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SMALL SCALE WASTE MANAGEMENT PROJECT

Design of Pressure Distribution Networks for Septic Tank-Soil Absorption Systems

by

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DESIGN OF PRESSURE DISTRIBUTION NETWORKS

FOR SEPTIC TANK-SOIL ABSORPTION SYSTEMS

By Richard J. Otis[1]

INTRODUCTION

Excessive costs of providing conventional public wastewater facilities, limitations on funding for local sewer extension projects and the increased demand for rural building sites, are forcing engineers and planners to reconsider the septic tank and other onsite systems as permanent rather than interim solutions to wastewater treatment and disposal problems in unsewered areas. This is being done with some reluctance because the septic tank system has had a reputation of poor reliability. Yet the system's low cost and simplicity of operation cannot be overlooked. With proper siting, design and construction, the system can serve single family homes, clusters of homes and small communities reliably.

Recent research in site evaluation, design, and construction techniques have improved the performance and will undoubtedly prolong the life of the septic tank-soil absorption field system. A part of this research has demonstrated that the method used to distribute the septic tank effluent within the soil absorption field can effect the system's performance (University of Wisconsin, 1978). Three generic methods have been identified (Otis et al., 1977). The simplist and most commonly method is gravity flow. With this method, wastewater is allowed to flow by gravity through large diameter pipes into the absorption field as wastewater is discharged from the septic tank. Distribution is usually localized in a few areas within the absorption field, which results in overloading of the infiltrative surface in these areas

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(Converse, 1974). This can lead to ground water contamination in coarse granular soils because of insufficient treatment (McCoy & Ziebell, 1975; Green, 1976) or more rapid and severe clogging in finer textured soils (Bouma et al., 1974; Bouma et al., 1975). The second method, dosing, overcomes some of the problems associated with gravity flow. This method requires a means of storing effluent following the septic tank for periodic discharge into the soil absorption field through large diameter pipe by a pump, siphon or other device. In this manner, the effluent is distributed over a larger portion of the absorption area and the period between doses allows the infiltrative surface to drain. Soil clogging does not seem to be as severe using this method (Bendixen et al., 1950; University of Wisconsin, 1978; Winneberger et al., 1960; Bouma et al., 1974). However, localized overloading still occurs (Converse, 1974). The third method, uniform application, solves this problem by applying effluent uniformly over the entire absorption area at a rate below the saturated hydraulic conductivity of the soil. This insures adequate treatment by the soil at all times and seems to reduce clogging (University of Wisconsin, 1978). However, achieving uniform application is difficult and can be costly. Therefore, uniform application is recommended as a distribution method only where the other methods are not acceptable (See Table 1).

Pressure distribution networks have been used as one means of approaching uniform application of septic tank effluent. These consist of a dosing chamber containing a pressurization unit, either a pump or siphon, where effluent is collected and periodically discharged into a network of small diameter perforated pipes designed to discharge equal amounts of effluent from each perforation. These networks have the advantages over other networks of providing dosing in addition to more uniform application, permitting irregular field configurations, providing equal division of flow between multiple trenches

TABLE 1

SUGGESTED METHODS OF EFFLUENT DISTRIBUTION FOR VARIOUS SYSTEM DESIGNS AND SOIL PERMEABILITIES[a]

Soil Permeability (Percolation Rate)	Trenches or Beds on Level Site	Trenches on Sloping Sites (>5%)
Very Rapid[b] <1 min/in (<0.04 cm/sec)	Uniform application[c] Dosing Gravity	Uniform application Gravity Dosing
Rapid 1-10 min/in (4-0.4 cm/sec x 10^{-2})	Uniform application Dosing Gravity	Gravity Uniform application Dosing
Moderate 11-60 min/in (4-0.7 cm/sec x 10^{-3})	Dosing Gravity Uniform application[d]	Gravity Uniform application Dosing
Slow 60 min/in (>0.7 x 10^{-3} cm/sec)	Not Critical Uniform application[d]	Not Critical

[a] Methods of application are listed in order of preference.

[b] Conventional soil absorption systems not recommended for these soils.

[c] Should be used exclusively in alternating field systems to ensure adequate treatment.

[d] Preferred method for large flows.

on level or sloping sites, and simultaneous application of effluent over large absorption areas. Originally developed for mound systems at the University of Wisconsin (Converse, 1974), their range of applications have increased to overcome problems with rapidly permeable soils, shallow water tables, bedrock or other restrictive horizons, steep slopes and large flows (Triangle J Council of Governments, 1979; State of Washington, 1980; State of Wisconsin, 1980; Carlile, 1980; Otis, 1978; U.S. EPA, 1980). Although their popularity is increasing, their use generally has been limited to small conventional systems because of the lengthy and tedious nature of their design. To realize the full potential of pressure networks, simplified design procedures are needed. The objectives of this paper are to review the principles of pressure distribution network design, discuss considerations that should be part of a good network design and to present a simplified design procedure. In addition, the design procedure is illustrated using three examples in Appendix I.

DESIGN PRINCIPLES

Pressure distribution networks usually consist of a solid pipe manifold that supplies effluent to a number of evenly spaced perforated laterals. To obtain uniform distribution, the amount of liquid discharged from each perforation within the network must be equal. The rate of effluent discharge from any perforation is a function of the area and characteristics of the opening and the liquid pressure behind it. This relationship can be described using the orifice equation:

$$q_j = Ca(2gh_j)^{1/2} \quad \text{.........................(1)}$$

in which q_j is the perforation discharge rate from the perforation in the j^{th} lateral segment, a is the area of the opening, g is the acceleration due to gravity, h_j is the liquid pressure behind the perforation, and C is a dimensionless coefficient that varies with the characteristics of the opening (Figure 1).

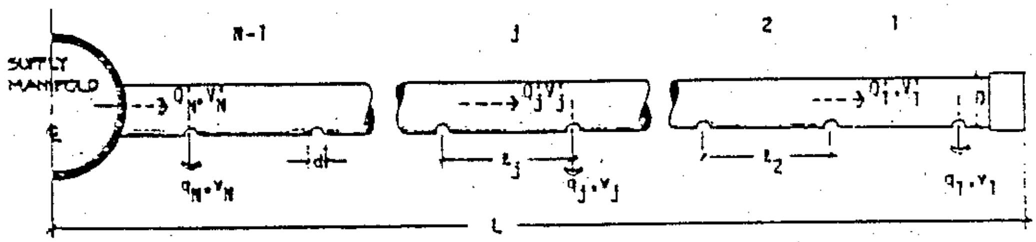


Figure 1: Flow Relationships within Typical Lateral

Energy losses incurred during delivery of effluent through each perforation alter the liquid pressure, h_j, at each perforation. Losses include pipe friction losses in the manifold and laterals, junction losses at each perforation, and entrance losses into each perforation. To maintain equal discharge rates from each perforation, the total of all losses incurred delivering effluent to each perforation must be equal.

Pipe friction losses are a function of the rate of flow and the pipe diameter. These can be calculated using the Hazen-Williams equation shown rearranged below:

$$\Delta h_j = 4.71\ \ell_j \left[\frac{Q'_j}{C_h\ D^{2.63}}\right]^{1.85} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ (2)$$

where Δh_j is the friction loss in lateral segment j, ℓ_j the length of the j^{th} lateral segment (or perforation spacing), Q'_j the sum of the discharge rates from the perforations downstream from the j^{th} lateral segment, D the lateral diameter, and C_h the Hazen-Williams friction factor. The friction losses in each segment add downstream in the lateral but at a decreasing rate as Q'_j decreases due to losses in flow through each perforation.

The discharge of effluent out each perforation reduces the total energy downstream in the lateral. The loss of energy at the junction causes a change in the liquid pressure within the lateral. The magnitude of change can be determined by equating the total energy upstream from the perforation to the energy in the liquid discharged through the perforation and the energy remaining in the lateral downstream of the perforation. The total energy in each liquid stream is the sum of its velocity head, pressure and elevation in relation to a fixed datum. Using this relationship and rearranging the change in pressure at the junction is:

$$\Delta h'_j = \frac{V'^2_j - V^2_{j-1}}{2g} - \frac{v^2_j}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

in which $\Delta h'_j$ is the change in pressure across the j^{th} perforation, V'_j the velocity in the lateral segment upstream from the perforation, V'_{j-1} the velocity in the lateral segment downstream from the perforation, and v_j the velocity through the perforation. Where V'_j is high near the inlet to the lateral, the change in pressure is positive but $\Delta h'_j$ becomes negative toward the distal end of the lateral. Thus, the in-line pressure increases slightly before decreasing downstream within the lateral.

The entrance loss into the perforation is a function of the relative velocities and directions of flow in the lateral and out the perforation. For sharp-edged circular perforations with axes perpendicular to the direction of flow in the lateral, the entrance loss, h'_e can be expressed by:

$$h'_e = \left[\frac{\phi V'^2_j}{v^2_j} + \theta \right] \frac{v_j^2}{2g} \quad \text{.......} \quad (4)$$

where ϕ and θ are dimensionless coefficients (Hudson et al., 1979). This shows that the entrance loss is the greatest in the perforation nearest the lateral inlet and the loss in each subsequent perforation decreases as V'_j decreases.

To properly design a distribution lateral, the total losses to each perforation must be balanced. This is a long reiterative process because the equations used to determine these losses are interdependent. The design can be greatly simplified if some of the losses can be ignored. An examination of equations 2, 3 and 4 shows that each of the losses are minimized when the ratio of the lateral diameter to the perforation diameter is large. At low lateral velocities, the entrance losses are nearly identical along the lateral so that they can be ignored. The junction losses are within construction elevation tolerances at low lateral velocities and pressures. Fortunately, this condition can be satisfied by minimizing friction losses. Therefore, the design is based upon minimizing the friction losses and ignoring the other losses.

