EFFLUENT DISTRIBUTION

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Member ASAE

A soil absorption system used for disposal of wastewater effluent must have the capability to absorb and adequately treat all wastewater it receives over a reasonable length of time. Proper sizing of the absorption area based upon soil and site characteristics, characteristics of the wastewater, and proper construction are obviously necessary if these criteria are to be met. However, once the system is put into service it also must be properly managed to maintain the soil's infiltrative surface if it is to have a long life of effective treatment and disposal. The method of wastewater application to the infiltrative surface is one important aspect of proper management.

Purification is the paramount objective when disposing of wastewater into the soil. Pollutants must not be allowed to reach the ground or surface waters in concentrations which would create health hazards or environmental degradation. This requires that the soil remain permeable to absorb all the liquid but not excessively permeable to allow pollutants to penetrate through the soil to the ground water.

When wastewater is applied continuously to a soil for a period of time a clogging mat usually forms at the infiltrative surface. The mat becomes a barrier to flow restricting the rate of infiltration. However, clogging *per se* is not synonymous with failure for flow through the mat will continue, albeit at a much reduced rate. In fact, some clogging is beneficial to enhance purification in rapidly permeable soils. Unsaturated conditions in the soil are created by the mat since the infiltration rate is reduced below the saturated soil's hydraulic conductivity. Thus, the wastewater is forced into the smaller pores which provide closer and longer soil-liquid contact for improved filtration, biochemical degradation and chemical retention of waste constituents. Studies have shown that where sufficiently unsaturated, 60 to 90 cm (2 to 3 ft) of soil is adequate to remove nearly all fecal indicator bacteria and viruses. If the soil is saturated or nearly saturated, removals become unacceptable (Tyler et al. 1977). The goal in management, therefore, becomes one of preventing excessive clogging while maintaining sufficiently unsaturated conditions for at least 90 cm (3 ft) below the infiltrative surface. This can be accomplished through the proper method of wastewater application to the infiltrative surface.

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GENERAL SOIL PROPERTIES EFFECTING WATER MOVEMENT

Properties of the soil determine to some extent the most appropriate method of wastewater application because of their effect on water movement. The general properties that should be considered are: (1) soil permeability, (2) depth to seasonal high water table, (3) depth to bedrock, (4) depth to cemented pan, (5) soil slope, and (6) stoniness. If any of these soil properties create a site limitation for soil absorption of wastewater, the limitation often can be overcome by selecting the proper method of wastewater application. A computerized inventory of these and other properties of the approximately 11,000 soil series in the United States is being prepared by the Soil Conservation Service of USDA (1975).

Soil Permeability

Soil permeability is defined as ''that quality of the soil that enables it to transmit water or air'' (Soil Survey Staff 1951). It can be measured quantitatively as the rate of water flow through a unit cross section of soil in unit time and, as such, is equivalent to hydraulic conductivity. The Soil Conservation Service (1971) has defined classes of soil permeability for which the class limit values represent saturated permeability which is the maximum permeability. If moisture content decreases, the larger pores empty first leaving only the smaller pores to conduct the water, thus resulting in a sharp drop in permeability. A soil material, therefore, can have an infinite number of permeabilities (hydraulic conductivities) corresponding to many moisture contents ranging downward from saturation. Permeability is specific for each kind of soil material because it is largely a function of the pore size distribution (Bouma 1973).

Bouma et al. (1974a) have described water movement in soil in terms of saturated and unsaturated hydraulic conductivity. The merits of unsaturated versus saturated flow of septic tank effluent in the absorption field have been discussed above. To have unsaturated flow, either a physical barrier to flow, such as a clogging mat, must be maintained at the surface of infiltration or the application rate must be lower than the saturated hydraulic conductivity. Figure 1 illustrates hydraulic conductivity curves for four different kinds of soil materials, each with its own characteristic pore size distribution. The sand is from the C horizon in Plainfield loamy sand. It has a relatively high saturated hydraulic conductivity because of its coarse porous nature. The unsaturated hydraulic conductivity drops sharply with decreasing moisture content because of the relatively few fine pores that conduct water. The clay is from the B2 horizon of Hibbing loam. The initial sharp drop in the unsaturated hydraulic conductivity results from dewatering of the relatively large pores between the peds (structural units) and root and worm channels. Because of the abundance of fine pores within peds, the hydraulic conductivity then decreases less rapidly with decreasing soil moisture content than the sand. The sandy loam and silt loam are from the IIC and B2 horizons, respectively, of Batavia silt loam and are intermediate in pore size distribution between the sand and clay.

The saturated permeability classes used by the Soil Conservation Service (1971) do not predict the unsaturated hydraulic conductivity that can be maintained or designed into absorption fields. They are useful, however, in stratifying soils according to their relative capacities to transmit liquid. Soils that have saturated permeability of less than 37 cm/day (0.6 in/hr) within a depth of 90 to 120 cm (3 to 4 ft) below the bottom of the absorption system generally require great care in the design, construction and management to prevent malfunction because of low permeability. Those with ''saturated'' permeability of more than 366 cm/day (6 in/hr) may also require special consideration to prevent too rapid percolation of effluent which permits the contamination of ground water. There is a need, however, for obtaining data on benchmark soil series concerning

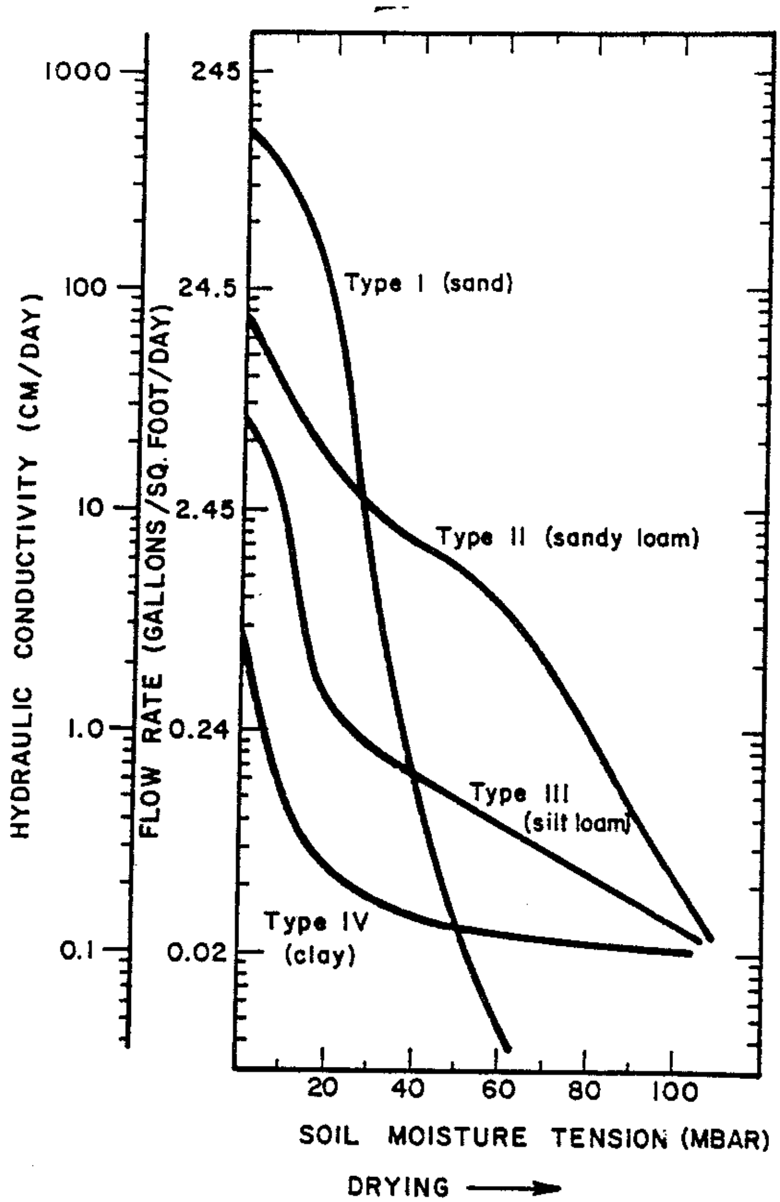


Fig. 1 Hydraulic Conductivity (K) as a Function of Soil Moisture Tension Measured in situ With the Crust-Test Procedure (Bouma et al. 1974a)

unsaturated permeabilities to make more accurate predictions about how well absorption fields function in different kinds of soils. Bouma et al. (1974a) gives a detailed description of the crust-test procedure for measuring hydraulic conductivities of unsaturated soil in situ which provides a means for obtaining this information.

