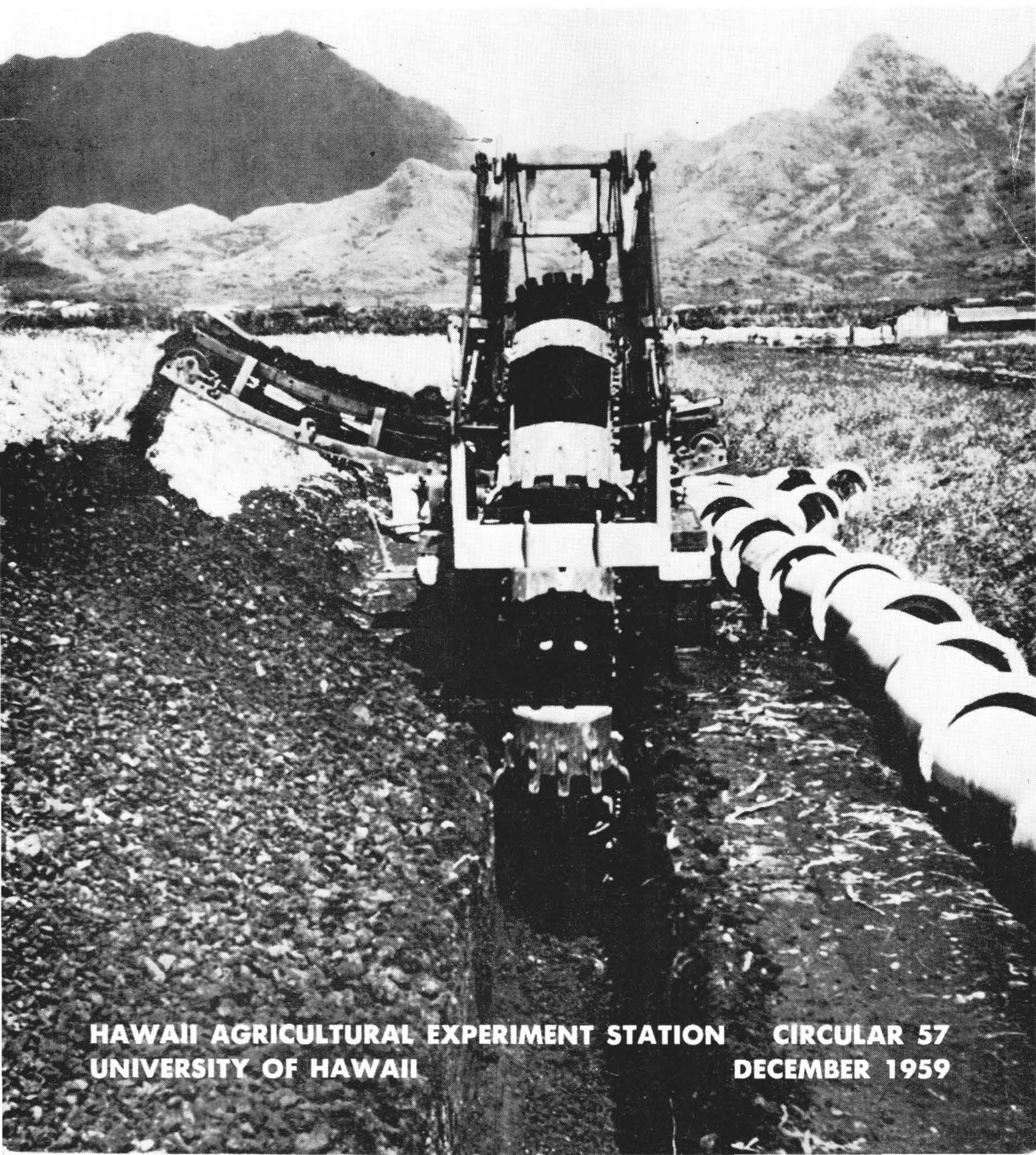
CONCRETE PIPE
FOR IRRIGATION IN HAWAII

Its selection, use, design, installation, and operation

Bessel D. van't Woudt



HAWAII AGRICULTURAL EXPERIMENT STATION CIRCULAR 57
UNIVERSITY OF HAWAII DECEMBER 1959

Concrete Pipe for Irrigation

HIGHLIGHTS

Concrete pipe systems are conveniently used with either furrow or sprinkler irrigation.

In designing a concrete pipe system, use should be made of the elevation head arising from differences in topography. However, on steep slopes pressure control is required to keep pressure within the limits afforded by concrete pipe.

Concrete pipelines should have ample vents for air escape.

Convenient regulatory devices are available for water diversion and for discharge from the pipelines.

Installing concrete pipelines properly is a key requirement for successful operation.

Advantages

Concrete pipelines permit:

1. Accessibility to cultivated fields by farm machinery
2. Seepage control
3. Weed control
4. Reduced space for water conveyance structures
5. Convenient control of water diversion
6. Water to move uphill

Disadvantages

Concrete pipelines require:

1. "Permanency" of farming layout
2. Trash control at pipe inlets

The cost of installing and using buried concrete pipelines versus that of ditches and flumes will determine which system is economically most attractive, taking the above factors into account.

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INTRODUCTION

Essentially, there are two systems of water conveyance: (1) open systems (rivers, canals, ditches, flumes, and furrows) and (2) closed systems (steel, aluminum, asbestos-cement, and concrete pipe). In modern irrigation, open systems are most commonly used for conveying large quantities of water from a source to an irrigated area, and for transient conveyance systems, such as temporary ditches. For permanent systems of a moderate and small capacity there has been an increasing tendency during the last 50 years to change over from open systems to closed ones. This is related to some advantages of closed over open systems which, for concrete pipe, are discussed on the following pages.

Concrete pipe has been widely used in many irrigated areas, largely on flat grades, but it has become increasingly attractive for use on steeper land due to improvements in pressure-control mechanisms.

Concrete pipe systems are conveniently used with border-, basin-, and check-irrigation and they can be adapted to sprinkler irrigation. With additional surface pipe, efficient furrow irrigation is possible.

To date, concrete pipe has found little use in irrigation in Hawaii. Much development work in Hawaii has been concentrated on improvements in open systems, notably flumes. It is not known in how far the advantages of concrete pipe over ditches and flumes would make concrete pipe attractive for expanded use in Hawaii, as this would be determined by economics. This has not been studied as this aspect falls beyond the scope of this report. However, it is believed that there are areas in Hawaii where an increased use of concrete pipe may be profitable.

This circular deals with the technical details of the design and installation of concrete pipe systems. Use has been made of knowledge gained on the U. S. mainland; available information has been adapted and extended for use under local conditions.

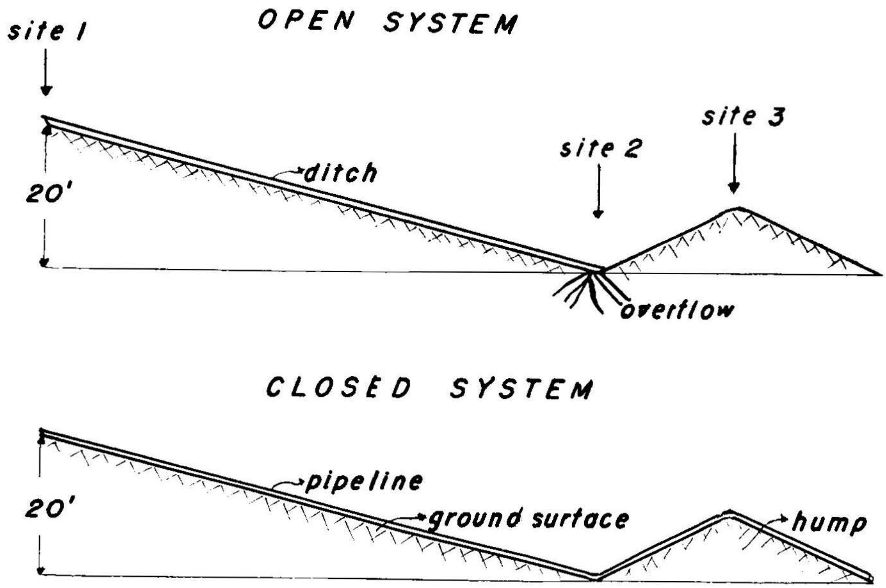


Fig. 1. In an open system (top) the elevation head due to the position of sites 1 and 2 cannot be utilized to bring water to the hilltop (site 3), as can be done in a closed system (bottom).

OPEN VERSUS CLOSED SYSTEMS OF WATER CONVEYANCE

Grade requirements

In order to appreciate the difference between these two systems, recognition should be made of the three forms of energy in a water body:

(1) Elevation energy, E_e, representing the energy derived from the position of water in relation to a reference level.

(2) Pressure energy, E_p, representing the thrust exerted by water against the walls of a vessel or conduit.

(3) Velocity energy, E_v, representing the kinetic energy of moving water.

Adopting the principle of conservation of energy, the total energy at any point in a system should be the same as at any other point, as is expressed by the Bernoulli equation:

For *liquids at rest:*

$$\begin{array}{ccc} \textit{Site 1} & & \textit{Site 2} \\ E_{e_1} + E_{p_1} & = & E_{e_2} + E_{p_2}, \end{array}$$

where subscripts 1 and 2 indicate sites 1 and 2.

For *liquids in motion* two other factors are introduced, E_v and E_f, E_f representing energy lost to the system by friction incurred during flow. The equation then becomes:

$$\begin{array}{ccc} \textit{Site 1} & & \textit{Site 2} \\ E_{e_1} + E_{p_1} + E_{v_1} & = & E_{e_2} + E_{p_2} + E_{v_2} + E_{f_{1-2}}, \end{array}$$

where subscript 1–2 represents friction losses between sites 1 and 2.

In open conveyance systems the pressure term is negligible for most practical purposes, and all elevation energy is converted into velocity energy or is dissipated as friction. The relationship here is $E_{e_{1-2}} = E_{v_2} + E_{f_{1-2}}$, where $E_{e_{1-2}}$ represents the energy derived from the difference in elevation between sites 1 and 2. In a closed system the three forms of energy, elevation, pressure, and velocity energy, are interconvertible.

In an open system, water will seek the lowest level, while in a closed system, energy due to elevation head or to applied pressure can be utilized to bring water uphill, as is illustrated in figure 1.

Discharge from open and closed systems

Difficulties have been experienced in Hawaii in diverting water from aluminum and concrete flumes on steep slopes into furrows and particularly in the regulation of such diversion. This is caused by the high velocity of water in these flumes (figure 2). Similar high velocities may also occur in closed systems (see table 9), but containment of water within a pipe results in pressure being exerted on its walls, assuming the pipe to flow full. This pressure results in water being positively pushed out through any openings provided. By contrast, water diverted from flumes has to be "caught" by receptacles placed in the path of the water stream.

Erosion and seepage control

An argument that closed systems and the use of lined ditches prevent erosion is not as valid in Hawaii as it is on the U. S. mainland. The maximum permissible velocities in unlined ditches safe against erosion advocated for U. S. mainland conditions (Houk, 1956) are generally well exceeded in Hawaii without harmful effects (Wadsworth, 1937). This is possible by the structural stability of most Hawaiian soils of a lateritic nature. However, merely to combat erosion on certain problem soils, concrete pipe has replaced open ditches on the sugar plantations of the Waialua Agricultural Company, Oahu, and the Pioneer Mill Company, Maui. In the first case the pipe was laid in an eroded ditch which was then filled (Rhea, 1929), and in the other case the pipe has been laid at the surface of the land.

Water losses by seepage from unlined canals and ditches tend to be greater in Hawaii than on the U. S. mainland. Percolation rates of Hawaiian soils (indicated in table 1) generally exceed those of soils of the temperate zone. For that reason lined ditches are extensive in Hawaii (Wadsworth, 1937). For seepage control, concrete pipelines would be as effective as lined ditches.

Fig. 2. The high velocity in concrete flumes on steep slopes makes it difficult to intercept water for diversion into the furrows.

Land occupation, land accessibility, and weed control

Including an assumed 10-foot-wide berm, a mile-long field ditch 6 feet wide occupies about 2 acres of land. The 173 miles of secondary ditches in Hawaii, reported by Wadsworth (1937), would take about 350 acres out of sugar cane production. This could be reduced to about 50 acres if underground pipe were used.

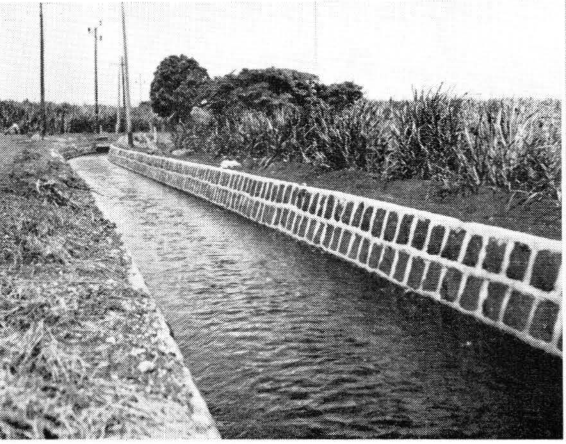


Fig. 3. Open ditches mean land occupation.

Fig. 4. Uncontrolled velocities on steep slopes cause erosion in irrigation ditches.

Aluminum and concrete flumes, widely used in sugar cane fields, are cemented in for the duration of the crop cycle. Being laid at the surface, they tend to obstruct mechanized field operations in the course of the crop cycle.

Figures 3, 4, 5, and 6 compare the problem of land occupation by ditches, erosion in unlined ditches, and weed growth in and around ditches with a situation where underground concrete pipe has been installed.

Advantages of open systems

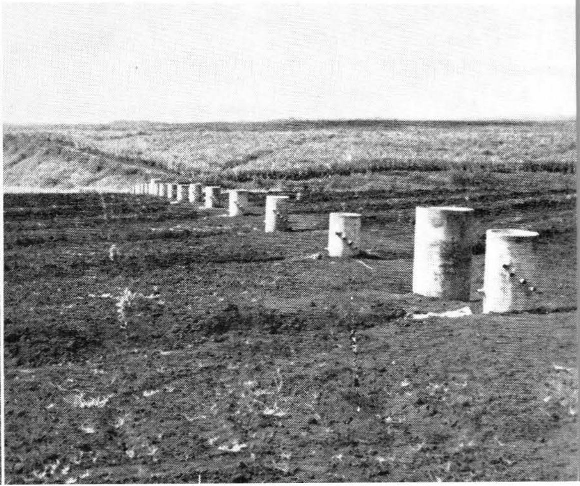
As against the points in favor of closed systems, open systems offer some advantages. Flumes and unlined ditches allow a farming layout to be altered, and few installations may be lost when crop land is abandoned. Trash poses less of a problem in open than in closed systems. Installation costs of open systems are usually less than those of closed systems.

Disadvantages of closed systems

The above remarks could virtually be reversed. Underground pipe systems reduce the flexibility of a farming layout and where fields are aban-

Fig. 5. Unlined irrigation ditches require weed control.

Fig. 6. An underground pipeline for irrigation water distribution eliminates erosion and seepage and occupies little land.



doned a permanent pipe installation may be lost. Entrapment of trash within a pipe system may harm the functioning of a closed system. Careful control of trash at pipe inlets is thus required.

THE SELECTION OF CONCRETE PIPE

Manufacturing processes

Pipes of a diameter of 18 inches or less are usually manufactured by either the packer-head or the tamping method. In either case a "dry" mixture is packed or tamped between a stationary mold and a revolving interior or exterior drum. The quality of the two resulting pipes is comparable. Pipes of larger diameter are generally made by a centrifugal process in which a "wetter" mixture is applied to an exterior revolving drum. The centrifugal force induces densification of the concrete. Excess water collected on the inside during spinning is drained away.

Pipes of 12 inches and smaller are usually nonreinforced. From 12 to 22 inches the pipe may or may not be reinforced, but most irrigation pipe in this range is nonreinforced. Pipes of 24 inches diameter and larger are usually reinforced by a metal grid embedded in the wall of the pipe (page 36).

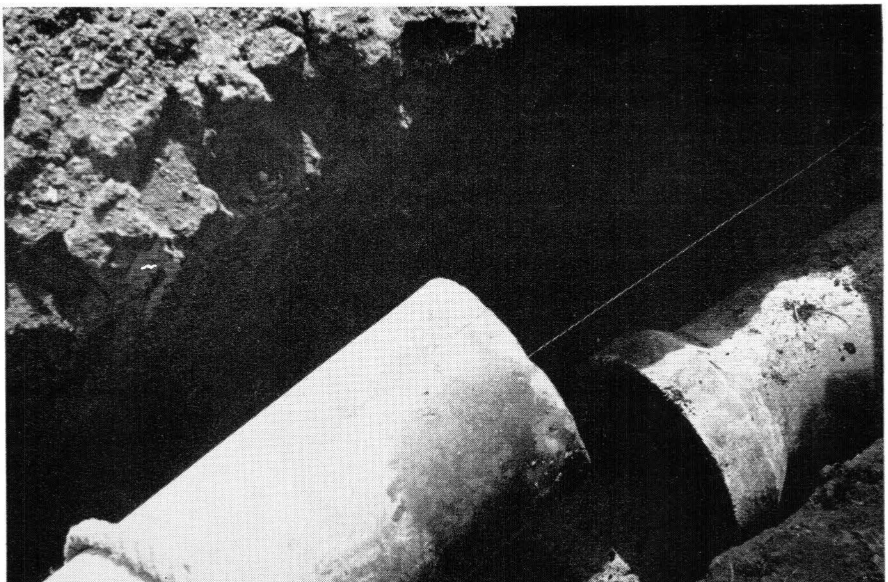


Fig. 7. The bell-and-spigot type of joint. The bell end (right) is embedded in a mortar-filled excavation in the trench bottom.

