Chapter X
PUMPS AND HYDRAULICS

TABLES

FIGURES

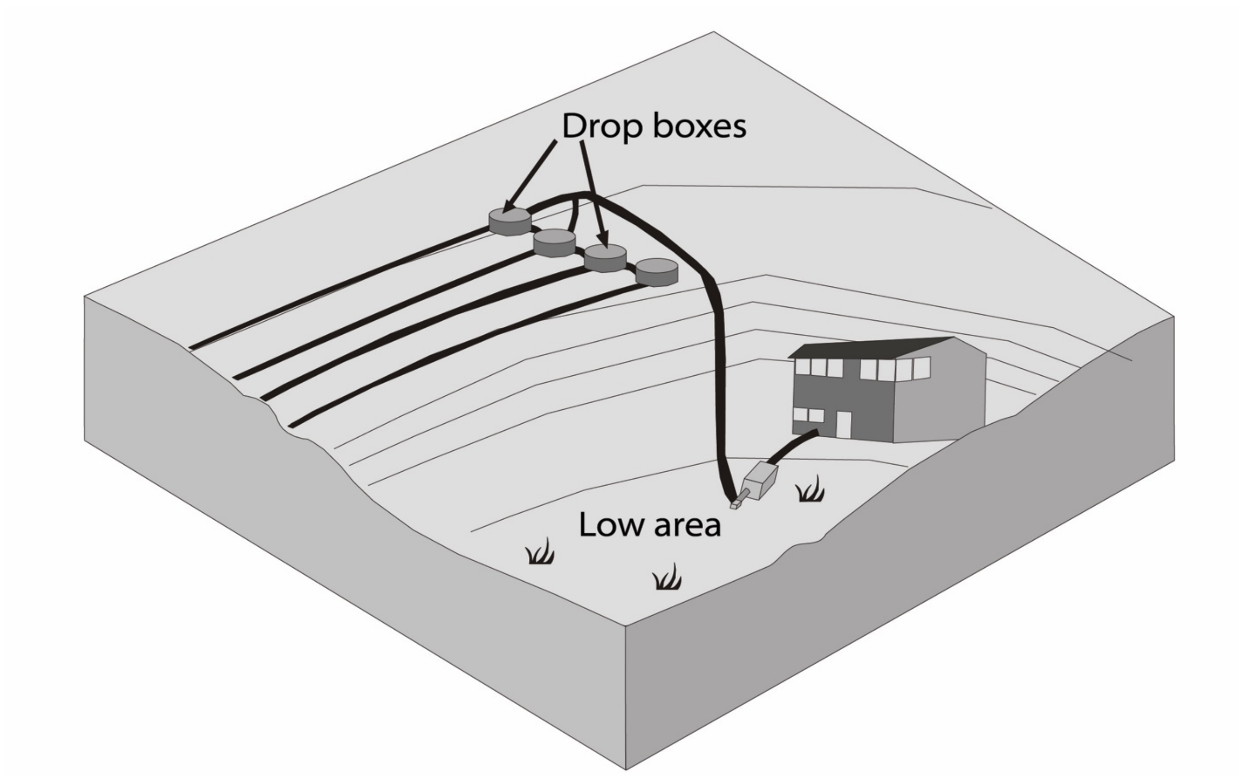
INTRODUCTION

Pumps are often utilized in on-site wastewater systems to —

- lift wastewater for improved system location options on a site
- distribute effluent uniformly
- time-dose an enhanced treatment or soil absorption system component
- inject air into an aerobic treatment unit (ATU)
- lift drainage water to lower a water table

Figure X-1 shows use of a pump to lift effluent to a soil-absorption area at higher elevation than the wastewater source. This use allows much greater flexibility in selection of the wastewater system and its location. Whenever a pump is required, maintenance and accessibility of the system are critical factors in its continued operation. Pump stations require careful design, installation, and maintenance by qualified technicians. Pump equipment and controls should be located in an accessible area protected from weather and vandalism. All components of the pumping station should be watertight and resistant to corrosion.