Any tolerance of variation of discharge rates between the first and last perforations in the lateral can be selected. In this paper, a 10 percent variation of discharge rates between perforations within the same lateral is allowed:

$$\frac{q_N}{q_1} = 1.1 \quad \text{.......} \quad (5)$$

Combining equations 1 and 5 and ignoring other losses:

$$\frac{q_N}{q_1} = \left[\frac{h_d + \sum_{j=1}^{N} \Delta h_j}{h_d} \right]^{1/2} = 1.1 \quad \text{.......} \quad (6)$$

where h_d is the in-line pressure desired at the distal end of the lateral. Thus,

$$\sum_{j=1}^{N} \Delta h_j = 0.21\ h_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

By selecting h_d, ℓ, D and d, the perforation discharge Equations 1 and 2 can be used to calculate the change in in-line pressure and the perforation discharge rate for each lateral segment. When Equation 7 is satisfied, the maximum lateral length for that set of conditions is reached.

Similarly, the selection of a suitable manifold diameter is a function of the discharge rates, and spacing of the laterals and the number of laterals or manifold length. Any tolerance of variation of discharge rates between the perforations of different laterals can be specified. In this paper 15 percent variation is allowed. This difference will be greatest between the perforation nearest the manifold in the nearest lateral, $q_{M,N}$ and the perforation furthest from the manifold in the furthest lateral, $q_{1,1}$:

$$\frac{q_{M,N}}{q_{1,1}} = 1.15 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

As with the lateral design, the junction and entrance losses into the laterals can be ignored by maintaining low manifold velocities. Therefore, combining Equations 1 and 8:

$$\frac{q_{M,N}}{q_{1,1}} = \frac{\left[h_d + 0.21h_d + \sum_{i=1}^{M} \Delta H_i\right]^{1/2}}{h_d^{1/2}} = 1.15 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

where ΔH_i is the friction loss in the i^{th} manifold segment, and M the number of manifold segments. Thus:

$$\sum_{i=1}^{M} \Delta H_i = 0.1\ h_d \quad \text{.......................................} \quad (10)$$

By assuming that the lateral discharge rates are equal along the length of the manifold and if losses of the manifold/lateral junctions are ignored, the manifold diameter can easily be calculated using Hazen-Williams equation until Equation 10 is satisfied:

$$\sum_{i=1}^{M} \Delta H_i = 4.71 \frac{\sum_{i=1}^{M} L_i Q_i^{1.85}}{(C_h D_m^{2.63})^{1.85}} \quad \text{............................} \quad (11)$$

where L_i is the length of the manifold segment or lateral spacing, Q_i is the flow rate in manifold segment i, and D_m the diameter of the manifold pipe. If all manifold segments are of equal length, Equations 10 and 11 can be combined and reduced to:

$$\sum_{i=1}^{M} F_i = 4.71 \sum_{i=1}^{M} Q_i^{1.85} = 0.1\ h_d \frac{(C_h D^{2.63})^{1.85}}{i \times L} \quad \text{............} \quad (12)$$

where F_i is a friction factor for the i^{th} manifold segment. By computing F_i values for each manifold segment and summing them over the length of the manifold, the manifold diameter can be quickly determined using Equation 12.

DESIGN CONSIDERATIONS

In-line Pressure

The in-line pressure at the distal end of the lateral, h_d, is important in the design of the network. At low pressures it is not significant in the network pipe sizing for level sites but a sufficiently high in-line pressure should be selected to minimize the effects of variations in the elevation of perforations that will occur constructing the system. An elevation difference of ±10 percent between perforation elevations will result in a 10 percent difference in discharge rates between the highest and lowest perforation. A minimum in-line pressure of 2.5 ft (0.76 m) permits a ±3 in (7.6 cm) construction tolerance. However, h_d should not be excessive because it affects the perforation discharge rate resulting in increased junction losses and pump or siphon capacity.

Perforation Spacing

Uniform distribution can be approached best by providing as many uniformly spaced perforations as practically possible. Smaller diameter perforations permit more perforations per unit length of lateral but holes smaller than 1/4 in (6.4 mm) in diameter are more likely to clog. Larger spacings between perforations permit longer laterals but spacings too great result in localized overloading of the soils infiltrative surface. Maximum spacings of 10 ft (3.05 m) are suggested here but smaller spacings are more desirable.

In bed systems lateral spacings equal to the perforation spacings are recommended. Perforations between any two laterals should be staggered so that they lie on the vertices of equilateral triangles. This arrangement will provide the most uniform distribution of liquid.

Since the laterals drain between doses, air must be vented from the laterals at the beginning of each dosing cycle. To facilitate venting, the perforation at the distal end of each lateral should be drilled horizontally in the end cap near the crown of the pipe.

Network Configurations

For any given absorption field, more than one network configuration may be acceptable. Since uniform distribution is the objective, the laterals may be oriented in any direction regardless of the original slope of the site.

The manifold may be located at one end, in the center, of off-center of the laterals as the situation demands. Central manifolds minimize lateral size because lateral sizing is based in part upon the length of lateral between the supply and distal ends. For very long narrow absorption areas, multiple manifolds may be used as long as the pressures at each manifold inlet are equal. Equal pressures can be insured by linking the laterals from each manifold into loops (Figure 2). Inlets to the manifold from the pressurization unit are typically located at one end of the manifold but they may be located at any point along the manifold. Central inlets minimize manifold size.

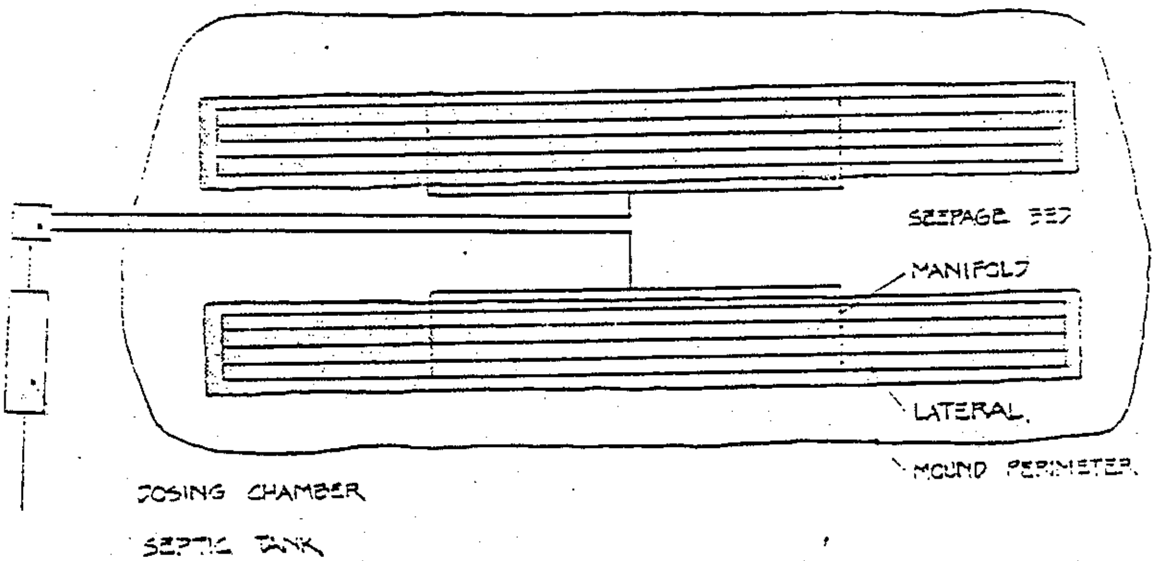
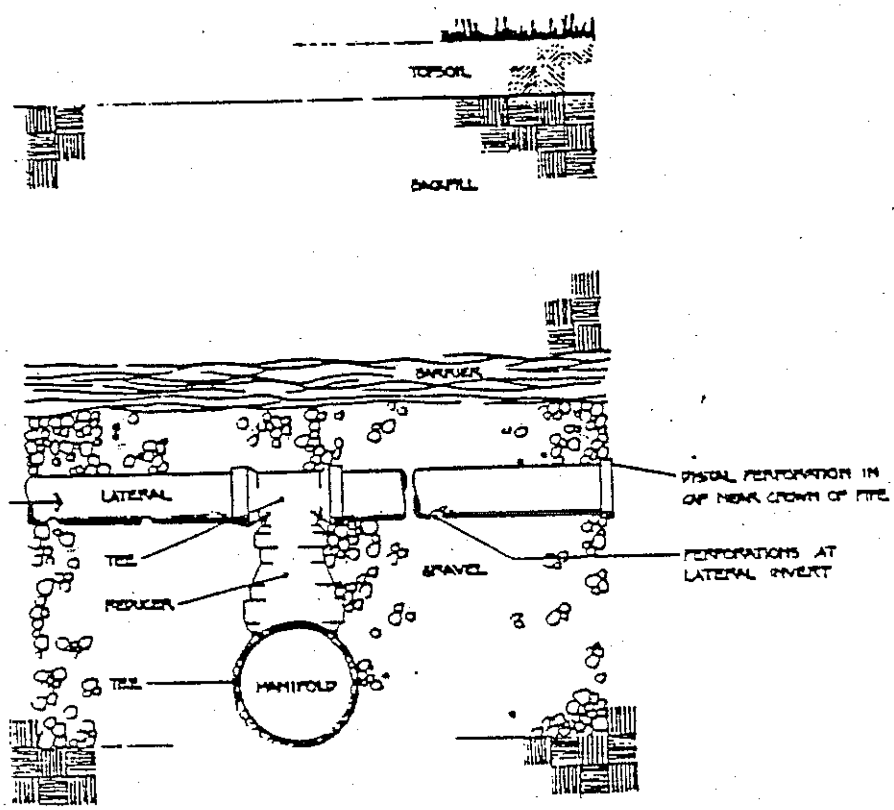


Figure 2. Looped Multiple Manifold for a Large Mound System

Leakage from the holes nearest the manifold will occur at the start of each dose while the network is filling. The amount of leakage can be minimized by locating the outlets to the laterals at the crown of the manifold using tee to tee construction (Figure 3). In this manner, the manifold will fill before discharging into the laterals. However, provisions must be made for draining the manifold in localities where freezing is a concern. Where pumps are used for pressurization, the manifold may be drained back into the dosing chamber. This is impractical with large volume manifolds and impossible if siphons are used. In such instances, the manifold should be insulated and provisions made for manual draining if the system is left idle for any extended period of time. In systems where the laterals are long in comparison to the manifold or where the outlets are located below the crown of the manifold, dosing volumes should be made large in relation to the pipe volume to reduce the significance of the leakage.