Fig. 8. The tongue-and-groove type of joint. The two pipes at bottom right show the groove end. (The groove corresponds to the bell.)

The quality of the pipe should conform to standards set by the American Society for Testing Materials (A.S.T.M.), 1916 Race Street, Philadelphia, Pa. These are given in specification A.S.T.M. C118-56 for nonreinforced pipe and by A.S.T.M. C361-55T for reinforced pipe. The standards are reviewed periodically for possible changes.

Types of joint

The bell-and-spigot type of joint (figure 7) has been most widely used in the past for both irrigation and sewage pipe, but the tongue-and-groove type of joint (figure 8) has gradually generally replaced bell-and-spigot pipe for irrigation on the U. S. mainland. The tongue-and-groove pipe has been found to have some advantages in that it is slightly cheaper to manufacture, is less bulky, and requires slightly less mortar for jointing pipe sections. Yet, it is generally believed that the individual pipe sections can be joined as firmly together as in bell-and-spigot pipe. For sewage pipe, however, bell-and-spigot pipe is still widely used.

In pipes of large diameter, 36 inches and up, and also in pipes of smaller diameter, where they are laid in soils subject to movement, flexible rubber-gasket (expansion) joints are used to connect pipe sections (figure 9). In the latter case expansion joints may be inserted in a mortar-jointed line

Fig. 9. Flexible rubber-gasket joints are being used in this 36-inch concrete pipe system for sewage conveyance.

at certain intervals. The required interdistance between the expansion joints has been calculated by Roberson, Tinney, and Sibley (1956), but the calculation requires a knowledge of the longitudinal compressive stress to which the pipe is subject and of the resistance between soil and pipe. Either of these factors is hard to evaluate under field conditions, but the above publication can be consulted for the solution of a sample case.

Length of pipe sections

Prefabricated pipe is usually manufactured in sections varying from 3 to 8 feet in length. Standard lengths are provided for certain pipe sizes, but there may be some flexibility in manufacturing pipe of lengths deviating from the standard ones. Longer pipe sections mean less jointing, but they entail more care in handling.

Jointless concrete pipe

In recent years pipelines have sometimes been cast in place in the field. The bottom of the trench is shaped to form the exterior lower mold. The exterior upper mold and the interior mold are provided by a concrete-mixing machine while it passes over and through the trench. This machine introduces concrete between the molds from a vibrating outlet. A revolving, tamping, packing, and centrifugal action used for making prefabricated pipe is hard to imitate under field conditions, and it is believed that the cast-in-place pipe is not of the same quality as that of prefabricated pipe. However, experimental data are not available in support of this belief. Nevertheless, this type of construction is gaining in popularity for straight, long lines on a uniform grade. At present the minimum size laid with this system is 24 inches.

THE USE OF CONCRETE PIPE SYSTEMS IN IRRIGATION

To convey irrigation water, concrete pipe systems can be used in the following ways:

A. For conveying water from a source to irrigated fields. This has recently been applied for developing irrigation water for the Waimea and Lalamilo areas on the island of Hawaii, and it has been planned for the Molokai project now under construction. Both projects are under the supervision of the Hawaii Water Authority. These systems can be designed according to circumstances.

   1. Provided the shape of the land surface permits, the pipelines can be laid on such a grade that friction losses balance elevation head, a procedure which has been possible in the design of the abovementioned Waimea line.

   2. On steeper grades water can be run through long lines without pressure control, provided the outflow at the ultimate end is free, for instance, into a reservoir. However, protective measures to prevent scouring at this end may have to be taken. Also, it will be necessary to introduce manholes at intervals so that the line is accessible for inspection and repair.

B. Where water needs to be taken out of the line at intervals, the introduction of gates for diversion is required and pressure control may be necessary. This situation is encountered where pipelines supply irrigated fields; the design of these lines is discussed on the following pages.

THE DESIGN OF CONCRETE PIPE SYSTEMS

Design based on water requirements

A pipeline should be designed with a knowledge of the flow required at the discharge ends. This is determined by the rate at which water should be applied to the land. For sugar cane this subject has been studied by Robert B. Campbell of the Experiment Station of the Hawaiian Sugar Planters' Association in collaboration with sugar cane plantations, by inflow and outflow tests in 100-foot furrow sections. This work has resulted in recommendations of required furrow application rates (also called furrow inflow rates, that is, quantities of water applied to the inlet of the furrow). These data have been made available for incorporation in this publication and are shown in table 2.

The values given in gallons per minute (g.p.m.) are for a typical application of 2.5 inches of water to the soil, to furrows on 2 percent grade, 100 feet long, and spaced at various interdistances. The table can be used for furrow lengths other than 100 feet and for furrow grades of 1 and 3 percent by applying the correction factors in the footnotes. The data apply to "clean" furrows and they may serve as a guidance for design whenever furrows can be maintained in that condition during the crop cycle. Their value for field conditions is insofar still limited in that quantitative control of discharge, whether from pipes or flumes, has not yet been achieved under field conditions (pages 7 and 47).

Additional information is still required on furrow application rates for older cane fields. Furrows in these fields are variably trash-clogged so that the hydraulics of flow is entirely different from that in "clean" furrows. It is common practice to increase the furrow application rates in trash-clogged furrows to as much as 50 or even 100 g.p.m. As the maximum water requirements determine the capacity of a system, for sugar cane the maximum discharge presently required in field practices should form the basis for design until further experimental information is available.

Furrow application rates cannot be reliably calculated in the absence of field data. U. S. mainland information cannot be applied to Hawaiian conditions because of the much higher infiltration rates observed in Hawaiian soils (table 1). Moreover, furrow application rates are generally limited on the U. S. mainland by erosiveness of the soil; under Hawaiian conditions the limitations imposed by this factor are less severe. For nonsugar cane crops no experimental data on furrow application rates are available yet. If table 2 were consulted for this purpose, caution should be practiced in applying the data. The reason for this is that at this stage it is not known what the effect on furrow application rate would be of furrows differing in shape from the typical trapezoidal ones used in sugar cane irrigation.

The interval between irrigations should also be taken into account in the design of a pipe system, as the minimum interval between irrigations determines the system capacity. The following relationship applies:

TABLE 1. Range of furrow infiltration rate observed in typical irrigated Hawaiian soil families

Furrow infiltration rate, inches per hour	Latosol series: Low humic	Latosol series: Humic	Latosol series: Humic ferruginous	Grey Hydromorphic series
0.5	Molokai			
1.0				
2.0	Lahaina			
3.0	Kahana			Honouliuli
4.0			Puhi	
5.0		Paauhau		
6.0	Kohala			

(Data made available by R. B. Campbell, Experiment Station, Hawaiian Sugar Planters' Association.)

TABLE 2. Recommended water application rates in gallons per minute to sugar cane furrows of 100 feet length* on a 2 percent grade† at various interdistances, to obtain a 2.5-inch application to the soil

Furrow infiltration rate, per hour	Time to apply 2.5 inches to soil	Interdistance between furrows in feet: 2	3	4	5	6	7
inches	*minutes*	*g.p.m. per 100 feet furrow length*					
0.5	300	1.0	1.6	2.1	2.6	3.1	3.6
1.0	150	2.1	3.1	4.2	5.2	6.2	7.3
2.0	75	4.2	6.2	8.3	10.4	12.5	14.5
3.0	50	6.2	9.3	12.5	15.6	18.7	21.8
4.0	38	8.3	12.5	16.6	20.8	24.9	29.1
5.0	30	10.4	15.6	20.8	26.0	31.2	36.4
6.0	25	12.5	18.7	24.9	31.2	37.4	43.6

*For furrows differing in length from 100 feet, increase or decrease g.p.m. values in accordance with furrow length.

†For 1 percent grade multiply g.p.m. values by 1.3 and application time by 0.8; for 3 percent grade use factors of 0.8 and 1.25, respectively.

NOTE: To insure a desired water application to the soil under all conditions, it is recommended to apply a safety factor of about 1.2 to the above values.

(Data made available by R. B. Campbell, Experiment Station, Hawaiian Sugar Planters' Association.)

$$\text{interval between irrigations in days} = \frac{\text{inches of water in root zone available to crop}}{\text{consumptive use by crop in inches per day}}$$

Table 3 shows average depth of water in inches available to crops in the top foot of soil for the majority of irrigated soil families in Hawaii. Available water in the second and third foot of soil is probably generally about 10 percent less, assuming soil to extend rather homogeneously to that level.

Few data on consumptive water use by crops in Hawaii have been published. Some information on water needs by sugar cane has recently appeared in the 1958 Report of the Experiment Station Committee of the Hawaiian Sugar Planters' Association. This shows that consumptive use varied from 0.10 inch per day during the winter months to slightly over 0.30 inch per day during the summer months in a central Maui location. Consumptive use of crops other than sugar cane should generally be lower, particularly that of pineapple, but no quantitative data are available yet.

Pipelines should be designed for the peak months of water use: the summer months. Away from the summer months the interval between irrigations should be longer, with the consequence that a pipe system is idle for varying lengths of time.

TABLE 3. Approximate depth of water in inches per foot of surface soil (difference between water held at ⅓ and 10 atmospheres soil moisture tension)*

Soil family	Soil description	† Pv	Bulk density*	Inches of water per foot of soil	Average from number of samples*
Molokai	Low humic latosol	7.5	1.26	1.1	26
Lahaina	Low humic latosol	7.3	1.32	1.2	51
Kahana	Low humic latosol	6.9	1.30	1.1	19
Kohala	Low humic latosol	5.5	1.31	.9	‡
Wahiawa	Low humic latosol	7.0	1.28	1.1	23
Waialua	Low humic latosol	12.0	1.26	1.8	‡
Puhi	Humic ferruginous latosol	7.0	1.04	.9	7
Haiku	Humic ferruginous latosol	5.5	1.06	.7	‡
Kalihi	Grey hydromorphic soils	8.0	1.31	1.3	‡
Kawaihapai	Alluvial soils	8.2	1.28	1.3	25

*From data provided by A. C. Trouse, Experiment Station, Hawaiian Sugar Planters' Association.
†Pv = moisture content as a percentage of the oven-dry weight of soil.
To convert Pv into inches of water per foot of soil, apply: Pv × bulk density × 12.
‡Data from Cornelison (1954).

The situation is somewhat different where not all land is in crop for which the system has been designed. This means smaller water requirements per unit time, resulting in pipes not flowing full.

Friction losses versus elevation head

Concrete pipelines can be used to convey water uphill by applying sufficient pressure to the water at the inlet, taking into account the pressure limitations of concrete pipe (page 23). However, most concrete pipelines are gravity systems. In these, water flows through the line by virtue of differences in elevation. The available head due to such differences determines the selection of the pipe size for a given flow. On flat grades little head loss is allowed and a large-size pipe must be used. As the elevation head increases on steeper land, higher friction losses in smaller pipe are allowable, and smaller pipe can and should be selected to save pipe cost. A selection of the most suitable pipe size can be made on the basis of table 4.

Topographic maps are useful for the planning of pipeline systems. The topographic detail on the map may vary according to the steepness of the land, as follows:

Slope of land	*Recommended vertical contour interval shown on map:*
Over 3 percent	5 feet
2–3 percent	2 feet
Under 2 percent	0.5 or 1 foot

From the topographic map a selection of the site of the lines is made. A field survey is then necessary to determine the exact differences in elevation of the selected sites.

Design for flat land

The process of balancing elevation head against friction losses in designing a system is best illustrated by examples. (1) Assume 2 cubic feet per second (c.f.s.) are to be distributed via a 2,000-foot line. Equal outflow is to take place via risers from 20 evenly spaced outlets. The elevation of the land decreases uniformly from the inlet to the ultimate end of the line by 5.5 feet; the available elevation head is thus 5.5 feet. Against this the friction losses for various pipe sizes have to be balanced. These losses are read from table 4.

Flow diminishes down the line as water is taken off through the outlets so that friction losses are reduced. To compensate for the reduced friction losses an empirical factor, F, shown in table 5, is applied to the friction losses given in table 4. For 20 outlets the F factor is 0.36. The calculation is then as follows:

friction losses in 2,000 feet of pipe carrying 2 c.f.s.:

	in 10-inch pipe	*in 12-inch pipe*	*in 14-inch pipe*
	16.0 feet	6.0 feet	2.8 feet
$\times$ 0.36	5.8 feet	2.2 feet	1.0 foot

TABLE 4. Estimated loss of head due to friction in feet per 1,000 feet of concrete irrigation pipe. (Corresponding velocities are shown in table 9.)

Flow:		Inside pipe diameter in inches								
g.p.m.	c.f.s.	6	8	10	12	14	16	18	20	24
90	0.2	1.1	0.2	......	......	......	......	......	......	....
180	0.4	4.6	1.0	0.3	......	......	......	......	......	....
270	0.6	10.4	2.3	0.7	0.3	......	......	......	......	....
360	0.8	18.4	4.1	1.3	0.5	0.2	......	......	......	....
450	1.0	28.8	6.4	2.0	0.8	0.4	......	......	......	....
540	1.2	42.0	9.2	2.8	1.1	0.5	0.2	......	......	....
630	1.4	56.0	12.5	3.8	1.5	0.7	0.3	......	......	....
720	1.6	74.0	16.3	5.1	2.0	0.8	0.4	0.2	......	....
810	1.8	93.0	20.7	6.5	2.4	1.1	0.5	0.3	......	....
900	2.0	......	25.4	8.0	3.0	1.4	0.7	0.4	0.2	....
990	2.2	......	30.8	9.5	3.7	1.6	0.8	0.4	0.3	....
1080	2.4	......	36.5	11.4	4.4	1.9	1.0	0.5	0.3	....
1170	2.6	......	43.0	13.3	5.1	2.3	1.1	0.6	0.4	....
1260	2.8	......	50.0	15.5	5.9	2.6	1.3	0.7	0.4	....
1350	3.0	......	57.3	17.8	6.8	3.0	1.5	0.8	0.5	0.2
1440	3.2	......	65.3	20.2	7.7	3.4	1.7	0.9	0.5	0.2
1530	3.4	......	73.5	22.8	8.8	3.9	1.9	1.0	0.6	0.2
1610	3.6	......	82.5	25.6	9.8	4.4	2.2	1.2	0.7	0.3
1700	3.8	......	92.2	28.5	10.8	4.9	2.4	1.3	0.8	0.3
1790	4.0	......	......	31.5	12.2	5.4	2.7	1.5	0.9	0.3
2020	4.5	......	......	39.7	15.3	6.8	3.4	1.9	1.1	0.4
2240	5.0	......	......	49.1	18.8	8.4	4.2	2.3	1.3	0.5
2470	5.5	......	......	59.6	22.8	10.2	5.0	2.7	1.6	0.6
2690	6.0	......	......	70.7	27.1	12.1	6.0	3.2	1.9	0.7
2920	6.5	......	......	82.7	31.8	14.2	7.1	3.8	2.2	0.8
3140	7.0	......	......	......	36.9	16.5	8.2	4.4	2.5	1.0
3360	7.5	......	......	......	42.4	18.9	9.4	5.1	2.9	1.1
3590	8.0	......	......	......	48.2	21.5	10.7	5.8	3.3	1.3
3870	8.5	......	......	......	54.4	24.3	12.1	6.5	3.7	1.4
4040	9.0	......	......	......	61.0	27.2	13.5	7.3	4.2	1.5
4260	9.5	......	......	......	68.0	30.3	15.1	8.2	4.7	1.8
4480	10.0	......	......	......	75.3	33.6	16.7	9.0	5.2	2.0
4930	11.0	......	......	......	......	40.7	20.2	10.8	6.3	2.4
5380	12.0	......	......	......	......	48.4	24.0	12.9	7.4	2.9
5880	13.0	......	......	......	......	56.8	28.2	15.2	8.8	3.4
6280	14.0	......	......	......	......	65.9	32.7	17.7	10.2	3.9
6730	15.0	......	......	......	......	......	37.5	20.3	11.7	4.5

Computation based on: $Q = C\,h^{0.5}\,d^{0.625}$, where Q = flow in c.f.s.; C = empirical constant 0.31; h = loss of head in feet per 1,000 feet of pipe; d = inside diameter of pipe (Scobey, 1920).

It is assumed that the center of the pipeline is all the way at 2.5-foot depth and that 2.5-foot head is available at the inlet which can be used to bring water from the line to the ground surface at the outlets. However, it should generally be attempted in designing a system to provide for 1-foot discharge head at the outlets. Allowance should also be made for minor head losses occurring at pipe inlets, bends, and outlets (page 45). Then:

	Required head: for 10-inch pipe	for 12-inch pipe	for 14-inch pipe
Friction losses	5.8 feet	2.2 feet	1.0 foot
Discharge head	1.0 foot	1.0 foot	1.0 foot
Minor head losses, assume	2.5 feet	2.0 feet	1.5 feet
	9.3 feet	5.2 feet	3.5 feet

Under these circumstances the 12-inch pipe would probably be selected. If the 10-inch pipe were chosen, additional head by a pump via a standpipe (figure 11 and page 36) should be supplied at the inlet of the line. The horsepower (h.p.) requirements are calculated by using the equations:

$$\frac{\text{g.p.m.} \times \text{head in feet (h)}}{3{,}960} = \frac{\text{c.f.s.} \times \text{h}}{8.8} = \text{water horse power (w.h.p.)}$$

$$\frac{\text{w.h.p.}}{\text{efficiency pump}} = \text{output h.p. motor}$$

(Note: 1 c.f.s. = 448.8 g.p.m.)

