

PRESSURE DISTRIBUTION TO IMPROVE SOIL ABSORPTION SYSTEMS

J. C. Converse
Assoc. Member ASAE

J. L. Anderson

W. A. Ziebell

J. Bouma

Disposal and proper purification of septic tank effluent is the primary function of the soil absorption system. Localized overloading, caused by poor distribution of effluent in the system, will result in saturated flow and probable poor purification of effluent, especially in sands. Once the effluent reaches the ground water, pathogenic pollution of private well water supplies may occur leading to public health problems. Another reason for system failure may be due to inadequate infiltration caused by either overloading of the soil absorption bed in relatively slow permeable soils, or by soil clogging, which strongly reduces infiltration rates. Soil clogging problems can be reduced by introducing dosing regimes (Popkin and Bendixen, 1968; McGauhey and Krone, 1967; Bouma et. al., 1972), in which the effluent should be distributed evenly over the entire seepage area during each dosage (McGauhey 1968).

The emphasis of this paper will be (i) on the aspects relating to purification as a function of different flow rates in soil and (ii) design criteria and evaluation of six pressure distribution systems installed in the field.

Types of Soils Most Applicable for Pressure Distribution

Not all types of soils require equal distribution and dosing to eliminate poor purification and soil clogging. This is evidenced by the fact that many systems have operated satisfactorily for many years without failure. Inadequate purification is of particular concern, in natural sands or in sand fills in mounds for a period of time after construction, before clogging occurs, which may take several years. During this period local overloading and poor purification occurs when the conventional system for effluent distribution is used.

The physical effect of clogging in weakly structured sandy loams and loams appears to be more severe than those occurring in well-structured silt loams and clays. Pressure distribution is, therefore, more relevant in these weakly structured soils than in the well-structured soils (Bouma 1974). In these soils, soil purification is much less of a problem than soil clogging because of the high absorptive capacity of the soil.

The authors are: J. C. Converse, Assistant Professor, Ag Engineering Dept.; J. L. Anderson, Research Assistant, Soil Science Dept.; W. A. Ziebell, Specialist, Bacteriology Dept.; J. Bouma, Associate Professor, Soil Science Dept. and Geological and Natural History Survey, University of Wisconsin - Extension; University of Wisconsin-Madison.

Relationships Between Loading Rates and Travel Times in Sand Fill

To evaluate the relationships between loading rates and travel times of effluent through sand fill underlying a seepage bed, a .61 m (2 ft.) high column was prepared. The column had a diameter of 10 cm. (4 in.) and was filled with a coarse sand simulating the fill occurring in mound systems (Bouma et. al., 1975, 1975).

Moisture retention characteristics of the coarse sand are shown in Fig. 1A. After the column was saturated and allowed to drain for 24 hours, equilibrium moisture contents were determined from the moisture retention curve and confirmed by in situ tensiometry (Fig. 1B) as described by Otis et. al. (1974).

The total volume of water present in the column at equilibrium was 720 cc., the volume of air-filled pores was 1420 cc. giving a total pore volume of 2140 cc.

Specific volume doses of 300 ppm chloride solution were applied to the column to test the effect of different loading rates on travel times of effluent through .61 m (2 ft.) of sand. The chloride content of column effluent was monitored until the 300 ppm level was reached. Repeated applications of chloride solution were made only after re-establishment of hydraulic equilibrium. The columns were washed with water before a new series of experiments were started by using dosages of different specific volumes of liquid.

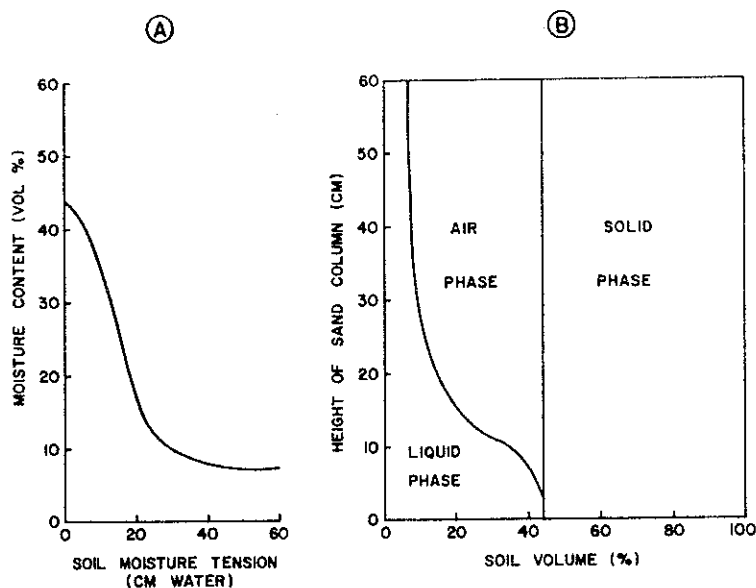


Figure 1. Moisture Retention Characteristics of the Coarse Sand Fill and a Representative of Equilibrium Moisture Contents.

