.0		16.1	110	13.9	95	8.0	55
26.0		14.4	100	12.5	85	7.2	50
32.5		12.9	90	11.2	75	6.4	45
41.0		11.4	80	9.9	70	5.7	40
51.0		10.2	70	8.8	60	5.1	35
64.0		9.1	65	7.9	55	4.5	30
81.0		8.1	55	7.0	50	4.0	30
93.5		7.5	50	6.5	45	3.2	20

$$* \quad P = V \left(\frac{3960\ ET}{et + 300{,}000D} \right)^{1/2}$$

where

P = surge pressure, psi
V = sudden change in velocity, ft/sec
E = modulus of elasticity of pipe material, psi
t = pipe wall thickness, inch
D = pipe inside diameter (ID), inch

See also: Seipt, W.R. 1974. Water hammer considerations for PVC pipeline in irrigation systems. TRANSACTIONS of the ASAE 17(3): 417-423.

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TABLE 4—PRESSURE RATING SERVICE FACTORS FOR TEMPERATURES FROM 23 TO 60 °C (73.4 TO 140 °F) FOR PVC AND PE PIPES*†

Temperature °C	°F	PVC factor	PE factor
23	73.4	1.00	1.00
26.7	80	0.88	0.92
32.2	90	0.75	0.81
37.8	100	0.62	0.70
43.3	110	0.50	
48.9	120	0.40	
54.4	130	0.30	
60.0	140	0.22	

*To obtain the pressure rating for a temperature above 23 °C (73.4 °F), multiply the pressure rating at 23 °C (73.4 °F) as given in Table 1 or Table 2 as appropriate by the corresponding service factor. For PE pipe having improved strength retention with an increase in temperature, and PE pipe used at temperatures exceeding 38 °C (100 °F), the manufacturer should be consulted for recommended service factors.

†For ABS pipe used at temperatures above 23 °C (73.4 °F) service factors recommended by the manufacturer should be used.

known, the working pressure shall not exceed the maximum allowable working pressure as given in Table 2 for the particular pipe and SDR or DR used. Maximum or critical pressure as a function of pipe SDR or DR is shown in Table 3 for thermoplastic pipe having different moduli of elasticity.

3.1.2 Service factor. All pressure ratings are determined in a water environment of 23 ± 2 °C (73.4 ± 3.6 °F). As the temperature of the environment or fluid increases, the pipe becomes more ductile. Therefore, the pressure rating must be decreased for use at higher temperatures to allow for safe operation of the pipe. The service factors for PVC and PE are shown in Table 4. For PE pipe having improved strength retention with an increase in temperature and PE pipe used at temperatures exceeding 38 °C (100 °F), the manufacturer should be consulted for recommended service factors. For ABS pipe, service factors recommended by the manufacturer should be used.

3.2 System capacity. The design capacity of the pipeline shall be sufficient to provide an adequate flow of water for all methods of irrigation planned.

3.3 Friction losses. For design purposes, friction head losses shall be no less than those computed by the Hazen-Williams equation using a flow coefficient (C) equal to 150.

3.4 Flow velocity. The design water velocity in a pipeline when operating at system capacity should not exceed 1.5 m/s (5 ft/s) unless special considerations are given to the control of surge or water hammer and adequate protection from these pressures is provided (see paragraph 3.1.1 and Table 3). Adequate pressure and/or air relief valves shall be used with all velocities.

3.5 Outlets. Outlets shall have adequate capacity at the pipeline working pressure to deliver the design flow to the distribution system at the design operating pressure of the respective systems, i.e., sprinklers, surface pipe, emitters, tricklers, etc.

3.6 Check valves. A check valve shall be installed between the pump discharge and the pipeline where detrimental back flow may occur. It shall be designed to close, without slamming shut, at the point of zero velocity before damaging reversal of flow can occur.

3.7 Pressure relief valves. These shall be installed between the pump discharge and the pipeline when excessive pressures can develop by operating with all valves closed. Pressure relief valves or surge chambers shall be installed on the discharge side of the check valve where back flow may occur and at the end of the pipeline when needed to relieve surge.

3.7.1 Low pressure systems. Pressure relief valves may be used as alternatives to serve the pressure relief functions of vents and stands open to the atmosphere. They do not function as air release valves and should not be substituted for such valves where release of entrapped air is required.

3.7.1.1 Pressure relief valves shall be large enough to pass the full pump discharge with a pipeline pressure no greater than 50 percent above the permissible working head of the pipe.

3.7.1.2 Pressure relief valves shall be marked with the pressure at which the valve starts to open. Adjustable valves shall be installed in such a manner to prevent changing of the adjustment marked on the valve.

3.7.2 High pressure systems. The ratio of nominal size pressure relief valves to pipeline diameter shall be no less than 0.25. Pressure relief valves shall be set to open at a pressure no greater than 34.5 kPa (5 psi) above the pressure rating of the pipe or the lowest pressure rated component in the system.

3.8 Air release and vacuum relief valves. Air release and vacuum relief valves shall be installed at all summits, at the ends, and at the entrance of pipelines to provide for air escape and air entrance. Combination air-vacuum release valves which provide both functions may be used.

3.8.1 Air flow capacity. Valves having large orifices to exhaust large quantities of air from pipelines when filling and to allow air to enter to prevent a vacuum when draining are required at the end and entrance of all pipelines. Valves intended to release entrapped air only may have smaller orifices and are required at all summits.

3.8.2 Low pressure systems (not open to the atmosphere).

3.8.2.1 Air-vacuum release valves shall be provided at each of the locations described in paragraph 4.5.3.

3.8.2.2 The size of valve outlet for low pressure systems shall be as specified in paragraph 4.6.2.

3.8.3 High pressure systems. The ratio of air release valve diameter to pipe diameter for valves intended to release air when filling the pipe should not be less than 0.1. However, smaller diameter valves may be used as a means of limiting water hammer pressures by controlling air release where filling velocities cannot be controlled. Equivalent valve outlet diameters of less than 0.1 are permitted for continuously acting air release valves. Adequate vacuum relief must still be provided. It is not only very important to select the correct air release or vacuum breaker valve, but also to select the right size and to locate valves properly at all places where needed. Air vacuum release valves shall be used as follows (all valve diameters refer to the total cross-sectional flow area of the vent or port outlet).

Pipe diameter mm (in.)	Minimum air-vacuum release valve outlet diameter mm (in.)
102 (4) or less	13 (0.5)
127-203 (5-8)	25 (1)
254-500 (10-20)	51 (2)
530 (21) or larger	0.1 pipe diameter

3.9 Draining. Provisions shall be made for draining the pipeline completely where a hazard is imposed by freezing temperatures, drainage is recommended by the manufacturer of the pipe, or drainage of the line is specified for any reason. Where provisions for drainage are required, drainage outlets shall be located at all low places in the line. The outlets may drain into dry wells or to points of low elevation. If drainage cannot be provided by gravity, provisions shall be made to empty the line by pumping.

3.10 Flushing. Where provision is needed to flush the line free of sediment, a suitable valve shall be installed at the distal end of the pipeline.

3.11 Gate stands and float valve stands. When these are used in low pressure pipelines not open to the atmosphere, refer to the criteria in paragraphs 4.4.1 and 4.4.2.

SECTION 4—SPECIAL DESIGN CRITERIA FOR LOW PRESSURE PIPELINE SYSTEMS OPEN TO THE ATMOSPHERE

4.1 Stands, general. Stands shall be used wherever water enters the pipeline to avoid entrapment of air, to prevent surge pressures, to avoid collapse due to negative pressures, and to prevent pressure from exceeding the head class of the pipe. Stands shall be supported on a base adequate to support the stand and prevent movement or undue stress on the pipeline. Stands shall be designed:

4.1.1 To allow at least 0.3 m (1 ft) of freeboard above design working head. The stand height above the centerline of the pipeline

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shall be such that neither the static head nor the design working head plus freeboard shall exceed the head class of the pipe.

4.1.2 With the top of each stand at least 1.2 m (4 ft) above the ground surface, except for surface gravity inlets, which shall be equipped with trash racks and covers.

4.1.3 With downward water velocities in stands not to exceed 0.6 m/s (2 ft/s). The inside diameter of the stand shall not be less than the inside diameter of the pipeline.

4.2 **Pump stands.** When the water velocity of an inlet exceeds three times the velocity of the outlet, the centerline of the inlet shall have a minimum vertical offset from the centerline of the outlet at least equal to the sum of the diameters of the inlet and outlet pipes. The cross-sectional area of the stands may be reduced above a point 0.3 m (1 ft) above the top of the upper inlet, but in no case shall the reduced cross section be such that it would produce an average velocity of more than 3 m/s (10 ft/s) if the entire flow was discharging through it.