Depth to Seasonal High Water Table

Unsaturated flow of the effluent through a thickness of at least 60 to 90 cm (2 to 3 ft) of soil below the absorption system is generally considered necessary for proper treatment of the effluent (Tyler et al. 1977). Therefore, depth to the seasonal high water table should be 1.5 to 1.8 m (5 to 6 ft). Ground water levels above this depth restrict the thickness of unsaturated flow. If wastewater is applied at too high a rate, mounding of the water table may occur reducing the unsaturated soil thickness. Included in the computerized inventory of soil properties that the Soil Conservation Service (1975) is making on the soil series of the United States is a record of depth to water table, kind of water table, and month(s) of occurrence.

Depth to Bedrock

If depth to bedrock is less than 1.5 to 1.8 m (5 to 6 ft), it is generally considered to present a site limitation that requires careful design of the system to overcome. If bedrock is fractured and there is insufficient soil depth above the bedrock to properly treat the effluent, the untreated or partially treated effluent may contaminate ground water. Unfractured or impermeable bedrock may cause mounding of the ground water below the absorption field, resulting in saturation of the treatment zone.

Depth to Cemented Pan

Cemented pans within the soil profile must be recognized because their effect on the performance of absorption fields is similar to that of shallow impermeable bedrock. Again, care must be used in the design of absorption systems if a cemented pan is shallower than 1.5 to 1.8 m (5 to 6 ft).

Soil Slope

Excessive slope may cause lateral seepage and surfacing of the effluent in downslope areas (Bouma et al. 1972) especially if slowly permeable layers are exposed down slope. Soil erosion and soil slippage may also be hazards where absorption fields are installed in sloping soils. Soils in the higher landscape positions may transmit water both by runoff and by internal lateral movement soils in lower landscape positions. Therefore, if absorption fields are placed in lower landscape positions, they must handle direct precipitation as well as the water they receive from soils at higher elevations, unless special designs are used to cut off surface or lateral subsurface flow (Ransom et al. 1975). Generally, special design is required on slopes greater than 15 percent.

Stoniness

Stoniness is a problem primarily because it can hinder construction of the system. However, if there are too few fine particulates (sand, silt, and clay) to fill the voids between the stones, the filtering capacity may be insufficient to treat the effluent.

METHODS OF WASTEWATER APPLICATION

To insure that the objectives of absorption and treatment are met over a long system lifetime, the method of wastewater application to the infiltrative surface must be compatible with the local soil and site characteristics. That is, suitable unsaturated conditions must exist for at least 90 cm (3 ft) below the infiltrative surface at all times without excessive clogging occurring. There are three basic methods of wastewater application for which a distribution network can be designed: (1) continuous ponding, (2) dosing and resting, and (3) uniform application without ponding. Each has its own advantages and disadvantages.

Continuous Ponding

As its name implies, this method of application maintains a head of wastewater above the infiltrative surface. The depth of liquid above the infiltrative surface can rise and fall but seldom, if ever, is the bottom surface exposed to air. Such a method has the advantage of increasing the effective infiltrative area by submerging the sidewall of th e absorption system. It also increases the hydraulic gradient across the infiltrative surface which may increase the infiltration rate.

If adequate treatment is to be achieved, this method requires a clogging

mat at the infiltrative surface to prevent saturated flow conditions in the underlying soil. With time, the mat develops but because none exists during initial operation, ground water contamination by pathogenic organisms and viruses can occur. This is particularly true in coarse grained soils such as sands (McCoy and Ziebell 1975, Green 1976). On the other hand, this method of application may be suitable prior to the formation of the clogging mat in more slowly permeable soils. A problem may arise, however, once the mat forms because it may become too restrictive since there is no aeration of the infiltrative surface. Despite these shortcomings, this method is usually employed.

Dosing and Resting

There is substantial evidence that continuous ponding leads to more severe clogging than if the clogging mat were to remain at least intermittently aerobic (Bendixen et al. 1960, Winneberger et al. 1960, and Thomas et al. 1966). To provide reaeration, periods of loading are followed by periods of resting with cycle frequencies ranging from hours to months. The resting phase allows the soil to drain and reaerate, thus encouraging rapid degradation of the clogging mat. This operation may extend the life of an absorption system or reduce the infiltrative surface area by keeping the clogging mat resistance to a minimum.

Early laboratory work with lysimeters showed repeatedly that reduction in the infiltrative capacity of the soil proceeds more slowly when periods of ponding were alternated with periods of aeration (Bendixen et al. 1950, Winneberger et al. 1960, and Thomas et al. 1966). Contrary to these findings, Kropft et al. (1975, 1977) report that total flow through the clogging mat remained higher in constantly ponded soil columns than in columns subjected to intermittent flooding. Similar results were obtained by Perry and Harris (1975) and Jawson (1976) when comparisons were made between soil columns aerated below the infiltrative surface and those that were not. The aerated columns showed that effluent application regimes characterized by short term alternating anaerobic-aerobic conditions may result in reduced infiltration associated with the formation of an intense clogging during the aerobic resting phase. Once clogged, restoration of the infiltrative surface by resting requires at least three to four weeks in sands (Perry and Harris 1975). The required resting period may be longer in finer-textured soils.

These results may not be as contradictory as they first seem. The oxidation-reduction potential in and around the clogging mat may be critical to maintaining high infiltration rates. Initially, cycles of dosing and resting maintain higher redox potentials in the soil than continuous ponding, which retards the development of the clogging mat as found by Jones and Taylor (1965). However, if clogging is allowed to proceed, the organics accumulated during periods of dosing may be too great for complete aerobic digestion during the resting phase. With an ample food supply, the aerobic and facultative organisms rapidly convert the clogging agents to new cell mass and slime which become new clogging agents. To prevent this, longer periods of aeration or more uniform distribution may be necessary to realize any benefits of dosing. This operation would require two alternating beds. Kropft et al. (1975) and Healy and Laak (1974) argue, however, that the total volume of liquid absorbed by two alternating systems operating at higher infiltration rates is no greater than that absorbed by one continuously ponded system of equal total area.

The laboratory results have not yet been validated in the field. Limited data from existing dosing systems indicate that the mechanisms may be even more complex than indicated. Bouma et al. (1975) and the University of Wisconsin (1977) reported that a system constructed in a silty clay loam soil with a percolation rate of 12 min/cm (30 min/inch) was still operating satisfactorily when dosed once daily at a loading rate one-third greater than that recommended by the Manual of Septic Tank Practice (U.S. Public Health Service 1967).

When excavations were made to determine the extent of clogging, evidence of worm activity was observed in the clogging mat which seemed to reopen the infiltrative surface. This activity could only occur during periods of rest. Obviously, more field work is required with different cycles of dosing and resting before conclusions can be drawn.

Uniform Application Without Ponding

The optimum method of application would seem to be one that distributes the liquid uniformly over the entire infiltrative surface at a rate lower than that at which the soil can accept liquid. The soil, therefore, would always remain unsaturated even during initial use of the system and aerobic conditions would always prevail at the infiltrative surface keeping the resistance of the clogging mat to a minimum. The sidewall is lost as an infiltration surface but this can be compensated for if higher infiltration rates can be maintained. Systems employing this method of application have not developed clogging mats after three years of use while maintaining adequate treatment (Converse et al. 1974, University of Wisconsin 1977).

RECOMMENDED LOADING METHODS FOR DIFFERENT SOIL AND SITE CONDITIONS

Uniform application without ponding would seem to be the best loading method for most soil and site conditions. In rapidly permeable soils, this method is essential to insure adequate treatment during initial operation when no clogging mat is present (Bouma 1975). In fine textured soils, however, it may not be possible to maintain a loading regime of uniform application without ponding. A system designed to utilize this method of application may revert to a continuously ponding regime due to excessive clogging. Continuous ponding may be necessary in fine textured soils to provide the necessary gradient across the clogging mat to absorb all the westewater. More research is needed to make this determination.

Dosing and resting loading regimes may be appropriate where absorption is the principal concern. This is true only if dosing and resting, either on a daily schedule or on a monthly or yearly schedule using alternating absorption systems, actually retard clogging. Again, more research is needed. Limited laboratory and field data seem to be contradictory on this point. Dosing and resting systems probably should not be used in highly permeable soils with a high water table unless small, frequent doses are applied each day (Bouma 1975). Long periods of aeration would permit the clogging mat to degrade, thus allowing pollutants to penetrate to the ground water, as is the case during initial operation of a system where no clogging mat is present.

Site limitations may be present which require loading methods that spread the wastewater over a large area. Examples of these site limitations are high ground water or shallow, impermeable bedrock or cemented pan where groundwater mounding may occur to reduce the unsaturated depth of soil. Uniform application without ponding would be best, although dosing and resting may be suitable if the dosing volume necessary to pond the whole infiltrative surface is not excessive. On steeply sloping sites, uniform application without ponding would be the most appropriate to prevent seepage downslope. Very stony sites should be avoided unless suitable filtering material is brought in.