Travel times associated with saturated flow were tested and those associated with the following dosages: 25 cm (10 in.); 10 cm (4 in.); 5 cm (2 in.); and 2.5 cm (1 in.).

Column effluents were monitored to determine the time of first appearance of chlorides in the effluent, and the times when the 150 ppm and 300 ppm level were reached. Soil moisture tensions were measured at a depth of 5 cm (2 in.) in the column. For the 5 cm (2 in.) dose, tensiometer measurements were made at four depths in the column 5 cm (2 in.), 10 cm (4 in.), 25 cm (10 in.) and 50 cm (20 in.) (Fig. 2). Cumulative outflows were measured for the time of first appearance of chloride in the effluent and for the time when the chloride concentration reached 300 ppm.

The times of appearance of chlorides in column effluent are transposed in Fig. 3 on the soil moisture tension readings for the different dosing rates. The 2.5 cm (1 in.) dose has the longest travel time, assuming that each cycle represents a once-a-day dosage. It has been shown that longer travel times result in more effective purification because of longer and better contact between effluent and soil particles (Bouma et. al., 1972). Seepage areas are designed to receive 5 cm (2 in.) of effluent per day (Bouma et. al., 1972). This 5 cm (2 in.) per day dosage corresponded with the second longest travel time (Fig. 3).

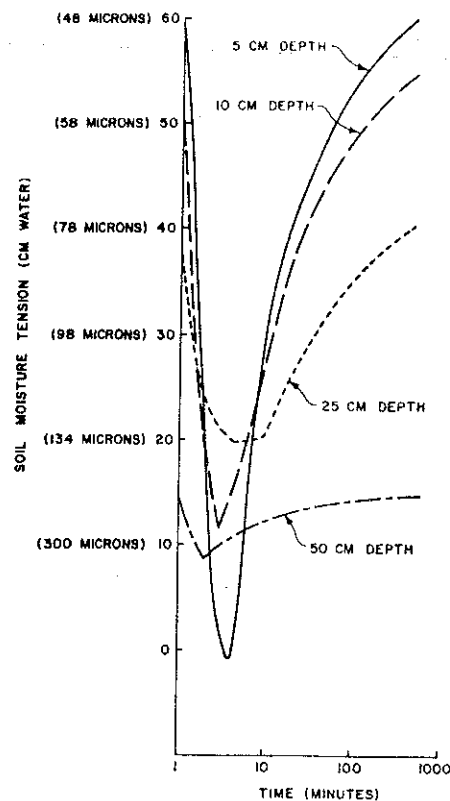


Figure 2. Tensiometer Measurements at Four Depths Over Time Following a 5 cm. Dose.

Cumulative outflows measured at the first appearance of chloride in the effluent and at 300 pp chloride, confirm the findings of Otis et. al. (1974) that approximately all of the water present in the column at equilibrium is displaced before the effluent added moves out of the column. This can be attributed to processes of capillarity where water applied to a soil will try to flow into the smaller pores, displacing the liquid there, because they exert the largest capillary forces. Under saturated conditions all pores are initially filled with water. There is a corresponding increase in the amount of water displaced before the effluent moves out of the column (Table 1).

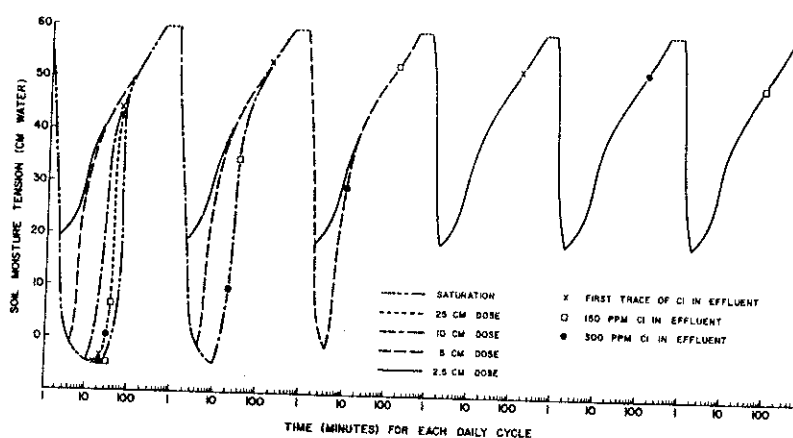


Figure 3. Tensiometer Measurements for Four Different Dosing Rates Over Time Assuming a Daily Dose. Travel Times of Effluent Through the Column are Illustrated for Each Dosing Rate.