4.2.1 **Types.** Pump stands shall be one of the following types:

4.2.1.1 Steel cylinder stands.

4.2.1.2 Concrete box stands with vertical sides, suitably reinforced.

4.2.1.3 Nontapered stands of concrete pipe, suitably reinforced.

4.2.1.4 Nontapered stands of concrete pipe, capped and having a vent pipe of a height exceeding the hydraulic gradeline plus freeboard.

4.2.2 **Vibration control.** Construction shall insure that the vibration from the pump discharge is not transmitted to the stand. Vibration control also applies to low-head pipelines not open to the atmosphere when pump stands are used.

4.3 **Sand traps.** Sand traps, when combined with a stand, shall have a minimum inside dimension of 762 mm (30 in.) and shall be constructed so that the bottom is at least 610 mm (24 in.) below the invert of the outlet pipeline. The downward velocity of flow of the water in a sand trap shall not exceed 0.08 m/s (0.25 ft/s). Suitable provision for cleaning sand traps shall be provided.

4.4 **Gate stands and float valve stands**

4.4.1 **Gate stands.** Gate stands shall be of sufficient dimension to accommodate the gate or gates, and shall be large enough to make the gates accessible for repair.

4.4.2 **Float valve stands.** Float valve stands shall be large enough to provide accessibility for maintenance and to dampen surge.

4.5 **Vent requirements.** Vents shall be designed into the system to provide for the removal of air and protection from surge.

4.5.1 Vents shall have a minimum freeboard of 0.3 m (1 ft) above the hydraulic gradeline. The maximum height of the vent above the centerline of the pipeline must not exceed the working head class of the pipe.

4.5.2 Vents shall have a cross-sectional area of at least one-half the cross-sectional area of the pipeline (both inside measurements) for a distance of at least one pipeline diameter up from the centerline of the pipeline. Above this elevation the vent may be reduced to 51 mm (2 in.) in diameter.

4.5.3 Vents shall be located as follows:

4.5.3.1 At the downstream end of each lateral.

4.5.3.2 At all summits of the line.

4.5.3.3 At points where there are changes in grade of more than 10 deg (18 percent) in a downward direction of flow.

4.5.3.4 Immediately below any stand if the downward velocity in the stands exceeds 0.6 m/s (2 ft/s).

4.6 **Air-vacuum release valves**

4.6.1 An air-vacuum release valve may be used in lieu of an open vent, but either a vent or an air-vacuum release valve shall be provided at each of the locations listed in paragraph 4.5.3.

4.6.2 Air-vacuum release valve outlets shall have a 51 mm (2 in.) minimum diameter. The valves shall be used as follows:

Pipe diameter mm (in.)	Minimum air-vacuum release valve outlet diameter mm (in.)
152 (6) or less	51 (2)
178-254 (7-10)	76 (3)
305 (12) or larger	102 (4)

NOTE: Air-vacuum release valves shall not replace the open stand required in paragraph 4.1.

SECTION 5—PIPE MATERIALS

5.1 **Compounds.** This Standard covers pipe made from the compounds that are listed and identified in this section by code classification and that are further defined and identified by hydrostatic design stress rating. The respective pipe compound shall have an established long term hydrostatic design stress rating as given in Table 5 when tested in accordance with paragraph 5.1.1. The compound shall meet the short term test requirement denoted by its code classification and defined in the relevant American Society for Testing and Materials Standards referenced in paragraph 5.2.

5.1.1 **Sustained pressure.** The pipe shall not fail, balloon, burst, or weep as defined in Section 4 of ASTM Standard D1598, Test for Time-to-Failure of Plastic Pipe Under Long-Term Hydrostatic Pressure. The pipe shall be treated in accordance with the following section of the applicable ASTM Standard:

5.1.1.1 PVC: Section 7.5 of ASTM Standard D2241, Specifications for PVC Plastic Pipe, at the appropriate test pressure given in Table 3 of that specification or Table 6 of this Standard.

TABLE 5 — MAXIMUM HYDROSTATIC DESIGN STRESS FOR THERMOPLASTIC PIPE

Compound	ASTM code classification	Type, grade	Standard code designation*	Hydrostatic design stress† psi	Hydrostatic design stress† MPa
PVC	12454-B	I, 1	PVC 1120	2000	13.8
PVC	12454-C	I, 2	PVC 1220	2000	13.8
PVC	14333-D	II, 1	PVC 2120	2000	13.8
PVC	14333-D	II, 1	PVC 2116	1600	11.0
PVC	14333-D	II, 1	PVC 2112	1250	8.6
PVC	14333-D	II, 1	PVC 2110	1000	6.9
PE	IVC-P34	III, 4	PE 3408	800	5.5
PE	IVC-P34	III, 4	PE 3406	630	4.3
PE	IIIC-P33	III, 3	PE 3306	630	4.3
PE	IIC-P23	II, 3	PE 2306	630	4.3
PE	IIC-P23	II, 3	PE 2305	500	3.4
ABS	3-5-5	I, 3	ABS 1316	1600	11.0
ABS	4-4-5	II, 1	ABS 2112	1250	8.6
ABS	5-2-2	I, 2	ABS 1210	1000	6.9

*Applies to compounds for pressure pipe

$\dagger \text{Hydrostatic design stress} = \frac{\text{long-term hydrostatic strength}\ddagger}{2.0}$

‡Long-term hydrostatic strength is determined by ASTM Standard D1598, Test for Time-to-Failure of Plastic Pipe Under Long-Term Hydrostatic Pressure, and ASTM Standard D2837, Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials.

NOTE: Recommended design stress values are issued by the Plastics Pipe Institute, New York, NY and are reissued periodically. Design stress values were issued in Technical Report TR-4, 1982.

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TABLE 6—SUSTAINED PRESSURE TEST CONDITIONS FOR PVC PLASTIC PIPE*†

SDR (DR)	Pressure required for test‡							
	PVC 1120 PVC 1220 PVC 2120		PVC 2116		PVC 2112		PVC 2110	
	psi	kPa	psi	kPa	psi	kPa	psi	kPa
51	170	1170	135	930	115	795	90	620
64	135	930	105	725	90	620	75	515
81	105	725	85	585	70	485	60	415
93.5	90	620						
50 ft head	83	570						

*Requirements in addition to those listed in ASTM Standard D2241, Specification for Poly (Vinyl Chloride) (PVC) and Chlorinated Poly (Vinyl Chloride) (CPVC) Plastic Pipe (SDR-PR), for SDR rated PVC plastic pipe.
†With water at 23 °C (73.4 °F).
‡The fiber stresses used to derive the test pressures are as follows:

	psi	MPa
PVC 1120, PVC 1220, PVC 2120	4200	29.0
PVC 2116	3360	23.2
PVC 2112	2800	19.3
PVC 2110	2300	15.9

5.1.1.2 **PE:** Section 7.7 of ASTM Standard D2239, Specifications for Polyethylene Plastic Pipe, at the appropriate test pressures given in Table 3 of that specification.

5.1.1.3 **ABS:** Section 7.4 of ASTM Standard D2282, Specifications for ABS Plastic Pipe, at the appropriate test pressures given in Table 3 of that specification.

NOTE: Tests of pipe made with different diameters and wall thicknesses but with the same material shall not be required to reestablish long-term hydrostatic design rating since this is a compound qualifying test.

5.2 Compound code classification

5.2.1 **PVC:** ASTM Standard D1784-12454 B (Type I, Grade 1)
12454 C (Type I, Grade 2)
14333 D (Type II, Grade 1)

5.2.2 **PE:** ASTM Standard D1248-II C-P23 (Type II, Grade 3, Class C)
III C-P33 (Type III, Grade 3, Class C)
IV C-P34 (Type III, Grade 4, Class C)

5.2.3 **ABS:** ASTM Standard D1788-5-2-2 (Type I, Grade 2)
3-5-5 (Type I, Grade 3)
4-4-5 (Type II, Grade 1)

5.3 **Rework materials.** Clean rework material generated from the manufacturer's own pipe production may be used by the same manufacturer, as long as the pipe produced meets all the requirements of this Standard.

5.4 Physical requirements

5.4.1 **Workmanship.** The pipe shall be homogeneous throughout and free from visible cracks, holes, foreign inclusions, or other defects. The pipe shall be as uniform as commercially practicable in color, opacity, density and other physical properties.

5.4.2 **Dimensions and tolerances**

5.4.2.1 **Wall thickness.** The wall thickness and tolerances shall be determined in accordance with the appropriate sections of ASTM Standard D2122, Determining Dimensions of Thermoplastic Pipe and Fittings, and shall be as shown in Tables 7, 8 and 9 of this Standard.