Sloping Sites

Special consideration is required for sloping sites where the infiltrative surfaces and distribution laterals are constructed at different elevations. The static heads between laterals installed at different elevations will vary thereby affecting the perforation discharge rates. If this is not properly compensated for, unequal distribution between the various infiltrative surfaces will result.

If the infiltrative surfaces are restricted to only two different elevations of equal loading, two separate distribution networks can be used with each network receiving alternate doses through the use of alternating pumps, valves, or siphons. Each network is designed separately. This arrangement provides the best assurance of an equal division of flow.

If the infiltrative surface areas are not equal or more than two levels are necessary, a single network can be designed by taking into account the differences in the total heads within each lateral (See Appendix I). However, if the network inlet is located at the upslope end and the manifold volume is large in comparison to each infiltrative surface, the lower surface may receive significantly more liquid while the network is filling and draining during each dose. The design should allow for this leakage.

Siphons or siphon breakers must be used in networks where the low water level in the dosing chamber is above the lateral inverts. If a pump without a siphon breaker is used in such an instance, a natural siphoning of liquid out the chamber will occur. A simple siphon break can be merely a small hole drilled in the discharge line at the highest point in the dosing chamber.

Dosing Chamber

The dosing chamber is a crucial component of the network design. It must discharge the appropriate volume at the required rate with each dose. Also, appurtances must be selected carefully to insure proper and reliable operation.

The chamber must be sized for the desired dosing volume plus a reserve volume if necessary. The dosing volume is controlled by the site conditions and the volume of pipe in the network. Site conditions will dictate the frequency of dosing. Soil texture is usually used to select the proper frequency as indicated in Table 2. The average, rather than peak flow should be used to determine the dosing volume. To minimize the significance of leakage out the holes nearest the manifold during filling, dosing volumes of five to ten times the pipe volume are suggested, but soil conditions will control. This volume should be in excess of any volume of liquid which drains back from the manifold after each dose.

TABLE 2. DOSING FREQUENCIES FOR VARIOUS SOIL TEXTURES (U.S. EPA, 1980)

Soil Texture	Dosing Frequency
Sand	4 doses/day
Sandy loam	1 dose/day
Loam	Frequency not critical*
Silt loam; silty clay loam	1 dose/day*
Clay	Frequency not critical*

* Long-term resting provided by alternating fields may be desirable.

A reserve capacity above the active dosing volume equal to one day's average flow should be provided if single pumps are used. This will provide one day or more for repairs with normal water fixture use in case of pump failure. A reserve volume is not needed if siphons are used because overflows by-passing the siphon are provided. However, high water alarms should be installed with siphons to alert the owner of any malfunctions.

Selection of the pressurization unit is based upon the needed capacity and the operating head. The capacity is determined from the in-line pressure and the number and diameter of perforations in the network. The pressure and perforation diameter will determine the perforation discharge rate which multiplied by the number of perforations in the network will give the needed discharge rate of the pump. The operating head is the sum of the in-line pressure desired, h_d, the network losses equal to 0.31 h_d (see Equations 7 and 10), the delivery losses, and the elevation difference between the low water level in the dosing chamber and the lateral inverts. Pumps are selected based on their head discharge curves to provide the needed capacity at the calculated operating head. Siphons are selected based on their rated capacity assuming a free discharge. The siphon discharge invert elevation is established by raising it a distance above the lateral inverts equal to the in-line pressure, h_d, plus network losses, 0.31 h_d, and delivery losses.

Necessary appurtances include level controls for pump systems, high water alarm switches, and suitable access to the pressurization unit for servicing. Other features may also be desirable (U.S. EPA, 1980). Switch selection and installation are extremely important because the most frequent cause of pump failure is switch malfunctions. The switches should be sealed from the corrosive atmosphere in the chamber and all electrical contacts and relays must be mounted outside the chamber. Provisions should be made to prevent gases in the chamber from following the electrical conduits into the control box. The high water alarm switch should be located 2-3 in (5-8 cm) above the pump or siphon activation level. This switch must be on a separate circuit from the pump level controls. Access for maintenance is best provided by a manway located over the pressurization unit .

DESIGN PROCEDURE

The design of even a small pressure distribution network can be very tedious. A simplified procedure is described here using graphs, tables and nomographs presented in Figures 4 through 11 and Tables 3 through 5. These were prepared using the equations presented above using an orifice coefficient, C, for sharp-edged orifices of 0.6 and a Hazen-Williams friction factor, C_h, for plastic pipes of 150. The tables and graphs can be used for any network configuration with design flows up to 25,000 gpd (95 m^3/d). Design examples illustrating this procedure are presented in Appendix I.

Step 1: Layout a network:

Use a manifold and lateral configuration that will provide uniform coverage of the infiltration surface.

Step 2: Select a perforation size and spacing:

In bed systems, the perforation and lateral spacing should be equal.

Step 3: Determine the appropriate lateral pipe diameter compatible with the chosen perforation size and spacing:

Use Figures 4 through 10.

Step 4: Calculate the lateral discharge rate:

From equation 1:

$$q = 11.79\ d^2 h_d^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

where q is the perforation discharge rate in gpm, d the perforation diameter in inches, and h_d the distal in-line pressure in ft. Table 3 presents q for various in-line pressures in place of using Equation 13. The lateral discharge rate is q times N, the number of perforations in the lateral.

Step 5: Calculate the appropriate manifold size based on the number, spacing and discharge rate of the laterals:

Determine the flow in each manifold segment by adding the discharge rates of the laterals downstream of the segment and calculate the F_i values in each manifold segment. Values of F_i can be taken from Table 4 or calculated using :

$$F_i = 9.8 \times 10^{-4} Q_i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

where Q_i is the manifold flow in gpm. The manifold diameter, D_M, can then be computed using:

$$D_M = \left[\frac{\sum_{i=1}^{M} L_i F_i}{f h_d}\right]^{0.21} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

where f is the fraction of the total headloss desired in that manifold segment or series of manifold segments. To maintain less than a 10 percent head loss, f must be less than or equal to 0.1. (This is used to design telescoping manifolds as illustrated in Appendix I.) If the manifold is to be a uniform size throughout the network, Table 5 can be used to select the appropriate manifold diameter based on the lateral spacing and lateral discharge rate.

Step 6: Determine the dose volume required:

The minimum dose volume is based on the pipe volume. It should be five to ten times the network pipe volume to minimize the significance of leakage during filling and draining of the network. The nomograph in Figure 11 can be used to calculate this volume. If the crown of the manifold lies below the lateral inverts, the manifold pipe volume does not need to be included. If this is not the case, however, the nomograph can be used to determine the manifold volume as well.

The required dose volume is calculated by dividing the average daily flow by the desired dosing frequency (Table 2). If the minimum dose based on pipe volume is larger than the required dose based on dosing frequency, a different network may have to be designed to reduce the pipe volume.

Step 7: Calculate the minimum pump or siphon discharge rate:

This is found by summing the perforation or lateral discharge rates.

Step 8: Calculate the total friction losses:

The total friction losses are the sum of the losses in the delivery pipe and the network losses. Using the minimum discharge rate computed in Step 7, determine the friction loss in the delivery pipe between the dosing chamber and the network inlet. This can be calculated using the Hazen-Williams equation:

$$\text{Friction loss} = L_d \left[\frac{3.55\, Q_m}{C_h D_d^{2.63}}\right]^{1.85} \quad \text{.................. (16)}$$

where L_d is the length of the delivery pipe from the dosing chamber to the network inlet in ft, D_d is the pipe diameter in in, and Q_m the discharge rate in gpm. If the delivery pipe is plastic (C_h = 150), Table 6 can be used. Add to this the network losses which are equal to 1.31 h_d.

Step 9: Select the pressurization unit:

Pump selection is based on the pumping head and discharge rate required for the network. The static lift, the difference in elevation between the low water level in the dosing chamber and the lateral inverts, must be added to the friction losses computed in Step 8 to obtain the total pumping head. Using the head-discharge curves supplied by the manufacturer, a pump able to efficiently discharge the minimum rate or more from Step 7 at the total pumping head is selected.

Siphons are selected based on the manufacturers stated average discharge rate for free discharge. This rate must be equal to or greater than the minimum discharge rate of the network. To function properly, the siphon discharge invert must be elevated above the lateral inverts a distance equal to the friction losses estimated in Step 8. The delivery pipe from the siphon should be one nominal size larger than the siphon to facilitate air venting.

Step 10: Size the dosing chamber:

The volume of the chamber is determined by the dosing volume computed in Step 6. If a single pump is to be used for pressurization, an additional storage volume equal to one day's average flow should be provided between the high water alarm switch and the chamber inlet invert.

CONSTRUCTION AND MAINTENANCE

The networks are usually assembled at the job site. However, drilling of the perforations is best done in the shop using a drill press to insure the hole axes are perpendicular to the pipe centerline and that all holes lie on the pipe invert. After drilling, any burrs left around the holes inside the pipe should be removed.

If the septic tank and dosing chamber are properly maintained, the network should require little maintenance. The dosing chamber should be inspected regularly. Switch operation should be tested and pump or siphon operation observed. The bottom of the chamber should be pumped whenever the septic tank is pumped. If solids do enter the network and plug the laterals the ends of the laterals can be cut and the lines rodded and flushed. The ends can be replaced with a slip coupling and solvent weld joint.