TABLE 1. DATA ON COLUMN STUDIES FOR DETERMINING TRAVEL TIMES AS A FUNCTION OF THE DOSING REGIME USING CHLORIDE.

Exp. No.	Vol. of Daily Dose cc (cm)	Time of First Chloride Appearance hr.	Cumulative Outflow to First Chloride Appearance cc	Time of Appearance 300 ppm Chloride hr.	Cumulative Outflow to 300 ppm Chloride cc
1	4800 (60) (Saturation)	.35	1200	.50	1870
2	2000 (25)	.36	690	.58	1140
3	800 (10)	1.5	618	25	1018
4	400 (5)	26	675	50	1025
5	200 (2.5)	74	652	121	1010

Tensiometer measurements at the four depths for the 5 cm (2 in.) dose, presented in Fig. 2, illustrate these phenomena. All pores, large and small, are filled in the top of the column shortly after the dosage with saturation occurring at 5 cm (2 in.) depth after five minutes. As the water moves downward, the small pores exert their greater capillary pull, which results in flow primarily through these pores. Therefore, minimum moisture tensions increase with greater depth in the column. For example, the lowest tension reached at 10 cm (4 in.) depth is 12 cm (4.8 in.) and at 25 cm (10 in.) depth is 20 cm (8 in.).

A uniform distribution system used in the field includes hydraulic conditions comparable to those induced by the different loading regimes in the columns except for two dimensional flow patterns encountered in the field. The flow in the columns is strictly one-dimensional. Travel times occurring under field conditions will then be longer than those obtained in these experiments. Uniform distribution avoids problems associated with local overloading which sharply reduces travel time as shown here, which, in turn, may result in poor purification as demonstrated in the next section.

Removal of Indicator Bacteria Through Soil with Various Loading Regimes

Evidence that sewage purification via soil percolation is related to the flow regime and liquid detention in soil was initially obtained from studies of septic tank-mound systems in Door County, Wisconsin. These studies will be reviewed and examples from two additional systems will be discussed. In Mound System I (Bouma et. al., 1972) localized overloading occurred near the mound distribution box when sewage was discharged by pumping via large 10 cm (4 in.) diameter perforated distribution pipes. Fecal indicator organisms were detected in the mound fill below the overloaded area (1000 fecal streptococci/100 ml and 2500 fecal coliforms/100 mls); whereas, these organisms were not detected in samples of fill taken from below the distribution system furthest from the initial points of distribution.

Sewage percolated rapidly through a very coarse sand fill in Mound 2 (Bouma et. al., 1972) resulting in high numbers of fecal indicator organisms (1800 fecal streptococci/100 ml and 17,000 fecal coliforms/100 ml) in liquid collected at the topsoil fill interface.

The third example is derived from parallel sand filters that were installed to evaluate the effects of loading rate on purification of septic tank effluent. Effluent was divided through a splitter box to give an approximate loading of 7.5 cm/day (1.8 gal/ft²/day) to Filter A and 15 cm/day (3.6 gal/ft²/day) to Filter B. Results of bacterial analyses on three samples taken over the three months of operation are given in Table 2.

Under relatively high loadings of septic tank effluent and with filtration through medium to coarse sand, five to ten times as many indicator organisms were found in effluent of the filter loaded at 15 cm/day as compared to the filter loaded at 7.5 cm/day.

The last example describes the application of septic tank effluent to a mound using a pressure distribution system (Bouma et. al., 1973). Liquid samples were collected after passing through .61 m (2 ft.) of mound fill; results of bacterial analyses on the septic tank effluent and mound drainage are given in Table 3.

TABLE 2. BACTERIAL ANALYSES ON EFFLUENT OF PARALLEL SAND FILTERS* LOADED AT DIFFERENT RATES WITH EFFLUENT FROM THE SAME SEPTIC TANK.