5.4.2.2 **Diameters.** The outside diameter or inside diameter of the pipe shall be determined in accordance with the appropriate sections of ASTM Standard D2122, Determining Dimensions of Thermoplastic Pipe and Fittings, and shall be as shown in Tables 10 and 11 of this Standard.

5.5 PVC pipe requirements

5.5.1 **Burst pressure.** The minimum burst pressure shall be determined in accordance with Section 7.5 of ASTM Standard D2241, Specifications for PVC Plastic Pipe, and as given in Table 4 of ASTM Standard D2241 or Table 12 of this Standard.

5.5.2 **Flattening.** There shall be no evidence of splitting, cracking, or breaking when the pipe is tested in accordance with Section 7.6 of ASTM Standard D2241, Specification for PVC Plastic Pipe.

TABLE 7—WALL THICKNESS AND TOLERANCE IN MILLIMETERS (INCHES) FOR PVC AND ABS PIPE, OD CONTROLLED

Nominal pipe size in.	mm*		50 ft head	DR 93.5	SDR 81	SDR 64	SDR 51	SDR 41	SDR 32.5	SDR 26	SDR 21	SDR 17	SDR 13.5
½	21	IPS											1.57 + 0.51 (0.062 + 0.020)
¾	27	IPS										1.57 + 0.51 (0.062 + 0.020)	1.98 + 0.51 (0.078 + 0.020)
1	33	IPS									1.60 + 0.51 (0.063 + 0.020)	1.96 + 0.51 (0.077 + 0.020)	2.46 + 0.51 (0.097 + 0.020)
1¼	42	IPS								1.63 + 0.51 (0.064 + 0.020)	2.00 + 0.51 (0.079 + 0.020)	2.49 + 0.51 (0.098 + 0.020)	3.12 + 0.51 (0.123 + 0.020)
1½	48	IPS								1.85 + 0.51 (0.073 + 0.020)	2.29 + 0.51 (0.090 + 0.020)	2.84 + 0.51 (0.112 + 0.020)	3.58 + 0.51 (0.141 + 0.020)
2	60	IPS							1.85 + 0.51 (0.073 + 0.020)	2.31 + 0.51 (0.091 + 0.020)	2.87 + 0.51 (0.113 + 0.020)	3.56 + 0.51 (0.140 + 0.020)	4.47 + 0.53 (0.176 + 0.021)
2½	73	IPS							2.11 + 0.51 (0.083 + 0.020)	2.79 + 0.51 (0.110 + 0.020)	3.48 + 0.51 (0.137 + 0.020)	4.29 + 0.51 (0.169 + 0.020)	5.41 + 0.66 (0.213 + 0.026)
3	89	IPS							2.74 + 0.51 (0.108 + 0.020)	3.43 + 0.51 (0.135 + 0.020)	4.24 + 0.51 (0.167 + 0.020)	5.23 + 0.64 (0.206 + 0.025)	6.58 + 0.79 (0.259 + 0.031)
3½	102	IPS							3.12 + 0.51 (0.123 + 0.020)	3.91 + 0.51 (0.154 + 0.020)	4.83 + 0.58 (0.190 + 0.023)	5.97 + 0.71 (0.235 + 0.028)	7.52 + 0.91 (0.296 + 0.036)
4	114	IPS				1.79 + 0.51 (0.070 + 0.020)		2.79 + 0.51 (0.110 + 0.020)	3.51 + 0.51 (0.138 + 0.020)	4.39 + 0.53 (0.173 + 0.021)	5.44 + 0.66 (0.214 + 0.026)	6.73 + 0.81 (0.265 + 0.032)	8.46 + 1.02 (0.333 + 0.040)
	105	PIP	1.65 + 0.51 (0.065 + 0.020)		1.65 + 0.51 (0.065 + 0.020)		2.06 + 0.51 (0.081 + 0.020)	2.57 + 0.51 (0.101 + 0.020)					
5	141	IPS						3.45 + 0.51 (0.136 + 0.020)	4.34 + 0.53 (0.171 + 0.021)	5.44 + 0.69 (0.214 + 0.021)	6.73 + 0.81 (0.265 + 0.032)	8.31 + 0.99 (0.327 + 0.039)	10.46 + 1.24 (0.412 + 0.049)
6	168	IPS				2.64 + 0.51 (0.104 + 0.020)		4.11 + 0.51 (0.162 + 0.020)	5.18 + 0.61 (0.204 + 0.024)	6.48 + 0.79 (0.255 + 0.031)	8.03 + 0.97 (0.316 + 0.038)	9.91 + 1.19 (0.390 + 0.047)	12.47 + 1.50 (0.491 + 0.059)
	156	PIP	1.78 + 0.51 (0.070 + 0.020)	1.78 + 0.51 (0.070 + 0.020)	1.93 + 0.51 (0.076 + 0.020)		3.05 + 0.51 (0.120 + 0.020)	3.81 + 0.51 (0.150 + 0.020)					
8	219	IPS				3.43 + 0.51 (0.135 + 0.020)		5.33 + 0.64 (0.210 + 0.025)	6.73 + 0.81 (0.265 + 0.032)	8.43 + 1.02 (0.332 + 0.040)	10.41 + 1.24 (0.410 + 0.049)	12.90 + 1.55 (0.508 + 0.061)	
	207	PIP	2.03 + 0.51 (0.080 + 0.020)	2.21 + 0.51 (0.087 + 0.020)	2.57 + 0.51 (0.101 + 0.020)		4.06 + 0.51 (0.160 + 0.020)	5.05 + 0.61 (0.199 + 0.024)					
10	273	IPS				4.27 + 0.51 (0.168 + 0.020)		6.65 + 0.79 (0.262 + 0.031)	8.41 + 1.02 (0.331 + 0.040)	10.49 + 1.27 (0.413 + 0.050)	12.98 + 1.55 (0.511 + 0.061)	16.05 + 1.93 (0.632 + 0.076)	
	259	PIP	2.54 + 0.51 (0.100 + 0.020)	2.77 + 0.51 (0.109 + 0.020)	3.20 + 0.51 (0.126 + 0.020)		5.08 + 0.61 (0.200 + 0.024)	6.32 + 0.76 (0.249 + 0.030)					
12	324	IPS				5.05 + 0.61 (0.199 + 0.024)		7.90 + 0.94 (0.311 + 0.037)	9.96 + 1.19 (0.392 + 0.047)	12.45 + 1.50 (0.490 + 0.059)	15.39 + 1.85 (0.606 + 0.073)	19.05 + 2.29 (0.750 + 0.090)	
	311	PIP	3.05 + 0.51 (0.120 + 0.020)	3.33 + 0.51 (0.131 + 0.020)	3.84 + 0.51 (0.151 + 0.020)		6.10 + 0.74 (0.240 + 0.029)	7.59 + 0.91 (0.299 + 0.036)					
14	363	PIP	3.56 + 0.51 (0.140 + 0.020)	3.89 + 0.51 (0.153 + 0.020)	4.50 + 0.53 (0.177 + 0.021)		7.11 + 0.86 (0.280 + 0.034)	8.89 + 1.02 (0.350 + 0.040)	11.20 + 1.27 (0.441 + 0.050)	13.97 + 1.78 (0.550 + 0.070)	17.30 + 1.90 (0.681 + 0.075)	21.36 + 2.41 (0.841 + 0.095)	26.92 + 3.18 (1.060 + 0.125)
15	389	PIP	3.81 + 0.51 (0.150 + 0.020)	4.17 + 0.51 (0.164 + 0.020)	4.80 + 0.58 (0.189 + 0.023)		7.62 + 0.91 (0.300 + 0.036)	9.47 + 1.14 (0.373 + 0.045)					
16	406	IPS	4.06 + 0.51 (0.160 + 0.020)	4.34 + 0.52 (0.171 + 0.021)	5.03 + 0.61 (0.198 + 0.024)		7.98 + 0.96 (0.314 + 0.038)	9.91 + 1.19 (0.390 + 0.047)	12.50 + 1.50 (0.492 + 0.059)	15.62 + 1.87 (0.615 + 0.074)			
18	466	IP		5.00 + 0.61 (0.197 + 0.024)	5.77 + 0.69 (0.227 + 0.027)		9.14 + 1.09 (0.360 + 0.043)	11.38 + 1.37 (0.448 + 0.054)	14.35 + 1.72 (0.565 + 0.068)				
18	475	PIP		5.08 + 0.61 (0.200 + 0.024)	5.87 + 0.71 (0.231 + 0.028)		9.32 + 1.12 (0.367 + 0.044)	11.58 + 3.23 (0.456 + 0.127)	14.61 + 1.75 (0.575 + 0.069)				
20	518	IP		5.54 + 0.66 (0.218 + 0.026)	6.40 + 0.76 (0.252 + 0.030)		10.16 + 1.22 (0.400 + 0.048)	12.65 + 1.52 (0.498 + 0.060)	15.95 + 1.91 (0.628 + 0.075)				
21	560	PIP		5.99 + 0.71 (0.236 + 0.028)	6.91 + 0.84 (0.272 + 0.033)		10.97 + 1.32 (0.432 + 0.052)	13.66 + 3.81 (0.538 + 0.150)	17.22 + 2.06 (0.678 + 0.081)				
24	610	IPS		6.53 + 0.79 (0.257 + 0.031)	7.52 + 0.90 (0.296 + 0.036)		11.96 + 1.44 (0.471 + 0.057)	14.86 + 1.78 (0.585 + 0.070)	18.75 + 2.25 (0.738 + 0.089)				
24	630	PIP		6.76 + 0.81 (0.266 + 0.032)	7.77 + 0.94 (0.306 + 0.037)		12.34 + 1.47 (0.486 + 0.058)	15.37 + 4.29 (0.605 + 0.169)	19.38 + 2.34 (0.763 + 0.092)				
27	710	PIP					13.92 + 1.68 (0.548 + 0.066)	17.32 + 4.83 (0.682 + 0.190)	21.84 + 2.62 (0.860 + 0.103)				