Figure X-1. Pump Used to Lift Wastewater to a Higher Elevation than the Source



PUMP APPLICATIONS

Pumping raw sewage requires a pump design to handle the solids found in raw wastewater. This type of pump has a non-clog impeller and many also have a grinder designed to handle tough and stringy solids such as rags or flushable wipes to avoid clogging the pump. Two common applications for pumping raw sewage are when a lagoon is at a higher elevation or plumbing fixtures in a basement must be lifted to the septic tank placed at the preferred shallow depth. A grinder pump might typically be chosen to handle all of the household sewage to pressurize or lift to a higher level. A non-clog sewage pump might be selected to serve fixtures in the basement such as a bath and laundry. These types of pumps are expensive and require frequent maintenance to meet service requirements. In addition, these pumps are typically low-head, and thus are designed for a limited lift or head of the wastewater.

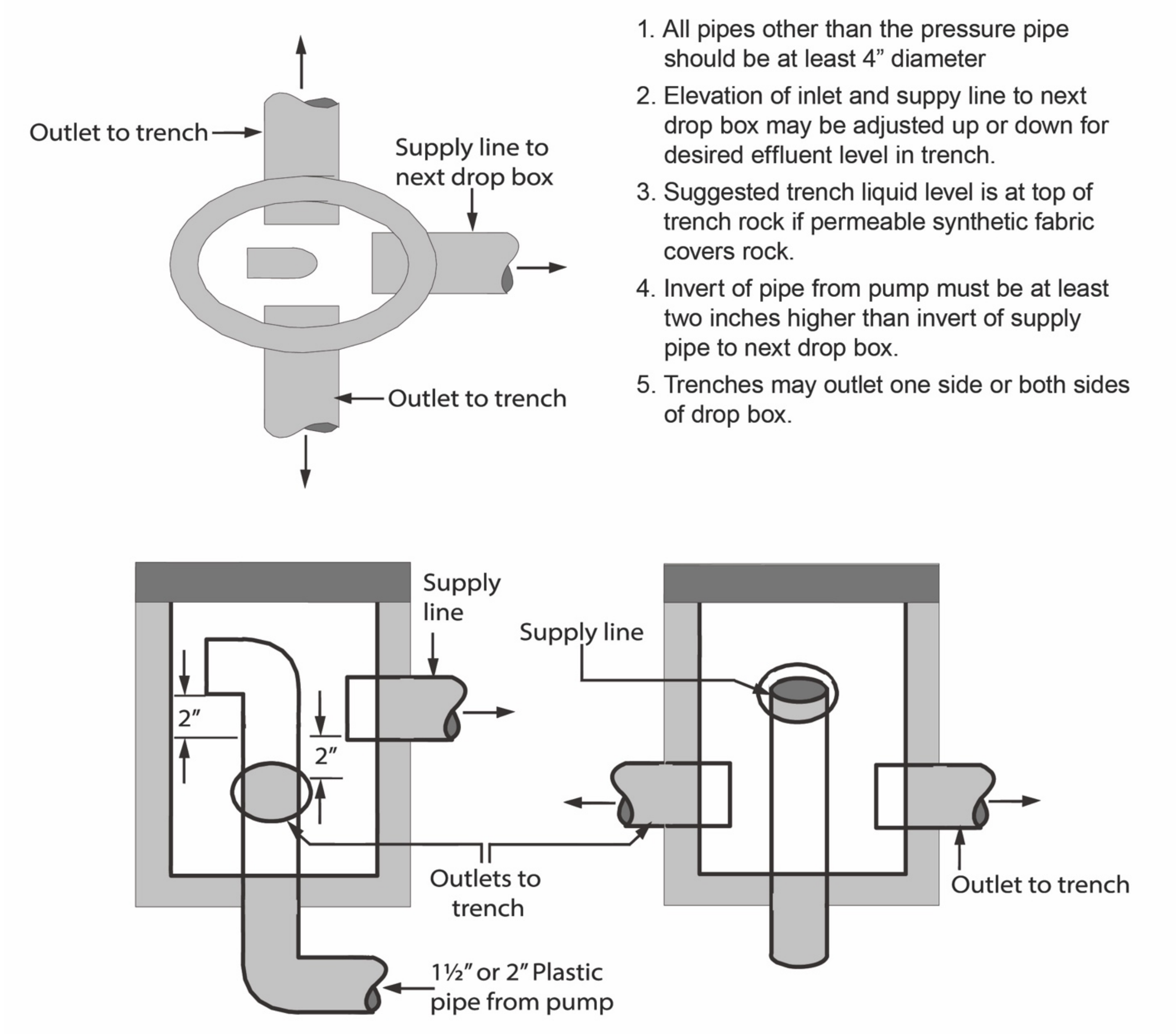
The vertical distance from the lowest water level at the pump to the highest point of the discharge is called lift or head (usually measured in feet). The cost to pump wastewater against high heads can become quite expensive. However, lifts of 10 to 20 feet with an efficient pump will be a modest price. Most on-site wastewater systems are designed to minimize the head required for the pump in order to keep the cost for the pump and its operation reasonable.