(2) Where a large area with variable and nonuniform grades needs to be covered, it may be found that several layouts are possible. They should be designed on paper and be compared, and that layout should be selected which is most economical and best suited to the farming layout. Because of cost, a system may be installed over a number of years. In that case the initial installation should be so designed that it is capable of extension.

Where possible, water should be introduced at a high point on the farm. Several sidelines may be required and standpipes with control gates

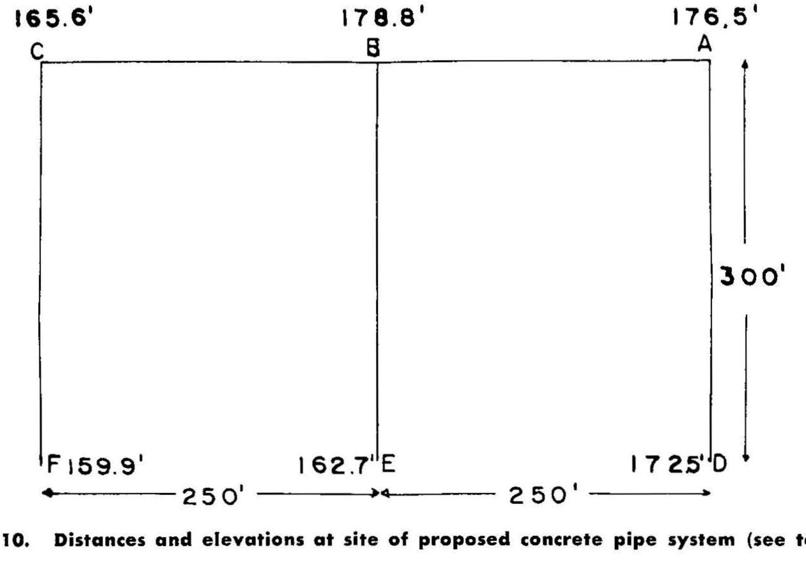


Fig. 10. Distances and elevations at site of proposed concrete pipe system (see text).

may have to be introduced for the control of water diversion. This is illustrated by the following example.

Figure 10 shows sites, and their elevation, where water is introduced into an area (site A) and needs to be made available (sites B to F). The assumption is made that the maximum flow will be 5 c.f.s. which is to be discharged wholly at any point away from A. As in the previous example it is assumed that the pipes are 2.5 feet below the ground surface and that a head of this magnitude is available at the inlet to bring water to the surface at discharge sites.

Using figure 10 and table 4 a comparison of available head and friction losses for 5 c.f.s. within the range of this head, adjusted for discharge head and minor head losses, is as follows:

Site of pipeline	*Length, feet*	*Available elevation head, feet*	*Pipe size, inches*	*Friction loss, feet*	*Head requirement, feet*
A–B	250	– 2.3	16	1.1	3.6
			14	2.1	4.6
B–C	250	13.2	12	4.7	7.7
			10	12.3	15.8
A–D	300	4.0	16	1.3	3.8
B–E	300	16.1	12	5.6	8.6
			10	14.7	18.2
C–F	300	5.7	14	2.5	5.0
			12	5.6	8.6

To convey 5 c.f.s. from A to B, 5.9 or 6.9 feet of head (or more head if a pipe size smaller than 14 inches were selected) need to be supplied via a pump at A, in a manner illustrated in figure 11. To supply 5.9 feet of head the water level within the standpipe should be 5.9 or 6 feet above the ground surface, as the ground surface forms the reference level. The standpipe should be 8 feet high, taking 2 feet of freeboard into account. It should conform to the requirements given on page 35. To keep the velocity within the stand to the required 1 foot per second, a 30-inch diameter standpipe should be adequate (table 6).

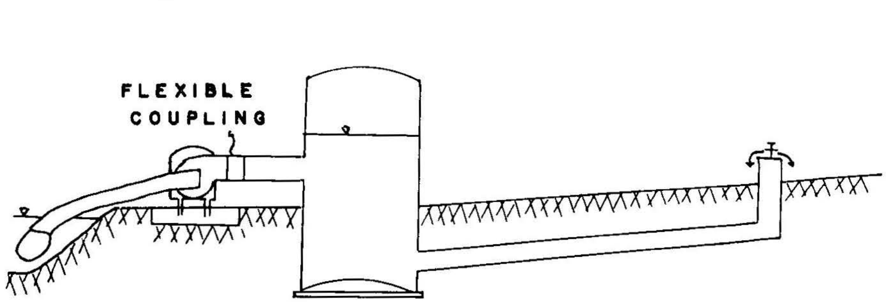


Fig. 11. A booster pump is connected to a concrete pipe system via a standpipe to provide additional pressure.

TABLE 5. Factors for determining head losses in pipelines with multiple outlets equally distributed along the line (from Christiansen, 1942)

No. of outlets	Factor	No. of outlets	Factor	No. of outlets	Factor	No. of outlets	Factor
1	1.00	5	0.44	9	0.39	20	0.36
2	0.63	6	0.42	10	0.39	30	0.35
3	0.52	7	0.41	12	0.38	50	0.34
4	0.47	8	0.40	15	0.37	200	0.33

TABLE 6. Minimum diameter of standpipes for maximum allowable velocities

Inside diameter of standpipe, inches	Maximum allowable velocities in feet per second					
	1 f.p.s.		2 f.p.s.		10 f.p.s.	
	Maximum discharge at velocities given					
	c.f.s.	g.p.m.	c.f.s.	g.p.m.	c.f.s.	g.p.m.
12	0.8	350	1.6	700	7.9	3,520
14	1.1	480	2.1	960	10.7	4,800
16	1.4	630	2.8	1,250	14.0	6,270
18	1.8	790	3.5	1,590	17.7	7,930
20	2.2	980	4.4	1,960	21.8	9,790
24	3.1	1,410	6.3	2,820	31.4	14,100
30	4.9	2,200	9.8	4,410	49.1	22,030
36	7.1	3,170	14.1	6,350	70.7	31,720
42	9.6	4,320	19.3	8,640	96.3	43,180
48	12.6	5,640	25.2	11,280	125.8	56,410

Computation based on: $q = av$, where q = flow in cubic feet per second; a = cross-sectional area of standpipe in square feet; and v = velocity in feet per second.

For the C–F line a 14-inch pipe would be selected. For the B–C and B–E lines a combination of two different pipe sizes is most economical. To calculate the desired length of each pipe size, a simultaneous equation is applied as follows, using the B–C line as an example:

elevation head at C	13.2 feet
less head for minor losses and discharge, assume, as before	3.0 feet
head available to dissipate friction	10.2 feet

Let the length of the 12-inch pipe be x and that of the 10-inch pipe be y, and given that the total length of the line is 250 feet, then the allowable friction losses are:

(take friction losses from table 4)

$$\frac{4.7x}{250} + \frac{12.3y}{250} = 10.2$$

$$= 4.7x + 12.3y = 2{,}550$$

divide by 4.7; hence

$$x + 2.6y = 543$$

but

$$x + y = 250$$

subtracting:

$$1.6y = 293$$

$$y = 180$$

The length of the 10-inch pipe y is thus 180 feet and that of the 12-inch pipe, 70 feet. Checking the friction losses, it is found that they total 10.2 feet in both pipes. The pipe of smallest diameter is laid nearest to the line terminus. A reducing section connects pipes of different sizes.

Before water would flow into the line B–C, the B–E line would fill up as this is at lower level. To facilitate diversion of water into the B–C line, a standpipe is introduced at B with a sliding gate on the inlet of the B–E line (see figure 12). In this case the height of the standpipe is determined merely by the requirement that it should be 4 feet above the ground

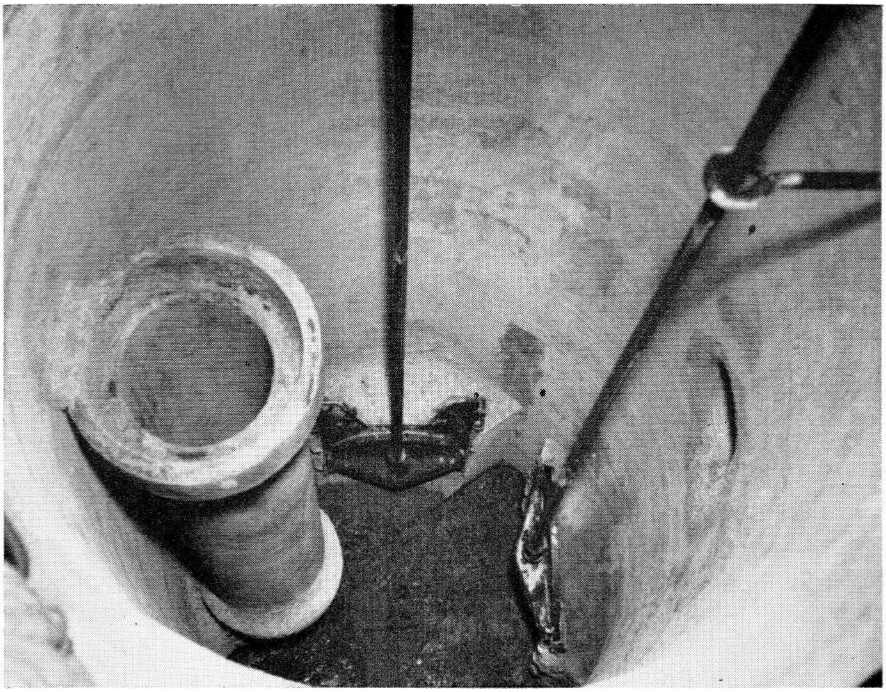


Fig. 12. The inside of this standpipe shows an overflow pipe attached to the upstream inlet (fig. 15*d*), a surface outlet opposite and below the level of the top of the overflow pipe, and two gates regulating inflow into outlet pipes.

surface (page 35). The diameter of the standpipe needs only be large enough to accommodate a 12-inch sliding gate. However, to allow accessibility to the bottom of the standpipe for repair, a 30-inch standpipe would be required.

If three lines took off from B, two gates would be required independent from the elevation of the second and third lines. The highest line would be left without a gate.

Design for steep land

Pressure control

On steep land pipelines should be laid, where possible, across-slope on a flat grade. However, where steep grades are unavoidable, water pressure within the pipeline should be controlled in ways discussed below. Nonreinforced concrete pipe should conform to the laboratory pressure tests described in the A.S.T.M. specification C118-56, referred to on page 11. The safety factor which should be applied to this test pressure, to arrive at allowable operating pressure for field conditions, is generally assumed to be 4; that is,

$$\text{allowable operating pressure} = \frac{\text{test pressure}}{4}.$$

Table 7 gives presently recommended allowable operating pressure for nonreinforced concrete pipe. For reinforced concrete pipe conforming to specification A.S.T.M. C361-55T, operating pressures should generally not exceed 100 feet.

TABLE 7. Recommended allowable operating pressure in nonreinforced concrete pipe conforming to A.S.T.M. specification C118-56

Inside diameter of pipe in inches	Wall thickness in inches	Operating pressure in feet
8	7/8	28
10	1	28
12	1 1/8	23
14	1 1/4	23
16	1 3/8	23
18	1 1/2	23
20	1 7/8	21
24	2 1/8	21

The safety factor of 4 for nonreinforced concrete pipe does not take into account instantaneous high pressure which may develop as a result of water hammer or surging. Its avoidance should be anticipated in the design:

(1) by laying straight lines on uniform grades to avoid accumulation of suspended matter and of "dead" water,
(2) by providing adequate opportunity for air escape (page 32),
(3) by using slowly-closing gates and valves, and
(4) by introducing standpipes with pressure control structures in the pipelines at adequate intervals, as discussed below.

Introducing standpipes at intervals limits the pressure which can develop within the line, as they limit the height to which water can rise above the line. This limit is set by the fact that a standpipe is open at the top so that excess intake into a line over outflow from it would cause a connected standpipe to overflow (figure 13).

Standpipes act as manometers, as the water level within them indicates the pressure in the line at their site. Connecting the water level in successive standpipes by a line gives the hydraulic grade line, the height of which above the pipeline gives the pressure head at any point in the intervening pipe section (figure 14). The degree of steepness or the slope of the hydraulic grade line gives the hydraulic gradient.

The interval between standpipes inserted in a line for pressure control is determined by the steepness of the land. The vertical difference in elevation in feet at the ground surface between two successive standpipes, after adjustment for any height of water within the standpipe during operation, should not exceed the recommended values given in table 7.

The continuity of a pipeline, interrupted by standpipes for pressure control, is achieved by placing either overflow structures or float-controlled valves within the standpipes.

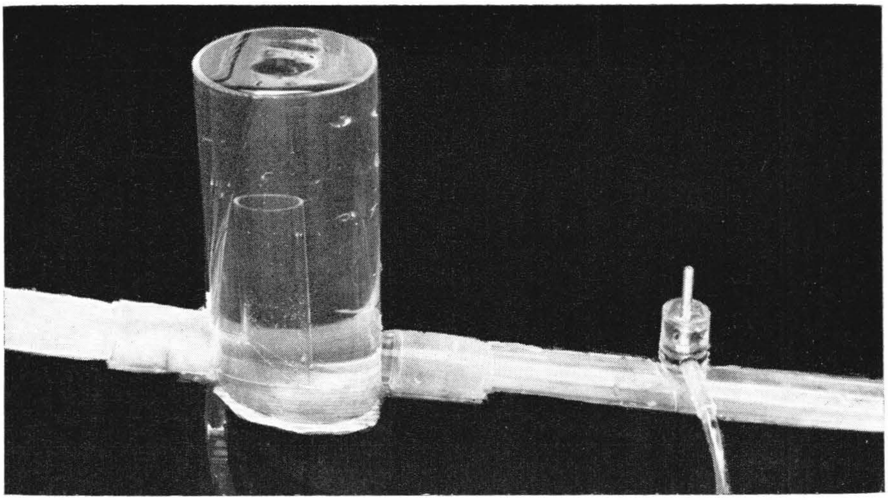


Fig. 13. If more water is applied to the inlet of a pipe system than is taken out of it, the lowest standpipe will overflow, as is illustrated in the above laboratory model. The height to which water can rise in a standpipe determines the maximum pressure which can develop in a pipeline at the site of the standpipe.

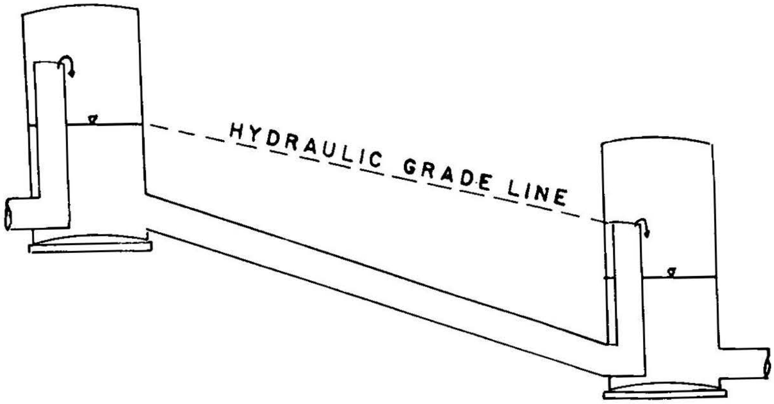


Fig. 14. The maximum pressure which can develop in a system with overflow structures is determined by the height of two consecutive standpipes.

Overflow structures within standpipes

Standpipes with overflow structures have been used for about 50 years. Their extended use indicates that they have served a purpose. Yet, difficulties have been encountered at an early date in that air tended to become entrained as water poured over baffles within the standpipe. The entrained air tended to cause water hammer and surging (Roberson, Tinney, and Sibley, 1956), particularly in long lines and pipe with low friction losses (Curtis, 1951; Hale, *et al.*, 1954).

Several overflow devices have been developed and figure 15 shows some of the most common ones in use. To reduce air entrainment a slanting section is introduced at the top of the overflow partition in structure (*a*), while a partially opened gate through the lower part of the partition wall in structure (*b*) attempts to maintain a depth of water on the downstream side of the partition above the downstream pipe inlet.

Both structures (*c*) and (*d*) have been used in an installation at the Waimanalo farm of the Hawaii Agricultural Experiment Station. Structure (*c*) is seen in operation under field conditions in figure 16, and air entrainment during overflow into the vertical pipe is demonstrated in a small laboratory model in figure 17. No difficulties have been encountered in using this structure under field conditions. This is probably due to the fact that the pipe used is one size larger in diameter than required. This causes pipes not to flow full and affords ample opportunity for air escape.