*Pipe is not currently available in metric sizes in the United States except for 475, 560, 630 and 710 mm sizes.

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TABLE 8—WALL THICKNESS AND TOLERANCE IN MILLIMETERS (INCHES) FOR PE PIPE, ID CONTROLLED

Nominal pipe size in	mm	SDR 15	SDR 11.5	SDR 9	SDR 7	SDR 5.3
½	16	1.52 + 0.51 (0.060 + 0.020)	1.52 + 0.51 (0.060 + 0.020)	1.75 + 0.51 (0.069 + 0.020)	2.26 + 0.51 (0.089 + 0.020)	2.97 + 0.51 (0.117 + 0.020)
¾	21	1.52 + 0.51 (0.060 + 0.020)	1.83 + 0.51 (0.072 + 0.020)	2.34 + 0.51 (0.092 + 0.020)	3.00 + 0.51 (0.118 + 0.020)	3.94 + 0.51 (0.155 + 0.020)
1	27	1.78 + 0.51 (0.070 + 0.020)	2.31 + 0.51 (0.091 + 0.020)	2.97 + 0.51 (0.117 + 0.020)	3.81 + 0.51 (0.150 + 0.020)	5.03 + 0.61 (0.198 + 0.024)
1¼	35	2.34 + 0.51 (0.092 + 0.020)	3.05 + 0.51 (0.120 + 0.020)	3.89 + 0.51 (0.153 + 0.020)	5.00 + 0.61 (0.197 + 0.024)	6.60 + 0.79 (0.260 + 0.031)
1½	41	2.72 + 0.51 (0.107 + 0.020)	3.56 + 0.51 (0.140 + 0.020)	4.55 + 0.51 (0.179 + 0.020)	5.84 + 0.71 (0.230 + 0.028)	7.72 + 9.91 (0.304 + 0.036)
2	52	3.50 + 0.51 (0.138 + 0.020)	4.57 + 0.56 (0.180 + 0.022)	5.84 + 0.71 (0.230 + 0.028)	7.49 + 0.89 (0.295 + 0.035)	0.91 + 1.19 (0.390 + 0.047)
2½	63	4.19 + 0.51 (1.65 + 0.020)	5.46 + 0.64 (0.215 + 0.025)			
3	78	5.21 + 0.51 (0.205 + 0.020)	6.78 + 0.81 (0.267 + 0.032)			
4	102	6.81 + 0.81 (0.268 + 0.032)	8.89 + 0.107 (0.350 + 0.042)			
6	154	10.26 + 1.22 (0.404 + 0.048)	13.38 + 1.60 (0.527 + 0.063)			

TABLE 9—DIAMETERS, WALL THICKNESSES AND TOLERANCES FOR PE PIPE IN IPS SIZING SYSTEM, OD CONTROLLED.*

	Pipe outside diameter				Minimum wall thickness†									
	Actual outside diameter		Tolerance (+) on average OD		SDR 32.5		SDR 26		SDR 21		SDR 17		SDR 11	
Nominal in.	in.	mm	in.	mm	in.	mm	in	mm	in.	mm	in.	mm	in.	mm
3	3.500	88.9	0.016	0.41	0.108	2.7	0.135	3.4	0.167	4.2	0.206	5.2	0.318	8.1
4	4.500	114.3	0.020	0.51	0.138	3.5	0.173	4.4	0.214	5.4	0.265	6.7	0.409	10.4
5	5.563	141.3	0.025	0.64	0.171	4.3	0.214	5.4	0.265	6.7	0.327	8.3	0.506	12.8
6	6.625	168.3	0.030	0.76	0.204	5.2	0.255	6.5	0.315	8.0	0.390	9.9	0.602	15.3
8	8.625	219.1	0.039	0.99	0.265	6.7	0.332	8.4	0.411	10.4	0.507	12.9	0.784	19.9
10	10.750	273.1	0.048	1.22	0.331	8.4	0.413	10.5	0.512	13.0	0.632	16.0	0.977	24.8
12	12.750	323.8	0.057	1.45	0.392	10.0	0.490	12.4	0.607	15.4	0.750	19.0	1.159	29.5
14	14.000	355.6	0.063	1.60	0.431	10.9	0.538	13.7	0.667	16.9	0.824	20.9	1.273	32.3
16	16.000	406.4	0.072	1.83	0.492	12.5	0.615	15.6	0.762	19.4	0.941	23.9	1.455	37.0
18	18.000	457.2	0.081	2.06	0.554	14.2	0.692	17.6	0.857	21.8	1.059	26.9	1.636	41.6
20	20.000	508.0	0.090	2.27	0.615	15.6	0.769	19.5	0.952	24.2	1.176	29.9		-
22	22.000	558.8	0.099	2.51	0.677	17.2	0.846	21.5	1.048	26.6	1.294	32.9		
24	24.000	609.6	0.108	2.74	0.738	18.7	0.923	23.4	1.143	29.0	1.412	35.9		-

*OD based PE pipe is also made in the metric sizing system, based on ISO 161 1 1978, Thermoplastics Pipes for the Transport of Fluids — Nominal Outside Diameters and Nominal Pressures — Part 1 Metric Series. Specifications for this pipe can be obtained from the Plastic Pipe Institute, New York, NY.

†Wall thickness variability in any diametrical cross section shall not exceed 12% when calculated in accordance with ASTM Standard D2122, Determining Dimensions for Thermoplastic Pipe and Fittings.

TABLE 10—OUTSIDE DIAMETER AND TOLERANCE FOR PVC AND ABS PIPE

					TOLERANCES(+)					
					For average OD		For maximum & minimum (out-of-round)			
							50 ft head DR 93.5 SDR 81 SDR 64 SDR 51		SDR 41 SDR 32.5 SDR 26 SDR 21	SDR 17 SDR 13.5
Nominal in.	Pipe size mm		Average OD in.	mm	in.	mm	in.	mm	in.	mm
½	21	IPS	0.840	21.34	0.004	0.10	0.015	0.38	0.008	0.20
¾	27	IPS	1.050	26.67	0.004	0.10	0.015	0.38	0.010	0.25
1	33	IPS	1.315	33.40	0.005	0.13	0.015	0.38	0.010	0.25
1¼	42	IPS	1.660	42.16	0.005	0.13	0.015	0.38	0.012	0.30
1½	48	IPS	1.900	48.26	0.006	0.15	0.030	0.76	0.012	0.30
2	60	IPS	2.375	60.32	0.006	0.15	0.030	0.76	0.012	0.30
2½	73	IPS	2.875	73.02	0.007	0.18	0.030	0.76	0.015	0.38
3	89	IPS	3.500	88.90	0.008	0.20	0.030	0.76	0.015	0.38
3½	102	IPS	4.000	101.60	0.008	0.20	0.050	1.27	0.015	0.38
4	114	IPS	4.500	114.30	0.009	0.23	0.050	1.27	0.015	0.38
	105	PIP	4.130	104.90	0.009	0.23	0.050	1.27	0.015	0.38
5	141	IPS	5.563	141.30	0.010	0.25	0.050	1.27	0.030	0.76
6	168	IPS	6.625	168.23	0.011	0.28	0.050	1.27	0.035	0.89
	156	PIP	6.140	155.96	0.011	0.28	0.050	1.27	0.030	0.76
8	219	IPS	8.625	219.08	0.015	0.38	0.075	1.90	0.045	1.14
	207	PIP	8.160	207.26	0.015	0.38	0.075	1.78	0.042	1.07
10	273	IPS	10.750	273.05	0.015	0.38	0.075	1.90	0.050	1.27
	259	PIP	10.200	259.08	0.015	0.38	0.075	1.90	0.050	1.27
12	324	IPS	12.750	323.85	0.015	0.38	0.075	1.90	0.060	1.52
	311	PIP	12.240	310.90	0.018	0.46	0.075	1.90	0.060	1.52
14	363	PIP	14.280	362.71	0.021	0.53	0.075	1.90	0.070	1.78
15	389	PIP	15.300	388.62	0.023	0.58	0.075	1.90	0.075	1.90
16	406	IPS	16.000	406.40	0.024	0.61	0.075	1.90		
18	466	IP	18.360	466.34	0.027	0.69	0.100	2.50		
18	475	PIP	18.701	475.00	0.028	0.71	0.100	2.50		
20	518	IP	20.400	518.16	0.030	0.76	0.100	2.50		
21	560	PIP	22.047	560.00	0.033	0.84	0.100	2.50		
24	610	IPS	24.000	609.6	0.036	0.91	0.125	2.50		
24	630	PIP	24.803	630.00	0.037	0.94	0.125	2.50		
27	710	PIP	27.953	710.00	0.047	1.19	0.125	2.50		