Effluent Location	Bacteria		
	Fecal Streptococci (FS)	Fecal Coliforms (FC)	Total Coliforms (TC)
Septic Tank	4,800**	350,000	1,700,000
Filter A	210	2,600	35,000
Filter B	1,500	17,000	170,000

*Filter size; area = 1.4 m^2 (16 ft^2), depth = 0.61 m (2 ft)
 Sand characteristics; effective size = 6.36 mm , uniformity coefficient = 3.0
 Operating temperature ranges; 5 to 10 c .

**Geometric mean number of bacteria/100 ml.

TABLE 3. BACTERIAL ANALYSES ON EFFLUENTS FROM A SEPTIC TANK-MOUND SYSTEM EMPLOYING PRESSURIZED DISTRIBUTION.

Effluent Location	Bacteria			
	Fecal Streptococci (FS)	Fecal Coliforms (FC)	Total Coliforms (TC)	<u>Pseudomonas aeruginosa</u>
Septic tank	2,000(27)*	270,000(28)	2,800,000(28)	92,000(2)
Mound **	45	310	2,500	19
% of Negative** Samples	22% (23)	33% (21)	6% (18)	0% (2)

*Geometric mean number of bacteria/100 ml. Numbers in parentheses indicate number of samples used in calculation.

**Geometric mean values for mound drainage samples in which organisms were detected are given with the percent of samples which were negative (less than 2 organisms/100 mls).

Effective bacterial removal was observed. Fecal streptococci and fecal coliforms were not detected in 22 and 33 percent, respectively, of the mound drainage samples. A higher frequency of total coliform organisms, principally non-fecal types, were detected in the drainage. These organisms generally survive longer in the natural environment as compared to the FS and FC, and are often part of the natural soil flora. High numbers of *Pseudomonas aeruginosa*, an opportunistic pathogen, were detected in the septic tank effluent. Percolation through the mound fill effects significant removal of this organism.

Samples of soil below the mound distribution bed were obtained on September 13, 1974 after two years of operation. None of the above organisms were detected in the clay subsoil (30 to 55 cm) below the mound base, indicating satisfactory purification results before the liquid reaches bedrock level (55 to 70 cm below the mound base).

Results obtained thus far indicate bacterial purification of septic tank effluent is greatly enhanced by unsaturated flow through soil which is associated with liquid movement through the smaller soil pores and increased liquid detention times, which, in turn, allows for indicator and pathogenic organism absorption, competition with antagonistic soil or sewage bacteria, and die off. Corresponding results are reported for virus by Green and Cliver (1974).

Field Evaluation of Seven Distribution Systems

The conventional distribution system consists of an interconnected network of four-inch perforated plastic or bituminous pipe with two parallel rows of 1.56 cm (5/8 in.) holes. The holes are spaced 7.6 cm. (3 in.) apart with the rows located downward 45° either side of a vertical line passing through the center. Converse (1974) has shown that this system, when fed by gravity or pump, does not distribute the effluent uniformly but results in overloading the bed in one or several small areas.

Converse (1974) described and evaluated three pressure distribution systems (Fig. 4; Systems 1, 3, 5) using small diameter PVC pipe with small diameter holes. Good distribution was achieved in each system. These and other systems have been installed in actual field systems for up to two years (Fig. 4). During September 1974, each system was evaluated for flow-rate, pressure system, pump capacity, hole clogging and sludge build-up.

The pumps used in these systems have been 1/3 hp submersible sump pumps with either pressure or mechanical switch control. Figure 5 shows the performance characteristics for four different 1/3 hp submersible sump pumps. These curves show a considerable performance difference from one pump to the next. In several of the systems described later, a pump with performance curves C or D would not do a satisfactory job of distributing the liquid. The six systems will now be discussed in more detail.

System 1. This distribution system for an elongated mound which was installed in June, 1972 consists of 22.2 m (73 ft.) of 2.5 cm (1 in.) PVC pipe with 25 holes spaced .92 m (3 ft.) apart. Hole diameters vary along the length of pipe from .437 to .795 cm (.172 to .313 in.) (Fig. 4).

Table 4 gives the flow rate, elevation and pressure at three locations in the distribution system and at the pump. Three holes (No. 1, 13, 25) were checked with no evidence of clogging. Pressure head at the far end of the pipe (D) was zero, but there was a trickle of water out the last hole. Seventy-four percent of the head was due to elevation difference between the pump and the distribution system thus giving little head to overcome friction and orifice head loss in the distribution system. For this particular system a 1/3 hp pump with performance curve of A instead of B would have given much better distribution. Had the elevation difference been considerably less, this pump would have been adequate. Very little black sludge has accumulated in the system.