DISCUSSION

The use of pressure distribution networks has been limited primarily to mound systems and large soil absorption fields. Their use is greatest in Wisconsin where the networks were first developed. Most of the netowrks are used in individual home systems but several large networks are employed in community and commercial systems designed for daily flows up to 30,000 gal (113,440 L). The length of service of these systems ranges from 9 years for the individual mound systems to 4 years for the large networks.

Experience with the networks has been good. Limited monitoring suggests that this method of distribution retards development of a clogging mat at the soil's infiltrative surface. Excavation of several systems did not reveal signs of a clogging mat even after two years of continuous operation. Mechanical failures have been infrequent. Of more than 500 networks in use in Wisconsin, less than 10 have malfunctioned, none employing siphons. Pump switch failure is the most common problem resulting in pump burnout. High quality corrosion proof switches have corrected this problem. In two other networks, the perforations have plugged. The perforation diameters were 1/4-in (6.4 mm) in both networks. In each case, the problem was traced to inadequate septic tank maintenance or design. One system serves a butcher shop in which the waste has a high grease content. The grease passed through the septic tank and into the distribution network where it plugged the laterals. More frequent septage removal has corrected this problem. The other system serves an office building. In this netowrk, the perforations had become plugged with fibers and adhesive strips from feminine hygiene products. Chambered septic tanks and more frequent septage removal have corrected these problems.

It is not expected that pressure distribution networks will replace conventional designs. However, they are recommended for use in very rapidly permeable soils, soils with shallow restrictive horizons as well as mounds and large absorption fields. The simplified design procedure presented here should remove one of the obstacles to their more widespread use.

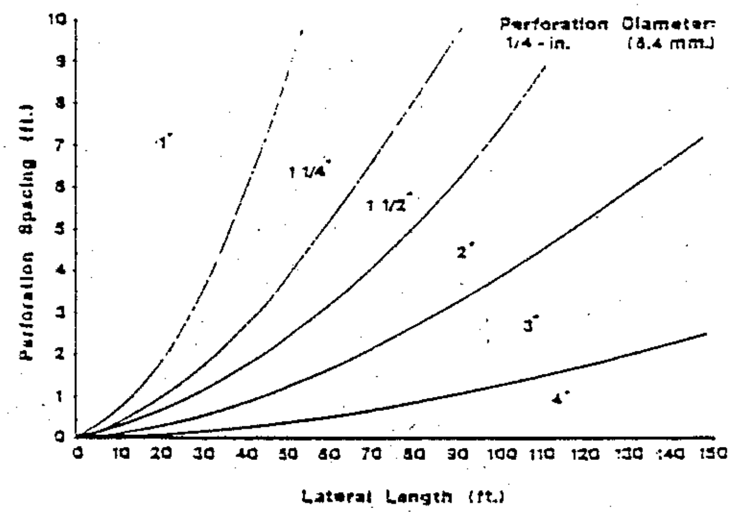


Figure 4. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 1/4-in (6.4 mm) Diameter Perforations (1 ft = 0.305 m)

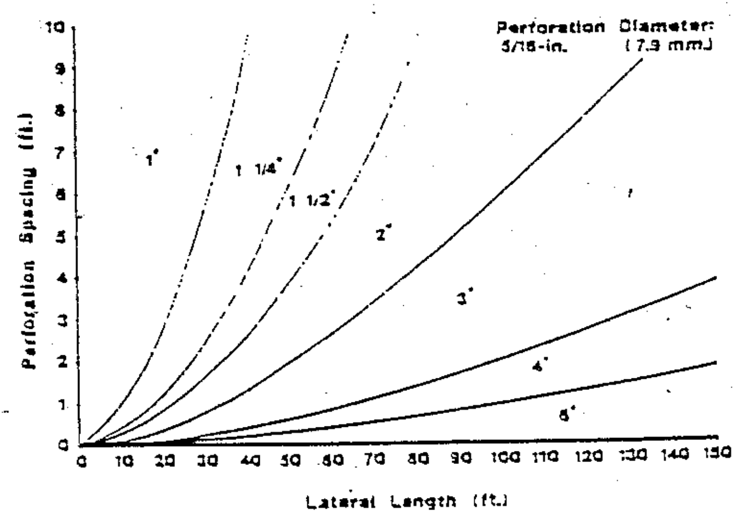


Figure 5. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 5/16-in (7.9 mm) Diameter Perforations

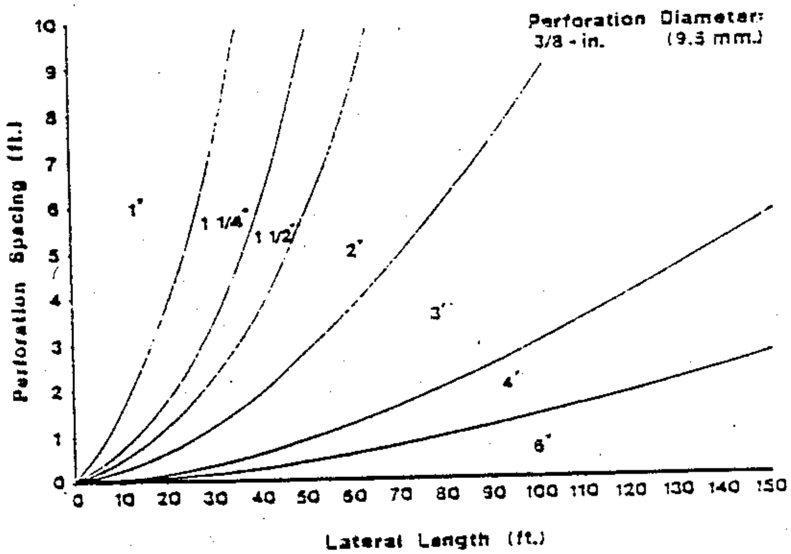


Figure 6. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 3/8-in (9.5 mm) Diameter Perforations (1 ft = 0.305 m)

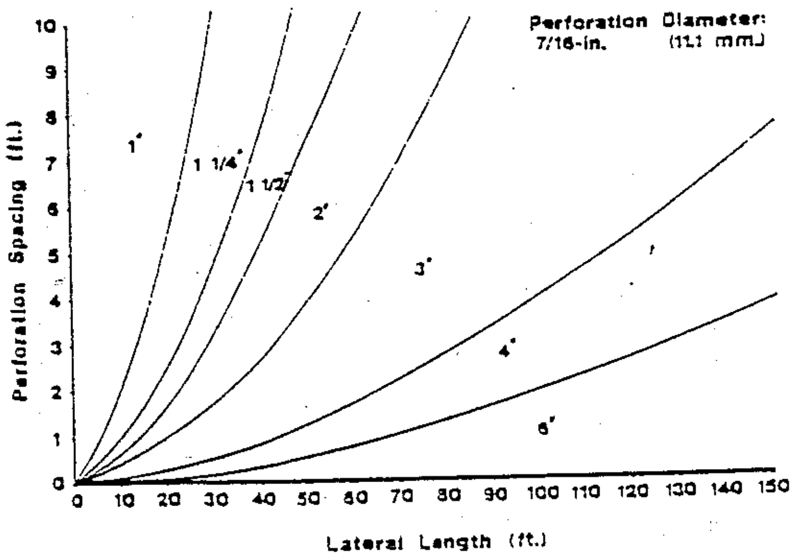


Figure 7. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 7/16-in (11.1 mm) Diameter Perforations (1 ft = 0.305 m)

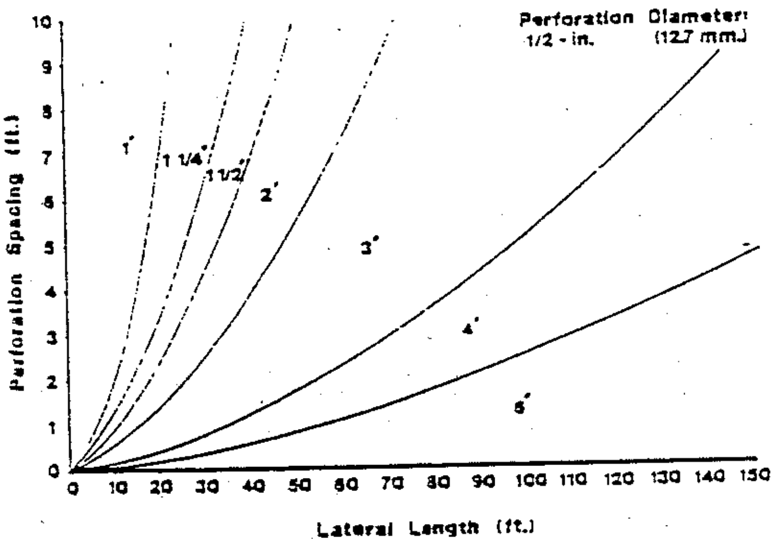


Figure 8. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 1/2-in (12.7 mm) Diameter Perforations (1 ft = 0.305 m)

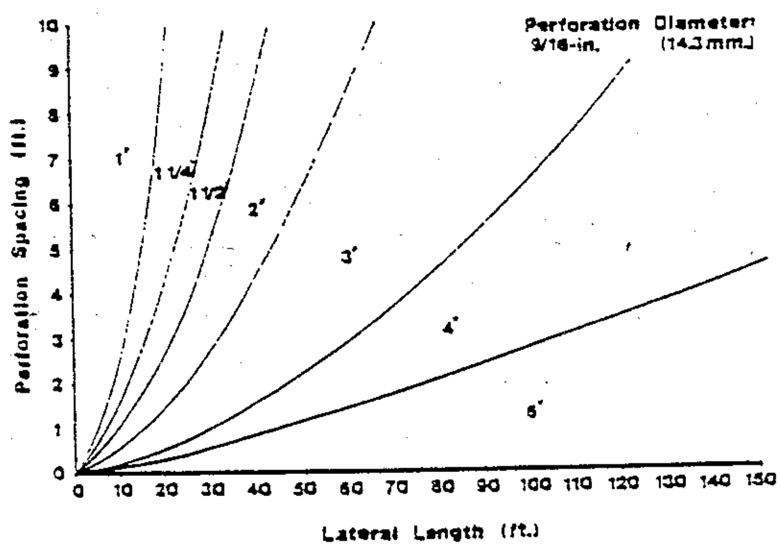


Figure 9. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 9/16-in (14.3 mm) Diameter Perforations (1 ft = 0.305 m)