Alternatively, provision for the escape of entrained air can be provided by placing a vent close to the standpipe on the downstream side of the line. The vent should not be too close, as an opportunity must be given for air bubbles to rise to the upper side of the water stream in the pipe, otherwise they are liable to be swept past the vent. The actual distance of the vent below the standpipe depends on the velocity of the pipe stream and the diameter of the pipe which determine the rise velocity of the entrained air bubbles. According to Roberson, Tinney, and Sibley (1956) the fol-

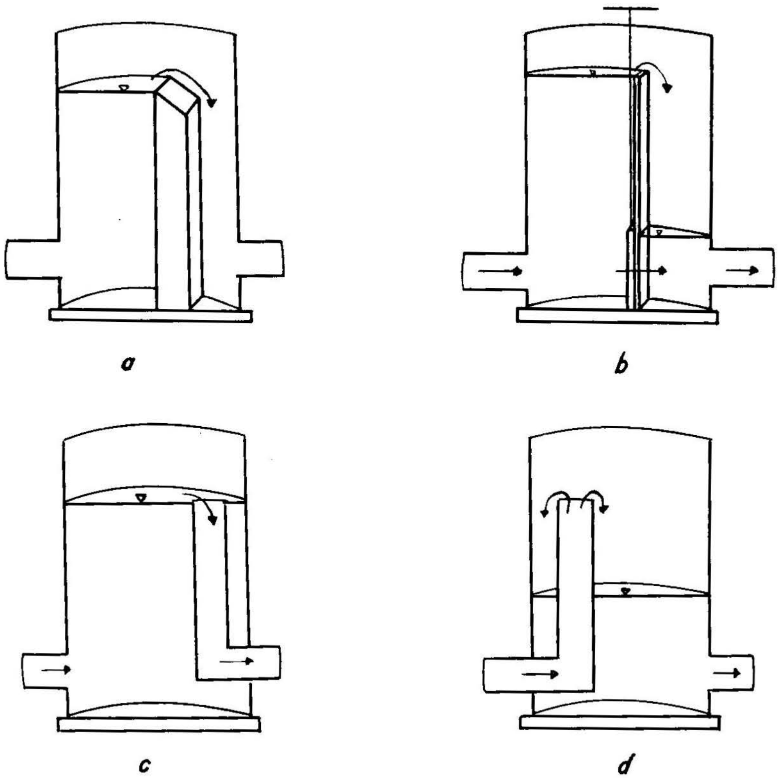


Fig. 15. Types of overflow structures in use within standpipes to control pressure in concrete pipelines on steep slopes.

lowing empirical relationship can be applied: $L = 1.76\ V_d D$, where: L = the length in feet downstream from the standpipe; V_d = the design velocity; and D = the diameter of the pipe.

The Bureau of Reclamation has experimented with standpipes closed at the top to limit the quantity of air available for inclusion in the water stream (Curtis, 1951). This, however, has caused some difficulties in regulating discharge on the upstream side of the standpipe because air trapped in the dome of the standpipe tends to change the hydraulic gradient upstream (Pillsbury, 1952).

Where a low-velocity head is assured, structure (*d*) is more satisfactory than structure (*c*), as air entrainment is avoided by the maintenance of a depth of water above the outlet pipe. This is shown in a laboratory model in figure 18. Under field conditions (figure 12) the system can be so regulated that the behavior is the same as shown in the model study.

The selection of the size and height of the standpipe and the height of the overflow structure is discussed on page 35.

Even where under field conditions no problem of air entrainment is encountered, a major problem in utilizing standpipes with overflow struc-

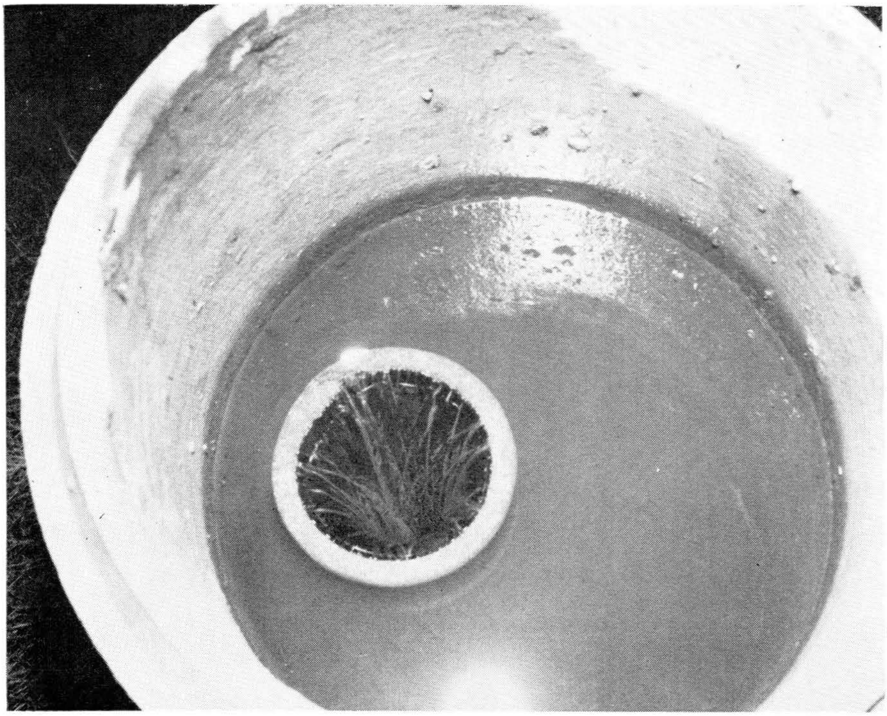


Fig. 16. The inside of a standpipe while water drops into a vertical overflow pipe attached to a downstream outlet pipe (fig. 15c). Air tends to be entrained during overflow.

tures is that inflow into a system cannot be readily matched by discharge from it. Where irrigation is changed from one field to another, one for which less discharge is required, inflow into the system needs to be reduced. Similarly, inflow into a system must be increased when larger discharge is required. An adjustment of inflow into a system during irrigating may be cumbersome and costly, particularly for large systems.

The described inflexibility tends to lead to malpractices in irrigating. Too large furrow application rates will be given where inflow into the system exceeds required discharge. Conversely, furrow application rates are too low where inflow into the system is less than required discharge.

Float-controlled valves within standpipes

A sounder approach towards the pressure problem is afforded by more recently developed float-operated valves of which there are a few makes available. Figure 19 shows an example of one type. The float-controlled valves are operated within standpipes and they regulate the inflow and outflow from them, as is illustrated in a small laboratory model shown in figure 20.

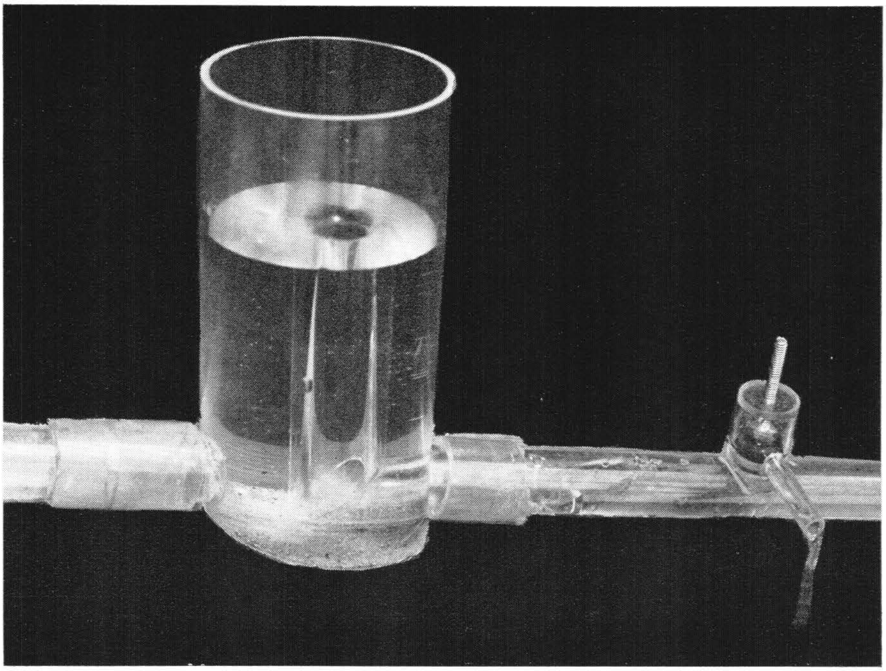


Fig. 17. Where the overflow pipe is attached to the outlet pipe within a standpipe, air is entrained as water drops into the vertical pipe. The entrained air collects in a pocket just below the standpipe but above the next outlet.

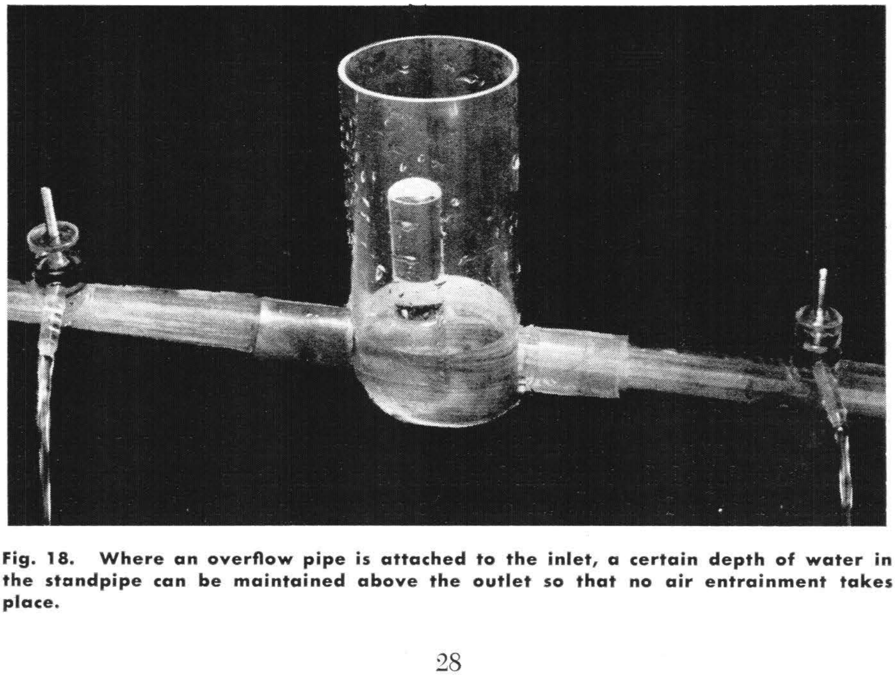


Fig. 18. Where an overflow pipe is attached to the inlet, a certain depth of water in the standpipe can be maintained above the outlet so that no air entrainment takes place.

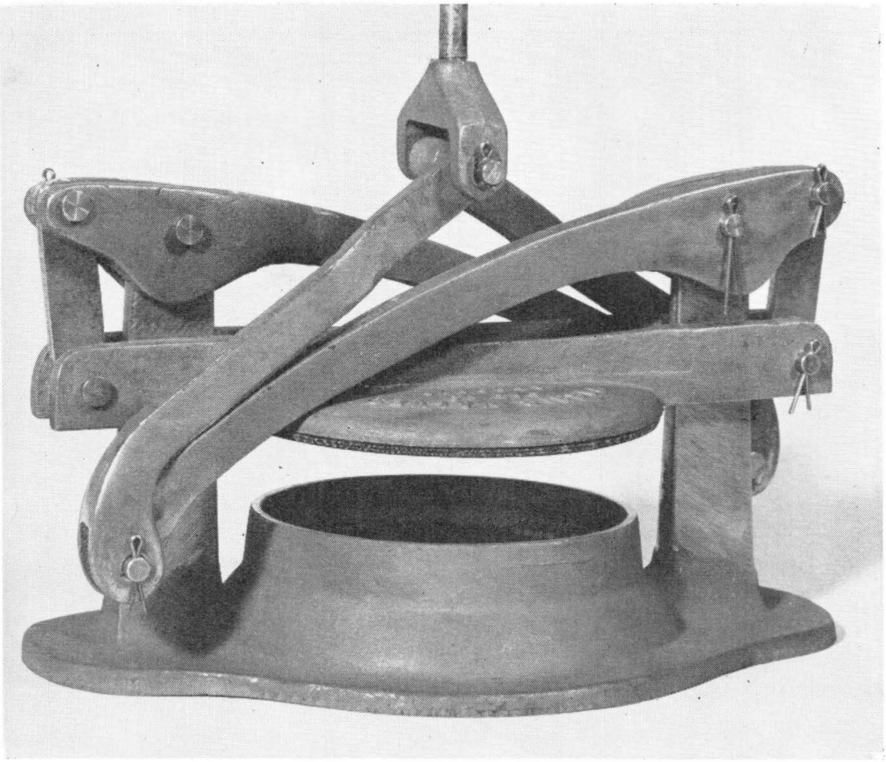


Fig. 19. A float-operated valve in an open position. The float attached to the spindle (top) upon being raised by a rising water level within the standpipe causes the valve (bottom) to be closed, thus allowing a readjustment of the water level within the standpipe.

Float-controlled valves eliminate air entrainment, as both inflow and outflow take place below the water surface within a standpipe.

Of more importance, however, is the fact that inflow into and outflow from a system are automatically balanced by the valves, *provided* that water can back up into the water source at the inlet of the system. This provision, however, eliminates the use of measuring devices at the inlet requiring a drop (page 47).

In a float-controlled system the quantity of water let into a standpipe is regulated by a valve on a vertical or horizontal inlet pipe. The opening of the valve is adjusted via a lever system by the position of a metal or styrafoam float at the water surface within the standpipe. A drop in water surface, resulting from increased discharge from the standpipe, causes the valve to open further so that more water is let into the standpipe. Conversely, decreasing the discharge from a downstream line causes the water level within the standpipe to rise so that the valve opening is decreased, till inflow into and outflow from the standpipe balance.

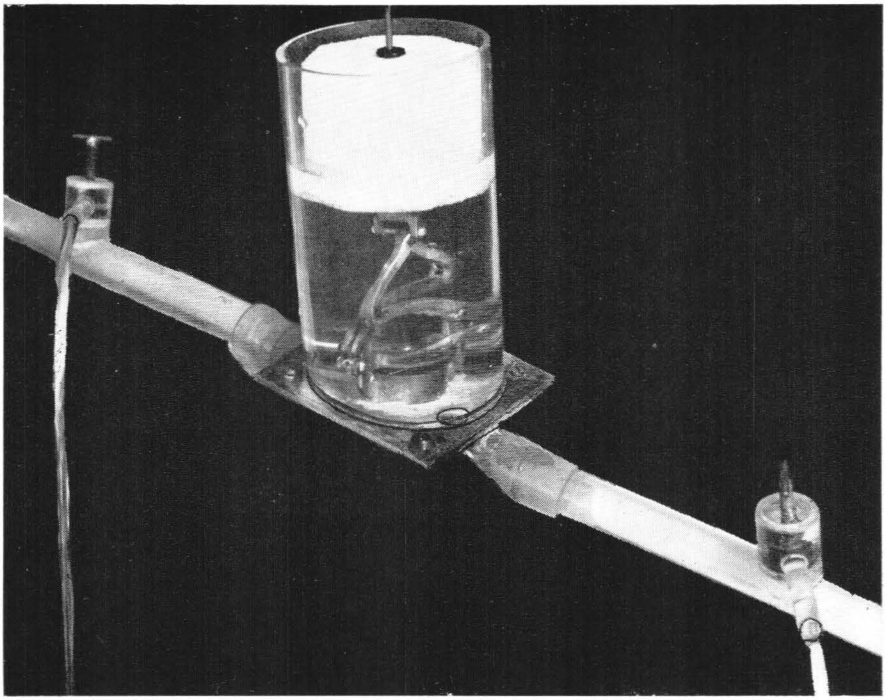


Fig. 20. In a laboratory model of a system using float-controlled valves for flow regulation, and styrafoam as a float, it is seen that both inflow and outflow take place below the water surface in the standpipe, thus preventing air entrainment.

To calculate the size of the float the following procedure is followed. Assume (1) that the float is of styrafoam with a water displacement of 55 pounds of water per cubic foot, (2) that an operating head of 23 feet is allowed in the concrete pipe, and (3) that a valve of 8 inches diameter is used.

The head of 23 feet corresponds to 10 pounds per square inch. The area of the valve is 50.2 square inches so that the valve is subject to a maximum thrust of 502 pounds. One cubic foot of styrafoam balances 55 pounds, which is magnified by a lever system connecting the float and the valve, say 5 times. Assuming that the float is designed for two-thirds submergence, 1 cubic foot of styrafoam balances $55 \times 5 \times 0.67 = 184$ pounds. Assuming further an arbitrary safety factor of 1.25, which includes provision for the weight of the float and the lever system, the size of the float selected would be

$$\frac{502 \times 1.25}{184} = 3.4 \text{ cubic feet.}$$

To reduce the bulkiness of the float and, therefore, the size of the standpipe, a balanced dual outlet (double-acting, or double disk) float-controlled

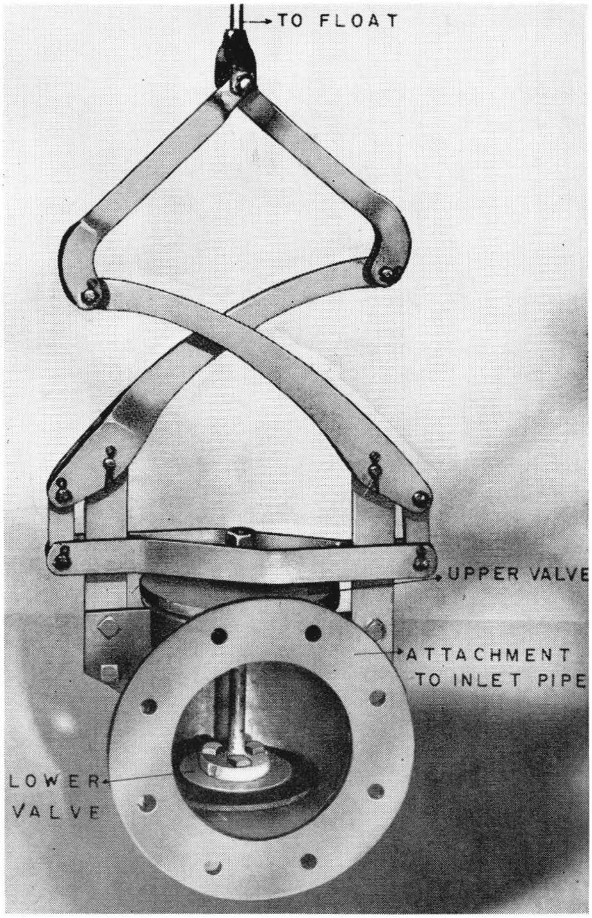


Fig. 21. In the balanced float-controlled valve shown, the float reacts to smaller differences in pressure, so that the float used can be smaller, reducing the size of the standpipe, where desirable. This valve is also used for controlling large pressures.

valve (figure 21) can be used. For this valve the float size is determined by the difference in thrust against the lower surface of an upper valve and the upper surface of a smaller, lower valve. This type of valve is also used for controlling large pressures, such as occur at the transition from a pressure system to a concrete pipeline. The spacing and size of the standpipes with float-controlled valves are the same as for systems controlled by overflow structures (see pages 24 and 35).