The most common type of pumping situation for on-site wastewater systems involves pumping the septic tank effluent after the solids have been removed and the effluent screened. For this application, the capacity to handle solids is not nearly as important as when pumping raw sewage. These pumps are classified as effluent pumps and usually use a turbine-type impeller. They are less expensive than non-clog or grinder pumps, not subject to clogging because of low solids, and some models are available to pump against a higher head.

An effluent pump may be used for a low-pressure pipe system, to reach an absorption area at a higher elevation for a drip-distribution system, sand or media filter, or simply to provide even distribution and/or dosing of the effluent. Effluent pumps usually have a screen that covers the pump intake and are preceded by a septic tank effluent filter. These filters must be kept clean to allow the effluent to enter the pump intake. Most of these pumps are designed to use water as a lubricant and coolant so the pump must not be operated under dry conditions. If screens are plugged and water cannot enter the intake, the pump may be damaged.

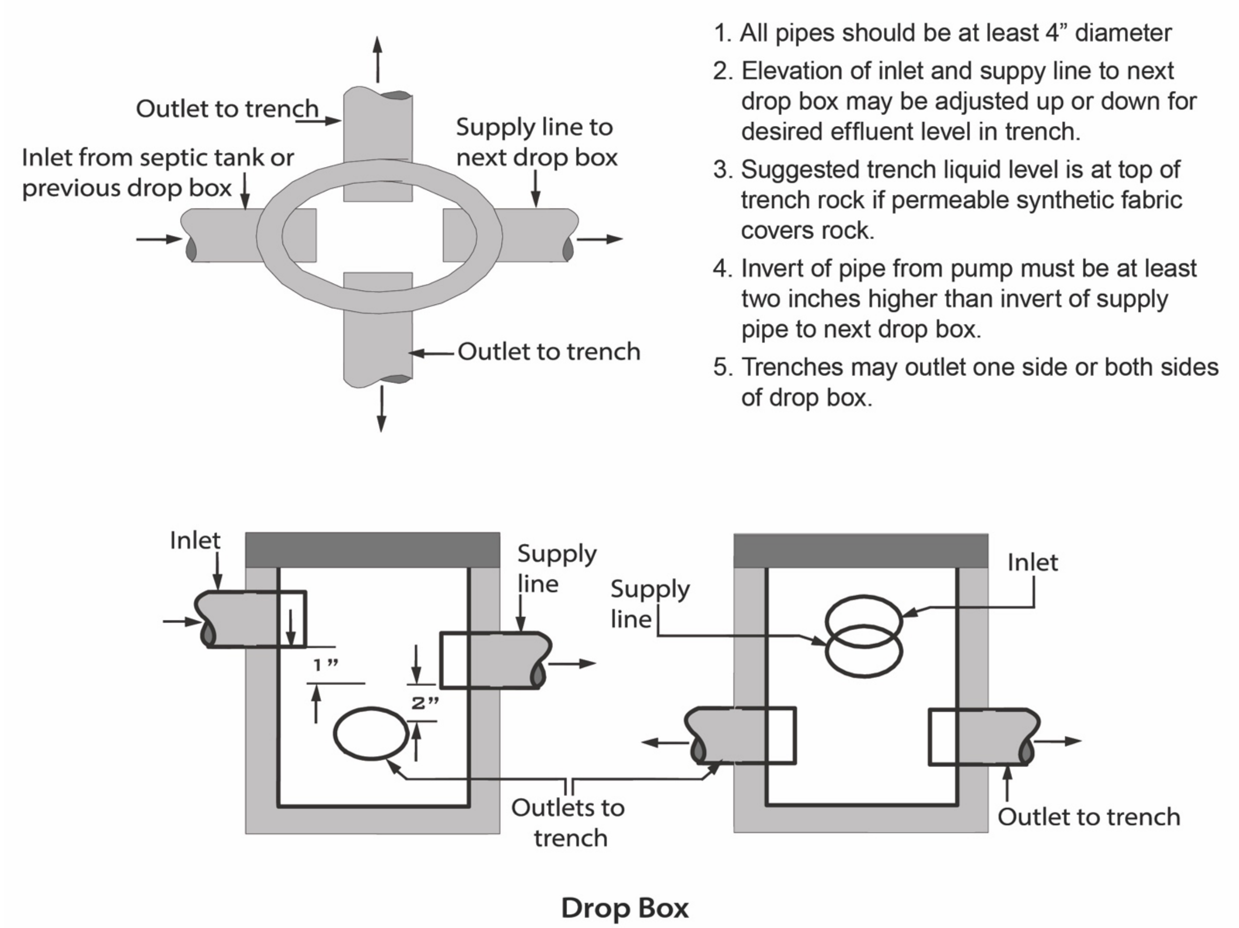
When an effluent pump is used to deliver wastewater to a conventional absorption field, flow may discharge to a drop box as shown in Figure X-2. On a slope, as shown in Figure X-1, water that upper laterals cannot absorb overflows via drop boxes, shown in Figure X-3, to lower laterals. In designing the drop-box system, the invert (bottom) of the discharge pipe from the pump must be at least 2 inches higher than the elevation of the supply line to the next drop box. **This arrangement will allow water in the discharge pipe to drain back to the pump tank but will not allow water from the rest of the absorption field to drain back into the pump.** The distribution box should be arranged so that effluent from the discharge pipe coming from the pump is directed to the wall of the drop box, opposite the supply line pipe. The outlet lines to the absorption field are then located on the sides of the box at a 90-degree angle from the inlet pipe. If this arrangement is not carefully designed, flow from the distribution box will not be evenly distributed. If additional trenches need to be supplied in a pumped system, drop boxes (see drop box design detail in Figure X-3) may be used in series, as shown in Figure X-1, to direct the flow through the absorption field.