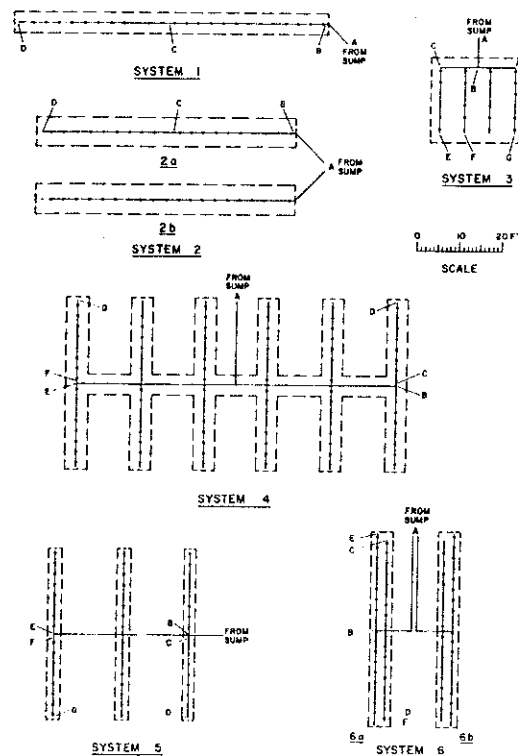


Figure 4. Schematic of the Six Field Installations. The Dashed Lines Represent the Borders of the Soil Absorption Bed. The Litters Represent Where Pressure Measurements were Made. Location A Was at Pump.

System 2. This distribution system for two subsurface trenches, which was installed in June 1973, consists of two 18.3 m (60 ft.) lengths of 2.5 cm (1 in.) PVC pipe with each lateral having 20 holes ranging in size from .476 to .635 cm (3/16 to 1/4 in.) (Fig. 4). For the past two years all the effluent went into lateral 2a. Table 4 gives the flow rate, elevations, and pressures at three locations for lateral 2a and also for laterals 2a when the valve for lateral 2b was open. This pump (curve B) had sufficient capacity to produce an adequate head at the far end of the line even

when effluent flowed into both laterals. Less than 50 percent of the head at the pump was attributed to elevation head in both cases. For some unknown reason, the head measured at the pump does not readily correspond to the performance curve of the pump (Fig. 5) for the given flow rate. Three holes (No. 1, 10, and 20) were checked with no evidence of clogging. No black sludge was observed in this system since the effluent is chlorinated.

System 3. This distribution system, which was installed in October, 1972 in a mound over creviced bedrock, consists of 2.5 cm (1 in.) PVC pipe with 24-.595 cm (15/64 in.) diameter holes spaced .76 m (30 in.) apart. The laterals are 4.1 m (13.5 ft.) long and spaced 1.4 m (4.5 ft.) apart. The elevation difference accounts for 55 percent of the head at the pump (Table 4). Good pressure is developed at the ends of the laterals using a 1/3 hp pump with curve B performance. Holes at position C, E, F, D and G were not plugged. A minimum of .68 m (2.25 ft.) of head was observed at each location. Very little head loss occurred in the laterals with one lateral gaining head which was probably due to misreading the fluctuating water level in the manometer. Dosing of the system occurs six times a day to eliminate localized overloading of the mound with only 24 holes. Doubling or tripling the number of laterals and holes would reduce the number of dosings per day but would require at least a 1/3 hp pump with curve A. Very little black sludge has accumulated in the pipes. Also, the head measured at this pump did not agree with performance curves of the pump at given flow rate (Fig. 5).