DISTRIBUTION NETWORKS DESIGNS

Many different distribution network designs have been used in soil absorption systems all with the intent of uniformly applying liquid over the entire infiltrative surface. This rarely is achieved, but it may not always be necessary. The designs include: large diameter perforated pipe networks, pressure distribution networks, and other proprietary designs. The choice of

one over the other depends upon the loading regime desired.

Large Diameter Perforated Pipe Networks

The conventional distribution system for seepage bed or trenches consists of agricultural drainage tile. In the past, short sections of 10 cm (4 in) pipe were used spaced about 1.3 cm (0.5 in) apart, with a paper covering the top half to keep the soil from entering the pipe. These were usually laid on a 0.167 to 0.33 percent slope. Effluent flowed into the pipe by gravity. The other type of pipe, which is more popular today, has two rows of holes near the invert 45° off vertical center. These holes are 1.3 to 1.6 cm (0.50 to 0.63 in) in diameter, and spaced 7.6 cm (3.0 in) apart. The pipe is laid level or on a 0.167 to 0.33 percent slope. In a trench system one pipe is normally laid down the center. In a bed system several pipes are laid on 0.9 to 1.8 m (3 to 6 ft) centers (Fig. 2). In a multi-trench or bed system, the pipes are interconnected by a common solid header pipe, drop box or distribution box (Fig. 3).

The purpose of laying the pipe on a true, prescribed slope and a prescribed distance apart is to get uniform distribution as the effluent trickles or flows in by gravity. However, this is not the case; McGauhey and Winneberger (1964) and Bouma et al. (1972) observed nonuniform distribution. As it flows into the pipe, effluent seems to exit out of a few holes either at the inlet area, middle, or far end of the trench. This causes localized overloading where small areas receive a more or less continuous trickle of effluent. Adequate treatment by the soil is not achieved because saturated flow conditions are created. Soon, biological clogging occurs and reduces infiltration below the rate at which effluent is discharged. The effluent is forced to flow along the bottom of the trench or bed until it reaches an unclogged area. This phenomenon, known as "creeping failure", continues until the

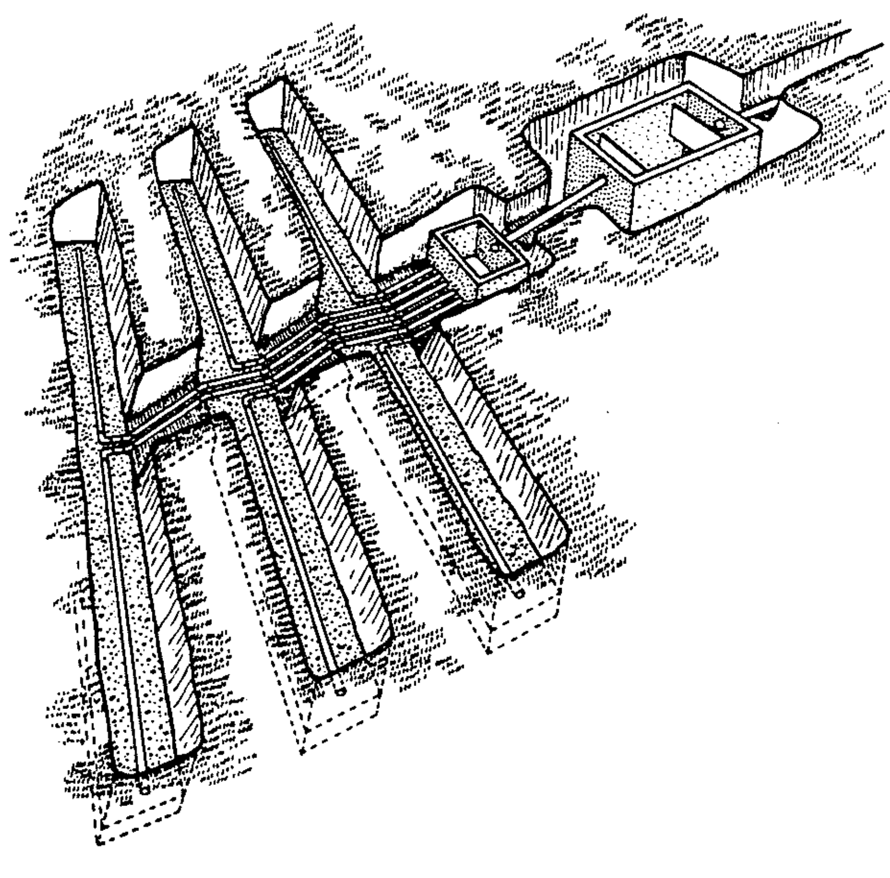


Fig. 2. A Trench System on a Sloping Site Utilizing a Distribution Box

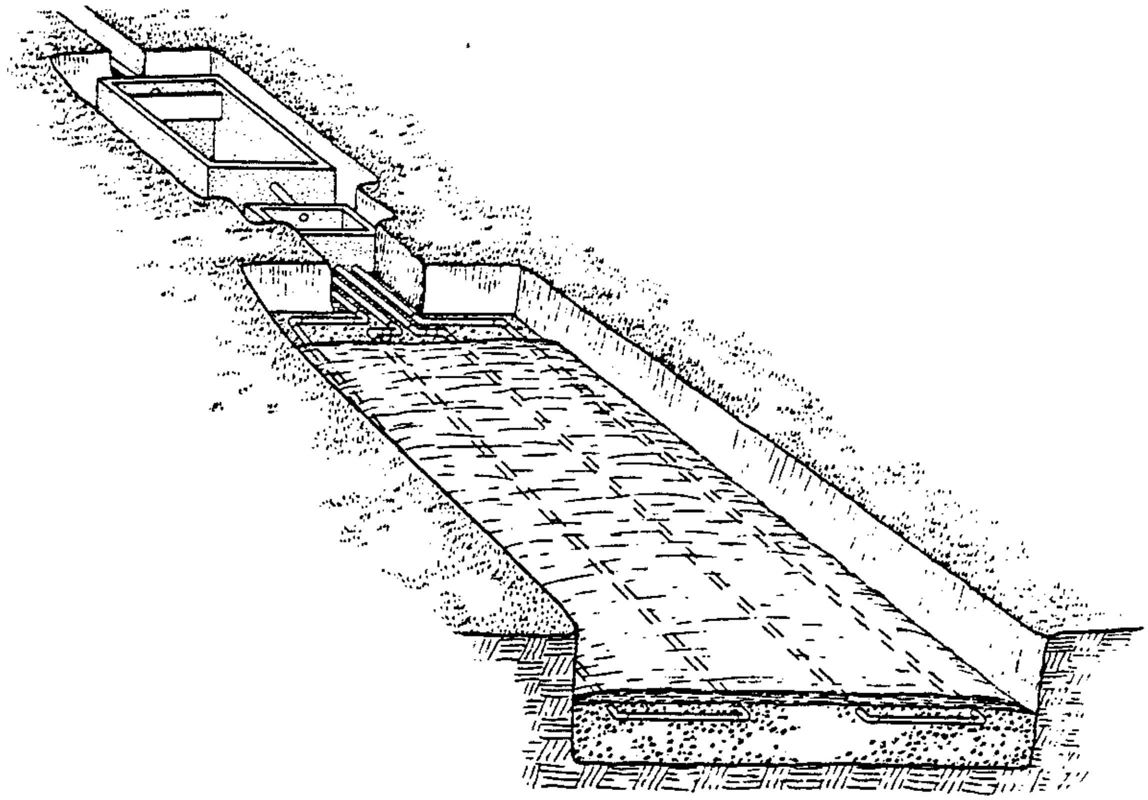


Fig. 3. A Soil Absorption Bed Using a Distribution Box

total bottom area of the system is clogged (Fig. 4). ''Creeping failure'' is somewhat of a misnomer, for if the system is sized such that the loading rate is less than or equal to the infiltration rate through the clogged soil, the system will not fail but will continue to function satisfactorily. However, when the loading rate exceeds the infiltration rate, surface seepage and failure result.