Figure X-2. Pump Discharge Delivery to a Gravity-Distribution, Lateral Drop Box



Drop Box for Pump Effluent

Figure X-3. Gravity Distribution Lateral Drop Box



PUMP TANKS

The pump tank must be watertight and corrosion resistant. Most pump tanks available today are concrete, fiberglass, or polyethylene. All openings to the tank must be sealed and watertight to prevent the flow of groundwater or surface water into the tank and also to prevent the flow of wastewater out of the tank, except through the pump discharge. The pump tank must contain a minimum volume of water at all times in order to prevent the tank from "floating" when the ground is saturated. The pump inside the tank should be elevated above the bottom of the tank to allow for unobstructed flow into the pump intake and to prevent solids from clogging the intake or plugging orifices. Note: Orifice is synonymous with perforation. The pump manufacturer will provide information on how high the pump intake should be from the bottom of the tank. Examples of pump tank configuration are shown in Figures X-4, X-5, and X-6.

Figure X-4. Gravity Serves Ground Floor and a Pump Lifts Sewage from Basement

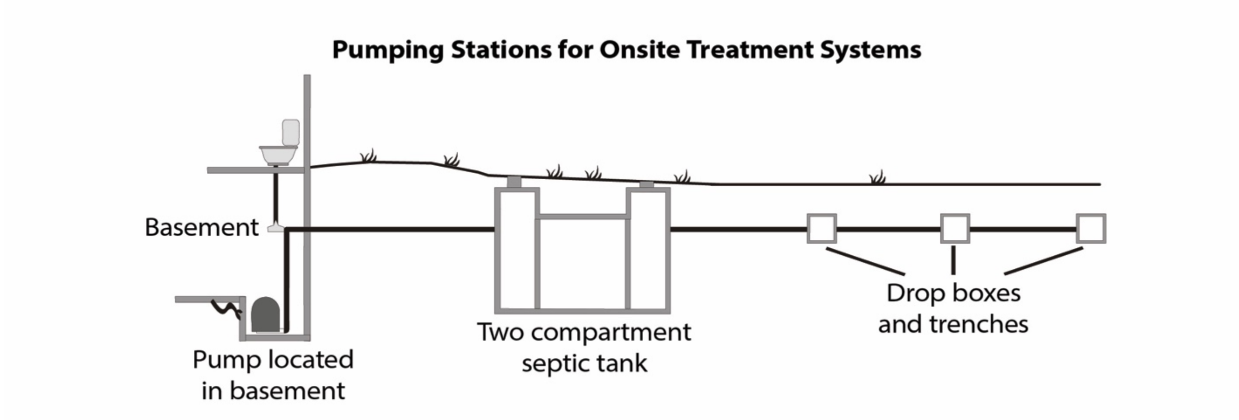


Figure X-5. Pump Lifts Tank Effluent to a Shallow Absorption Field

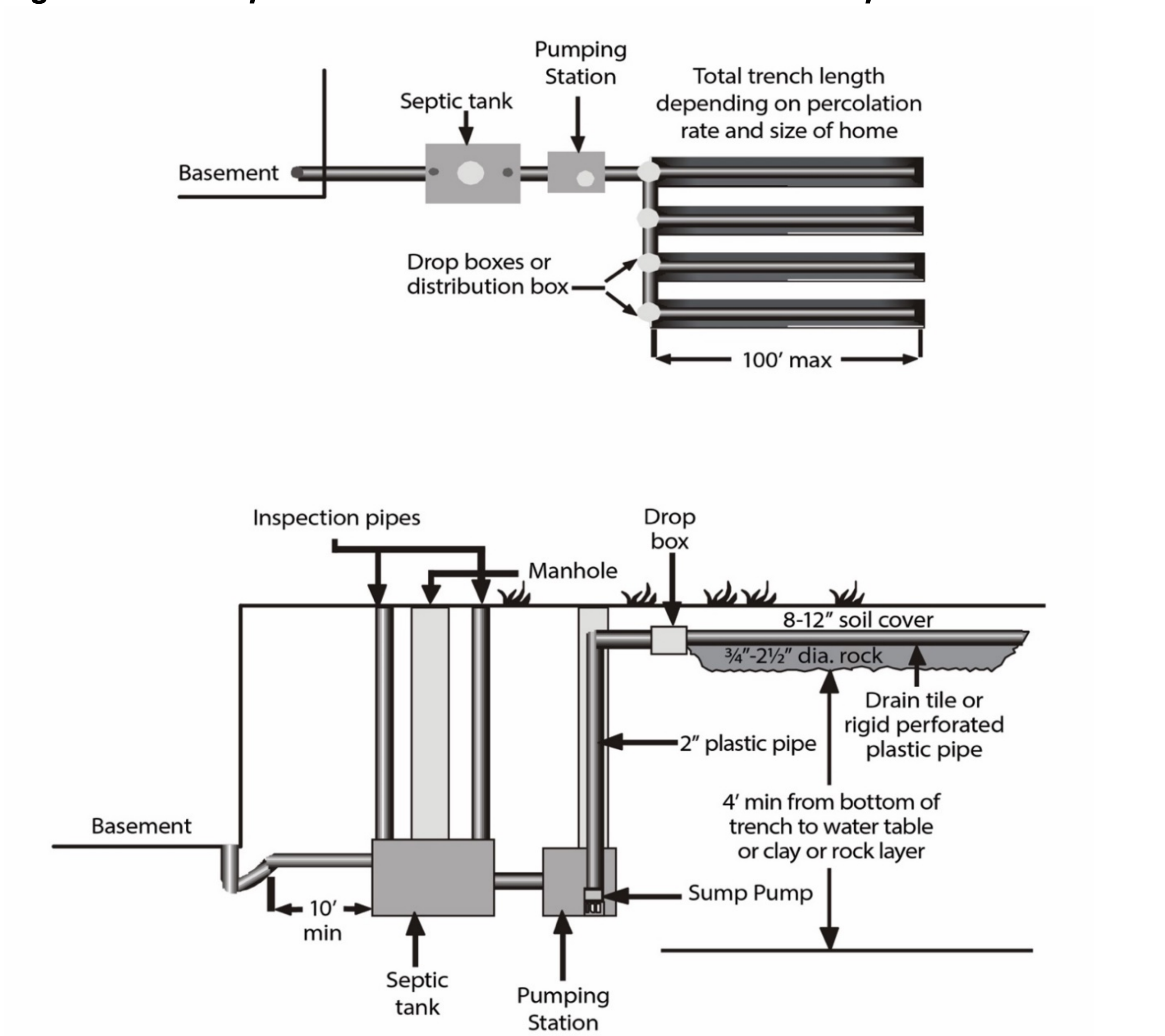
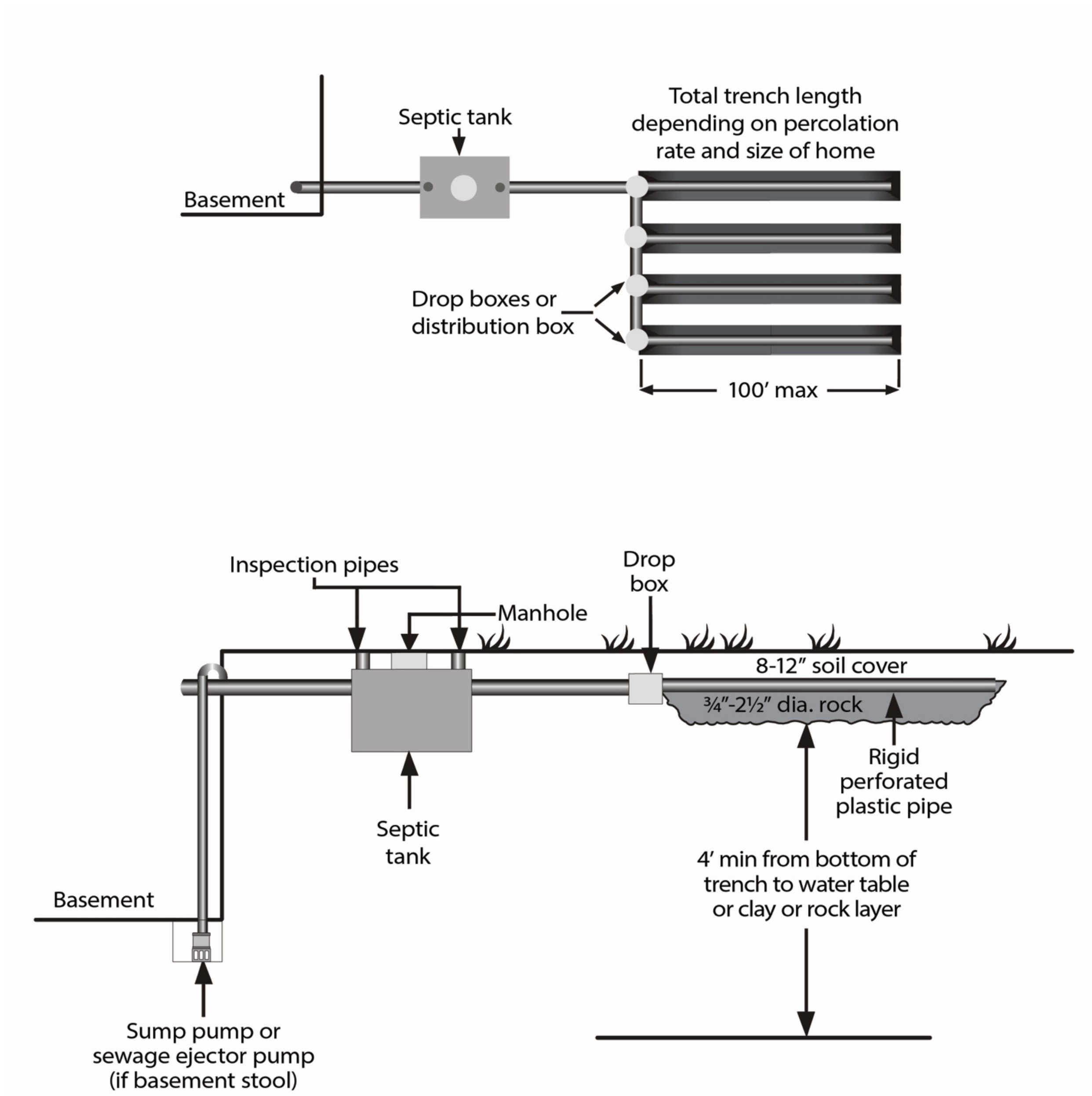