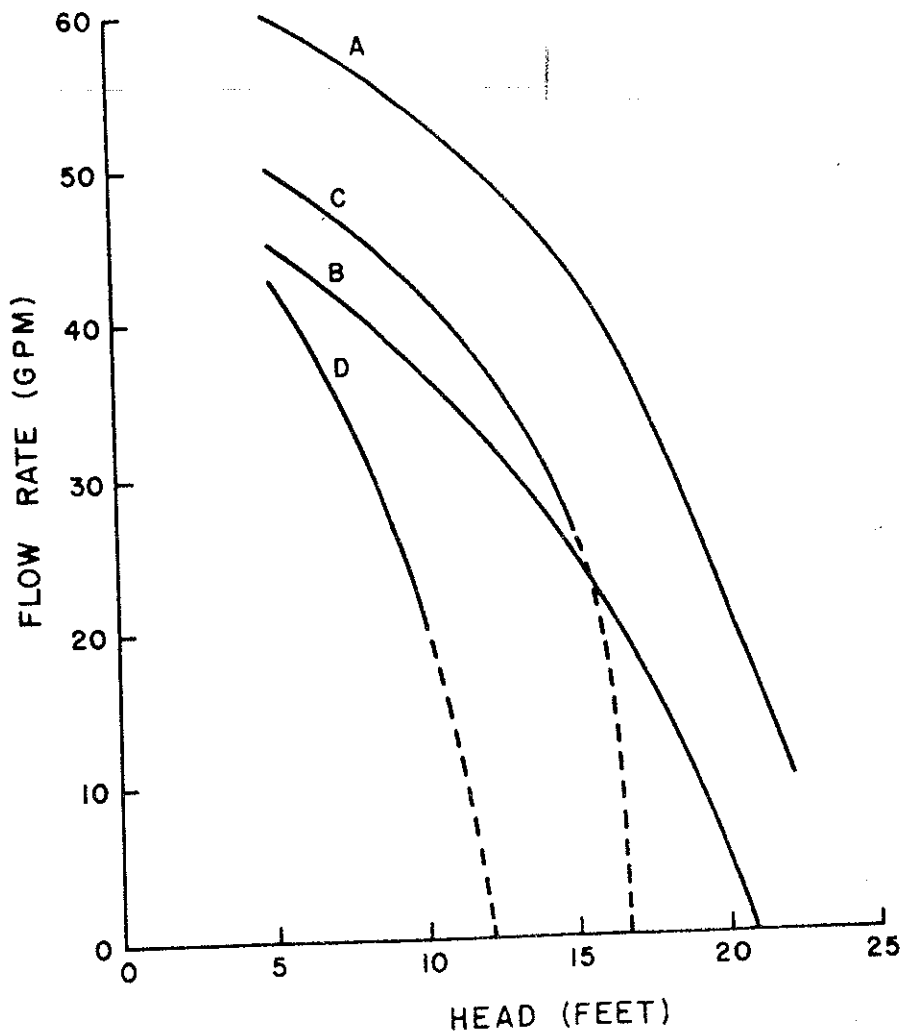


Figure 5. Performance Characteristic Curves for Four Makes of 1/3 hp Submersible Sump Pumps.

System 4. This large distribution system, constructed in summer, 1972, consists of a 7.6 cm (3 in.) PVC manifold and 12 2.5 cm (1 in.) diameter laterals each 6.1 m (20 ft.) long with 8-.635 cm (1/4 in.) holes spaced .76m (30 in.) apart (Fig. 4). Manometers were placed at the pump (A), at the end of each main B and E and at the front end of two laterals (C, D, F, and G) (Table 4). The elevation difference accounted for about 38 percent of head at the pump. The pressure at the end of the laterals were 1.2 and 2.23 cm (.48 and .88 ft.) which is low. At position D, black sludge was forced out the holes while hole G was plugged. A 1/3 hp pump with curve B is not large enough to provide the necessary pressure at the ends of the laterals for this large system. A higher performance 1/3 hp pump (curve A) or a 1/2 hp pump with a better performance curve is required for these large systems. Otis et. al., (1974) described the design criteria for this system.

System 5. This distribution system, constructed in October 1973, consists of a 7.6 cm (3 in.) PVC manifold and six 2.5 cm (1 in.) diameter laterals, each lateral with eight .635 cm (1/4 in.) holes located .76 m (30 in.) apart (Fig. 4). In April 1974, the two laterals furthest from the inlet were plugged due to grease and sand which was blamed on poor construction of the pumping chamber and runoff entering the pumping chamber (Otis et. al., 1974). The lateral ends were opened and the material removed. Evaluation of this system was difficult because the pump (curve D) was vastly undersized. The evaluation difference accounted for over 95 percent of the head at the pump (Table 4). No effluent came out the hole at D but some did at hole G because it was about 15 cm (6 in.) lower. This system would require at least a 1/3 hp pump with performance curve A or B. Some black sludge was evident in the system when the end cap of the manifold was removed.

System 6. This system is designed for a mound on top of a crest and installed in August, 1974. Each distribution system has its own line back to the pump for research purposes. The distribution system consists of a .1 cm (2 in.) manifold with four 2.5 cm (1 in.) PVC laterals. Two laterals are 7.0 m (23 ft.) long with two laterals 6.4 m (21 ft.) long. The .635 cm (1/4 in.) diameter holes are spaced .76 m (30 in.) apart (Fig. 4). The purpose of the two rows of laterals is to give better distribution in the 1.2m (4 ft.) wide bed. Elevation differences between pump and distribution bed is 2.6 m (8.5 ft.). The pressures in the pipe were less than .31 m (1 ft.) which indicates that when the total system is used, the pressure will not be sufficient to give good distribution. A 1/3 hp or larger pump with at least a performance capacity of curve A is recommended.