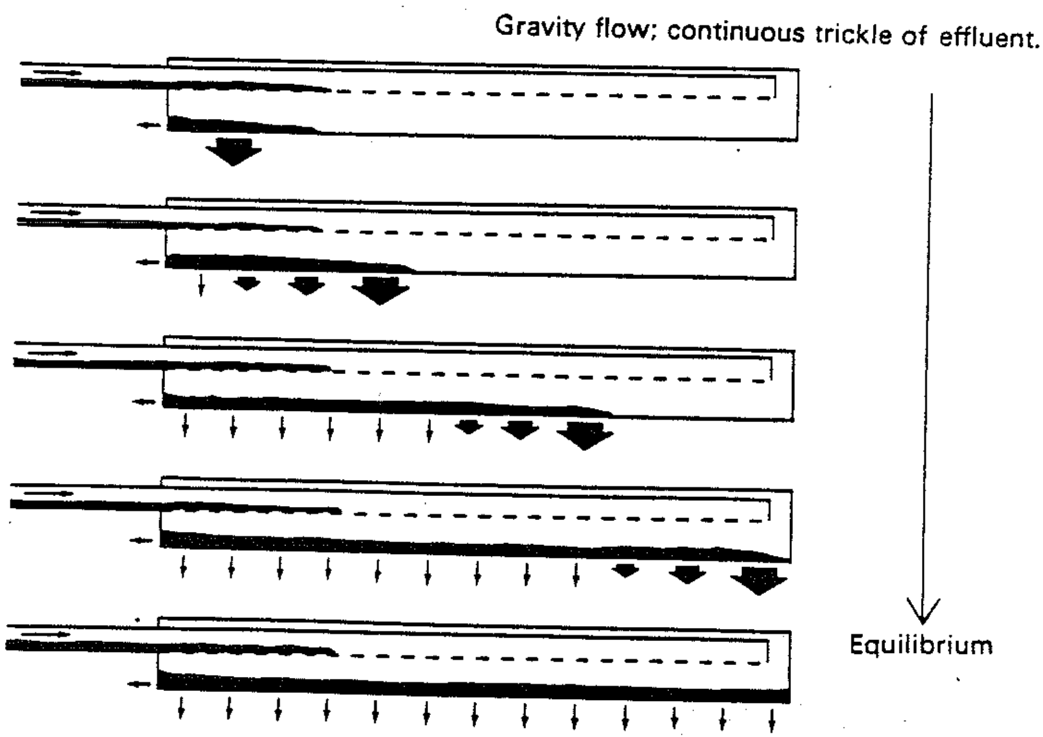


Fig. 4. Progressive Clogging of the Infiltrative Surfaces of Subsurface Absorption Systems (Bouma et al. 1972)

To verify the field observations of poor distribution, a full-size gravel trench was set up in the laboratory (Converse 1974). The gravel trench was constructed so that water passing through the gravel could be measured at each 45 cm (18 in) segment along the length of the trench. Gravity flow and pressure flow were measured.

Figure 5 shows the distribution of effluent by gravity through a 10 cm (4 in) bituminous pipe with 2 rows of holes located near the invert. Very poor distribution resulted with most of the water leaving the pipe at the inlet. When the water was pumped in this pipe at the rate of 48 L/min (13 gpm), 97 percent of the water was distributed over 53 percent of the bed with the majority in the first 3.1 m (10 ft) of the 14.6 (48 ft) long pipe (Fig. 6). Laying the pipe on a flatter slope resulted in less than 45 percent of the trench receiving effluent.

Rotating the pipe 180° so that the two rows of holes were upward did not improve distribution much, especially with gravity flow. Gravity distribution through a 10 cm (4 in) pipe with one row of holes located at the crown gave poor distribution when holes were spaced 7.6 cm (0.25 ft) apart. Distribution uniformity increased as hole spacing increased to 90 cm (3 ft) but still was dependent on hole elevation. The holes of lower elevation discharged more water (Fig. 7). Dosing into this same pipe with the holes at the crown and spaced 45 cm (1.5 ft) and 90 cm (3.0 ft) gave good distribution along the length of pipe (Fig. 8). A minimum flow rate of 95 L/min (25 gpm) for at least 2.5 minutes was recommended for hole spacing of 90 cm (3.0 ft).

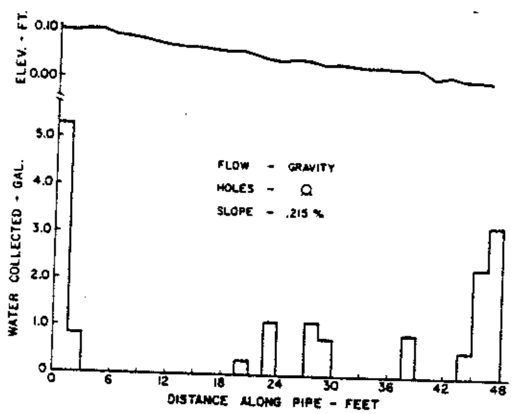


Fig. 5 Distribution of Water Flowing by Gravity Along 4 in Perforated Bituminous Pipe (Converse 1974)

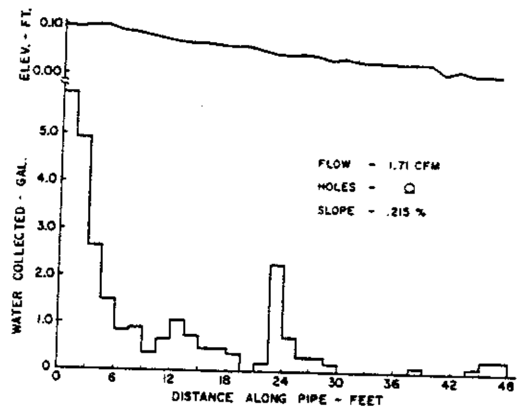


Fig. 6. Distribution of Water Pumped at 1.71 cfm Along a 4 in Perforated Bituminous Pipe (Converse 1974)

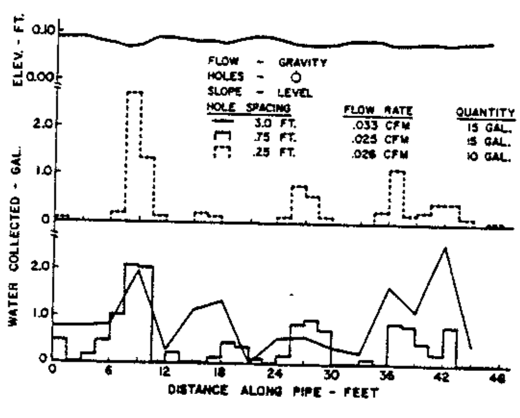


Fig. 7. Distribution of Water Flowing by Gravity Along a 4 in Perforated Bituminous Pipe with the Holes at the Crown (Converse 1974)

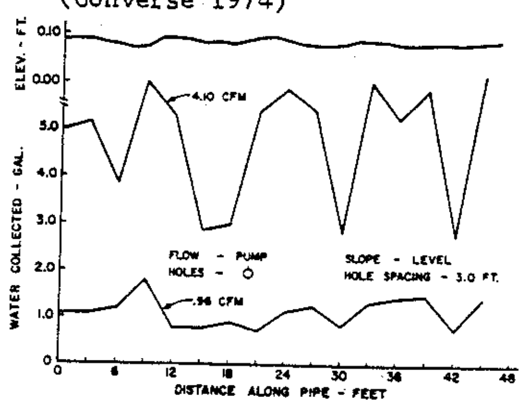


Fig. 8. Distribution of Water Pumped at 0.96 and 4.1 cfm Along a 4 in Perforated Bituminous Pipe with Holes at Crown (Converse 1974)

These results indicate that the function of the 10 cm (4 in) perforated pipe is merely to convey the effluent to the trench or bed. Laying the pipe at a prescribed uniform slope and spacing is of no value because the effluent will exit the hole of lowest elevation. Dosing does not greatly improve distribution because of the large number of holes.

When more than one distribution pipe is required, some means of dividing the flow between pipes is necessary. Interconnecting the pipes in closed loops is used but would seem to be ineffectual since the liquid quickly exits the pipe and travels along the infiltrative surface. Interconnecting trenches so that the infiltration surfaces are continuous and at the same elevation is certainly wise, but the additional pipe seems unnecessary.

Distribution boxes have also proven to be ineffectual (Coulter and Bendixen 1958). The distribution laterals all enter the distribution box which receives the effluent as it flows from the septic tank. If the laterals are laid on identical slopes and the inverts remain at the same elevation, equal distribution between the pipes should occur. In practice, however, it does not. One lateral usually receives most of the flow due to poor construction or differential settling of the box. When clogging causes the trench to pond, the flow backs up into the box and forces liquid into some other pipe. The U.S. Public Health Service (1967) discarded this technique because poor distribution is obtained and the full capacity of the system is not used.

Further Public Health studies indicated that for sloping sites serial distribution offers many advantages (Sullivan et al. 1959). A series of trenches is used with each trench constructed along the contour at successively lower elevations. All the effluent is discharged to the first trench. An overflow line is arranged so that the trench is forced to pond to the full depth of the gravel before the liquid flows to the next lower trench. This can be done by providing relief lines or drop boxes (Fig. 9). This design uses the full absorptive capacity of each trench sidewall and promotes the maximum hydrostatic head to force water into the surrounding soil. However, serial distribution systems may result in more severe clogging of the first trench in the system because of heavier solids load and deeper ponding in the trench. This problem can be alleviated if drop boxes are used, which permit any trench in the system to be shut off for resting and rejuvenation of the system. Serial distribution should not be used in rapidly permeable soils because of the poor filtering capacity of heavily loaded trenches.