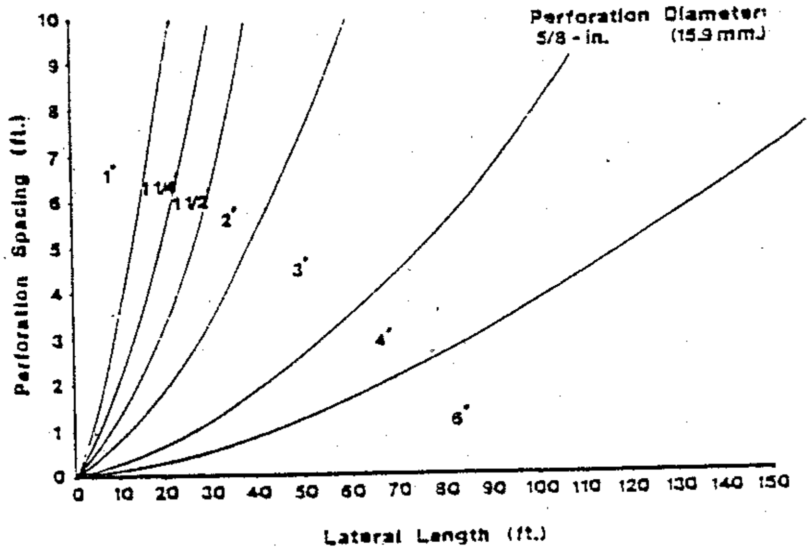


Figure 10. Minimum Lateral Diameter for Plastic Pipe (C_h = 150) Versus Perforation Spacing and Lateral Length for 5/8-in (15.9 mm) Diameter Perforations (1 ft = 0.305 m)

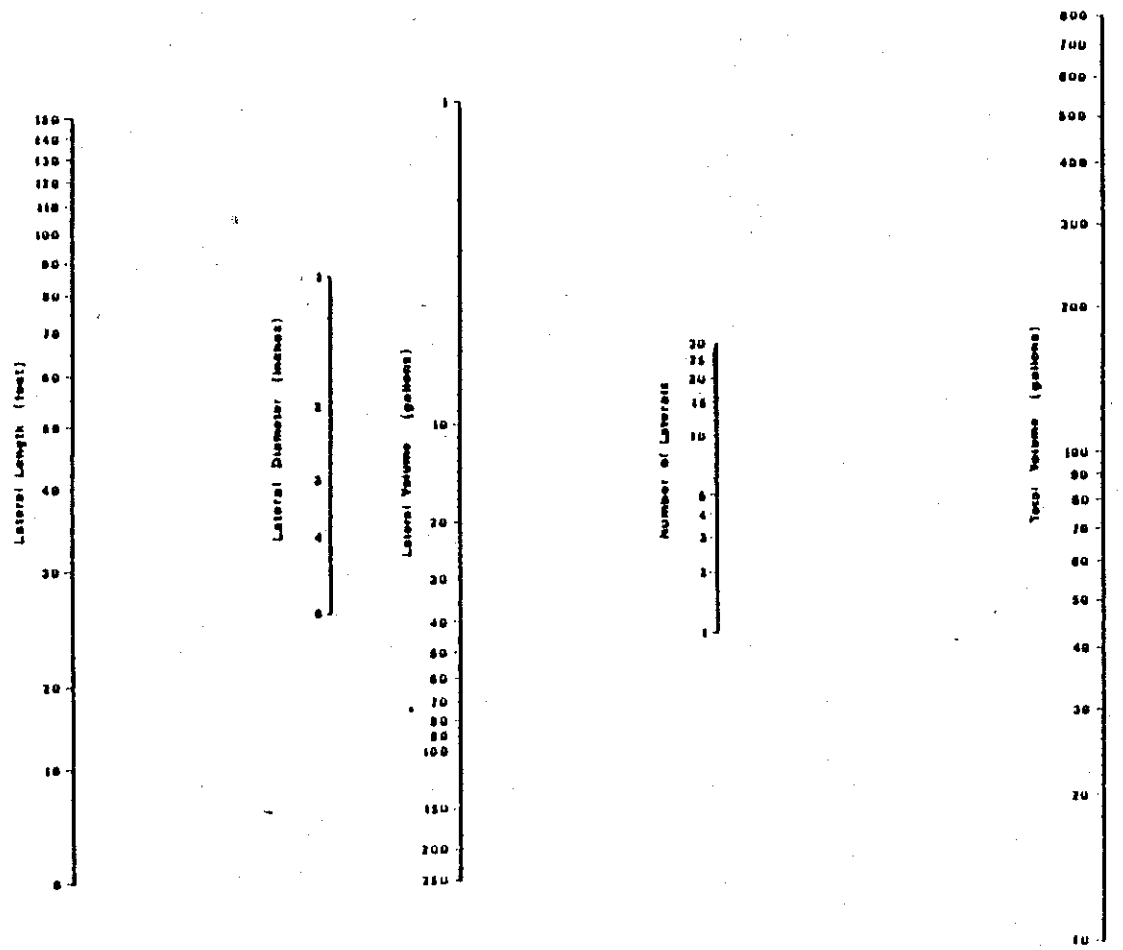


Figure 11. Nomograph for Determining the Total Pipe Volume Given the Diameter, Length, and Number of Laterals (Manifolds). (1 in = 25.4 mm, 1 ft = 0.305 m, 1 gal = 3.785 L)

Table 3. Perforation Discharge Rates in Gallons per Minute Versus Perforation Diameter and In-Line Pressure (1 in = 25.4 mm, 1 ft = 0.305 m, 1 gpm = 0.063 L/s)

In-Line Pressure (ft)	Perforation Diameter (in) 1/4	5/16	3/8	7/16	1/2	9/16	5/8
	gpm						
1.0	0.74	1.15	1.66	2.26	2.95	3.73	4.60
1.5	0.90	1.41	2.03	2.76	3.61	4.57	5.64
2.0	1.04	1.63	2.34	3.19	4.17	5.27	6.51
2.5	1.17	1.82	2.62	3.57	4.66	5.90	7.28
3.0	1.28	1.99	2.87	3.91	5.10	6.46	7.97
3.5	1.38	2.15	3.10	4.22	5.51	6.98	8.61
4.0	1.47	2.30	3.31	4.51	5.89	7.46	9.21
4.5	1.56	2.44	3.52	4.79	6.25	7.91	9.77
5.0	1.65	2.57	3.71	5.04	6.59	8.34	10.29

Table 4. F_i Values for Individual Manifold Segments Versus Manifold Segment Flow Rates[1]
(1 gpm = 6.31 x $10^{-5} m^3/s$)

Q_i' (gpm)	F_i	Q_i' (gpm)	F_i	Q_i' (gpm)	F_i	Q_i' (gpm)	F_i	Q_i' (gpm)	F_i	Q_i' (gpm)	F_i	Q_i' (gpm)	F_i
5	0.02	105	5.38	205	18.53	310	39.83	510	100.1	875	271.6	1750	979.1
10	0.07	110	5.86	210	19.38	320	42.24	520	103.7	900	286.1	1800	1032
15	0.15	115	6.36	215	20.24	330	44.72	530	107.4	925	301.0	1850	1085
20	0.25	120	6.88	220	21.12	340	47.26	540	111.2	950	316.2	1900	1140
25	0.38	125	7.42	225	22.02	350	49.86	550	115.1	975	331.8	1950	1196
30	0.53	130	7.98	230	22.93	360	52.53	560	119.0	1000	347.7	2000	1254
35	0.70	135	8.56	235	23.86	370	55.26	570	122.9	1050	380.6	2050	1312
40	0.90	140	9.15	240	24.81	380	58.05	580	126.9	1100	414.8	2100	1372
45	1.12	145	9.77	245	25.77	390	60.91	590	131.0	1150	450.3	2150	1433
50	1.36	150	10.40	250	26.76	400	63.83	600	135.1	1200	487.2	2200	1495
55	1.63	155	11.05	255	27.75	410	66.82	625	145.7	1250	525.4	2250	1559
60	1.91	160	11.72	260	28.77	420	69.86	650	156.7	1300	565.0	2300	1623
65	2.21	165	12.40	265	29.80	430	72.97	675	168.0	1350	605.8	2350	1689
70	2.54	170	13.11	270	29.80	430	76.14	700	179.7	1400	648.0	2400	1756
75	2.88	175	13.83	275	31.91	450	79.37	725	191.8	1450	691.4	2450	1825
80	3.25	180	14.57	280	33.00	460	82.67	750	204.2	1500	736.2	2500	1894
85	3.64	185	15.33	285	34.09	470	86.02	775	217.0	1550	782.2	2550	1965
90	4.04	190	16.10	290	35.21	480	89.44	800	230.1	1600	829.6	2600	2037
95	4.47	195	16.90	295	36.34	490	92.92	825	243.6	1650	878.2	2650	2110
100	4.91	200	17.71	300	37.49	500	96.45	850	257.4	1700	928.0	2700	2184

[1] $F_i = 9.8 \times 10^{-4} Q_i(\text{gpm})^{1.85}$ for plastic pipe (C_h = 150).