General Recommendations

1. The pumping chamber should be 1845 to 2842 l. (500 to 750 gal.) capacity to provide once or twice a day dosing of the system and also to provide some reserve capacity in case of electrical or pump failure. More frequent dosings lead to early clogging in some soils and poorer distribution of the effluent.
2. Alarm systems consisting of a pressure switch in the pumping chamber and alarm in the house should be required.

TABLE 4. SYSTEM CHARACTERISTICS SHOWING FLOW RATE, PUMP CURVE AND HEAD CHARACTERISTICS. SYSTEM NUMBERS REFER TO FIGURE 4 AND PUMP CURVE REFERS TO FIGURE 5.

System No.	Flow Rate gpm	Pump Curve	Position	Head Characteristics		
				Elevation ft.	Head ft.	Total Head ft.
1	10.5	B	A	0	19.13	19.13
			B	14.08	1.72	15.80
			C	14.18	.25	14.43
			D	14.24	0	---
2a ($\frac{1}{2}$ system)	20.18	B	A	0	14.73	14.73
			B	4.17	5.92	10.09
			C	4.30	2.38	6.68
			D	4.16	1.70	5.86
2a (full system)	22.20		A	0	12.60	12.60
			B	4.17	3.77	7.94
			C	4.30	1.29	5.59
			D	4.16	.83	4.99
3	12.83	B	A	0	15.91	15.91
			B	8.73	3.30	12.03
			C	8.66	2.95	11.61
			D	8.73	2.98	11.71
			E	8.91	2.25	11.16
			F	9.03	3.16	12.19
			G	8.90	2.75	11.65
4	35.48	B	A	0	9.68	9.68
			B*	3.88	2.31	6.19
			C	3.75	2.00	5.75
			D	3.60	.48	4.08
			E*	3.84	2.92	6.76
			F	3.71	2.92	6.00
			G	3.75	.88	4.63
5	8.25	D	A	0	6.90	6.90
			B*	6.31	.25	6.56
			C	6.37	.19	6.56
			D	6.67	0	---
			E*	6.12	.46	6.58
			F	6.20	.08	6.28
			G	6.17	.25	6.42
6b ($\frac{1}{2}$ system)	22.50	B	A	0	15.75**	15.75
			B	8.42	.92	9.34
			C	8.52	.42	8.94
			D	8.54	.75	9.29
			E	8.50	.50	9.00
			F	8.51	.58	9.09

*Measured on top of 7.6 cm (3 in.) manifold, all the rest measured from top of 2.5 cm (1 in.) laterals.

**Taken from performance curve at 22.5 gpm flow from Fig. 5 instead of measured in situ.

Conversion: 1 ft = .305 m, 1 gpm = 3.79 liters per minute.