Large diameter perforated pipe networks are best suited for continuously ponded or dosing and resting loading regimes. However, requirements for uniform slopes and specific spacings of the pipe seem unwarranted, only adding to the cost of the system. For dosing applications, the critical factor is to discharge by pump or siphon a sufficiently large volume of liquid with each dose to submerge the entire infiltrative surface. This is contrary to most guidelines which recommend dosing volumes based on pipe volume alone.

Pressure Distribution Networks

For uniform application the distribution network must be designed so that the volume of water passing out every hole within the network is identical. This design permits much better control of application rates and prevents local saturated conditions.

This is most easily done by putting the network under pressure and sizing the pipe and hole diameters to balance the headlosses to each hole. Rules of thumb used are: (1) to assume at least 60 to 90 cm (2 to 3 ft) of head at the terminal end of each lateral, (2) to assume that 65 to 85 percent of the total headloss in the network occurs crossing the orifice, and (3) to assume that 10 to 15 percent of the total headloss occurs in delivering the liquid to each hole. The remaining headlosses would occur through fittings. By

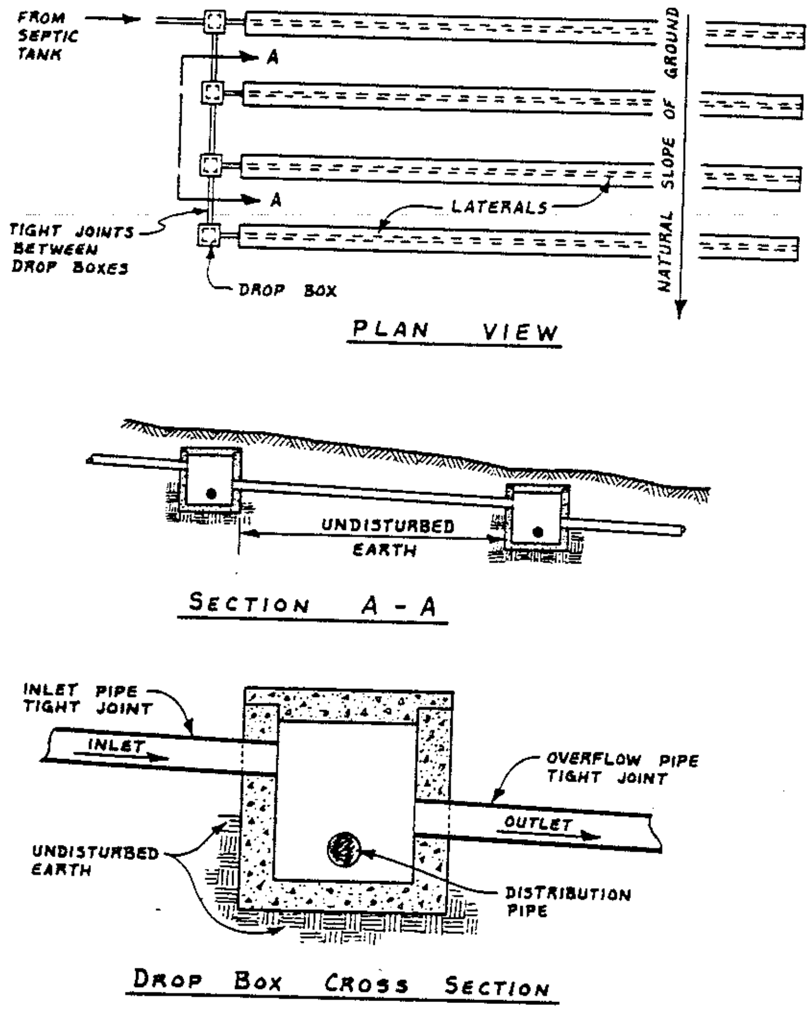


Fig. 9. A Serial Distribution System for a Sloping Site With Drop Boxes

using the sharp-edged orifice discharge equation, the headlosses are balanced throughout the system by beginning at the most remote lateral of the network and accumulating flows and headlosses. A design example is given in the appendix.

When this analysis is complete, a total headloss within the system and the required flow rate are known. This information is used to size the pump or siphon. To further insure uniform distribution the total dosing time should be 8 to 10 times longer than that necessary to fill the pressure network at the operating discharge rate.

Machmeier (1975) developed a computer program to determine pump size and maximum lateral lengths for various pipe diameters and layouts. Uniform distribution criterium used was to limit variation in discharge between the holes at the supply and distal ends of the lateral to 10 percent. Pipe diameters of 2.5 cm (1 in), 3.2 cm (1.25 in) and 3.8 cm (1.5 in) and hole diameters of 0.48 cm (0.19 in), 0.56 cm (0.22 in) and 0.64 cm (0.25 in) were evaluated. Only single lateral networks were considered.

Table 1 gives maximum lateral length for the various diameter laterals for three hole diameters and two hole spacings. For the 2.5 cm (1 in) pipe with 0.64 cm (0.25 in) holes spaced 75 cm (30 in) apart, a maximum lateral length of 7.5 m (25 ft) is recommended. Based on their field work, Converse et al. (1974) also recommended a maximum length for 2.5 cm (1 in) diameter laterals

TABLE 1. Allowable Lateral Lengths for Three Pipe Diameters, Three Perforation Sizes and Two Perforation Spacings (Machmeier 1976)

Perforation		Pipe diameter		
Spacing cm (in)	Diameter cm (in)	2.5 cm (1 in)	3.2 cm (1-1/4 in)	3.8 cm (1-1/2 in)
		m (ft)		
75 (30)	0.48 (0.19)	10.4 (34)	15.6 (52)	21.3 (70)
	0.56 (0.22)	9.1 (30)	13.7 (45)	17.4 (57)
	0.64 (0.25)	7.6 (25)	11.6 (38)	15.2 (50)
90 (36)	0.48 (0.19)	11.0 (36)	18.3 (60)	22.9 (75)
	0.56 (0.22)	10.1 (33)	15.5 (51)	19.2 (63)
	0.64 (0.25)	8.2 (27)	12.8 (42)	16.5 (54)

of 7.5 m (25 ft). Thus, the maximum length of trench or bed can be 16 m (53 ft) if the supply manifold is set in the center.

The discharge rate per unit area of infiltration surface from the lateral, assuming 90 cm (3 ft) lateral spacing, versus the head of water at the distal ends of the lateral is given in Table 2. It is plotted for the various hole and pipe diameters with 75 cm (30 in) hole spacing in Fig. 10. This figure can be used to size the pump or siphon necessary to pressurize the network. For example, if 60 cm (2 ft) of head is to be maintained at the distal end of a lateral with 0.56 cm (0.22 in) diameter holes spaced 75 cm (30 in) apart, an absorption area of 33.5 m^2 (360 ft^2) requires a pump able to discharge

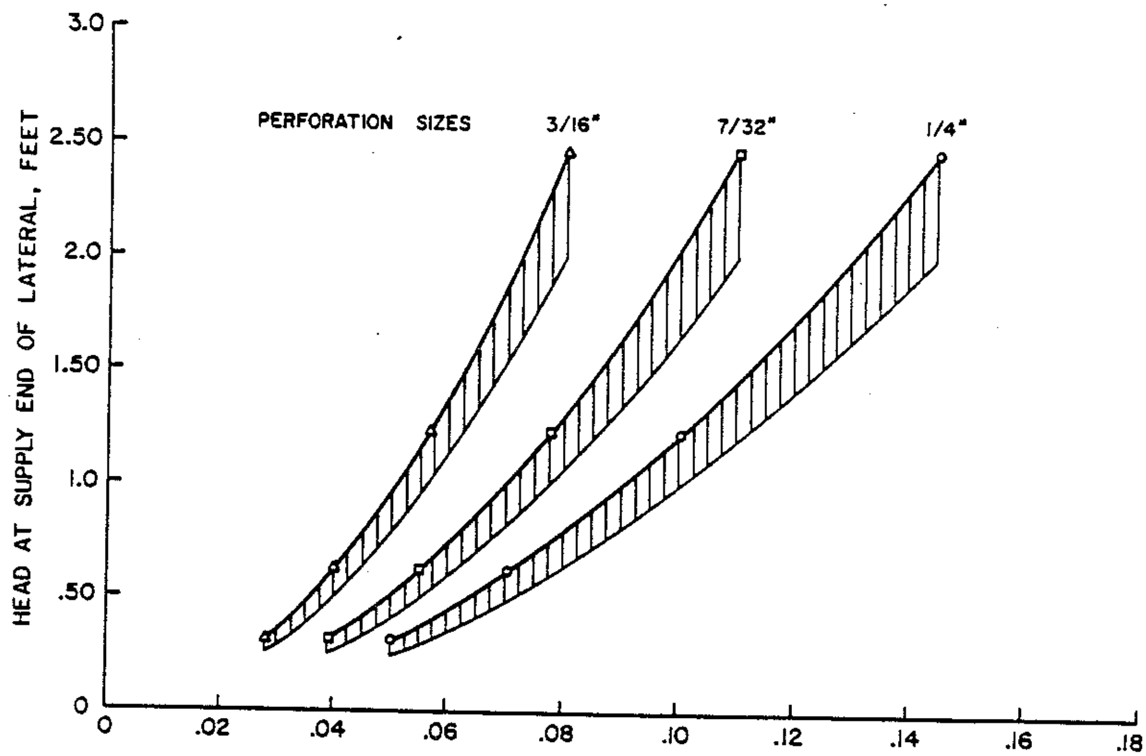


Fig. 10. Flow Rate Versus Head for Various Size Perforations Spaced 2.5 ft Apart with 3 ft Lateral Spacing (Machmeier 1975)