On steep land much head loss can be allowed and small valves can be selected. Figure 22 gives some laboratory determinations of the discharge-head loss relationship. Some manufacturers provide data on valve capacities at various heads.

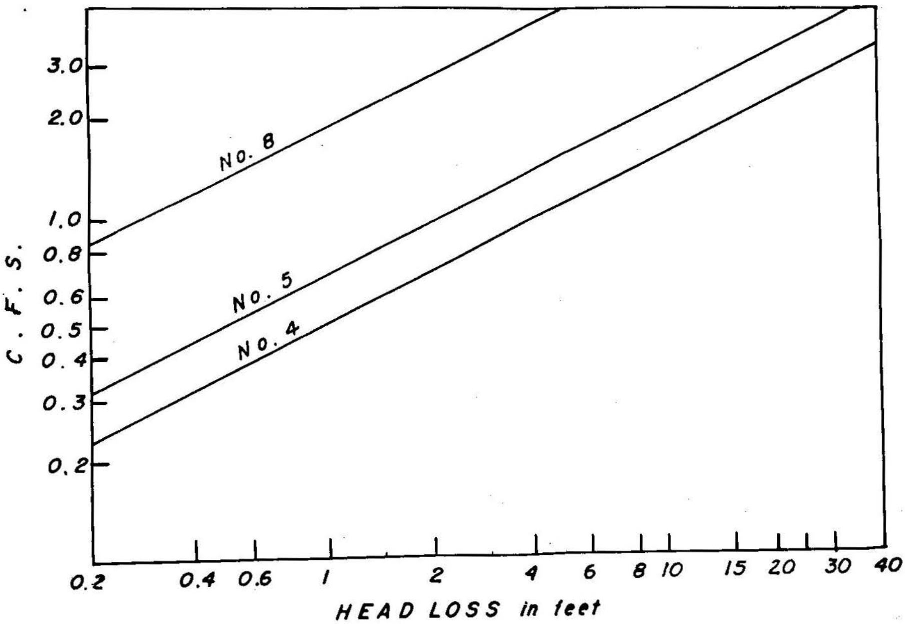


Fig. 22. Head loss versus discharge from nonbalanced, fully open, common commercial float valve (after Taylor and Pillsbury, 1953).

Float-controlled valves have been in use on the U. S. mainland for several years, and reports indicate that they function satisfactorily under field conditions.

On land that is so steep that standpipes are required at unduly close interdistances, pressure pipe of reinforced concrete (designed for about 100 feet of operating pressure), metal, or asbestos-cement should be considered.

AIR RELEASE FROM CONCRETE PIPE SYSTEMS

As concrete pipelines are subject to pressure limitations, ample opportunity should be provided for air to escape as the lines fill with water.

At the end of any line air must be able to escape in advance of water inflow, and intermediate escape vents should be provided for in long lines. The spacing between air vents should not exceed 1,000 feet (Texas Engineering Handbook, 1958). At high points and at bends air may be trapped and its release should be provided for by inserting vents at these points. For their design and installation the recommendations of the American Society of Agricultural Engineers (1958), quoted below, should be followed.

"*Vent Requirements*

(a) *Locations*

Vents shall be placed

(1) at the downstream end of each lateral (that is, just before termination of a line, figure 23);

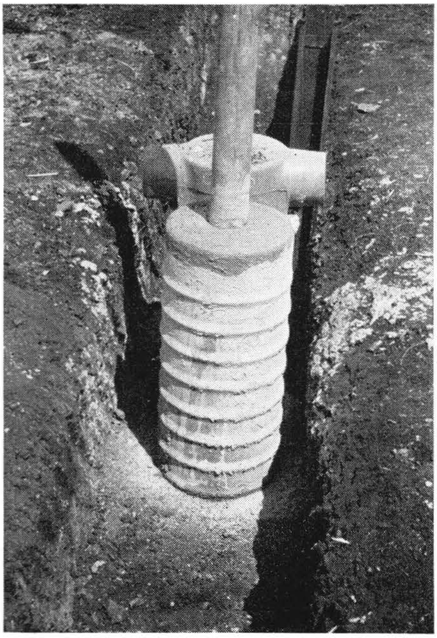


Fig. 23. An air vent is installed just before the end of a line, in front of the terminal riser and outlet. Note large-size diameter of bottom part of vent topped by 3-inch galvanized metal pipe.

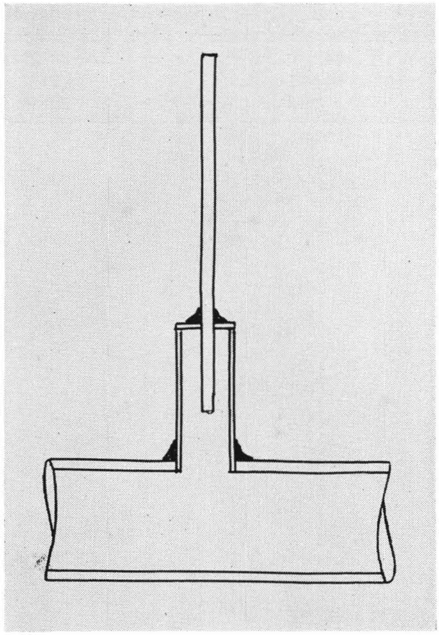


Fig. 24. General construction of an air vent. The lower portion has a large diameter to prevent air from being swept past.

(2) immediately downstream from where there is opportunity for air entrainment and inadequate opportunity for escape of that air;
(3) at high points wherever there are changes in grade downward in the direction of flow of more than ten degrees; and
(4) at all turns of 90 degrees or more with the exception of lines not more than 50 feet in length.

Any stand shall substitute for a vent. In all lines along which there are outlets at periodic intervals, an open upstream vent should be within sight of the irrigator who might be operating such outlets. Vents shall be spaced as necessary for successful operation at design capacity.

There shall be considered to be 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 down-draft of air into the well casing while the pump is in operation, the well shall be considered to pump air. In such case, steel cylinder pump stands shall not be used unless, (1) a vent is placed immediately downstream therefrom, or (2) the average downward velocity all the way from the pump discharge to the pipeline does not exceed one foot per second (table 6).

TABLE 8. Minimum inside diameter of air vent for size of pipeline given

Inside diameter of pipeline, inches	Minimum inside diameter of lower portion of air vent, inches	Minimum inside diameter of upper portion of air vent, inches
6	4	1
8	6	1
10	8	2
12	10	2
14	10	2
16	12	2
18	14	2.5
20	14	3
24	18	3
30	24	4
36	30	5
42	30	6
48	36	8

(b) *Size*

The cross-sectional area of the vent shall be 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 center line of the pipeline (table 8). Above, the vent may reduce to one sixtieth the cross-sectional area of the pipeline, but not less than two inches diameter pipe shall be used (figures 23 and 24). Vents shall have a freeboard above the hydraulic gradient of one to five feet with about two feet being preferred.

(c) *Air release valves*

Where the hydraulic gradient is more than 20 feet above the ground surface, air release valves can be used in place of a tall vent pipe, except in lines where outlets are located between vent and next downstream stand. The sizes of the air release valve shall be the same nominal size as called for by a tall vent pipe."

Figure 24 shows an air vent in cross-section. Some buffering action is afforded by inserting the upper narrow pipe below the top of the lower wide pipe. The large inlet prevents air from being swept past. The protruding upper part of the vent may be of concrete, asbestos-cement, or metal. Recommended pipe sizes are shown in table 8.

The air release valves mentioned refer to a plastic, rubber, or stainless steel ball snugly fitting into an upper seat in which it is pushed by water pressure, thus forming a seal. The ball drops into a lower chamber in the absence of water pressure allowing air to escape. These valves are commercially available; ordinary air vents are made up locally. As has been mentioned on page 25, special precautions have to be taken for air escape where there is an opportunity for air entrainment from overflow structures within standpipes.

FLOW-REGULATING DEVICES

Standpipes and gates

In a system consisting of more than one pipeline, linegates (Appendix) can be used to divert flow into one or more lines. Linegates are of the same diameter as the pipeline in which they are inserted. On closing, they block downstream flow, allowing diversion into a sideline at higher elevation.

However, standpipes are most widely used for flow regulation, and in addition to this function they serve as air vents (page 33), sand traps, where required, and as pressure-control devices (page 24). They also allow discharge at the ground surface. Standpipes are, moreover, used in conjunction with a pump and are then referred to as pump stands (figure 11 and page 19).

Water let into a standpipe from an inlet pipe can be diverted into one or more outlet pipes by using gates (figure 12 and page 22). The width of the standpipe is determined by its function: the number of pipes taking off from it, the number of gates, the space required for an overflow or float-controlled device, and the space needed for repairs in place. In addition, the diameter of the standpipe should be such that the recommended velocities are not exceeded. To this end, table 6 should be consulted. The height of the standpipe should be at least 4 feet above the ground surface to be clearly visible in farm operations. Any greater height required is determined by the water level needed in the standpipe to bring water to the end of any sideline. For instance, where the ground surface at the end of a line is 4 feet higher than at the site of the standpipe, 4 feet of head above the ground surface needs to be maintained within the standpipe merely to overcome this difference in elevation. For discharge to take place at the end of this line, additional head in the standpipe is to be provided to cover friction losses and discharge head. In irrigation practice, standpipes are commonly not more than 12 feet above the ground surface, but standpipes up to 15 feet above the ground surface have been installed.

A standpipe can be round or square, of reinforced concrete, asbestos-cement, or corrugated galvanized metal. Its base is set in concrete a few inches below the depth of the line.

The following are the recommendations of the American Society of Agricultural Engineers, 1958, which should be followed:

"*Stand Requirements*

(a) All stands serve as vents in addition to their other functions.

(1) They will avoid entrainment of air.

(2) They will allow one to five feet of freeboard.

(3) If constructed of concrete pipe they will be constructed of 1500-D-Ultimate pipe as specified in the latest revision of A.S.T.M. designation C76, if of greater diameter than 24 inches.

(4) If cast in place, they will contain steel reinforcing on not more than one foot centers to provide steel areas equal to or greater than the least values specified for 1500-D-Ultimate pipe in the latest revision of A.S.T.M., designation C76.

(5) The height of all stands shall be at least four feet above the ground surface.

Pump Stands

(b) Pump stands will be:

(1) Concrete box stands with vertical sides.

(2) Non-tapered stands of concrete pipe.

(3) Non-tapered concrete pipe stands, capped and having a vent pipe of such height to take care of hydraulic gradients plus freeboard, or

(4) Steel cylinders mortared to a single piece of concrete pipe. The center line of the pump discharge pipe shall have a minimum vertical offset from the center line of the outlet pipe equal to the sum of the diameters of the inlet and outlet pipes. Weighted flap 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. Construction should be such as to insure that the vibration from the pump discharge pipe is not carried to the stand.

Velocities in Stands

(1) Downward velocities will not exceed one foot per second in concrete stands or two feet per second in steel stands. In no case will such velocity exceed the average pipeline velocity.

(2) If the size of the stand is decreased above the pump discharge pipe, the top 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 ten feet per second." (See table 6.)

To obtain discharge from a standpipe, holes are chiseled, hammered, or cut into the wall and a short metal pipe length is mortared or welded in just above the ground surface or at any other height desired (figure 25). Gates on downstream outlets are closed to back up water in the standpipe so that surface discharge can be obtained. Alternatively, the gates can be so regulated that simultaneously surface discharge and diversion into one or more downstream pipelines take place. When an overflow device is used within the standpipe, structure (*d*) of figure 15 should be used in combination with gates on downstream outlets. Structure (*c*) of figure 15 can be conveniently used for obtaining surface discharge from a stand-

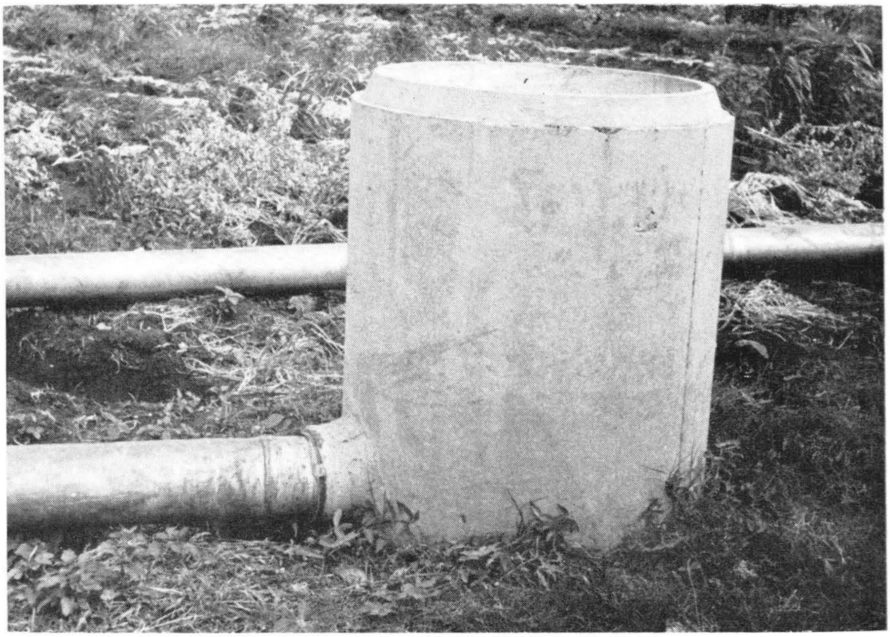


Fig. 25. Standpipes can be used to discharge water at the ground surface by chipping a hole in the side and mortaring in a short end of pipe to which a gated pipe can be coupled.

pipe without the use of a gate (assuming one outlet pipe), but involves danger of air entrainment (page 26).

Pressure gates are installed on the upstream (pressure) side, and sliding gates on the downstream side. Pressure gates are regulated via a rotating stem; sliding gates, by pushing the stem up and down (figure 12). Pressure gates are fastened by wires through the wall of the standpipe and are then mortared in place; sliding gates are merely mortared in. Several makes of these gates are available which are designed for the tongue-and-groove type of pipe. It should be attempted to design a system using sliding gates only.

DISCHARGE FROM CONCRETE PIPELINES

Discharge devices and their operation

For discharge from a pipeline to take place, water must be brought from the level of the underground pipe to just above the ground surface. As has been discussed in the preceding section, this can be done via standpipes where they happen to be installed, for the purposes described, at sites where discharge is required.

At other sites, riser pipes are inserted in the pipeline. These can be of two types:

(1) *Concrete risers* are formed by the right-angle arm of a tee-piece (figure 26). The horizontal arms of the tee-piece are of the same diameter

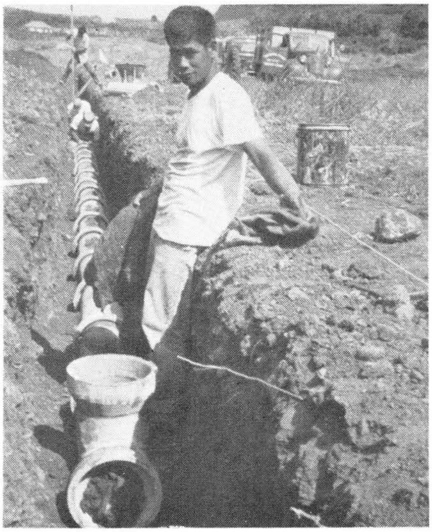


Fig. 26. Risers can be installed by inserting tee-sections in the line. Note burlap in end of pipe to prevent air circulation while the mortar is curing.

Fig. 27. A furrow valve installed on top of a bell at the end of a line. A coupling is welded on to either outlet to which surface pipe is attached.

as the pipeline, but the vertical arm can be varied from the same size as the line to smaller sizes as required.

Discharge from the risers is controlled by mortaring in valves near the top of the riser pipe. Alfalfa valves are commonly used for check irrigation, orchard and furrow valves for furrow irrigation. A furrow valve (figure 27) has one or two outlets to which a gated pipe, distributing water to the inlet of furrows, can be fastened (figure 28). Alfalfa and orchard valves are designed to discharge water at the ground surface but they can be adapted for use with surface pipe by placing over them a portable hydrant with one or two outlets for pipe attachment. In designing a system it should be attempted to discharge water at 1 foot head through these valves; approximately 0.5 foot head is allowed for head losses upon discharge and 0.5 foot allows water to pond to that height above the soil surface. Where this head is assured, a lightweight hydrant, open at the top, can be used (figure 29). However, on sites where 1 foot discharge head may be exceeded, as on steeper slopes, a closed hydrant of sturdier construction (Appendix) may be used. Yet, under any condition, it should be attempted to dissipate head in excess of 1 foot by throttling back the valves.