Table 5. Maximum Manifold Length (ft) for Various Manifold Diameters Given the Lateral Discharge Rate and Lateral Spacing (1 in = 2.54 cm, 1 ft = 0.305 m, 1 gpm = $6.31 \times 10^{-5} m^3/s$)

Lateral Discharge Rate (gpm)		Manifold Diameter = 1 1/4"					Manifold Diameter = 1 1/2"					Manifold Diameter = 2"					Manifold Diameter = 3"					Manifold Diameter = 4"					Manifold Diameter = 6"				
		Lateral Spacing (ft)					Lateral Spacing (ft)					Lateral Spacing (ft)					Lateral Spacing (ft)					Lateral Spacing (ft)					Lateral Spacing (ft)				
End Manifold	Central Manifold	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
10	5	4	8	6	8	10	10	8	12	16	20	12	16	24	24	30	26	40	48	56	70	42	64	84	96	110	84	136	174	208	240
20	10	4	4	6			4	4	6	8	10	6	8	12	16	20	16	24	30	32	40	26	40	54	64	70	54	84	106	128	150
30	15	2					2	4	6			4	8	6	8	10	12	16	24	24	30	20	28	36	48	50	42	64	84	96	110
40	20											4	4	6	8	10	10	12	18	16	20	16	24	30	32	40	34	52	64	80	90
50	25											2	4	6	8		8	12	12	16	20	14	20	24	32	40	30	44	60	72	80
60	30											2	4				6	8	12	16	20	12	16	24	24	30	26	40	48	64	70
70	35											2	4				6	8	12	8	10	10	16	18	24	30	24	36	44	56	60
80	40											2					6	8	6	8	10	10	12	18	16	20	22	32	42	48	60
90	45											2					4	8	6	8	10	8	12	18	16	20	20	28	42	48	50
100	50											2					4	4	6	8	10	8	12	12	16	20	18	28	36	40	50
110	55																4	4	6	8	10	8	12	12	16	20	16	24	36	40	40
120	60																4	4	4	8	10	6	8	12	16	10	16	24	30	32	40
130	65																4	4	6	8	10	6	8	12	16	10	14	24	30	32	40
140	70																2	4	6	8		6	8	12	8	10	14	20	24	32	40
150	75																2	4	6			6	8	12	8	10	14	20	24	32	30
160	80																2	4	6			6	8	6	8	10	12	20	24	32	30
170	85																2	4	6			4	8	6	8	10	12	20	24	24	30
180	90																2	4				4	8	6	8	10	12	16	24	24	30
190	95																2	4				4	8	6	8	10	12	16	18	24	30
200	100																2	4				4	4	6	8	10	10	16	18	24	30

Table 6. Friction Losses in Plastic Pipe (C_h = 150) Versus Flow Rate and Pipe Diameter (1 m = 2.54 cm, 1 ft = 0.305 m, 1 gpm = 6.3 x $10^{-5}m^3/s$)

Flow gpm	Pipe Diameter (in.) 1	1 1/4	1 1/2	2	3	4	6	8	10	12	Flow gpm
	ft/100 ft										
1	0.10										1
2	0.35	0.12									2
3	0.75	0.25	0.10								3
4	1.28	0.43	0.13								4
5	1.93	0.65	0.27	0.07							5
6	2.70	0.91	0.38	0.09							6
7	3.59	1.21	0.50	0.12							7
8	4.60	1.55	0.64	0.16							8
9	5.72	1.93	0.80	0.20							9
10	6.95	2.35	0.97	0.24							10
11		2.80	1.15	0.28							11
12		3.29	1.35	0.33							12
13		3.31	1.57	0.39							13
14		4.37	1.80	0.44	0.06						14
15		4.97	2.05	0.50	0.07						15
16		5.60	2.31	0.57	0.08						16
17		6.27	2.58	0.64	0.09						17
18		6.96	2.87	0.71	0.10						18
19			3.17	0.78	0.11						19
20			3.49	0.86	0.12						20
25			5.27	1.30	0.18						25
30				1.32	0.25	0.06					30
35				2.42	0.34	0.08					35
40				3.10	0.43	0.11					40
45				3.35	0.54	0.13					45
50				4.68	0.65	0.16					50
60					0.91	0.23					60
70					1.21	0.30					70
80					1.55	0.38					80
90					1.93	0.46	0.07				90
100					2.35	0.58	0.08				100
125					3.55	0.88	0.12				125
150					4.97	1.23	0.17				150
175						1.63	0.23	0.06			175
200						2.09	0.29	0.07			200
250						3.16	0.44	0.11			250
300						4.42	0.61	0.15			300
350							0.82	0.20	0.07		350
400							1.05	0.26	0.09		400
450							1.30	0.32	0.11		450
500							1.58	0.39	0.13		500
600							2.72	0.55	0.18	0.08	600
700							2.95	0.73	0.25	0.10	700
800							3.77	0.93	0.31	0.13	800
900							4.69	1.16	0.39	0.16	900
1000								1.41	0.47	0.20	1000

APPENDIX I - DESIGN EXAMPLES

Example 1: Level Site

A pressure network is to be designed for an absorption field constructed in sandy loam soil. It is to receive an average flow of 250 gpd. The field consists of 5 trenches each 3 ft wide and 40 ft long spaced 9 ft on center. The dosing chamber is located 50 ft from the first lateral.

Step 1: Layout network

Two layouts would be suitable for this system. The distribution laterals can be fed either by an end or a central manifold. With an end manifold, 5 laterals are required, while a central manifold requires 10 laterals (See Figure I-1). An end manifold will be used in this example.

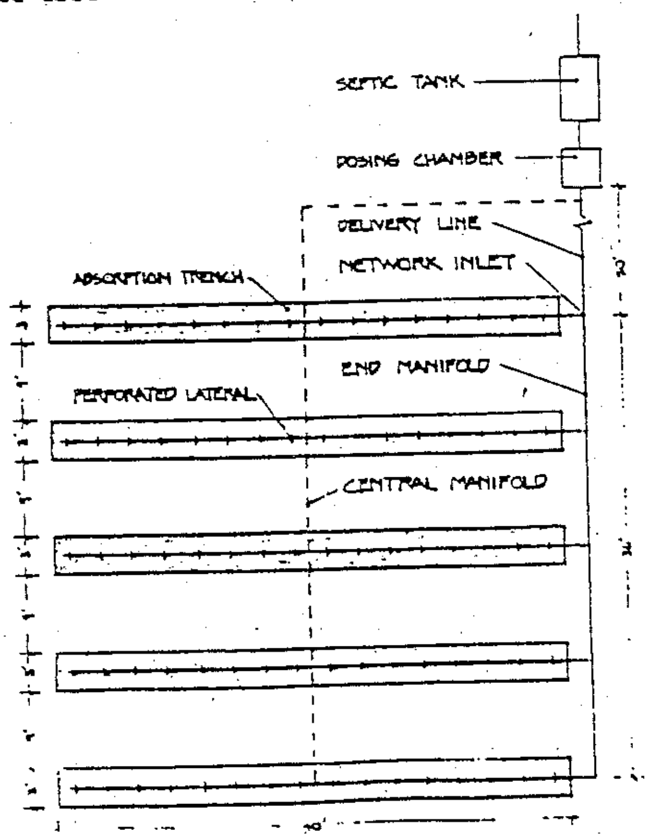


Figure I-1. End and Central Manifold Configurations for a Trench System

Step 2: Select perforation size and spacing

Perforations 1/4-in in diameter spaced 2.5 ft will be used. (Other combinations may be just as suitable.)

Step 3: Select lateral diameter

To provide the most uniform effluent application over the trench bottom, the first and last perforations in the lateral will be located one-half the perforation spacing from either end of the trench. Therefore:

Lateral Length = 40 ft - 1/2 x 2.5 = 38.75 ft

From Figure 4 (for 1/4-in diameter perforations), the minimum lateral diameter for a 38.75 ft lateral with a 2.5 ft perforation spacing is 1 1/2-in.

Step 4: Calculate the lateral discharge rate

A minimum in-line pressure of 2.5 ft is desired. From Table 3, a 1/4-in perforation will discharge 1.16 gpm at this pressure.

$$\text{No. of Perforations/Lateral} = \frac{40}{2.5} = 16 \text{ perforations}$$

Lateral Discharge Rate = 16 x 1.16 = 17.5 gpm

Step 5: Calculate the manifold size

The manifold diameter is to be uniform along its length to simplify construction.

Manifold Length = 4 x 9 = 36 ft

From Table 5, an end manifold with lateral discharge rates of 17.5 gpm and lateral spacings of 9 ft can have a maximum length of 20 ft for a 2-in diameter or 43 ft for a 3-in diameter. Therefore a 3-in diameter is necessary.

Step 6: Determine dose volume

The crown of the manifold is to be located below the lateral inverts and the manifold drained back into the dosing chamber at the end of each dose. Therefore, the minimum dose volume is based on lateral pipe volume only. Using the nomograph in Figure 11, a straightedge is placed at 38.75 ft on the Lateral Length scale and at 1 1/2-in on the Lateral Diameter scale. The straightedge crosses the Lateral Volume scale at about 3.5 gal. Maintaining this point on the Lateral Volume scale, the straightedge is rotated to align with 5 on the Number of Laterals scale. The straightedge crosses the Total Pipe Volume scale at 17.5 gals. A minimum dose volume of 5 to 10 times the total pipe volume or 90 to 175 gal should be used.

The required dosing frequency taken from Table 2 is 1 dose/day for sandy loam. Therefore:

$$\text{Required Dosing Volume} = \frac{250 \text{ gpd}}{1 \text{ dose/day}} = 250 \text{ gal/dose}$$

The minimum dose is less than the required so the network is satisfactory. Since the manifold will drain back to the dosing chamber, the dose volume must be increased in volume equal to that in the manifold and delivery line. If a 50 ft 3-in delivery line is used, the volume increase is equal to 50 ft + 36 ft or 86 ft of 3-in pipe. Using the nomograph in Figure 11, this volume is determined to be approximately 32 gals. Therefore,

$$\text{Dosing Volume} = 250 + 32 = 282 \text{ gal}$$

Step 7: Calculate the minimum discharge rate

$$\text{Minimum Discharge Rate} = 17.5 \times 5 = 87.5 \text{ gpm}$$

Step 8: Calculate total friction losses

From Table 6:

Delivery Losses $= 1.64 \text{ ft}/100 \text{ ft}^{*} \times 50 \text{ ft} \quad = 0.82 \text{ ft}$

*(50 ft of 3-in pipe at 87.5 gpm)

Total Network Losses $= 1.31\ h_d = 1.31 \times 2.5 \quad = 3.28 \text{ ft}$

Total Losses = 4.10 ft

Step 9: Select pressurization unit

In this instance, a pump is to be used.