TABLE 11—INSIDE DIAMETER AND TOLERANCE FOR PE PIPE

Nominal pipe size in.	mm		ID in.	mm	Tolerance in.	mm
½	16	IPS	0.622	15.80	+0.010	0.25
¾	21	IPS	0.824	20.93	+0.010	0.25
					−0.015	0.38
1	27	IPS	1.049	26.64	+0.010	0.25
					−0.020	0.51
1¼	35	IPS	1.380	35.05	+0.010	0.25
					−0.020	0.51
1½	41	IPS	1.610	40.89	+0.010	0.25
					−0.020	0.51
2	52	IPS	2.067	52.50	+0.015	0.38
					−0.020	0.51
2½	63	IPS	2.469	62.71	+0.015	0.38
					−0.025	0.64
3	78	IPS	3.068	77.93	+0.015	0.38
					−0.030	0.76
4	102	IPS	4.026	102.26	+0.015	0.38
					−0.035	0.89
	102	PIP	4.000	101.6	+0.020	0.51
					−0.020	0.51
6	154	IPS	6.065	154.05	+0.020	0.51
					−0.035	0.89
	152	PIP	6.000	152.4	+0.025	0.64
					−0.025	0.64
8	203	PIP	8.000	203.2	±0.040	1.02
10	254	PIP	10.000	254.0	±0.040	1.02
12	305	PIP	12.000	304.8	±0.040	1.02
15	381	PIP	15.000	381.0	±0.040	1.02

TABLE 12—BURST PRESSURE REQUIREMENTS FOR PVC PLASTIC PIPE*†

SDR (DR)	Minimum burst pressure‡ PVC 1120, PVC 1220, PVC 2120 psi	kPa	PVC 2116, PVC 2112, PVC 2110 psi	kPa
51	260	1790	200	1380
64	200	1380	160	1105
81	160	1105	125	860
93.5	140	965		
50 ft head	127	875		

*Requirements in addition to those listed in ASTM Standard D-2241, Specification for Poly(Vinyl Chloride)(PVC) and Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe (SDR-PR), for SDR rated PVC plastic pipe.
†With water at 23 °C (73.4 °F).
‡The fiber stresses used to derive the test pressures are as follows:

	psi	MPa
PVC 1120, PVC 1220, PVC 2120	6400	44.1
PVC 2116, PVC 2112, PVC 2110	5400	34.5

5.5.3 Extrusion quality. The pipe shall not flake or disintegrate when tested in accordance with ASTM Standard D2152, Test for Quality of Extruded PVC Pipe by Acetone Immersion.

5.5.4 Impact resistance. The pipe shall be tested in accordance with ASTM Standard D2444, Test for Impact Resistance of Thermoplastic Pipe and Fittings by Means of a Tup (Falling Weight), using a 89 N (20 lb), Type B tup with a flat plate at 23 ± 2 °C (73.4 ± 3.6 °F) and shall meet the test levels shown in Table 13 of this Standard. The impact test shall be made on new production pipe at the time of manufacture.

5.6 PE pipe requirements

5.6.1 Thickness of outer layer. For pipe produced by simultaneous multiple extrusion, that is, pipe containing two or more concentric layers, the outer layer shall be at least 0.51 mm (0.020 in.) thick.

5.6.2 Bond. For pipe produced by simultaneous multiple extrusion, the bond between the layers shall be strong and uniform. It shall not be possible to separate any two layers with a probe or a point of a knife blade so that the layers separate cleanly at any point.

5.6.3 Carbon black. The pipe extrusion compound shall contain at least 2 percent carbon black when tested in accordance with Section 7.5 of ASTM Standard D2239, Specifications for Polyethylene (PE) Plastic Pipe. For pipe produced by simultaneous multiple extrusion, this requirement shall apply to the outer layer.

5.6.4 Density. The polyethylene base resin (uncolored PE) in the pipe compound shall have a density in the range from 0.926 to 0.940 Mg/m³ for pipe made from Grade P23 and 0.941 to 0.965 Mg/m³ for pipe made from Grade P33 and Grade P34 of ASTM Standard D1248, Specifications for Polyethylene Plastic Molding and Extrusion Materials, when determined in accordance with Section 7.6 of ASTM Standard D2239, Specifications for Polyethylene Plastic Pipe.

5.6.5 Burst pressure. The minimum burst pressure for PE plastic pipe shall be determined in accordance with Section 7.8 and Table 4 of ASTM Standard D2239, Specifications for Polyethylene Plastic Pipe.

TABLE 13—IMPACT REQUIREMENTS FOR PVC AND ABS PIPE*

Nominal pipe size in.	mm		50 ft head ft·lbf	Nm	DR 93.4 ft·lbf	Nm	SDR 81 ft·lbf	Nm	SDR 64 ft·lbf	Nm	SDR 51 ft·lbf	Nm	SDR 41 ft·lbf	Nm	SDR 32.5 ft·lbf	Nm	SDR 26 ft·lbf	Nm	SDR 21 ft·lbf	Nm	SDR 17 ft·lbf	Nm	SDR 13.5 ft·lbf	Nm
½	21	IPS																					25	35
¾	27	IPS																			36	50	26	50
1	33	IPS																	38	50	38	50	38	50
1¼	42	IPS															50	70	50	70	50	70	50	70
1½	48	IPS													50	70	50	70	50	70	50	70	50	70
2	60	IPS													63	85	63	85	63	85	63	85	63	85
2½	73	IPS													63	85	63	85	63	85	63	85	63	85
3	89	IPS													75	100	75	100	75	100	75	100	75	100
3½	102	IPS													75	100	75	100	75	100	75	100	75	100
4	114	IPS							50	70					80	110	100	135	100	135	100	135	100	135
	105	PIP	30	40			30	40			70	95	70	95										
5	141	IPS													90	120	110	150	110	150	110	150	110	150
6	168	IPS							60	80					100	135	120	165	120	165	120	165	120	165
		PIP	30	40	30	40	30	40			80	110	80	110										
8	219	IPS							70	95					100	135	130	175	130	175	130	175	130	175
	207	PIP	30	40	30	40	30	40			90	120	90	120										
10	273	IPS							80	110					110	150	110	150						
	259	PIP	60	80	60	80	60	80			100	135	100	135										
12	324	IPS							100	135					110	150	150	205						
	311	PIP	100	135	100	135	100	135			110	150	110	150										
14	363	PIP	110	150	100	135	110	150			120	165	120	165	120	165	150	205	150	205	150	205	150	205
15	389	PIP	110	150	110	150	110	150			120	165	120	165										
16	406	IPS	110	150	110	150	110	150			120	165	150	205	120	165	150	205						
18	466,475	IP PIP			110	150	110	150			120	165	150	205	150	205								
20	518	IP			110	150	110	150			120	165	150	205	150	205								
21	560	PIP											150	205										
24	610,630	IPS,PIP			110	150	110	150			120	165	150	205	150	205								
27	710	PIP											150	205										

*When tested in accordance with ASTM Standard D2444, Test for Impact Resistance of Thermoplastic Pipe and Fittings by Means of a Tup (Falling Weight), using an 89 N (20 lb) type B tup with a flat plate at 23 °C (73.4 °F) on new production pipe at the time of manufacture.

5.6.7 **Environmental stress cracking.** There shall be no loss of pressure in the pipe when tested in accordance with Section 7.9 of ASTM Standard D2239, Specifications for Polyethylene Plastic Pipe.