In doing so, discharge takes place through a small orifice with large head losses. In this way, any undesirable high head at the outlet site can be dissipated. The small orifice allowed causes the velocity upon discharge to be high because of the $q = av$ relationship. A low velocity is restored as the water enters into a wider pipe and when the flow is known, the ultimate attained velocity can be read from table 9, or can be calculated from $v = \frac{q}{a}$. Upon discharge at the ground surface, velocity should not exceed 1 foot per second. Where outflow takes place from a gated pipe situated on a steep slope, the high head will also necessitate the closing back of the gate to obtain any desired outflow (table 2). As the discharge will be proportional to the square root of the head (table 11), discharge will similarly take place at high velocities which should be dissipated either by an energy dissipator (figures 28 and 30, and Tribble, 1950), or by guiding the flow through a sleeve of a cross-sectional area larger than that of the gate opening. This subject is further discussed quantitatively in the next section.

A reduction in velocity and discharge at the ground surface can also be obtained by placing a wide pipe (often called "pot") over the valve as illustrated in figure 30. Several outlets within the walls of the pipe allow discharge to several furrows. Additional small surface flumes can be used to distribute water to adjacent furrows. These flumes are provided with upturned terminal lips to dissipate excess head. This system tends to become replaced by gated pipe, while gated rubber tubing has recently also been tried. This tubing is still under experimental tests.

(2) *Metal risers* can be used for:

(a) small outflows to adjacent furrows,

(b) diverting water from underground pipe for sprinkler irrigation, and

(c) where high heads at the discharge site should be dissipated.

These metal risers can be inserted into chipped holes in the pipeline and be mortared in. In doing so, attention should be paid to the possibility of

Fig. 28 (left). Gated pipe taking off from a furrow valve. Note erosion controller at outlet from gated pipe.

Fig. 29 (right). A lightweight hydrant, open at the top, is placed over alfalfa or orchard valves where low heads are assured. It allows surface discharge and conveyance to distant furrows by additional surface pipe. Outflow from the hydrant into either surface pipe is controlled by a sliding gate.

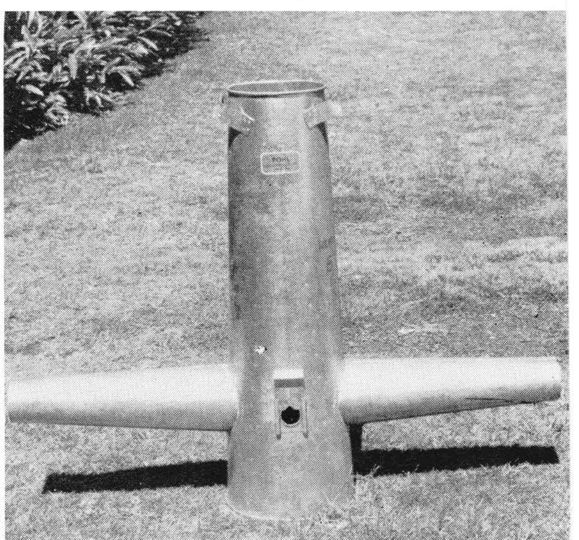


TABLE 9. Velocity in feet per second for various rates of flow in pipe of various sizes (corresponding friction losses are shown in table 4)

Flow:		Inside diameter of pipe in inches								
		6	8	10	12	14	16	18	20	24
g.p.m.	c.f.s.	Cross-sectional area in square feet								
		.1963	.3491	.5454	.7854	1.069	1.396	1.768	2.162	3.142
90	0.2	1.0	0.6	....	....	....	....	....	....	....
180	0.4	2.0	1.1	0.7	....	....	....	....	....	....
270	0.6	3.1	1.7	1.1	0.8	....	....	....	....	....
360	0.8	4.1	2.3	1.5	1.0	0.7	....	....	....	....
450	1.0	5.1	2.9	1.8	1.3	0.9	....	....	....	....
540	1.2	6.1	3.4	2.2	1.5	1.1	0.9	....	....	....
630	1.4	7.1	4.0	2.6	1.8	1.3	1.0	....	....	....
720	1.6	8.1	4.6	2.9	2.0	1.5	1.1	0.9	....	....
810	1.8	9.2	5.1	3.3	2.3	1.7	1.3	1.0	....	....
900	2.0	....	5.7	3.7	2.5	1.9	1.4	1.1	0.9	....
990	2.2	....	6.3	4.0	2.8	2.1	1.6	1.2	1.0	....
1080	2.4	....	6.9	4.4	3.1	2.2	1.7	1.4	1.1	....
1170	2.6	....	7.4	4.8	3.3	2.4	1.9	1.5	1.2	....
1260	2.8	....	8.0	5.1	3.6	2.6	2.0	1.6	1.3	....
1350	3.0	....	8.6	5.5	3.8	2.8	2.1	1.7	1.4	1.0
1440	3.2	....	9.2	5.9	4.1	3.0	2.3	1.8	1.5	1.0
1530	3.4	....	9.7	6.2	4.3	3.2	2.4	1.9	1.6	1.1
1610	3.6	....	10.3	6.6	4.6	3.4	2.6	2.0	1.6	1.1
1700	3.8	....	10.9	7.0	4.8	3.6	2.8	2.1	1.7	1.2
1790	4.0	....	......	7.3	5.1	3.7	2.9	2.3	1.8	1.3
2020	4.5	....	......	8.2	5.7	4.2	3.2	2.5	2.1	1.4
2240	5.0	....	......	9.2	6.4	4.7	3.6	2.8	2.3	1.6
2470	5.5	....	......	10.1	7.0	5.1	3.9	3.1	2.5	1.7
2690	6.0	....	......	11.0	7.6	5.6	4.3	3.4	2.7	1.9
2920	6.5	....	......	11.9	8.3	6.1	4.7	3.7	3.0	2.1
3140	7.0	....	......	......	8.9	6.5	5.0	4.0	3.2	2.2
3360	7.5	....	......	......	9.5	7.0	5.4	4.2	3.4	2.4
3590	8.0	....	......	......	10.2	7.5	5.7	4.5	3.7	2.5
3870	8.5	....	......	......	10.8	7.9	6.1	4.8	3.9	2.7
4040	9.0	....	......	......	11.5	8.4	6.4	5.1	4.1	2.9
4260	9.5	....	......	......	12.1	8.9	6.8	5.4	4.4	3.0
4480	10.0	....	......	......	12.7	9.4	7.2	5.7	4.6	3.2
4900	11.0	....	......	......	......	10.3	7.9	6.2	5.0	3.5
5380	12.0	....	......	......	......	11.2	8.6	6.8	5.5	3.8
5880	13.0	....	......	......	......	12.2	9.3	7.4	6.0	4.1
6280	14.0	....	......	......	......	13.1	10.0	7.9	6.4	4.5
6730	15.0	....	......	......	......	14.0	10.7	8.5	6.9	4.8

Computation based on: $v = \frac{q}{a}$, where v is velocity in feet per second; q is flow in cubic feet per second; and a is cross-sectional area of pipe in square feet.

corrosion of metal in contact with soil. One type of riser with built-in hydrant has recently been used at the University's Experimental Farm at Waimanalo (figures 31 and 32); other types to be used with or without hydrants or pots are shown in figure 33.

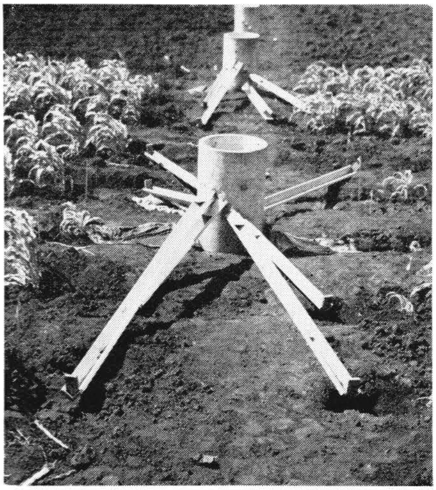


Fig. 30. A "pot" distributor can be placed over a riser and valve to guide water to adjacent furrows. Additional small aluminum flumes can be used. These flumes have an upturned end for energy dissipation to control erosion.

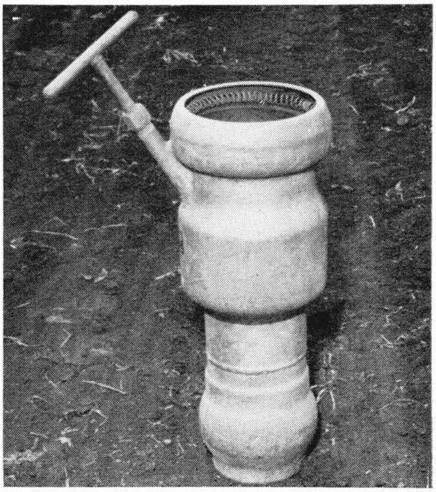


Fig. 31. A metal hydrant section could readily be adapted for use as a riser, valve, and adjustable outlet. The footing requires modification.

Fig. 32. Outflow from a metal riser is controlled by turning the valve handle. The elbow outlet can be adjusted to any desired direction. The hydrant can be provided with multiple outlets.

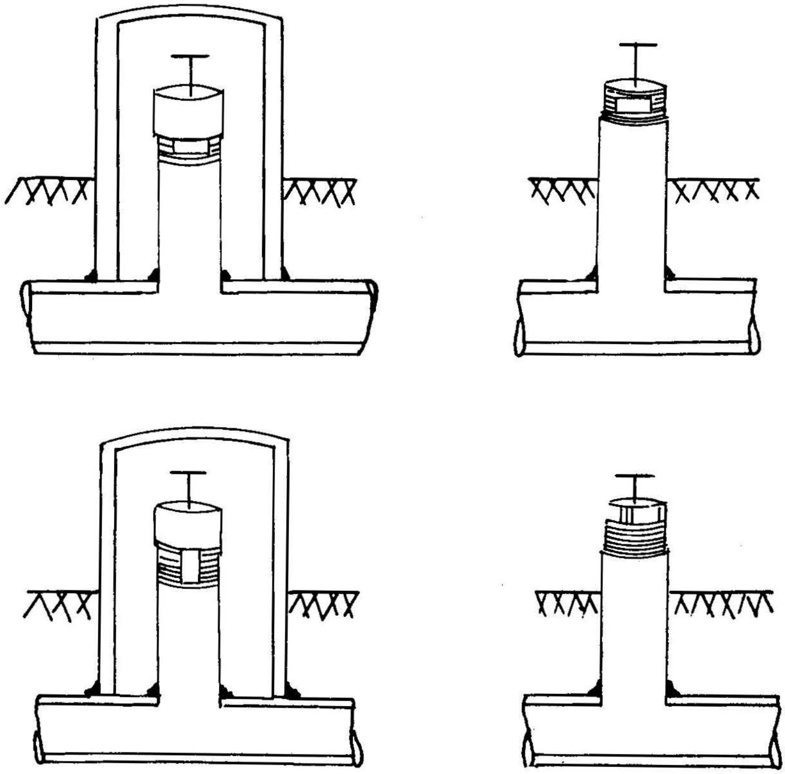


Fig. 33. Examples of metal risers with control valves for distributing water from underground pipe. The use of additional concrete risers is optional (after Lincoln, 1949).

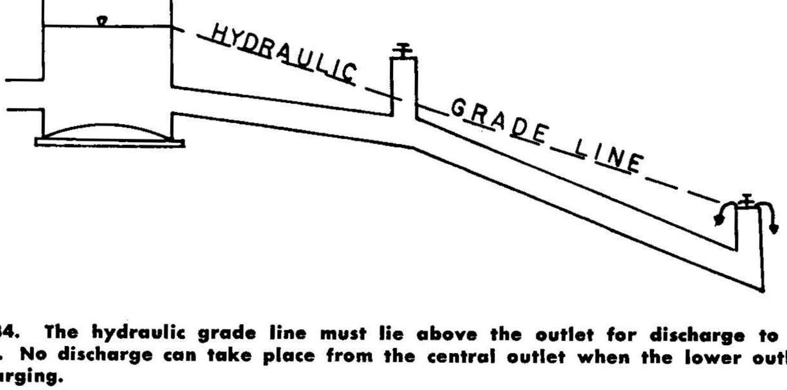


Fig. 34. The hydraulic grade line must lie above the outlet for discharge to take place. No discharge can take place from the central outlet when the lower outlet is discharging.

In designing systems attention should be given to the position of the hydraulic grade line. On a nonuniform grade a downstream outlet at low level may during operation so depress the hydraulic grade line that no water can simultaneously be obtained from a higher outlet. The hydraulic grade line then runs below the higher outlet; in other words, water does not rise to the height of that outlet (figure 34). This should be watched in particular where outlets are planned on elevated sites. On the other hand, where outlets occur in depressions the hydraulic grade line may lie so high that the head at the outlet is well in excess of a desirable 1 foot. This problem should be resolved in planning the system.

Capacity of discharge valves

It may be assumed that the major interest in Hawaii lies in guiding discharge from concrete pipelines by means of gated pipe to contour fur-

TABLE 10. Average measured head loss in alfalfa valves and risers (after Scott, 1950)

Flow	0.5 225	1.0 450	1.5 675	2.0 c.f.s. 900 g.p.m.
Valve size, inches		*Head loss in feet*		
8	0.05	0.2	0.49	0.85
10	0.02	0.07	0.17	0.33
12	0.01	0.03	0.08	0.16

TABLE 11. Calculated discharge from fully opened alfalfa and orchard valves at 1-foot discharge head, allowing 0.5-foot ponding of water above valve

Alfalfa valves			Orchard valves		
Inside diameter of riser, inches	c.f.s.	g.p.m.	Inside diameter of valve outlet, inches	c.f.s.	g.p.m.
6	0.8	348	3.5	0.23	101
8	1.4	618	5	0.46	207
10	2.2	966	6	0.66	296
12	3.1	1390	8	1.18	530
14	4.3	1890			
16	5.5	2470			
18	7.0	3130			
20	8.6	3860			

Calculated from: $q = ca\sqrt{2gh}$, where q = flow in c.f.s.; c = coefficient of discharge; a = cross-sectional area of orifice in square feet; g = gravitational constant, 32 feet per second per second; and h = head of water at outlet in feet.

row inlets. Costwise, the use of either alfalfa or orchard valves in combination with a lightweight hydrant with one or more pipe outlets, or the use of a furrow valve can be considered (see Appendix).

Measured head losses against discharge from alfalfa valves are shown in table 10. To extend the useful range of values, discharge has been calculated at 1-foot discharge head for various sizes of alfalfa and orchard valves which are given in table 11. In making the calculations a coefficient of discharge (c) of 0.7 and 0.6 for alfalfa and orchard valves, respectively, has been adopted, following the recommendations of Pillsbury (1952).

No experimental data are available on discharge against head losses for furrow valves. These valves lack a central valve web which obstructs

TABLE 12. Recommended coefficients (k) for use in $h_0 = kl \frac{v^2}{2g}$ to calculate head losses in pipeline sections (Texas Engineering Handbook, 1958)

Pipe material	Nominal size, inches	k	Pipe material	Nominal size, inches	k
Steel	5	0.0842	Asbestos-cement:		
welded	6	0.0654	100, 150	4	0.0801
	8	0.0461	100, 150	6	0.0467
	10	0.0339	200	6	0.0480
	12	0.0267	100, 150	8	0.0318
			200	8	0.0325
			200	10	0.0236
Steel	4	0.0811	100, 150, 200	12	0.0185
welded, coated	5	0.0597		14	0.0151
with coal	6	0.0466		16	0.0126
tar enamel	8	0.0326		18	0.01078
	10	0.0237		20	0.00938
	12	0.0186		24	0.00735
				30	0.00546
				36	0.00428
Aluminum or	2	0.2165			
galvanized iron	3	0.1226	Concrete,	6	0.0789
with flexible	4	0.0823	standard	8	0.0537
couplers	5	0.0611	irrigation	10	0.0399
	6	0.0478		12	0.0313
	8	0.0326		14	0.0255
	10	0.0243		16	0.0213
				18	0.0182
				20	0.0158
				24	0.0124
				30	0.0092
				36	0.0072

In above formula: h_0 = head loss in feet; l = length in feet; v = velocity in feet per second; and g = 32.2 feet per second per second.

TABLE 13. Recommended coefficients (k) for use in $h_0 = k\frac{v^2}{2g}$ to calculate minor losses in bends, entrances, and similar situations (Texas Engineering Handbook, 1958)

Nature of resistance	k
Square-edged entry	0.50
Projecting pipe at entry	1.00
90° large radius bends	0.25
90° short radius bends	0.50
Tees or crosses, straight flow	0.10
Tees or crosses, angle flow	1.50
Taper reducer (refer to smaller pipe)	0.25
Taper increaser (refer to smaller pipe)	0.15
Sudden contraction (refer to smaller pipe)	0.35
Sudden enlargement (refer to smaller pipe)	0.20

In above formula: h_0 = head loss in feet; v = velocity in feet per second; g = 32.2 feet per second per second.

flow in alfalfa valves, so that discharge at a given head through furrow valves should be larger. The few data available (Scott, 1950) indicate that discharge from furrow valves at a given head should not be more than 10 percent higher than from alfalfa valves.

Outflow from any discharge valve can be calculated by using the formula in table 11, if the head at the discharge site and the coefficient of discharge are known. To arrive at the head, deduct from the available elevation head, the head losses in the pipeline, read from table 4, the head losses in risers, computed from table 12, and the minor losses, derived from table 13. For most practical purposes a coefficient of discharge of 0.6 for square-edged orifices may be assumed (King, 1954, sections 3–5).

Where excessive head at the discharge site needs to be dissipated, the size of the riser and of the discharge device can be selected from the above data. This is illustrated by the following example.