Total Pumping Head = Static Head + Friction Losses

If the low water level in the dosing tank is 5 ft below the lateral inverts, the total pumping head is:

5 ft + 4 ft (friction losses from Step 8) or 9 ft.

Using head-discharge curves provided by manufacturers, a pump able to discharge at least 80 gpm against 9 ft of head is selected.

Step 10: Size the dosing chamber

Since only one pump is to be used, a reserve volume equal to one day's average flow is necessary in case of pump failure. Therefore, a volume of 280 gals (dose volume) + 250 gals (average daily flow) or 530 gals must be provided between the pump off switch and the dosing chamber inlet invert. (The high water alarm switch is located just above the pump on switch.)

Example 2: Large Bed System

A pressure network for an absorption field serving a small community is to be designed. The field is to be a 100 ft x 130 ft bed constructed in a sandy soil receiving an average daily flow of 15,000 gals. The dosing chamber is to be located 200 ft from the network inlet.

Step 1: Layout network

A central manifold configuration is selected as shown in Figure I-2.

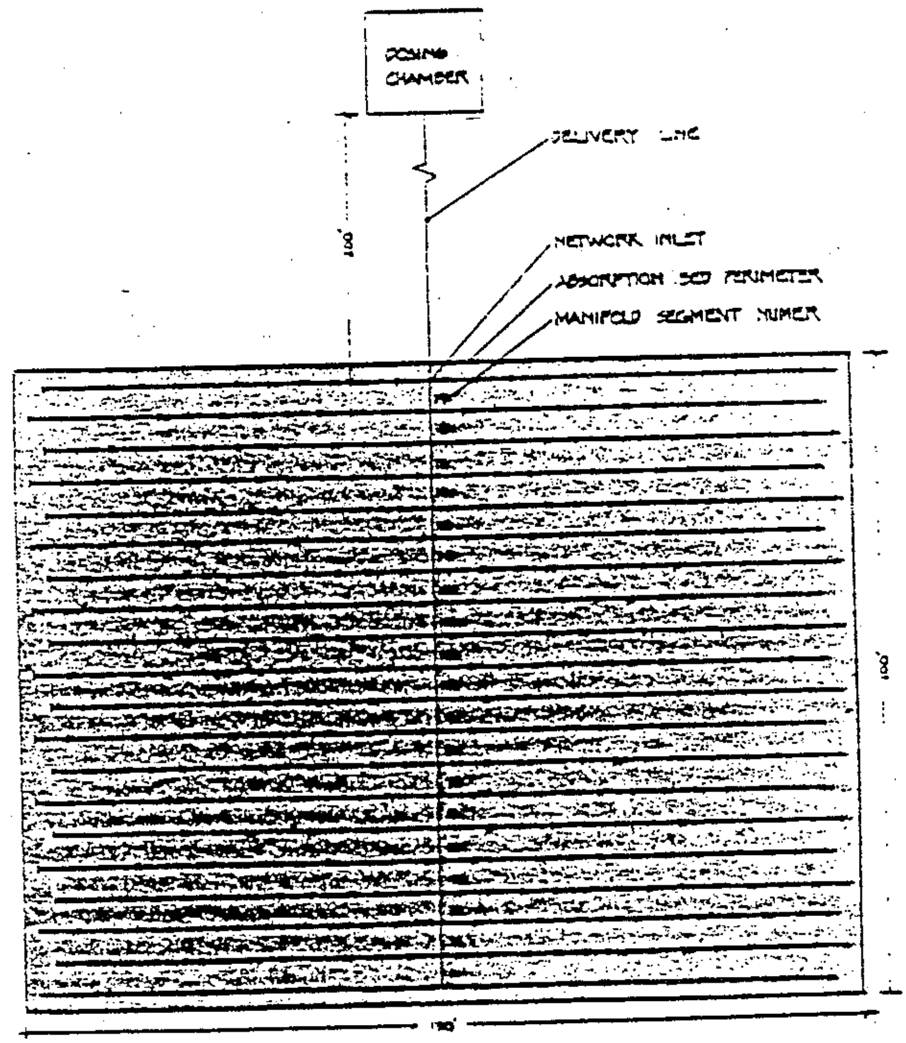


Figure I-2. Central Manifold Network Configuration for a Community-Sized Absorption Field (Example 2).

Step 2: Select perforation size and spacing

Perforations are to be 3/8-in diameter spaced 5 ft apart. The perforations are to be staggered between laterals to provide more uniform distribution (Figure I-2).

Step 3: Select lateral diameter

From Figure 6: 2-in laterals required.

Step 4: Calculate the lateral discharge rate

A minimum in-line pressure of 2 ft is used. From Table 3:

Perforation Discharge Rate = 2.34 gpm

$$\text{No. of Perforations} = \frac{65}{5} = 13$$

Lateral Discharge Rate = 13 x 2.34 = 30 gpm/lateral

Step 5: Calculate Manifold size

This network is too large to determine the manifold size from Table 5. Therefore, the F_i values from Table 4 or calculated from Equation 14 are used.

$$\text{Number of Manifold Segments} = \frac{100}{5} - 1 = 19 \text{ segments}$$

Table I-1. Results of Calculations to Determine Manifold Segment Diameters

Segment No.	Q_i (gpm)	F_i	ΣF_i	D_m (in)	Segment No.	Q_i (gpm)	F_i	ΣF_i	D_m (in)
1	60[a]	1.91	1.91		11	660	161.2	704.99	
2	120	6.88	8.79	4.98[b]	12	720	189.4	894.39	9.02
3	180	11.57	23.36		13	780	224.9	1119.29	
4	240	24.81	48.17	6.15	14	840	251.9	1371.19	9.55
5	300	37.49	85.66		15	900	286.1	1657.29	
6	360	52.53	138.19	7.05	16	960	322.4	1979.69	10.04
7	420	69.86	208.05		17	1020	360.9	2340.59	
8	480	89.44	297.49	7.80	18	1080	401.1	2741.69	10.48
9	540	111.2	408.19		19	1140	443.2	3184.89	10.70
10	600	135.1	543.79	8.44	Inlet	1200	-	-	-

[a]2 laterals x 30 gpm/lateral
[b]From Equation 15

Allowing 0.1 h_d loss of head in the manifold, the necessary manifold diameter can be calculated using Equation 15.

$$D_m = \left[\frac{L \sum_{i=1}^{M} F_i}{0.1\ h_d}\right]^{0.21} = \left[\frac{5 \times 3184.89}{0.1 \times 2}\right]^{0.21} = 10.7\text{-in or } 12\text{-in}$$

By this method a 12-in manifold would be required.

A uniform sized manifold is not necessary. To save expense and to provide more uniform distribution by reducing the difference between lateral entrance losses, the manifold should be telescoped to smaller diameters downstream. The same method as above may be used to determine the proper diameters for each segment if the allowable headloss in the manifold is assumed to be linear along its length. Making this assumption, each segment may account for $\frac{0.1}{19} h_d$ of the manifold friction loss. Calculated diameters of the even numbered segments appear in Table I-1. For example, the diameter for segment 2 is:

$$D_2 = \left[\frac{5 \times 8.79}{2 \times \frac{0.1}{19} \times 2}\right]^{0.21} = 4.98 \text{ in or } 6 \text{ in}$$

From Table I-1 the nominal manifold diameters are selected:

Manifold segments:	1-3	6-in
	4-8	8-in
	9-16	10-in
	16-19	12-in

Step 6: Determine dose volume

The crown of the manifold is to be located below the lateral elevation. A manual drain valve will be installed on the manifold to drain the manifold when the network is out of service. From Figure 11:

Minimum Dose Volume = 10.5 gal/lateral x 40 x (5 to 10) = 2100 to 4200 gals

From Table-2:

$$\text{Required Dose Volume} = \frac{15{,}000 \text{ gpd}}{4 \text{ dose/day}} = 3750 \text{ gals}$$

This is satisfactory.

Step 7: Calculate the minimum discharge rate

Minimum Discharge Rate = 30 gpm/lateral x 40 = 1200 gpm

Step 8: Calculate total friction losses

From Equation 16:

$$\text{Delivery Losses (200 ft of 12-in)} = 200 \times \left[\frac{3.55 \times 1200}{150 \times (12)^{2.63}}\right]^{1.85}$$

$$= 0.55 \text{ ft}$$

Total Losses = 3.17 ft

Step 9: Select pressurization unit

A 12-in siphon with a manufacturer's average discharge rating of 1200 gpm is selected. The discharge invert must be elevated a minimum of 3.2 ft above the lateral inverts.

Step 10: Size the dosing chamber

A dosing volume of 3750 gals is to be used. The siphon has a 30-in draw. No reserve volume is necessary since the siphon has an overflow. A high alarm switch is necessary, however to alert the owner to an overflow condition.