5.7 ABS pipe requirements

5.7.1 **Burst pressure.** The minimum burst pressure shall be determined in accordance with Section 7.6 and Table 4 of ASTM Standard D2282, Specifications for ABS Plastic Pipe.

5.7.2 **Impact resistance.** The pipe shall be tested in accordance with ASTM Standard D2444, Test for Impact Resistance of Thermoplastic Pipe and Fittings by Means of a Tup (Falling Weight), using a 89 N (20 lb) type B tup with a flat plate at 23 ± 2 °C (73.4 ± 3.6 °F) and shall meet the test levels shown in Table 13 of this Standard. The impact test shall be made on new production pipe at the time of manufacture.

5.8 Joints

5.8.1 **General.** All joints shall be constructed to withstand the design maximum working pressures for the pipeline without leakage, and without internal obstruction which could reduce its capacity below design requirements, except that insert fittings for joining PE pipe are permitted. Manufacturer's recommendations for joining pipe shall be used when not in conflict with requirements of paragraph 5.8.

5.8.2 **Sockets and couplings.** The integral bell or separate coupling shall meet the same strength requirements as the pipe. When joint assembly requires use of separate couplings, one such coupling of the same class and size shall be furnished with each length of pipe.

5.8.3 **Solvent cements.** Solvent cements only for use with PVC pipe and fittings shall meet the requirements of ASTM Standard D2564, Specifications for Solvent Cements for PVC Plastic Pipe and Fittings. Solvent cements only for ABS pipe and fittings shall meet the requirements of ASTM Standard D2235, Specifications for Solvent Cement for ABS Plastic Pipe and Fittings. The pipe manufacturer should be consulted for the type of cement recommended for joining large diameter pipes. Safe handling of solvent cements shall conform to ASTM Standard F402, Recommended Practice for Safe Handling of Solvent Cements Used for Joining Thermoplastic Pipe and Fittings.

5.8.4 **Rubber gasket joints.** Rubber gasket joints shall conform to ASTM Standard D3139, Specifications for Joints for Plastic Pressure Pipes Using Flexible Elastomeric Seals.

5.8.5 **Plastic risers.** Plastic risers shall have at least the same strength as the pipe, including risers with use limited to subsurface attachment.

5.9 Fittings

5.9.1 **General.** All fittings, such as couplings, reducers, bends, tees and crosses shall be made of material that is recommended for use with the pipe and shall be installed in accordance with the recommendations of the manufacturer. Where fittings made of steel or other materials subject to corrosion are used in the line, they shall be adequately protected by wrapping with plastic tape or by coating with high quality corrosion preventatives. Where plastic tape is used, all surfaces to be wrapped shall be thoroughly cleaned and then coated with primer compatible with the tape prior to wrapping.

5.9.2 **Requirements.** Fittings for IPS sized pipe shall meet all the dimensional and quality requirements given in the following ASTM Standards:

- ASTM Standard D2468, ABS Plastic Pipe Fittings, Socket-Type, Schedule 40
- ASTM Standard D2469, ABS Plastic Pipe Fittings, Socket-Type, Schedule 80
- ASTM Standard D2609, PE Plastic Insert Fittings
- ASTM Standard D3261, Butt Heat Fusion for PE Plastic Pipe and Tubing
- ASTM Standard D2672, Bell-End PVC Plastic Pipe
- ASTM Standard D3036, PVC Plastic Line Couplings, Socket-Type
- ASTM Standard D2466, PVC Plastic Pipe Fittings, Socket-Type, Schedule 40
- ASTM Standard D2467, PVC Plastic Pipe Fittings, Socket-Type, Schedule 80

SECTION 6—MARKING

6.1 **General.** The pipe shall be marked at intervals of not more than 5 ft (1.5 m). (The metric marking shall not be required until it becomes a national standard. In this section customary units are shown first.)

Marking shall include the following:

6.1.1 The nominal pipe size, e.g. 4 in. (114 mm).

6.1.2 The pipe OD sizing system when applicable (IPS, IP, or PIP), e.g. PIP.

6.1.3 The ASTM Standard numeric designation for sizing systems other than PIP, e.g. ASTM Standard D2241.

6.1.4 The type of plastic pipe material in accordance with the designation code, e.g. PVC 1120.

6.1.5 **Pressure rating**

6.1.5.1 **Low pressure pipe.** The pressure rating shall be shown in psi and/or in feet of head: e.g. 22 psi (152 kPa) 50 ft (15.2 m) head.

6.1.5.2 **SDR pipe.** The pressure class rating in psi for water at 73.4 °F (23 °C); e.g. 200 psi (1379 kPa) @ 73.4 (23), or the standard dimension ratio as calculated in paragraph 2.8; e.g. SDR 21, or both: e.g. 200 psi (1379 kPa) @ 73.4 (23) SDR 21.

6.1.6 The manufacturer's name or trademark and code.

6.1.7 Pipe intended for the conveyance of potable water shall also include the seal or mark of the laboratory making the evaluation for this purpose, spaced at intervals specified by the laboratory.

SECTION 7—INSTALLATION REQUIREMENTS

7.1 **General.** The thermoplastic pipe shall be installed in accordance with the manufacturer's recommendations. If these are not available, then for pipe 152 mm (6 in.) diameter or less, ASTM Standard D2774, Recommended Practice for Underground Installation of Thermoplastic Pressure Piping, or this Standard shall be followed. Recommendations in ASTM Standard D2321, Underground Installation of Flexible Thermoplastic Sewer Pipe, may also be followed.

7.2 Trench construction

7.2.1 **Trench bottom.** The trench bottom should be continuous, firm, relatively smooth and free of rocks or other hard objects larger than 13 mm (0.5 in.) in size. Where ledge rock, hard pan or boulders are encountered, the trench bottom shall be undercut and filled with bedding material, using sand or compacted fine-grained soils to provide a minimum depth of bed between the pipe and rock of 100 mm (4 in.). Where unstable trench bottom conditions are encountered, stabilizing methods and materials to provide adequate and permanent support shall be used.

7.2.2 **Trench width.** The width of the trench at any point below the top of the pipe should not be greater than necessary to provide adequate room for joining the pipe and compacting the initial backfill. The trench width should be sufficient to provide adequate room for joining the pipe in the trench, if this is necessary; filling and compacting the side fills and snaking the pipe from side-to-side along the bottom of the ditch, if recommended by the pipe manufacturer. Trench widths above the top of the pipe should not be greater than 0.6 m (2 ft) wider than the pipe diameter, except that in unstable soils where sloughing or caving may occur or where required by regulations or local conditions, the sidewalls above the top of the pipe may be sloped.

7.2.2.1 **Low pressure pipe.** Maximum and minimum trench widths below the top of the pipe for low pressure pipe shall be as follows:

Pipe size		Approximate trench width minimum		Approximate trench width maximum	
in.	mm	in.	mm	in.	mm
4	102	16	400	30	760
6	152	18	450	30	760
8	203	20	510	30	760
10	254	22	560	30	760
12	305	24	610	30	760
14	356	26	660	30	760
15	381	27	690	30	760
18	457-475	30	760	36	910
20	508	32	810	36	910
24	610-630	36	910	42	1070
27	710	40	1020	46	1170

7.2.3 Trench depth. The trench depth should be determined with consideration given to requirements imposed by trench bottom, pipe size and cover conditions (see paragraphs 7.2.1 and 7.7). The depth shall be sufficient to ensure placement of the top of the pipe 0.25 m (10 in.) below the frost line unless the requirements of paragraph 3.9 are satisfied.

7.2.4 Safety. Provisions shall be made to insure safe working conditions where unstable soil, trench depth or other conditions impose a safety hazard to personnel working in the trench.

7.3 Placement

7.3.1 General. Special handling and an awareness of temperature effects on thermoplastic pipe are needed to prevent permanent distortion and pipe damage when handling during unusually warm or cold weather. Prior to any backfilling beyond light backfill for shading, and prior to connecting to other facilities, the pipe shall be allowed to come to within a few degrees of the temperature it will reach after complete covering. The pipeline shall be installed to provide protection from hazards imposed by traffic crossing, farming operations, freezing temperatures, or soil cracking. If the pipe is assembled above ground, it should be lowered into the trench with care to prevent dropping or damaging the pipe or its joints. Treatment such as dragging or excessive bending which could cause excessive joint stressing, displacement or pull-out should be avoided.

7.3.2 Deflection and bending. The pipe shall be installed in a manner to ensure that excessive deflection in elastomeric seal joints and excessive bending of the pipe do not occur during installation. Bending stresses should be avoided and at no time should the pipe be blocked or braced to hold a bend. The pipe manufacturer should be consulted for maximum permissible deflection limits and minimum pipe bending radii.