Assume 25 g.p.m. are to be discharged into each of 30 furrows spaced at 5 feet interdistance. Therefore, 750 g.p.m. or 1.7 c.f.s. are required. This quantity is to be discharged at the end of an 8-inch concrete pipeline at a site which is at 11.5-foot lower elevation than a standpipe 300 feet away. During operation there is 3 feet of head in the standpipe above the center of the pipeline. This balances a lift of 3 feet at the discharge end, as the center of the pipe is 2 feet below the ground surface and water is to be discharged with 0.5-foot head and 6 inches above the ground surface. Gated pipe distributes water from the concrete pipe to the furrows on a 5 percent slope. The question is: What size riser and discharge device can be used and in what manner can water be discharged into the furrow inlet at a velocity of 1 foot per second?

The elevation head at the site of discharge is 11.5 feet. Friction losses for 1.7 c.f.s. in a 300-foot 8-inch concrete pipe (table 4) are $0.3 \times 17.8 =$ 5.3 feet. For minor losses at the pipe and riser inlet, apply $h_0 = k\frac{v^2}{2g}$

(table 13, k = 0.50), giving $2 \times 0.50 \times \frac{4.8^2}{64.4} = 0.4$ foot. The 4.8 feet per second velocity was read from table 9. The available head at the riser inlet is thus $11.5 - (5.3 + 0.4) = 5.8$ feet. A riser pipe and discharge mechanism can thus be selected which dissipates 5.8 feet head. Assume a 3-foot long metal riser pipe of 4 inches diameter. The head losses in this pipe are $h_0 = kl\frac{v^2}{2g}$ (table 12, k = 0.0811) $= 0.0811 \times 3 \times \frac{19.5^2}{64.4} = 1.4$ feet. The velocity in this pipe was calculated from $v = \frac{q}{a} = \frac{1.7}{0.0873}$. Thus, $5.8 - 1.4 = 4.4$ feet is available to dissipate head losses on discharge. To calculate the opening in the discharge device allowed at this head, apply $q = ca\sqrt{2gh}$ (table 11), and solve for a. From this, a cross-sectional area of 0.16 square foot is calculated which is close to that of a 6-inch pipe. This cross-sectional area will allow a discharge of 1.7 c.f.s. at 4.4 feet head.

Making use of generally available tables of friction losses in aluminum pipe, it is found that a 6-inch gated pipe is adequate. The head at the gates is large enough for 30 g.p.m. to be discharged while the gate is set at a small opening. Because of this small orifice, velocity on discharge from

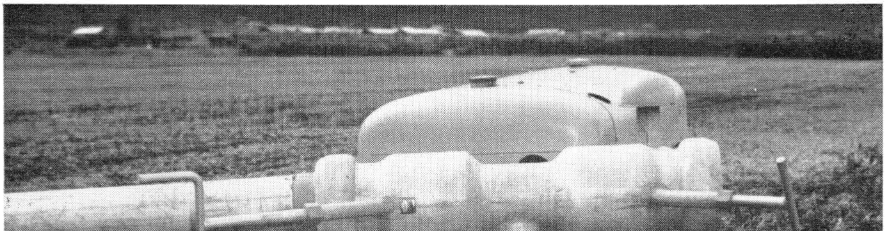


Fig. 35. Water for sprinkler irrigation can be pumped from a concrete pipe system as long as flow matches capacity of pump. To take vibration from pumping unit away from riser, a short length of rubber hose should be inserted between riser and pump.

the gate will be high, but this can be reduced to 1 foot per second by either allowing outflow into the furrow inlet via a 3.5-inch diameter sleeve, or by using the energy-dissipating device recommended by Tribble (1950). Quantitative control of discharge from gates under field conditions is still hard to achieve, even though this can be accomplished experimentally (Hansen, 1954).

SPRINKLER IRRIGATION WITH CONCRETE PIPELINES

Sprinkler irrigation using buried concrete pipe as supply line has been successfully practiced for a number of years at the University's Waimanalo Experimental Farm (figure 35). In this case water is carried under gravity through the pipe system and at the point of discharge a booster pump is installed. This contrasts with the situation where pressure lines (aluminum, steel, or asbestos-cement) are used as here water is conveyed under pressure to the sprinkling system.

The discharge from a concrete pipeline should match the capacity of the pump. Where standpipes with overflow structures are installed within the system this is difficult to achieve. There will be a tendency for the supply to fall below the pump capacity so that some air is sucked through the concrete pipe system. This is due to the fact that any excess of supply over discharge from the pump would cause overflow from the lowest standpipe. This difficulty is not encountered in systems using float-controlled valves for flow control. In that case the required flow would be delivered as long as the initial inflow into the system is adequate (page 29).

Provision should be made for the installation of a vacuum switch on the inlet of the pump so that the pump shuts off automatically when the water supply is cut off.

Priming the pump is readily achieved where the head at the concrete pipe outlet allows water to ascend within the pump, a situation encountered at the site of the illustration (figure 35). Otherwise, a self-priming pump should be used.

THE MEASUREMENT OF WATER INTAKE INTO PIPE SYSTEMS

Water measurement at the inlet of the pipe system may be required to control inflow or to allow charges to be made for supplied water.

In a system where ample head loss can be allowed, one may install weir boxes, but the measuring device has to be rather elaborate to reduce the velocity of approach (figure 36), or the introduced error will be large. Parshall flumes do not require the same provision for approach velocity and can be operated with less drop at the discharge end than weirs. However, as was pointed out on page 29, a measuring system requiring a drop cannot be used in conjunction with concrete pipelines with float-controlled valves, because of the necessity that water must be able to back up into the source of supply.

Fig. 36. Water is taken from a ditch (left) and discharged via a rectangular weir into buried concrete pipe. This weir requires an elaborate approach channel, as shown.

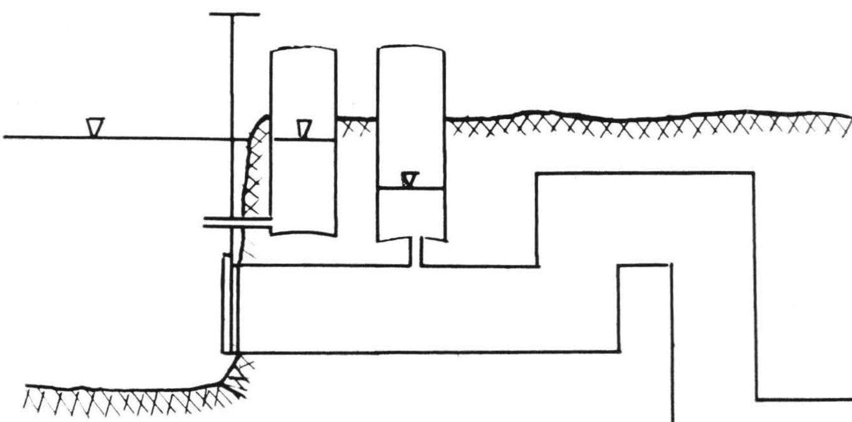


Fig. 37. A metergate for controlling and measuring outflow from a reservoir or ditch into a pipeline. The upward section on the right assures submergence.

One solution is to use a commercial metergate which is supplied in combination with a sliding gate to control inflow (figure 37). The water level in the source of supply is measured in one vertical water level recorder, while an adjacent recorder measures the water level in the initial section of the pipe. The device has been calibrated and discharge can be read from tables.

The outlet section of this device is turned up to insure submergence. To connect the initial pipe section to an underground pipeline, three additional 90-degree elbows are required as shown in figure 37. Under most circumstances, it will be necessary to prevent or reduce trash intake by providing grids and screens in front of the sliding gate.

Another commercial device is based on a propeller principle. It can be used at the inlet or anywhere in the center of a line. The flow is read from a calibrated counter.

THE INSTALLATION OF CONCRETE PIPE SYSTEMS

The significance of the proper installation of concrete pipe systems cannot be overstressed. Undoubtedly, some disappointing experiences in the past have been due to inadequate or faulty installation. Proper grade control and good bed preparation can do much to prevent leakage and breakage of installed pipelines at a later date.

The following recommendations have, in part, been based on those made by the Soil Conservation Service for U.S. mainland conditions (Texas Engineering Handbook, 1958). Those recommendations have been modified and extended to suit local conditions.

Depth of pipeline

The recommendations for U.S. mainland conditions read as follows: "Pipelines shall be a minimum of two feet below the ground surface in loose porous soils and three feet in heavy soils subject to considerable volume change on wetting and drying. Shallower coverings may be specified for rocky areas or other local conditions. Where shallower coverings are specified provisions must be made to protect the line from vehicular traffic and farming operations." (Texas Engineering Handbook, 1958). Most concrete pipelines at the University's Waimanalo Experimental Farm have been installed with 2 feet of soil above the pipe. No evidence has been collected that normal farming operations and the use of trucks and tractors cause damage to the lines. It is believed that this depth of installation will be satisfactory for most Hawaiian conditions, except in situations where deep tillage is practiced.

Trenching and bedding

(1) Before trenching, the tract of land at the pipeline location should be leveled and provided with alignment guides (figure 38). Grade control at the land surface means good grade control in the trench bottom. The prior elimination of high and low spots at the surface will prevent their reflection in the trench bottom. After trenching, the uniformity of the grade should be checked by a trench level (figure 39) and be adjusted by

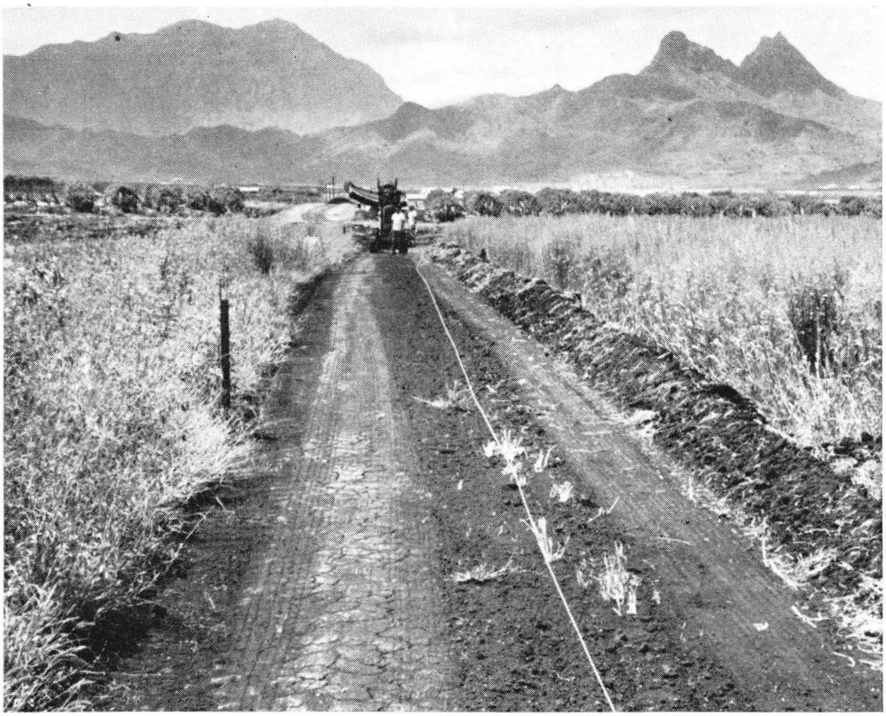


Fig. 38. After insertion of survey stakes (left center), the future site of a concrete pipeline is smoothed by tractor with bulldozer blade. String guides trench digger (in distance).

hand where necessary (figure 40). A shovel used for this purpose should have a curvature similar to that of the pipe, so that the bed can be shaped to fit the pipe snugly.

(2) The trench should not be less than 12 inches nor more than 18 inches greater than the external diameter of the pipe, and should be deep enough to allow a minimum cover of 18 inches over the pipe. If the trench is too narrow, there is insufficient room to make the joints properly. If the trench is too wide, the bed stability is weakened. Where the trench digger is too narrow, a wider trench can be made with two passes.

(3) Foundations for the pipe should be firm, but should yield sufficiently to provide a uniform bearing for the bottom half of the pipe throughout the length of the line. When an unstable foundation cannot be avoided, it should be consolidated or removed and replaced by sand, gravel, or other suitable material to avoid uneven settlement and leaks. On a rocky trench bottom an even load distribution may be hard to obtain, and an equalizing bed of sand, gravel, or suitable earth free of rock may have to be provided. This embedment should have a thickness directly under the center of the pipeline ranging from 2 inches for 6-inch pipe to 4 inches for 24-inch pipe.

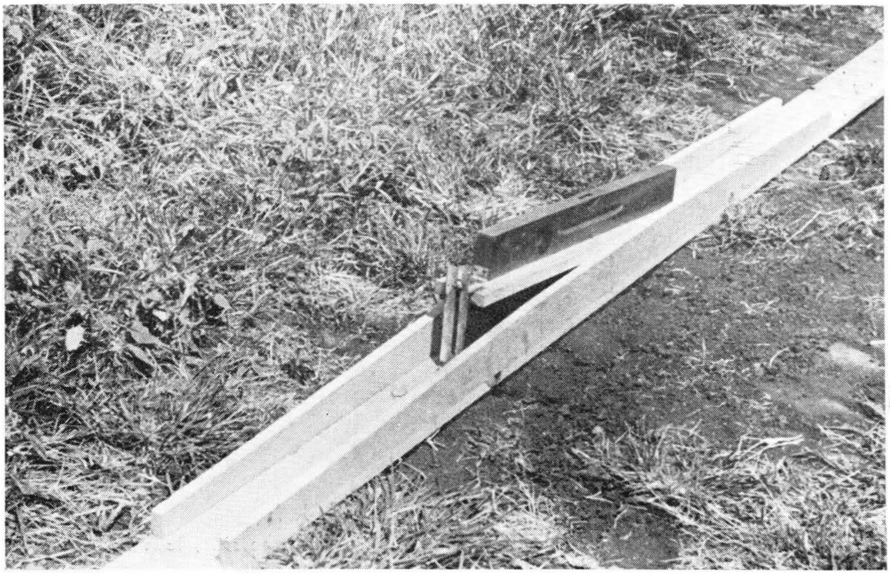


Fig. 39. A trench level is made up by fastening a carpenter's level on a small board which, while being hinged, can be set at variable angles. The angle is given on the sliding frame and is taken from the survey map for a particular tract. The level is pulled through the trench by a string, and the spots are marked which require grade adjustment.

Fig. 40. After trench digging and before checking on the final grade by the trench level, the bed is cleaned by a shovel with a curvature similar to that of the pipe to assure a snug fit of the pipe in the prepared bed.

Fig. 41. On preparing the trench, the earth is dumped on one side, leaving the other side clear for handling the pipe.

(4) On installing the line, water must be removed from the trench so that joints can be properly made and inspected.

(5) All excavated material should be placed on one side of the trench, and the pipe delivered on the other (figure 41).

(6) In a few instances "cradles" have been used in Hawaii in attempts to give added stability to concrete pipelines. These cradles are concrete blocks shaped to fit in the trench bottom. They are rounded at the top to accommodate the pipe and are placed at one or two points below each pipe.

It is believed that under most conditions in Hawaii the use of this type of cradle is unwarranted. Firstly, most Hawaiian soils are more than usually stable, by virtue of which they should make good pipe beds. Secondly, in order to make cradles effective, the cradles themselves should be well anchored on concrete piles or substantial blocks of masonry and be continuous all along the pipe length. Cradles are used on the U.S. mainland under conditions where no other means is available for providing a stable pipe bed, but only then where the cradles themselves are well founded, necessarily involving considerable cost. It is believed that cradles loosely fitting into the trench bottom and supporting a pipe at one or two points only are likely to provide reduced rather than increased pipe support.

Laying and jointing the pipe

(1) In order to provide maximum bed stability and an opportunity for a firmly tamped initial pipe cover, it is generally recommended to lay the pipe in moist soil. Therefore, the pipes should be laid shortly after rain or, where necessary, the soil should be irrigated at the site of the pipeline a few days in advance of the laying.

Fig. 42. As the pipe is lowered into the trench by a crane and tongs, the jointing ends are cleaned and wetted by brush.

(2) A small crane and tongs should be available to handle and lower the pipe in the trench bottom (figures 42 and 50).

(3) The installation should start from low points in the line so that the weight of the pipes will tend to strengthen the joints.

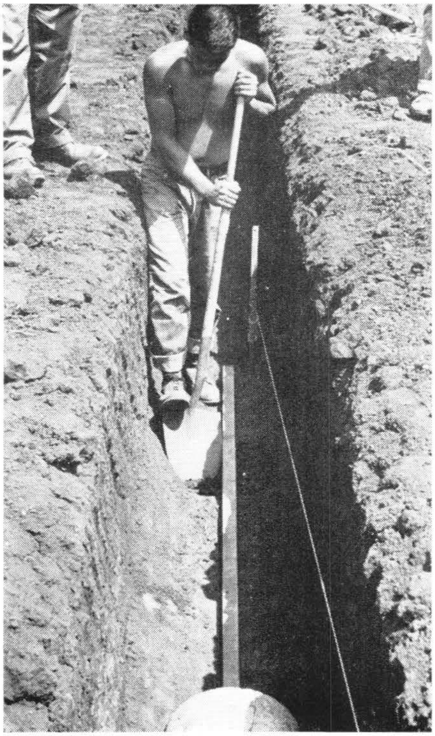
(4) The bell end of the bell-and-spigot pipe should face uphill and toward the layer, but in the case of the tongue-and-groove pipe either the tongue or the groove end is placed in that position. However, an upstream position of the groove end is recommended.

(5) The terminal pipe should be anchored by angle iron stakes and mortar.

(6) Where abrupt changes in grade are unavoidable, the two pipes adjoining the change in grade should similarly be stabilized by anchor blocks.

(7) Pipes adjoining a sudden change in horizontal direction should be anchored in the same manner.

(8) The ends of the pipe to be joined should be cleaned with a wet brush (figure 42). (Note: Illustrations show bell-and-spigot type of pipe, but the principles shown also apply to tongue-and-groove pipe. The bell corresponds to the groove end and the spigot to the tongue end of the pipe. See figures 7 and 8).