Example 3: Sloping Site

A pressure network for a series of absorption trenches constructed on a sloping site is to be designed. The system is to receive an average flow of 200 gpd from a 4 bedroom home (peak flow = 150 gal/bedroom x 4 = 600 gpd). Five trenches of unequal lengths are to be excavated at different elevations as shown in Figure I-3. The elevations of the distribution lateral inverts are to be as follows:

Lateral 1:	873.0 ft
Lateral 2:	873.5 ft
Lateral 3:	874.0 ft
Laterals 4 & 5:	874.5 ft
Laterals 5 & 6:	875.5 ft

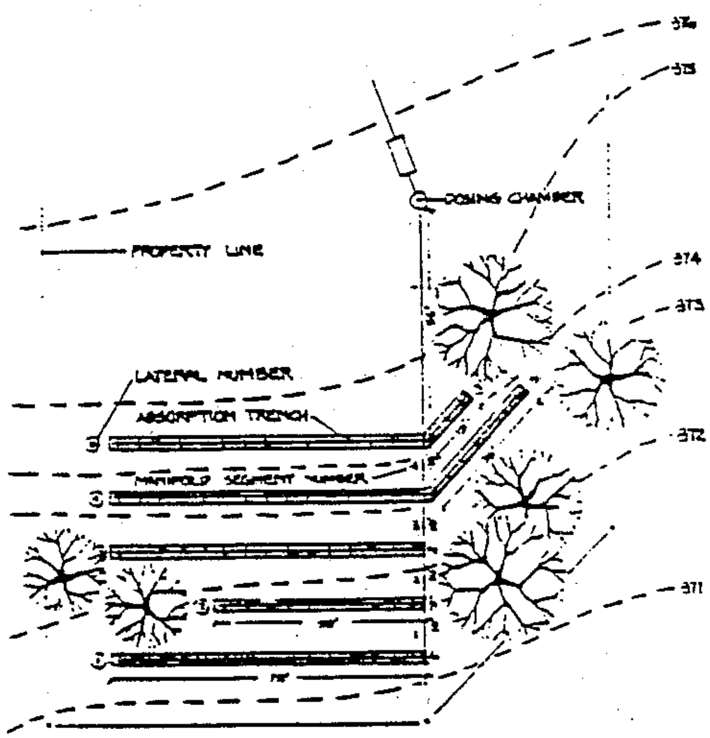


Figure I-3. Network Layout for a Trench System on a Sloping Site (Example 3)

Step 1: Layout network

A layout as shown in Figure I-3 is selected to conform to the trench layout.

Step 2: Select the perforation size and spacing

Because the static heads in the laterals in each trench will vary, either the perforation diameter or the perforation spacing must be changed to maintain uniform application of effluent to each of the infiltrative surfaces. It is most practical to change the spacing, since the perforation diameters normally can only change by nominal drill bit sizes.

A perforation diameter of 1/4-in is selected with a maximum spacing of 5 ft. Since the lateral at the lowest elevation will static head, the maximum spacing is to be used in this lateral.

To determine the spacing for the remaining laterals, it is necessary to compute the fraction of the dosage rate that is directed into each lateral to provide uniform distribution. Knowing this and the in-line pressure, the perforation discharge rates can be determined for each lateral and thence, the perforation spacing.

To calculate the lateral discharge rates, the discharge rate of the lowest lateral must be calculated first based on the perforation diameter and spacing selected. To do this, a minimum in-line pressure in the upper most lateral must be selected. Then the minimum in-line pressure in the lower most lateral is equal to the minimum in-line pressure in the uppermost lateral plus the elevation difference between the two laterals less the upstream manifold losses. Therefore in Lateral 1:

Minimum In-Line Pressure =

$$2.5 + (875.5 - 873.5) - (\frac{4}{4} \times 0.1) \times (2.5) = 4.75 \text{ ft}$$

From Equation 13:

$$\text{Perforation Discharge Rate} = 11.79(1/4)^2(4.75)^{1/2} = 1.60 \text{ gpm}$$

$$\text{Lateral Discharge Rate} = \frac{75}{5} \times 1.6 = 24 \text{ gpm}$$

Knowing that the ratio of the lateral discharge rates to the total trench loading in each trench must be equal to maintain uniform distribution, the remaining lateral discharge rates, in-line pressures, and perforation discharge rates can be calculated (See Table I-2). The perforation spacing is determined by first dividing the lateral discharge rate by the perforation discharge rate to obtain the number of perforations per lateral and then dividing this into the trench length. Table I-2 presents the results of these calculations.

Step 3: Select lateral diameter

Figure 4 is used to select the lateral diameter. The diameters obtained from Figure 4 appear in Table I-2. To reduce the number of different pipe diameters, larger nominal diameters may be ultimately chosen. For instance, laterals 1 through 4 could be 1 1/2-in pipe.

Step 4: Calculate the lateral discharge rate

This was done in Step 2 of this example. See Table I-2.

Step 5: Calculate the manifold size

The manifold is to be a uniform diameter throughout.

$$\text{Manifold Length} = 4 \times 10 = 40 \text{ ft}$$

Since the lateral discharge rates vary, Table 4 is used to make this calculation:

Manifold Segment No.	Accumulative Flow gpm	F_i
1	24	0.35
2	40	0.90
3	64	2.15
4	99	4.82
	TOTAL	8.22

$$D_m = \left[\frac{10 \times 8.22}{0.1 \times 2.5}\right]^{0.21} = 3.38 \text{ in or } 4 \text{ in}$$

Step 6: Determine dose volume

Since the manifold must fill entirely before the upper laterals are filled, the lateral and manifold pipe volume must be included in the calculation of the minimum dose volume. Figure 11 is used

Table I-2. Determination of Lateral Diameters on a Sloping Site (Example 3)

Lateral No.	Trench Length (ft)	Trench Width (ft)	Loading Rate (gpd/ft^2)	Trench Loading (gpd)	% Total Loading	In-Line Pressure (ft)	Perforation Diameter (in)	Perforation Discharge Rate (gpm)	Lateral Discharge Rate (gpm)	No. Perforations	Perforation Spacing (ft)	Lateral Length (ft)	Lateral Diameter (in)
1	75	3	0.5	112.5[a]	18.8[b]	4.75	1/4	1.60	24	15	5.0	72.5	1 1/2
2	50	3	0.5	75.0	12.5	4.31	1/4	1.53	16[c]	10[d]	5.0[e]	47.5[f]	1 1/4
3	75	3	0.5	112.5	18.8	3.88	1/4	1.45	24	17	4.4	72.8	1 1/2
4	75	3	0.5	112.5	18.8	3.44	1/4	1.37	24	18	4.2	72.9	1 1/4
5	35	3	0.5	52.5	8.8	3.44	1/4	1.37	11	8	4.4	32.8	1
6	75	3	0.5	112.5	18.8	2.50	1/4	1.17	24	21	3.5	73.3	2
7	15	3	0.5	22.5	3.8	2.50	1/4	1.17	5	4	3.8	13.1	1
Total	400	-	-	600	100.3	-	-	-	120	-	-	-	

[a] 75 ft x 3 ft x 0.5 gpd/ft^2 = 112.5 gpd

[b] (112/5 ÷ 600) x 100 = 18.8%

[c] (24 ÷ 18.8) x 12.5 = 16 gpm

[d] 16 ÷ 1.53 = 10 perforations

[e] 50 ÷ 10 = 5 ft

[f] 50 - (5 ÷ 2) = 47.5 ft

to make this calculation.

Lateral Volume:	275 ft of 1 1/2-in	=	26 gals
	75 ft of 2-in	=	12
	50 ft of 1-in	=	2
Manifold Volume:	40 ft of 4-in	=	26
			66 gals

Five to 10 times the pipe volume gives a minimum dose volume of 330 to 660 gals/dose equal to about 1 dose per day. If a 330 gal dose is used, at average daily flow 1 dose will occur every 1 1/2 days. This is satisfactory.

Step 7: Calculate the minimum discharge rate

This is the sum of the lateral discharge rates equal to 128 gpm from Table I-2.

Step 8: Calculate total friction losses

From Equation 16:

$$\text{Delivery Losses} = 20 \times \left[\frac{3.55 \times 128}{150 \times 4^{2.63}}\right]^{1.85} = 0.18 \text{ ft}$$

$$\text{Network Losses} = 1.3 \times 2.5 = 3.25 \text{ ft}$$

$$\text{Total Losses} = 3.43 \text{ ft}$$

Step 9: Select pressurization unit

In this case, a siphon can be used. It would be selected on the basis of the average rated discharge. The discharge invert would be set at a minimum of 3.43 ft above the uppermost lateral invert.

Step 10: Size the dosing chamber

The draw of the siphon and size of the dose selected, 330 gal, is sufficient to size the dosing chamber.

APPENDIX II - REFERENCES

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APPENDIX III - NOTATION

The following symbols are used in this paper:

A = lateral cross-sectional area

C = orifice coefficient

C_h = Hazen-Williams friction factor

D = lateral pipe diameter

D_d = delivery pipe diameter

D_m = manifold pipe diameter

F_i = friction factor for i^{th} manifold segment

ΔH_i = friction headloss in i^{th} manifold segment

L = lateral spacing

L_d = length of delivery pipe

L_i = length of i^{th} manifold segment

M = number of laterals

N = number of lateral segments or perforations in lateral

Q_i = flow rate in i^{th} manifold segment

Q_m = flow rate at manifold inlet

Q'_j = flow rate in j^{th} lateral segment

R = hydraulic radius

V'_j = flow velocity in j^{th} lateral segment

a = perforation cross-sectional area

d = perforation diameter

f = acceleration due to gravity

h_j = in-line pressure in lateral segment j

h = in-line pressure or head

h_d = in-line pressure at distal end of lateral

h'_e = perforation entrance headloss

Δh_j = friction loss in j^{th} lateral

$\Delta h'_j$ = in-line pressure change across the j^{th} perforation

i = lateral segment number

j = manifold segment number

ℓ = perforation spacing of length of lateral segment

ℓ_j = length of j^{th} lateral segment

q_j = perforation discharge rate in j^{th} lateral segment

q_N = perforation discharge rate in N^{th} lateral segment

$q_{M,N}$ = perforation discharge rate in N^{th} lateral segment of M^{th} lateral (nearest perforation to network inlet)

v_j = flow velocity through perforation in j^{th} lateral segment

ϑ = entry loss coefficient

ϕ = entry loss coefficient