7.3.3 Connection to a rigid structure. Where differential settlement could create a concentrated loading on a pipe or joint, as at the connection of a buried pipe to a rigid structure such as a stand, extra care should be taken to compact the foundation and bedding adjoining the structure. A supporting structure beneath the joint and the pipe or a flexible joint also may be used.

7.3.4 Bell holes for rubber gasket joints. When the pipe being installed is provided with rubber gasket joints, bell holes shall be excavated in the bedding material to allow for the unobstructed assembly of the joint. Care should be taken that the bell hole is no larger than necessary to accomplish proper joint assembly. When the joint has been made, the bell hole should be carefully filled with initial backfill material to provide adequate support of the pipe throughout its entire length.

7.4 Thrust blocking

7.4.1 General. Thrust blocking prevents the line from moving and is required primarily with rubber gasket joints. Unequal forces due to water pressure at changes in pipeline alignment result in thrust loads. The thrust block transfers this load from the pipe to a wider load bearing surface. Thrust blocks are required at the following locations:

7.4.1.1 Where the pipe changes the direction of the water (i.e., ties, elbows, crosses, wyes and tees).

7.4.1.2 Where the pipe size changes (i.e., reducers, reducing tees and crosses).

7.4.1.3 At the end of the pipeline (i.e., caps and plugs).

7.4.1.4 Where there is an in-line valve.

7.4.2 Placement. The thrust block must be formed against a solid trench wall that has been excavated by hand. Damage to the bearing surface of the trench wall may result from excavation by mechanical equipment. The size and type of thrust block depends on pipe size, line pressure, type of fitting, degree of bend and type of soil. Thrust block size can be calculated by the procedures shown in Table 14.

7.4.3 Side thrust on curves. An outward pressure exists on all deflections from a straight line. Generally, good soil properly tamped in sufficient to hold side thrust. If the soil is unstable, blocking should be placed against the pipe on the outside radius on each side of a gasketed coupling. Do not thrust block the coupling itself.

7.4.4 Construction of thrust blocks. Thrust blocks are anchors placed between the pipe or fittings and the solid trench wall. The recommended blocking is concrete having a calculated compressive strength of at least 13.8 MPa (2000 psi). The concrete mixture is one part cement, two parts washed sand and four parts gravel. Thrust blocks should be constructed so the bearing surface is in direct line with the major force created by the pipe or fitting (see Table 14). The earth bearing surface should be undisturbed with only the simplest of forms required.

7.5 Initial backfill

7.5.1 General. The pipe should be uniformly and continuously supported over its entire length on firm stable material. Blocking should not be used to change pipe grade or to intermittently support pipe across excavated sections.

7.5.2 Special considerations. Special consideration must be given to soils, backfilling, and bedding procedures for 457 mm (18 in.) diameter and larger low pressure pipe to ensure protection of the pipe under the maximum loading conditions to which it may be subjected. Special engineering design and soils analysis may be needed to determine the supportive strength of the soils intended for use as backfill.

7.5.3 All low pressure pipelines shall be water-strutted or filled with water prior to backfilling. The backfill must be compacted to the required or on adequate density for all low pressure pipe. Either the water packing method or hand or mechanical backfilling methods may be used for backfill consolidation.

7.5.3.1 Water packing. When water packing is used, the pipeline must first be filled with water, all air removed, and the pipe kept full during the backfill operation. The initial backfill material shall be as specified in paragraph 7.5.3.2. The backfill, before wetting, shall be 300 to 450 mm (12 to 18 in.) deep over the top of the pipe. Water packing is accomplished by adding water in such quantity as to thoroughly saturate the initial backfill. While saturated, rods, shovels, concrete vibrators or other means may be used to help consolidate the backfill around the pipe, taking care not to float the pipe. After saturation, the pipeline shall remain full until after final backfill is made. The wetted fill shall be allowed to dry until firm enough to walk on before final backfill is begun.

7.5.3.2 Hand or mechanical backfilling. The initial backfill in contact with the pipe and immediately surrounding it shall be of fine-grained material free from rocks, stones, or clods greater than approximately 19 mm (0.75 in.) diameter and earth clods greater than approximately 50 mm (2 in.) diameter. The backfill shall be tamped in layers not to exceed 150 mm (6 in.) lift and compacted firmly around the pipe and up to at least 152 mm (6 in.) above the top of the pipe. The backfill material shall be sufficiently damp to permit thorough compaction under and on each side of the pipe to provide support free from voids. Care should be taken to avoid deforming, displacing, or damaging the pipe during this phase of the operation.

7.6 Final backfill

7.6.1 After pipeline testing, final backfill shall be placed and spread in approximately uniform layers in such a manner as to fill the trench completely so that there will be no unfilled spaces under or around rocks or lumps of earth in the backfill. Final backfill shall be free of large rocks, frozen clods and other debris greater than 75 mm (3 in.) in diameter.

7.6.2 Rolling equipment or heavy tampers should *not* be used to consolidate the final backfill until *after* the minimum depth of cover has been placed and then only with pipe having wall thicknesses greater than that of SDR-41.

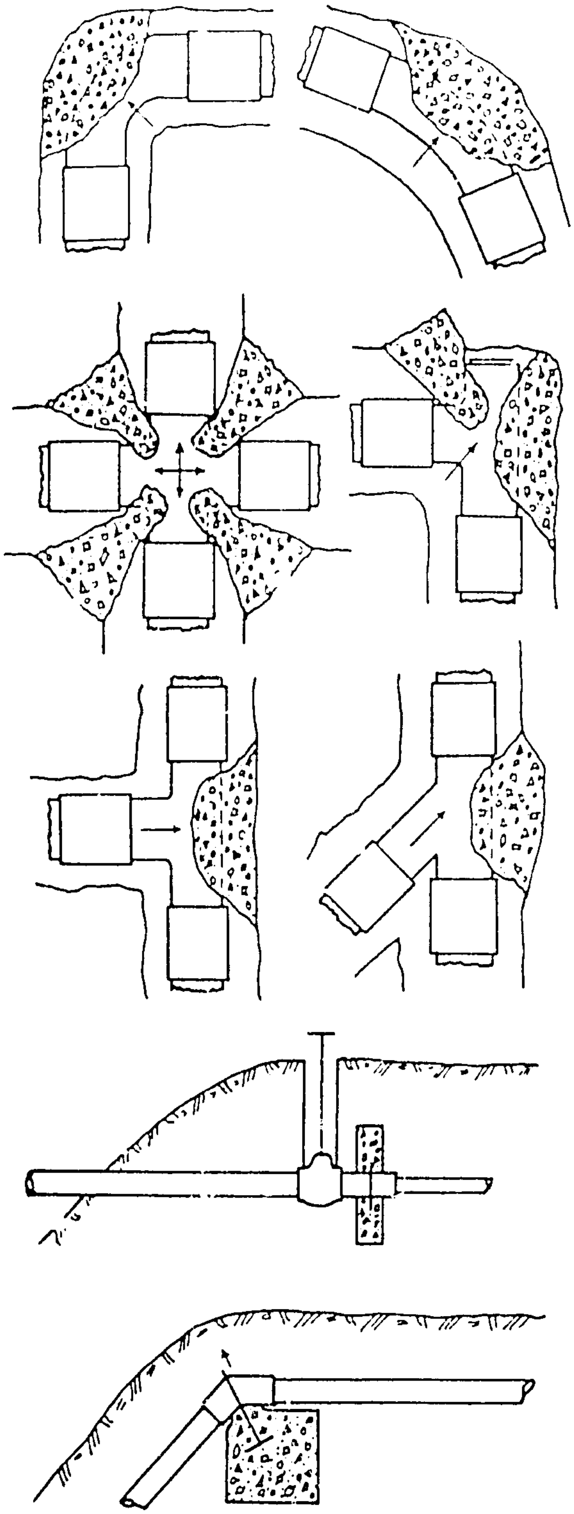
7.7 Minimum depth of cover

7.7.1 General. At low places on the ground surface, extra fill may be placed over the pipeline to provide the minimum depth of cover. In such cases, the top width of the fill shall be no less than 3 m (10 ft) and the side slope no steeper than 4 horizontal to 1 vertical. The minimum depth shall be as follows:

Pipe size mm (in.)	Minimum depth of cover mm (in.)
13-64 (0.5-2.5)	460 (18)
76-102 (3-4)	610 (24)
>102 (>4)	760 (30)

7.7.2 Minimum cover for load applications. At least 760 mm (30 in.) cover over the top of the pipe shall be provided before the trench is wheel-loaded for both low pressure and high pressure pipe.