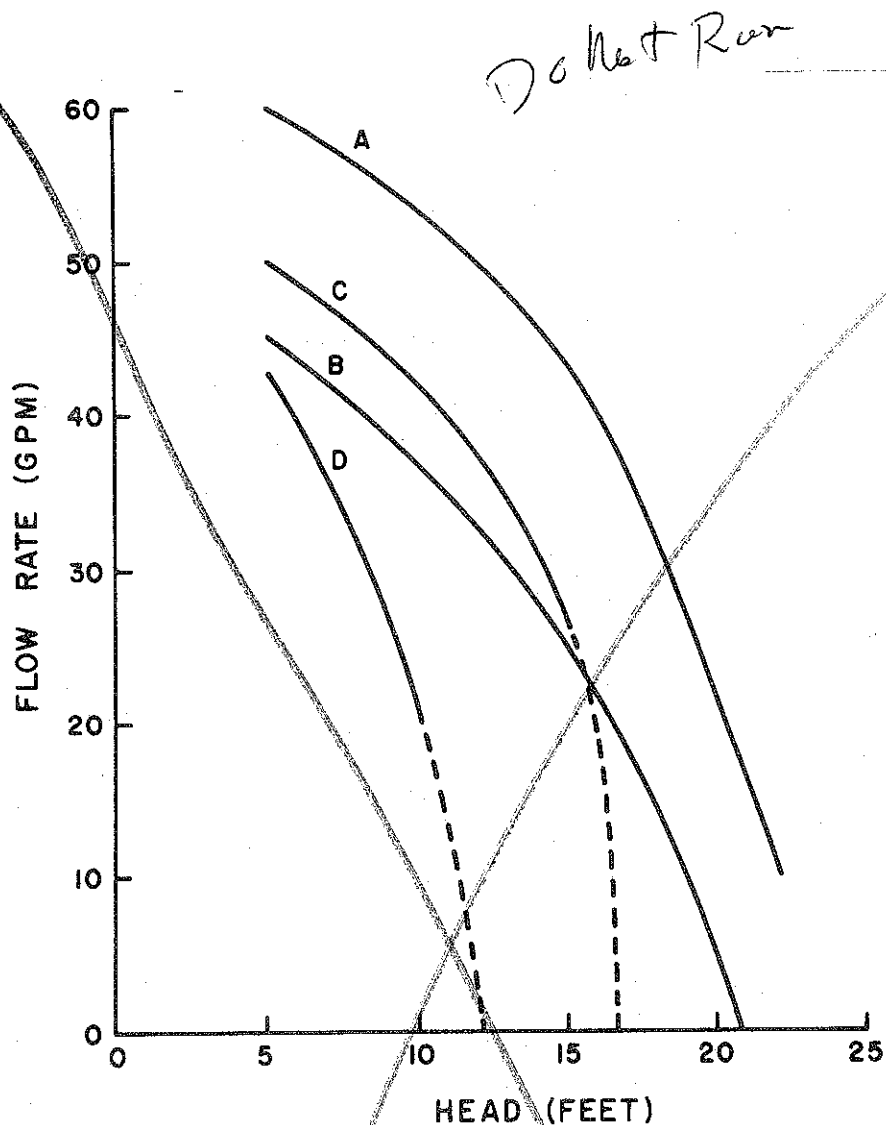


Figure 5. Performance Characteristic Curves for Four Makes of 1/3 hp Submersible Sump Pumps.

System 4. This large distribution system, constructed in summer 1972, consists of a 7.6 cm (3 in.) PVC manifold and 12 2.5 cm (1 in.) diameter laterals each 6.1 m (20 ft) long with 8-.635 cm (1/4 in.) holes spaced .76 m (30 in.) apart (Fig. 4). Manometers were placed at the pump (A), at the end of each main B and E and at the front and end of two laterals (C, D, F, and G) (Table 4). The elevation difference accounted for about 38 percent of head at the pump. The pressure at the end of the laterals were 1.2 and 2.23 cm (.48 and .88 ft) which is low. At position D, black sludge was forced out the holes while hole G was plugged. A 1/3 hp pump with curve B

3. The elevation difference between pump and distribution system should be as small as possible.
4. The septic tank and bottom of pumping chamber should be pumped every three years to minimize solids carry over to soil absorption system.
5. Proper pump sizing is necessary to provide adequate pressure in the distribution lines. A 1/3 hp pump with a flow rate of 75.7 l/m (20 gpm) under a head of 7.1 m (20 ft.) is recommended in most systems. Large systems may require a larger pump. Pumps rated at the same hp will vary considerably in their performance capabilities. It is better to oversize the pump than undersize it.
6. A pressure at the ends of the laterals of .71 m (2 ft.) of water is desirable to assure adequate flow to the far end of the system.
7. Pumps with pressure switches instead of mechanical float switches have caused fewer problems.
8. The pump should be set on a concrete block in the pumping chamber to avoid pumping solids which carry over from the septic tank.
9. All parts of the distribution system (manifold and laterals) should be sloped slightly toward the inlet to avoid freezing and ponding of water in the system between dosing. A slight slope in the lines will result in less sludge build-up in the distribution system as effluent will drain from the pipe. Freezing has not been a problem even though temperatures surrounding the distribution system have been below freezing. It is difficult to lay the 2.5 cm (1 in.) pipe on a uniform slope, but fortunately, this is not as essential for good distribution as it is for gravity flow in larger diameter pipes.

Summary

Uniform distribution of effluent in soil absorption systems gives better purification of effluent and reduces clogging in certain soils. Sands need uniform distribution during early system life to avoid improper purification until clogging results at which time unsaturated flow occurs with good purification.

Five pressure distribution systems have been evaluated after two years of field operation. Proper sizing of the pumps is a critical factor in satisfactory operation of these systems. Pumps rated at the same hp do not give the same performance curve, thus undersizing can result but pumps with performance curve A (Fig. 5) or greater are recommended for all but the smallest systems.

Some hole clogging and accumulation of black sludge has occurred in some of these systems. It appears that proper sizing of the pump will minimize these problems. Further evaluation over a longer period of time is required before a general recommendation can be given.

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