TABLE 2. Pumping Rates Per Unit Area of Infiltration Surface (gpm/ft^2) for Various Perforation Diameters and Spacing with 3 ft Lateral Spacing (Machmeier 1975)

Head at Supply end	Head at Distal end	Perforation Diameter 3/16-in	7/32-in	1/4-in
ft	ft	---------gpm/ft^2---------		
		Perforations spaced 2.5 ft apart		
0.31	0.25	0.0286	0.0391	0.0516
0.62	0.50	0.0409	0.0557	0.0728
1.23	1.00	0.0578	0.0787	0.1020
2.45	2.00	0.0817	0.1110	0.1450
		Perforations spaced 3.0 ft apart		
0.31	0.25	0.0240	0.0327	0.0428
0.61	0.50	0.0340	0.0462	0.0605
1.22	1.00	0.0481	0.0655	0.0854
2.43	2.00	0.0679	0.0926	0.1210

150 l/min (40 gpm) against a head of 75 cm (2.5 ft) plus the elevation difference between the pump and the lateral invert and any losses incurred delivering the liquid to the lateral. If only 30 cm (1 ft) of head is desired at the distal end, a pump able to pump 136 l/min (36 gpm) lateral would be required. Machmeier (1975) recommends maintaining a minimum head of 15 cm (0.50 ft) of water at the supply end of the lateral. However, this is too small because a slight change in lateral elevation will greatly affect flow. Instead, a head of at least 60 cm (2 ft) of water should be maintained at the distal end of the lateral. This will require a larger pump. If the pump used is not capable of supplying the desired flow rate against the total design head, it may be appropriate to use a smaller hole size. The charts given by Machmeier (1975) work well for small systems but they need to be expanded to size the manifold in networks with more than one lateral and for larger systems where the hole size and lateral spacing may be greater. In the larger systems, the holes in adjacent laterals should be located at the vertices of equilateral triangles.

Several pressure distribution systems were constructed to evaluate their performance for uniform distribution (Converse 1974). Figures 11 and 12 show the plan view and the distribution of effluent along each lateral, respectively, of one of the systems evaluated in the laboratory. Irregularities in holes due to drilling caused some variation in flow. The results demonstrate that distribution was much better than that provided by conventional methods.

Six pressure network systems were evaluated under field conditions (Converse et al. 1974). The principal problem encountered was undersizing of the pump which did not give sufficient head and flow. Hole plugging was not a problem with a properly sized pump.

Recently, one large network was designed and installed for an absorption bed serving the small community of Westboro, Wisconsin (Otis 1976). The

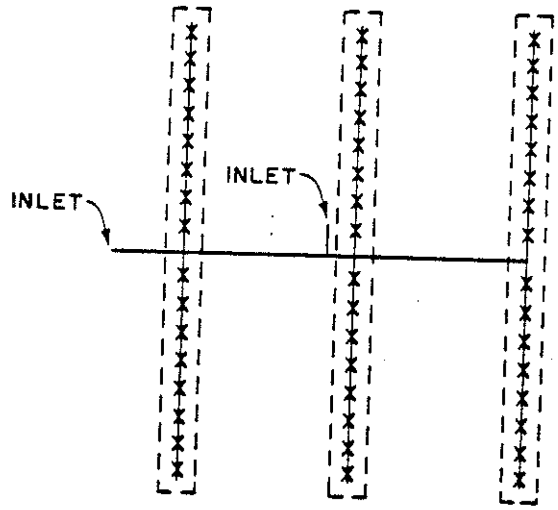


Fig. 11. A Pressure Distribution Network with a 7.5 cm (3 in) PVC Manifold with Six 2.5 cm (1 in) PVC Laterals. Each Lateral Has Eight 6.4 cm (0.25 in) Holes Spaced 7.5 cm (30 in) Apart. The Inlet is on the End or Center. Each 6 m (20 ft) Lateral is Spaced 4.5 m (15 ft) Apart. (Converse 1974)

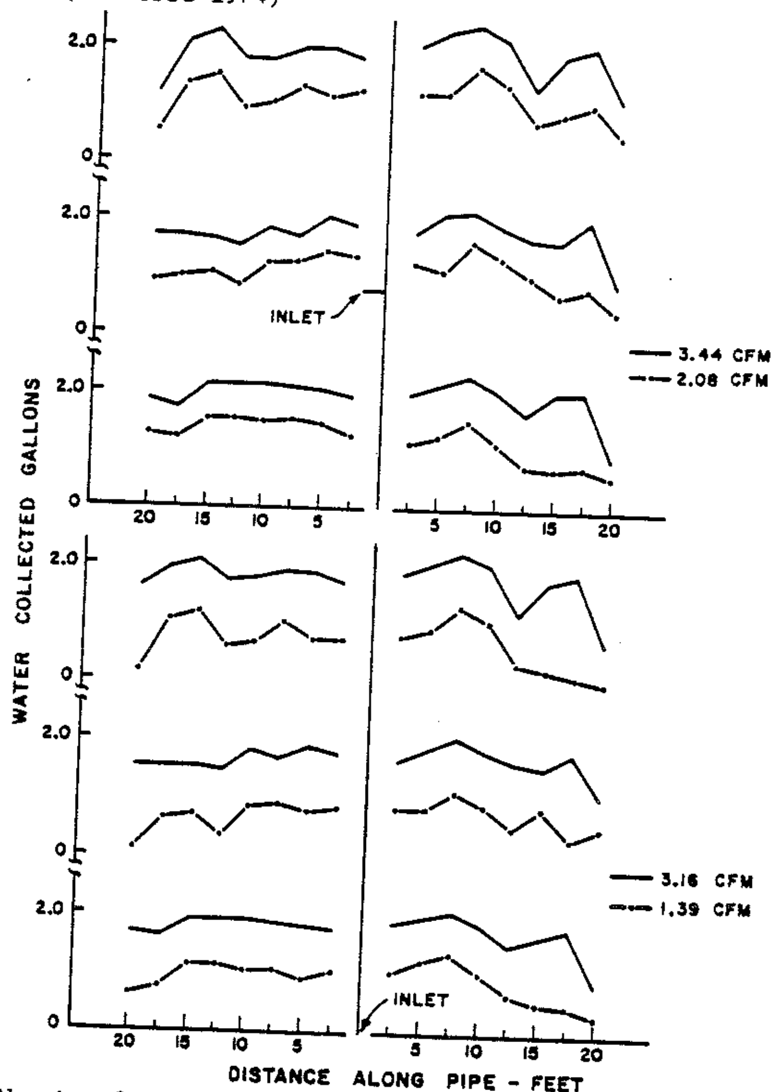


Fig. 12. Distribution for the Network Illustrated in Fig. 11. The Top Portion is the Distribution Pattern for a Center Inlet for Two Flow Rates. The Bottom Portion is the Distribution Pattern for an End Inlet for Two Flow Rates (Converse 1974)

bed has a total absorption area of 1210 m^2 (13,000 ft^2) and is dosed twice daily with 30 280 l (8,000 gal). Because of the large absorption area and concern for preventing groundwater contamination, a pressure distribution network was constructed. The network consists of two 20 cm (8 in) diameter manifolds that telescope down to 10 cm (4 in) over their 30.5 m (100 ft) length. Each manifold supplies nineteen laterals extending 11 m (36 ft) on both sides of the two manifolds. The laterals are 7.6 cm (3 in) in diameter with 1.2 cm (0.47 in) diameter holes spaced 2 m (6.5 ft) apart. The two 20 cm (8 in) manifolds are joined by a 30 cm (12 in) pipe leading from the dosing chamber. A 25.4 cm (10 in) siphon is used to dose and pressurize the network. Two identical networks are dosed by alternating siphons. Field tests have shown that distribution is uniform and no ponding occurs.