TABLE 14—THRUST BLOCKING AND ANCHORS FOR UNDERGROUND IRRIGATION PIPELIN[illegible]



Step 1. Multiply the working pressure by the appropriate value shown in the following table to obtain total thrust in N (lb):

PIPELINE THRUST FACTORS* †

Pipe size in.	mm	Dead end or tee	90° Elbow	45° Elbow	22½° Elbow
1½	48	2.94	4.16	2.25	1.15
2	60	4.56	6.45	3.50	1.78
2½	73	6.65	9.40	5.10	2.60
3	89	9.80	13.9	7.51	3.82
3½	102	12.8	18.1	9.81	4.99
4	114	16.2	25.0	12.4	6.31
5	141	24.7	35.0	18.9	9.63
6	168	34.8	49.2	26.7	13.6
8	219	59.0	83.5	45.2	23.0
10	273	91.5	130.0	70.0	35.8
12	324	129.0	182.0	98.5	50.3
14	363	160.2	226.5	122.6	62.6
15	389	183.9	260.0	140.7	71.9
16	406	201.1	284.4	153.8	78.6
18	475	274.7	388.4	210.1	107.4
20	518	326.9	462.2	250.1	127.8
21	560	381.8	539.9	292.1	149.3
24	630	483.2	683.2	369.6	188.9
27	710	612.7	867.8	469.5	239.9

*Based on thrust per kPa (psi) pressure
†Blocking for cross may not be needed with long branch lines.

Step 2. Determine the bearing strength of the soil from the table below:

BEARING STRENGTH OF SOILS

Soils and safe bearing loads	lb/ft^2	kPa
Sound shale	10 000	500
Cemented gravel and sand difficult to pick	4 000	200
Coarse and fine compact sand	3 000	150
Medium clay-can be spaded	2 000	100
Soft clay	1 000	50
Muck	0	0

Step 3. Divide the total thrust obtained in Step 1 by the bearing strength of the soil to get the area needed, m^2(ft^2).

SIDE THRUST ALTERNATIVE PROCEDURE

Pipe size in.	mm	Side thrust-per degree* lb	N
1½	48	5.1	22.7
2	60	7.9	35.1
2½	73	11.6	51.6
3	89	17.1	76.1
3½	102	22.4	99.6
4	114	28.3	125.9
5	141	43.1	191.7
6	168	60.8	270.5
8	219	103.0	458.2
10	273	160.0	711.7
12	324	225.0	1000.8
14	363	278.2	1237.4
15	389	319.6	1421.6
16	406	349.3	1553.7
18	475	477.3	2123.0
20	518	568.0	2526.5
21	560	663.6	2951.7
24	630	839.6	3734.5
27	710	1066.2	4742.5

*Based on side thrust per 689 kPa (100 psi) pressure per degree of deflection.

NOTE: Multiply side thrust from table by degrees of deflection times kPa (psi) divided by 100 to obtain total side thrust in N (lb).

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7.8 Maximum depth of cover. The maximum depth of cover for low pressure pipe shall be 1.2 m (4 ft). For other classes of pipe, the pipe manufacturer should be consulted for maximum depths of cover greater than approximately 1.2 m (4 ft).

SECTION 8—TESTING

8.1 General. Low pressure pipelines shall be thoroughly and completely tested for pressure strength and leakage before backfill operations are undertaken. If it is necessary to partially backfill the line before testing to hold the line in place, the partial backfill shall be undertaken as specified in paragraph 7.5. Only the body of the pipe sections shall be covered with all joints and connections left uncovered for inspection. High pressure pipelines may be tested after backfilling.

8.2 Filling. The line shall be slowly filled with water. The velocity of the water input shall not exceed 0.2 m/s (1 ft/s). Adequate provision shall be made for air release while filling, taking care to bleed all entrapped air in the process. The pressure shall be slowly built up to the maximum design working pressure. Pressurizing should take at least ten minutes for pipelines 102 mm (4 in.) and smaller in diameter and having a test pressure of 690 kPa (100 psi) in a test section of 305 m (1000 ft). For larger diameters, longer lines and higher pressures, proportionately longer build-up times shall be used.

8.3 Inspection. The pipeline shall be inspected in its entirety while the maximum working pressure is maintained. Where leaks are discovered, they shall be promptly repaired and the line retested.

8.4 Flow capacity. It shall be demonstrated by testing that the pipeline will function properly at design capacity. At or below design capacity, there shall be no objectionable surge or water hammer.

8.5 Objectional flow conditions. Objectional flow during testing conditions shall include continuing unsteady delivery of water, damage to the pipeline, detrimental overflow from vents or stands, or sudden or rapid changes in flow velocity at either start-up or shutdown including emergency shut-off, particularly in lines appreciably longer than 305 m (1000 ft).

SECTION 9—BASIS OF ACCEPTANCE

9.1 Requirements. The acceptability of the pipeline shall be determined by inspections to check compliance with all the provisions of this Standard with respect to the design of the line, the pipe and appurtenances used, and the minimum installation requirements.

SECTION 10—CERTIFICATION

10.1 General. All materials shall conform to these minimum requirements and to the tests prescribed in the applicable ASTM Standards.

10.2 Certification. When required, the pipe shall be certified by a qualified testing laboratory for compliance with the requirements set out in this Standard.

SECTION 11—PLANS AND SPECIFICATIONS

11.1 General. Plans and specifications for construction of underground thermoplastic irrigation pipelines shall be in keeping with this Standard and shall describe the requirements for application of the practice to achieve its intended purpose.

Cited Standards:

ASTM D1248, Specification for Polyethylene Plastics Molding and Extrusion Materials
ASTM D1598, Test for Time-to-Failure of Plastic Pipe Under Constant Internal Pressure
ASTM D1784, Specification for Rigid Poly (Vinyl Chloride) Compounds and Chlorinated Poly (Vinyl Chloride) Compounds
ASTM D1788, Specification for Rigid Acrylonitrile-Butadiene-Styrene (ABS) Plastics
ASTM D2122, Determining Dimensions of Thermoplastic Pipe and Fittings
ASTM D2152, Test for Quality of Extruded PVC Pipe by Acetone Immersion
ASTM D2235, Specification for Solvent Cement for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe and Fittings
ASTM D2239, Specification for Polyethylene (PE) Plastic Pipe (SDR-PR)
ASTM D2241, Specification for Poly (Vinyl Chloride) (PVC) and Chlorinated Poly (Vinyl Chloride) (CPVC) Plastic Pipe (SDR-PR)
ASTM D2282, Specification for Acrylonitrile-Butadiene Styrene (ABS) Plastic Pipe (SDR-PR)
ASTM D2321, Underground Installation of Flexible Thermoplastic Sewer Pipe
ASTM D2444, Test for Impact Resistance of Thermoplastic Pipe and Fittings by Means of a Tup (Falling Weight)
ASTM D2466, Specification for Socket-Type Poly (Vinyl Chloride) (PVC) and Chlorinated Poly (Vinyl Chloride) (CPVC) Plastic Pipe Fittings, Schedule 40
ASTM D2467, Specification for Socket-Type Poly (Vinyl Chloride) (PVC) and Chlorinated Poly (Vinyl Chloride) (CPVC) Plastic Pipe Fittings, Schedule 80
ASTM D2468, Specification for Socket-Type Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe Fittings, Schedule 40
ASTM D2469, Specification for Socket-Type Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe Fittings, Schedule 80
ASTM D2564, Specification for Solvent Cements for Poly (Vinyl Chloride) (PVC) Plastic Pipe and Fittings
ASTM D2609, Specification for Plastic Insert Fittings for Polyethylene (PE) Plastic Pipe
ASTM D2672, Specification for Bell End Poly (Vinyl Chloride) (PVC) Pipe
ASTM D2774, Recommended Practice for Underground Installation of Thermoplastic Pressure Piping
ASTM D2837, Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials
ASTM D2855, Standard Recommended Practice for Making Solvent Cemented Joints with Polyvinyl Chloride Pipe and Fittings
ASTM D3036, Specification for Poly (Vinyl Chloride) (PVC) Plastic Line Couplings, Socket-Type
ASTM D3139, Specification for Joints for Plastic Pressure Pipes using Flexible Elastomeric Seals
ASTM D3261, Butt Head Fusion for PE Plastic Fittings for PE Plastic Pipe and Tubing
ASTM F402, Recommended Practice for Safe Handling of Solvent Cements Used for Joining Thermoplastic Pipe and Fittings
ASTM F412, Definitions of Terms Relating to Plastic Piping Systems
ASTM F477, Specification for Elastomeric Seals (Gaskets for Joining Plastic Pipe)
ISO 161/1-1978, Thermoplastic Pipes for the Transport of Fluids—Nominal Outside Diameters and Nominal Pressures—Part 1: Metric Series