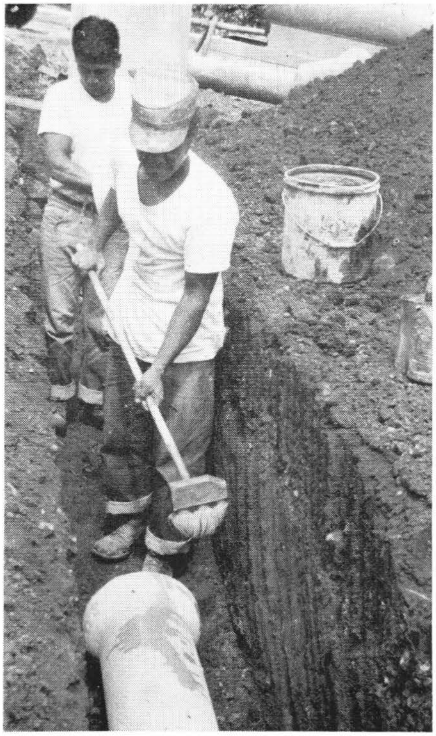


Fig. 43 (left). A mark on the stick lying in the trench bottom indicates the site of the next joint. An excavation is made here which is filled with mortar. On placing the pipe, good contact should be established between the groove or bell end and the mortar in the excavation as this will become the bottom section of the mortar band surrounding the joint. (See fig. 7.)

Fig. 44 (right). After a pipe has been jointed to the previous one, a swab with long handle removes any mortar squeezed out of the joint to the inside of the pipe.

(9) A shallow excavation is made in the trench bottom at the site where the next joint is to be accommodated (figure 43), this excavation to be filled with mortar (figure 7).

(10) When placing the pipe in position, mortar is applied to the pipe end which, when laid, should be in good contact with the mortar in the prepared excavation (figure 7). (It is here that leaks tend to occur when the pipe is laid by inexperienced men.)

(11) The tongue or spigot end of the pipe is to be firmly pressed into the end of the pipe already in position until the mortar is squeezed out from the inner and the outer surfaces.

(12) The interior of the pipe should be clean and smooth at its completion, free from excess mortar and foreign materials. If the pipe size permits, the inner side of the joint should be inspected to assure that the joint is filled with mortar before the outer band is placed.

To clean the inside joints of small-diameter pipe, place a large swab of burlap or other material inside the first pipe. A handle or chain

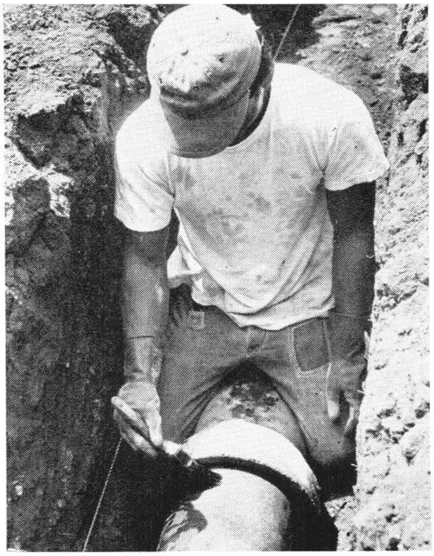


Fig. 45. Before banding two pipes together, the concrete is wetted by brush. Mortar is then worked in and smoothed off by hand to provide a solid band around the joint. Note use of rubber gloves.

Fig. 46. Moist soil is packed around the pipe and the joint immediately after the mortaring.

attached to the swab allows it to be pulled past the joint to wipe off surplus mortar (figure 44).

(13) After wetting with a brush, the outside of the joint is surrounded by a mortar band (figure 45). The band should not be less than ½ inch thick and 3 inches wide. The banding is carried out immediately behind the laying operation.

Mortar

(1) The mortar for joints and bands should, unless otherwise specified, consist of one part of Portland cement to two parts of fine, clean sand. For this purpose "nonshrinking" mortar has been successfully used.

(2) The joint mortar should be of such consistency that it will adhere readily to the pipe and squeeze easily out of the joints.

(3) The band mortar should be plastic and of such consistency that the band will not sag.

(4) All mortar should be used within 30 minutes after mixing with water, or sooner where "nonshrinking" mortar is used.

(5) "Nonshrinking" mortar has proven particularly useful for repair work on pipelines.

(6) Where alkali or sulfates are present in the soil, a sulfate-resistant mortar should be used.

Curing and backfilling

The completed joints should be immediately protected from air and sun with an initial covering of moist earth, sand, or burlap. To prevent air

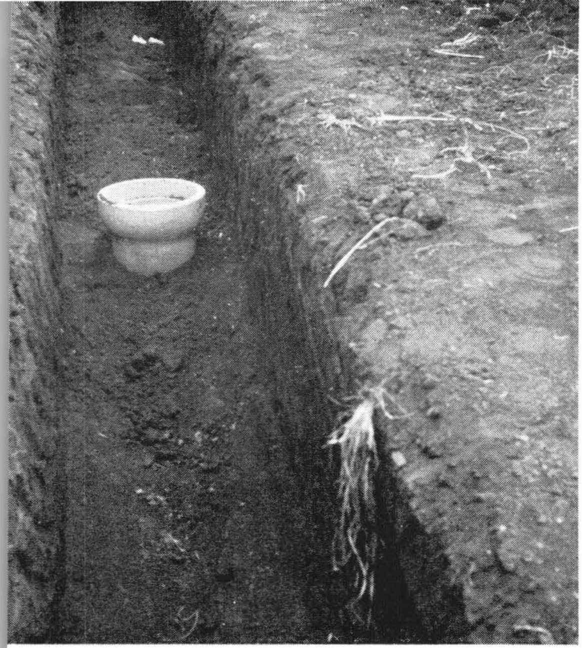


Fig. 47. The initial backfill is done by shovel and is packed well around the pipe by tamping.

Fig. 48. The final backfilling is done by a tractor with bulldozer blade. Note that the tractor moves across the trench.

circulation within the pipe, the ends of the pipe should be kept sealed as much as possible (figure 26).

Initial backfilling is done by hand with moist earth or sand. It should not be more than a few pipes behind the one which is being laid. The initial fill is about 6 inches thick and is carefully tamped (figures 46 and 47). This operation is just as important as initial bed preparation for ultimate bed stability.

Final backfilling is done by tractor (figure 48) and should be completed within 24 hours after laying.

Fig. 49. Before installing standpipes, holes for pipe inlets are chipped out.

Fig. 50. The standpipe is now ready for installation. The margin below the pipe inlets allows the standpipe to be set in a concrete footing.

Standpipe construction

Before installing a prefabricated standpipe, holes for pipe inlets are chipped into the side walls (figures 49 and 50). There must be sufficient margin below these holes to allow the construction of a concrete footing. The concrete floor prepared after the standpipe is in place should be at least 4 inches thick, and around the outside base of the standpipe a thick mortar band should be applied. If the standpipe is to act as a sand trap, the margin below the inlets should be greater according to the quantity of sand which is expected to be trapped. Where required, a second pipe section can be placed and mortared in on top of the base section.

Trash interception

Trash needs to be intercepted before it enters a pipe system. Grids and screens (in series where required) protecting the pipe inlet may be effective as long as they are kept clean. Some devices developed for open sys-

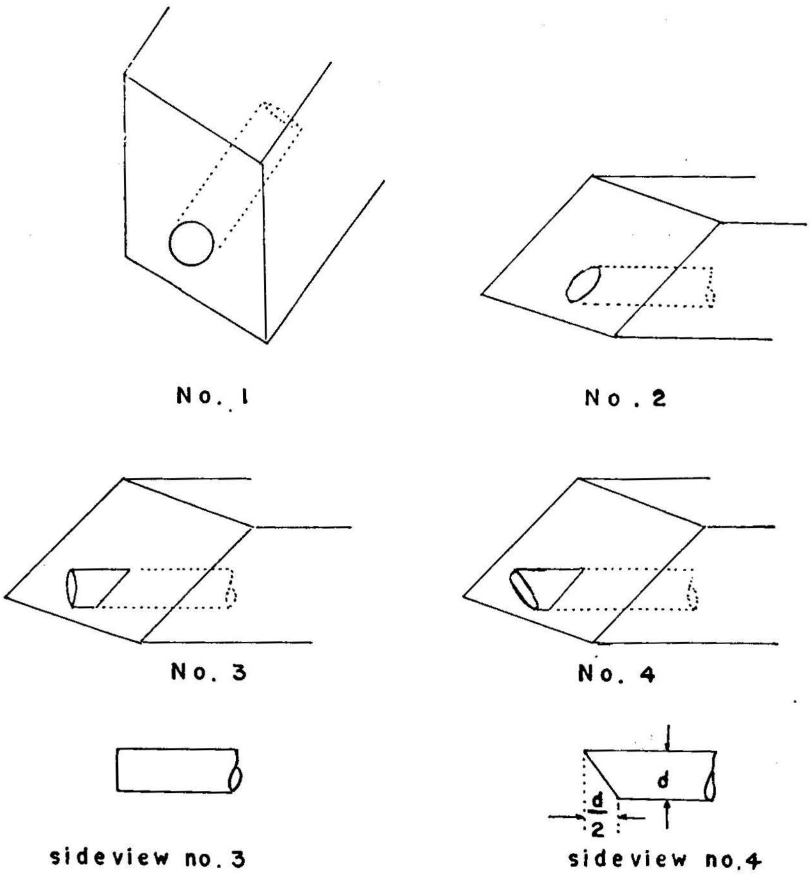


Fig. 51. Pipes with inlets 1, 2, and 3 were found not to flow full. Full pipe flow was obtained by using inlet 4 (see text).

tems (Texas Engineering Handbook, 1958) could probably be adapted for closed systems.

Pipe inlets

Experimental work on inlets of pipe culverts under roadways (Karr and Clayton, 1954) has drawn attention to the shape of a pipe inlet required to insure full flow in the pipe. Inlets 1, 2, and 3 of figure 51 were found to allow air to be drawn in (by vortex formation) from the surface of the water in the reservoir or ditch supplying the pipeline, unless (as was shown for inlet 3) a head of more than 1⅛ the pipe diameter was available above the top of the inlet or unless the inlet was artificially primed. This behavior was ascribed to air filling the space created by contraction at the inlet which caused the pipes not to flow full. Contraction at the upper side of the inlet was prevented by using inlet 4 of figure 51. This inlet caused pipes to flow full, but the flow was slightly smaller than where inlet 3 was used after this had been artificially primed. The greater flow recorded by using inlet 3 under these conditions was ascribed to entrance losses of inlet 3 being lower than those of inlet 4. It is believed that there is sufficient justification to apply these results to inlets of concrete irrigation pipe systems, and inlets of shape 4 are therefore recommended.

CAUSES OF FAILURE OF CONCRETE PIPE SYSTEMS

One common source of trouble, that of large variations in soil temperature, is of no great concern in Hawaii. A second common source of trouble may, however, apply—that of the effect of alternating wetting and drying of the pipe. It is, therefore, recommended to attempt to keep the pipeline as full as possible between irrigations. Where the lines are nonleaking, this is in part achieved by overflow pipes in standpipes and by the use of float valves.

Lack of proper bed preparation has been a major source of trouble. The best bed preparation is achieved by laying the pipe in moist soil and backfilling with moist soil. By doing so, and by proper tamping of the soil around the pipe, the greatest bed stability is assured.

In an early system of the University's Experimental Farm at Waimanalo, difficulties were encountered in that a number of pipes cracked (figure 52). It is probable that this early pipe, obtained from an unconventional source, was not up to the standard which is now set for concrete pipe and, furthermore, that the pipe was not properly installed. At the time one cradle for each pipe was fitted loosely into the trench bottom without further support. Pipes of satisfactory quality recently installed without cradles but in a properly prepared bed in the same soil have not shown signs of failure.

As mentioned, surging and water hammer have been related to air entrainment. The use of overflow structures involves the risk of air entrainment, but the trouble seems to have been mainly observed in pipes of large diameters, 18 inches and up, and in long lines where surging is accentuated via a number of standpipes (page 25).

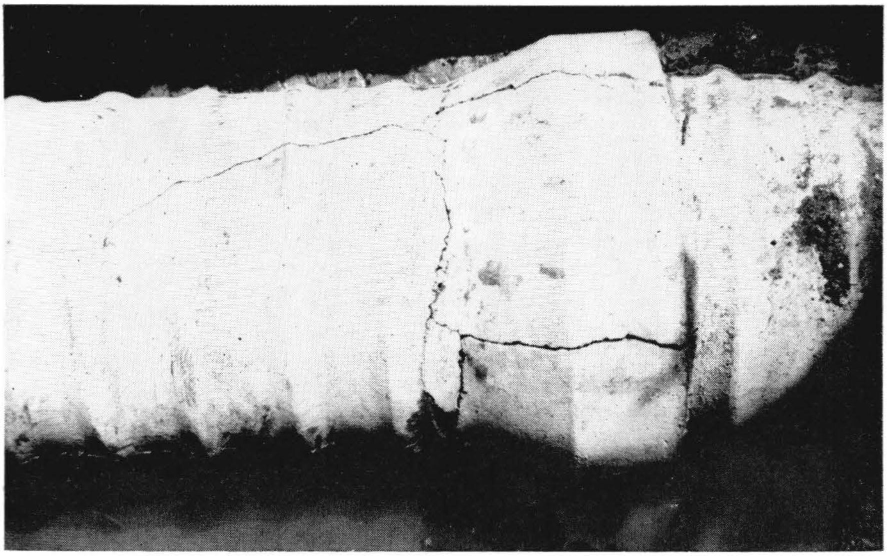


Fig. 52. Longitudinal and circumferential cracking occurred in an early concrete pipeline in Hawaii. The trouble was probably partly due to poor quality of the pipe and inadequate bed preparation.

Water hammer and, therefore, instantaneous high pressure in the line can be created by a sudden discharge into a line, or by a sudden shut-off of valves. Slowly-closing gates and valves are recommended for use with concrete pipe systems.

Newly-installed pipes become more impervious with time, presumably because of a filling up of pores by dirt. Initial "sweating" of pipe may disappear after a few months' use.

Testing of pipelines can be done by either following the drop in water level in a standpipe above a full pipe or by inflow and outflow tests. Leakages can often be detected from wet spots at the soil surface.

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APPENDIX

Estimated Cost of Concrete Irrigation Pipe and Accessories

(1) Approximate cost of material

Nonreinforced concrete pipe		*Reinforced concrete pipe*		*Tees*		*90-degree elbows*	
INCHES	PER FOOT	INCHES	PER FOOT	INCHES	EACH	INCHES	EACH
4	$0.55	12	$1.20	4	$ 3.20	4	$2.20
6	0.65	15	2.60	6	4.65	6	2.85
8	0.70	18	3.00	8	5.65	8	3.55
10	1.00	21	3.70	10	7.30	10	4.55
12	1.35	24	4.70	12	10.30	12	6.40
18	2.70						
24	4.30						

(NOTE: inches represent inside diameter of pipe)

Float valves		*Orchard valves*		*Alfalfa valves*		*Furrow valves*	
INCHES	EACH	INCHES	EACH	INCHES	EACH	INCHES	EACH
4	$ 30	6	$2.75	8	$10	6	$30
5	40	8	3.50	10	12	8	35
8	54	10	5.20	12	15	10	40
12	90	12	6.50				
16	120						

(NOTE: inches denote pipe size on which they fit)

Hydrant—heavy type			*Hydrant—lightweight, low-head type*		
INCHES FOOTING	SINGLE OUTLET EACH	DOUBLE OUTLET EACH	INCHES FOOTING	SINGLE OUTLET EACH	DOUBLE OUTLET EACH
8	$61	$68	8	$13	$19
10	72	82	10	16	21
12	80	90	12	19	23

Sliding gates		*Pressure gates*		*Line gates*	
INCHES	EACH	INCHES	EACH	INCHES	EACH
8	$22	10	$40	10	$ 90
10	26	12	50	12	105
12	33	14	70	14	130

(NOTE: inches denote pipe size on which they fit)

(2) Estimated cost of installation

Up to 12-inch pipe, *trench digger* $25 per hour; estimated length of digging 100 to 300 feet per hour, depending on soil and actual size of trench.

Small crane for lowering pipe into trench estimated at $15 per hour.

Estimated *length of pipe laid per hour:*

Pipe size	*Length of pipe section*	*Number of men*	*Length of line laid per hour*
INCHES	FEET		FEET
4–8	8	3	125
10–12	6	3	100
14–18	6	4	50
24	6	5–7	50

Backfilling by tractor: 50 feet per hour, up to 12-inch trench.

Installation of standpipe of 24 or 36 inches diameter: 2 men 4 hours, plus ½ hour crane.

Gate, overflow, and valve installation: 2 men 3 hours per unit.

Estimated *length of pipeline jointed and banded per bag of cement,* for sections of 6 or 8 feet, as mentioned above:

Pipe size	*Length pipeline*	*Pipe size*	*Length pipeline*
INCHES	FEET	INCHES	FEET
6	176	12	84
8	138	24	36
10	102	36	24

ACKNOWLEDGMENTS

Grateful acknowledgment is made (1) to Robert B. Campbell and A. C. Trouse of the Experiment Station, Hawaiian Sugar Planters' Association, Honolulu, for making some experimental data available for inclusion in this publication; (2) to Ronald I. Blewitt of the U.S. Soil Conservation Service, Honolulu, for helpful comments and information; and (3) to the Honolulu Construction & Draying Company, Ltd., for a grant in support of this publication.

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