Pressure distribution networks may be the only alternative where rapidly permeable soils are used for absorption in areas where groundwater contamination is possible. Further field demonstrations are necessary to determine their value in other settings.

Other Distribution Techniques

Several other distribution systems have been developed in recent years that attempt to improve distribution to enhance soil absorption of effluent.

Case System: The Case System was developed in North Carolina and is being promoted for use in slowly permeable soils by the manufacturer. The Case System distributes septic tank effluent through a series of porous cement blocks. The blocks are laid end to end in a trench, cemented together for a distance of 12 to 18 m (40 to 60 ft) and covered with earth backfill. No gravel is used in the trench. Effluent moves in the hollow chambers of the blocks until it diffuses through the porous block walls into the surrounding soil. However, this system will not work where the soil remains wet much of the time or has very low hydraulic conductivity.

As part of the Case System, a two compartment septic tank is used. The first compartment serves as a traditional solids retention and stabilization compartment, and the second serves as a settling and dosing chamber. A siphon in the second compartment doses approximately 120 l (30 gal) of effluent at a time, thus giving greater circulation of effluent throughout the block trench and maximizing surface absorption area.

The Case System attempts to achieve uniform application. If successful, the loading to the soil would be more uniform than can be achieved with a pressurized network having widely spaced holes. However, failures have occurred when the block does not drain between doses. Therefore, the system must be carefully sized and constructed to insure complete absorption of effluent between doses. This is a difficult problem since the natural soil must be compacted around the block. Excessive compaction or puddling can result, thus significantly changing the hydraulic conductivity of the surrounding soil.

Panel System: A similar system is the prefabricated panel system recently introduced in North Carolina. It is composed of porous cement panels 3 m (10 ft) long, 1.2 m (4 ft) high, and 0.2 m (8 in) wide. The panel is a network of vertically spaced chambers with interconnected tubes at one end. The top wall of the panel has a removable cover for access and inspection of the interior. The panels are placed in a trench 30 cm (1 ft) wide and 120 cm (4 ft) deep. A bed of medium sand is laid in the bottom of the trench for the panel to rest on, and sand is packed between the sidewall of the panel and the trench. A small cap of surface soil is placed over the top of the panel at the ground surface. Effluent enters the top chamber of the panel and diffuses through the bottom and sidewall of the chamber.

As this chamber fills with effluent, an overflow tube passes the liquid into lower chambers. A typical arrangement of the system is a series of five connected panels in each of two trenches fed by a distribution box.

This system is designed to provide uniform distribution of effluent over the full length of the panel to minimize the clogging potential at the soil's infiltrative surface. As in the Case System, the clogging potential is further reduced by the retention of solids in the cement pores, but additional removals are achieved by absorption and aeration of the organic components in the sand backfill around each panel. Over the long term, this system would seem to give uniform application without ponding of the soil surface with the advantage that dosing equipment is not necessary.

Ameration Chamber: Another system that does not use gravel within the drainfield is the Ameration system. This system consists of concrete chambers with open bottoms that interlock to form an underground cavern 45 cm (18 in) high over the exposed infiltrative surface. No pipe or stone is used. The septic tank effluent is discharged into the cavern through a central weir, trough, or splash plate and allowed to flow over the surface in any direction. Vents provide a free flow of air directly to the soil surface between doses. Manholes in the roof of the chamber allow visual inspection of the soil surface and access for necessary maintenance.

The Ameration system provides a dosing and resting loading regime. Its advantages over the conventional system are that the infiltrative surface is more readily exposed to air and access to the surface is provided. However, in soil other than very coarse textured soils there would seem to be the danger of severe clogging from fine particles migrating during dosing.

Fuldos System: The Fuldos System is another system recently developed for effluent dosing. In this system, a 90 cm (3 ft) trench is dug and a concrete chamber approximately 45 cm (18 in) square is placed on a bed of stone and backfilled with stone. Effluent is dosed into the stone fill outside the chamber at such a rate that it will completely fill the void spaces in the stone. Any excess liquid flows through an inlet at the top of the concrete chamber and is stored until the liquid level drops in the trench. A oneway release outlet at the bottom of the storage chamber allows the liquid to flow out of storage as the level drops in the trench.

This system is designed to maximize sidewall absorption under a dosing and resting loading regime. However, the cost is about 4 to 5 times higher than conventional systems and may not be feasible for individual home owners. This system is best for larger operations, especially where flow is quite variable and the storage chamber can be better utilized.

SUMMARY

Laboratory and field studies have demonstrated conclusively that saturated soil conditions in shallow soils or in areas with high water tables can lead to ground water contamination. Continuously ponded or dosing and resting loading methods initially can create this hazard. After a clogging mat forms, however, unsaturated conditions prevail despite the loading method. Thus, the risk of a health hazard from ground water contamination over an initial start-up period must be carefully considered before a loading method is selected.

Improved methods, such as dosing and resting and uniform distribution without ponding, may reduce the resistance of the clogging mat. This would

be economically attractive because the size of the absorption system could be reduced or its life significantly prolonged. However, laboratory and field studies conflict on this point and further research is needed. Uniform distribution without ponding appears to be the best application method under most conditions. If the system is sized according to the soil's hydraulic conductivity, unsaturated flow through the soil is insured from the first day of operation. In addition, uniform distribution without ponding maximizes the aeration time of the clogging mat.

Regardless of the application method chosen, well established network designs exist. Other proprietary systems are available, each with its own advantages, but controlled testing of these has not been done.

APPENDIX

PRESSURE DISTRIBUTION NETWORK DESIGN PROCEDURE

1. Determine the total absorption area required and its dimensions from the estimated daily flow and soil conditions.

2. Lay out a network configuration. Laterals may be spaced from 2.5 to 10 feet apart, depending on the size of the system. The feeder manifold can be located at the end or the center of the laterals. The center is usually best to minimize pipe size when balancing headlosses. Orifices can be spaced 2.5 to 10 feet apart. At wide spacings, place them so that holes of adjacent laterals form vertices of equilateral triangles.

3. Size the orifice diameters and determine the discharge rate using the orifice discharge equation:

$$Q = CA\sqrt{2gh}$$

   where Q is the flow rate (gpm), C is the orifice discharge coefficient (0.6 for sharp-edged orifices), A is the hole area (ft^2), g is the acceleration due to gravity (32.2 ft/sec^2), and h is the headloss through the orifice. One to two psi should be maintained in the terminal end of each lateral. This sets h. By selecting an orifice size, Q can then be computed. Usually 1/4 in orifices are sufficient. The larger the hole, the larger the pump and pipe required, but in larger systems this may be desirable to keep discharge rates to the absorption system ahead of inflow to the dosing tank.

4. Size the lateral diameter. Starting at the terminal end of the lateral, work upstream computing headlosses in each section as flow is added.

5. Size the manifold diameter in the same manner as the laterals.

6. Check the difference between total headlosses through the first and last orifice in the network by adding headlosses for each section of manifold and lateral, starting at the upstream end. The difference between the total losses through any two orifices should be less than 15 percent. Fitting losses can be ignored.

7. Size the pump or siphon. The pump or siphon should be capable of supplying the necessary flow against the network losses plus the elevation head and delivery losses.

6. Determine the dosing volume. The volume discharged per dose should be 10 times the total pipe volume to minimize differences in the volume discharged from orifices while the network is being filled.

Design Example

Problem: To design a subsurface soil absorption system in sand to handle 15,000 gpd.

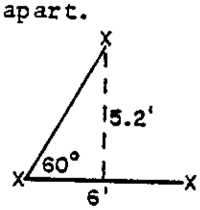
1. Required Bed Area & Dimensions

(Equilibrium infiltration rate for sands = 1.2 gpd/ft^2)

Area = 15,000 gpd ÷ 1.2 gpd/ft^2 = 12,500 ft^2
Dimensions: 100 ft x 125 ft = 12,500 ft^2

2. Network Layout

Place orifices at vertices of equilateral triangles spaced 6 ft apart.



Lateral spacing, $x = 6 \cos 30° = 5.2$ ft

Set lateral spacing at 5 ft for convenience.
Number of laterals = 125 ft ÷ 5 ft = 25

(Laterals could lie in the other direction in which case there would be 100 ft ÷ 5 ft = 20 laterals. This orientation could increase the lateral diameter required. The diameter of the lateral will vary directly with the total area of the orifices. Therefore, orifice diameter, spacing and number can be increased or decreased to increase or decrease the lateral diameter.)

3. Size Orifices and Calculate Discharge Rate

Select 3/8-in. diameter orifices (The diameter of the lateral will vary directly with the total area of all the orifices in each lateral. Thus, the lateral diameter is a function of the orifice size and number. The number will vary according to the spacing and length of the lateral).

Table 3 gives orifice discharge rates for various heads and orifice diameters using the sharp-edge orifice equation.

Maintain 1 psi (2.3 ft) of pressure in the end of each lateral.

$$Q = CA\sqrt{2gh} = (0.6)\left[(3/8)(1/12)(1/2)\right]^2 \pi \left[(2)(32.2)(2.3)\right]^{1/2} = 2.5 \text{ gpm}$$

(Assume this flow out each orifice. This is incorrect but changes in head between holes should be less than 15% affecting Q by $\sqrt{15\%}$ or less than 4%.)

TABLE 3. Orifice Discharge Rates for Various Orifice Diameters

Pressure ft.	Pressure psi	Orifice Diameter (inches) 1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2
		---gpm---							
1	0.434	0.05	0.18	0.41	0.74	1.15	1.66	2.26	2.95
2	0.867	0.07	0.26	0.59	1.04	1.63	2.34	3.19	4.17
3	1.301	0.08	0.32	0.72	1.28	1.99	2.87	3.91	5.10
4	1.734	0.09	0.37	0.83	1.47	2.30	3.31	4.51	5.89
5	2.168	0.10	0.41	0.93	1.65	2.57	3.71	5.04	6.59
6	2.601	0.11	0.45	1.01	1.80	2.82	4.06	5.53	7.22
7	3.035	0.12	0.49	1.10	1.95	3.05	4.39	5.97	7.80

4. Size Lateral Diameter

Use central manifold with lateral extending to either side. Therefore, only one half the lateral, from the manifold to one end, need be considered for sizing.

Lateral length = 100 ft

Number of orifices per 1/2 lateral = 100 ft ÷ (2)(6 ft)
= 8.33 or 8 orifices

Determine the total headlosses expected through perforated laterals of various diameters by developing Table 4. Use Table 6 to estimate headlosses by beginning with the distal end of the lateral. In this case, select the 2-in diameter lateral because of the excessive headlosses incurred in the 1.5-in diameter lateral (Table 4).

TABLE 4. Total Headlosses through 1.5-in and 2-in Diameter Laterals

Orifice No.[a]	Total Q in lateral segment	Headlosses in 1.5-in lateral ft/6 ft[b]	Headlosses in 1.5-in lateral Total	Headlosses in 2-in lateral ft/6 ft[b]	Headlosses in 2-in lateral Total
	gpm	ft	ft	ft	ft
1	2.5	0.0	0.0	0.0	0.0
2	5.0	0.01	0.01	0.0	0.0
3	7.5	0.02	0.03	0.01	0.01
4	10.0	0.04	0.07	0.01	0.02
5	12.5	0.07	0.14	0.02	0.04
6	15.0	0.09	0.23	0.03	0.07
7	17.5	0.12	0.35	0.04	0.11
8	20.0	0.15	0.50	0.04	0.15

[a] Orifice #1 is at distal end of lateral
[b] Ft/6 ft is friction loss in a 6 ft pipe sequence between orifices. This was computed using Table 6 which gives headlosses in ft/100 ft of pipe length.

5. <u>Size Manifold Diameter</u>

Number of Laterals = 125 ft ÷ 5 ft = 25

Determine headlosses in manifold in the same manner as the laterals by developing Table 5. Ignore fitting losses.

TABLE 5. Total Headlosses Through Various Size Manifolds

Manifold Segment[a]	Total Q in Segment[b]	Headlosses in Manifold							
		4-in diam		6-in diam		8-in diam		10-in diam	
		ft/5 ft[c]	Total	ft/5 ft	Total	ft/5 ft	Total	ft/5 ft	Total
	gpm	ft							
1	40	0.0	0.0		0.0				
2	80	0.02	0.02		0.0				
3	120	0.04	0.06	0.01	0.01				
4	160	0.07	0.13	0.01	0.02				
5	200			0.01	0.03	0.00			
6	240			0.02		0.01	0.04		
7	280			0.03		0.01	0.05		
8	320			0.03		0.01	0.06		
9	360			0.04		0.01	0.07	0.0	
10	400			0.05		0.01	0.08	0.0	
11	440			0.06		0.02		0.01	0.09
12	480					0.02		0.01	0.10
13	520					0.02		0.01	0.11
14	560					0.02		0.01	0.12
15	600					0.03		0.01	0.13
16	640					0.03		0.01	0.14
17	680					0.03		0.01	0.15
18	720							0.01	0.16
19	760							0.01	0.17
20	800							0.02	0.19
21	840							0.02	0.21
22	880							0.02	0.23
23	920							0.02	0.25
24	960							0.02	0.27
25	1000							--	--

[a] Manifold segment 1 is at distal end of manifold.
[b] Each half lateral accounts for 20 gpm.
[c] ft/5 ft is friction loss in a 5 ft pipe segment between laterals. This was computed using Table 6, which gives headlosses in ft/100 ft of pipe length.

Manifold diameters can be selected from Table 5. Because of the large variation of flow along the length of the lateral it is most cost effective to reduce the size of the manifold as the flow is reduced. In this case, 75 ft of 10-in, 25 ft of 8-in, and 25 ft of 6-in pipe is used, producing a total headloss of 0.27 ft.

6. <u>Check Difference in Headlosses through First and Last Orifices</u>

Loss to first orifice = 2.3 ft (no manifold or lateral loss, only orifice loss)
Loss to last orifice = 0.27 ft(manifold) + 0.15 ft(lateral) + 2.3 ft(orifice)
= 2.72 ft
Percent difference = (2.72 ft - 2.3 ft) ÷ 2.3 ft x 100 = 18%

This is greater than 15%, but not excessively so. To reduce the difference in this example it would be most appropriate to reduce the orifice diameter rather than increase the manifold or lateral diameter.

7. <u>Size Pump or Siphon</u>

Total headloss in network = 2.72 ft
Discharge rate = 1000 gpm

Pump or siphon would be sized to deliver 1000 gpm against 3 ft of head at the network inlet. Elevation and friction losses incurred before the network inlet must be added to the 3 ft pumping head.

8. <u>Determine Dosing Volume</u>

Dosing volume should be at least 10 times the total pipe volume to minimize in discharge volumes from first and last orifices in laterals during filling.

Total pipe volume

$$= (7.5\ \text{gal/ft}^2)(\pi \div 4)\ (20)(100\ \text{ft})(3 \div 12)^2\ + 40\ \text{ft}(8 \div 12)^2 + 30\ \text{ft}(10 \div 12)^2$$
$$= 1180\ \text{gal}$$

Dosing volume = 10 x 1180 gal = 11,800 gal

This example demonstrated the design of a network where all the lateral inverts would be installed at the same elevation. If the network were to be installed on a sloping site, the differences in elevations of the laterals must be taken into account. This can be done best by using different hole diameters in each of the laterals so that the discharge rate from all holes are identical.

TABLE 6. Friction Loss in Schedule 40 Plastic Pipe (C = 150)

	Pipe Diameter (in)								
Flow	1	1 1/4	1 1/2	2	3	4	6	8	10
gpm	ft/100 ft								
1	0.07								
2	0.28	0.07							
3	0.60	0.16	0.07						
4	1.01	0.25	0.12						
5	1.52	0.39	0.18						
6	2.14	0.55	0.25	0.07					
7	2.89	0.76	0.36	0.10					
8	3.63	0.97	0.46	0.14					
9	4.57	1.21	0.58	0.17					
10	5.50	1.46	0.70	0.21					
11		1.77	0.84	0.25					
12		2.09	1.01	0.30					
		2.42	1.17	0.35					
14		2.74	1.33	0.39					
15		3.06	1.45	0.44	0.07				
16		3.49	1.65	0.50	0.08				
17		3.93	1.86	0.56	0.09				
18		4.37	2.07	0.62	0.10				
19		4.81	2.28	0.68	0.11				
20		5.23	2.46	0.74	0.12				
25			3.75	1.10	0.16				
30			5.22	1.54	0.23				
35				2.05	0.30	0.07			
40				2.62	0.39	0.09			
45				3.27	0.48	0.12			
50				3.98	0.58	0.16			
60					0.81	0.21			
70					1.08	0.28			
80					1.38	0.37			
90					1.73	0.46			
100					2.09	0.55	0.07		
125						0.85	0.12		
150						1.17	0.16		
175						1.56	0.21		
200							0.28	0.07	
250							0.41	0.11	
300							0.58	0.16	
350							0.78	0.20	0.70
400							0.99	0.26	0.90
450							1.22	0.32	0.11
500								0.38	0.14
600								0.54	0.18
700								0.72	0.24
800									0.32
900									0.38
1000									0.46

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