Figure X-6. Pump Lifts Sewage to Shallow Septic Tank Placement for Easier and Much Less Costly Maintenance



Pumping station for homes (pump in basement)

The pump tank must be accessible for maintenance. A manhole with a minimum diameter of 24 inches must be provided into the pump tank. The access opening into the tank itself must be a minimum of 20 inches. A larger manhole is preferred and may be required depending on the type of pump used. The manhole cover must have a lock mechanism to prevent unauthorized persons from opening the tank. An unsecured manhole cover is a serious safety hazard and may become a target for vandalism. The pumping station is a confined space and may contain dangerous gases. The manhole must provide access for maintenance and servicing the pump but in no case should the pump tank be entered unless all OSHA safety regulations regarding a confined space entry are observed. The pump should be installed with a quick-release discharge that is accessible at the top of the manhole. This will allow the pump to be serviced or replaced as needed. All electrical connections should be made outside the pump tank in approved waterproof connection boxes. The pump station must have an electrical disconnect that is located outside the house and is accessible for a service technician to cut the power to the system before any maintenance work is begun.

The pump tank should have a volume adequate to provide the minimum volume needed to keep the pump intake submerged, the pump-down volume of one pump cycle, and a reserve capacity of 75% of the daily flow in case the pump fails. Most pump stations are equipped with an alarm to indicate when the water level is rising above the normal volume. Once the alarm is activated, the pump tank should have enough reserve to handle 75% of the daily flow. However, in many situations even this reserve volume may not allow enough time to get the pump operational. Some pump stations are now equipped with an additional power outlet to allow the pump tank to operate using a portable pump. This will provide additional capacity until the pump can be repaired.

Most pump stations are designed so that the discharge line from the pump will drain back into the pump tank by gravity when the pump shuts off. The pump used in this application must be designed to allow this drain-back feature, which will make the impeller turn backwards. This feature will help prevent freezing of the lines in the absorption field. In addition, the pump discharge line should be fitted with a quick disconnect or union so the pump can be easily removed for repair. Easy access to the pump tank for maintenance and repair is a critical factor in the design of a pump station. The pump station may need to have some type of built-in rail or guide to allow the pump to be easily installed or removed.

If the pump tank is not concrete, then the tank may need to be bedded in concrete or anchored to prevent floatation when the tank is nearly empty. Fiberglass and polyethylene tanks will need to have some type of anti-floatation design incorporated into the installation of the tank. Usually the manufacturer will include installation specifications designed to prevent floatation. If the tank has two or more compartments, at least one of the compartments should be full enough to prevent the possibility of floatation.

Depending upon the application, the pump station may require a filter. If the pumped effluent is being discharged into a soil absorption field, then the pump station will usually have some type of effluent filter to keep solids out of the absorption field. Effluent filters should also be used in pump stations that dose a media filter, such as sand or textile filters. If the pump station is using any type of high-head effluent pump, then a filter and pump screen are mandatory. A variety of different sizes and configurations are available for these effluent filters. The type of filter used will depend upon the application, the tank, and the type of pump being used. The filter will need to be accessible for cleaning and should be designed for easy removal.

PUMP TYPES

Recently, the grinder pump has become more common for use in individual homes. It is equipped with blades mounted on the impeller that literally grind and shred solids before they enter the impeller. The grinder pump must grind solids to a size that will not clog the impeller. Two types are currently available: a centrifugal grinder pump and a positive displacement grinder pump. The centrifugal grinder pump is more common, but is usually not capable of pumping against a high head. However, the positive displacement grinder pump can handle a high-head application. Regardless of the type of grinder pump used, the blades will need to be replaced and the pump will require maintenance. Some grinder pumps require a great deal of maintenance and repairs can be very expensive. The grinder pump is most commonly used to convey wastewater into a small diameter sewer that serves some type of central wastewater treatment facility.

A pump that handles solids, or an ejector pump, is commonly used to deliver wastewater from a basement into a septic tank. This type of pump system must be designed to deliver the required volume during each pump cycle, without any drain-back from the line into the septic tank.

Ejector pumps are usually equipped with a check valve to prevent this from happening. When sewage is pumped into a septic tank, turbulence is created in the septic tank. For this reason, the septic tank should have at least two compartments, or have two tanks installed in series, in order to minimize the turbulence and allow the septic tank to function properly in removing solids and scum. The volume of wastewater delivered in one pump cycle should be no greater than 1% of the volume of the first compartment. If two septic tanks are used in series, the volume of the pump cycle may be increased to 5% of the volume of the first tank

PUMP CONTROLS

The pump is usually controlled by floats suspended in the pump tank. These may be mechanical floats and may be rated as “normally on” or “normally off.” In either case, the float is designed to hang vertically so that as the water level in the tank rises, the float gradually becomes suspended until the water completely covers it and it is inverted in the water. When the float is inverted, it will throw an electrical circuit either open or shut, depending upon the wiring configuration. Many pump stations have three floats: a low-water alarm, the on/off float, and a high-water alarm. In the three-float system, the low-water alarm float is always submerged unless the water level falls below the low-water alarm level.

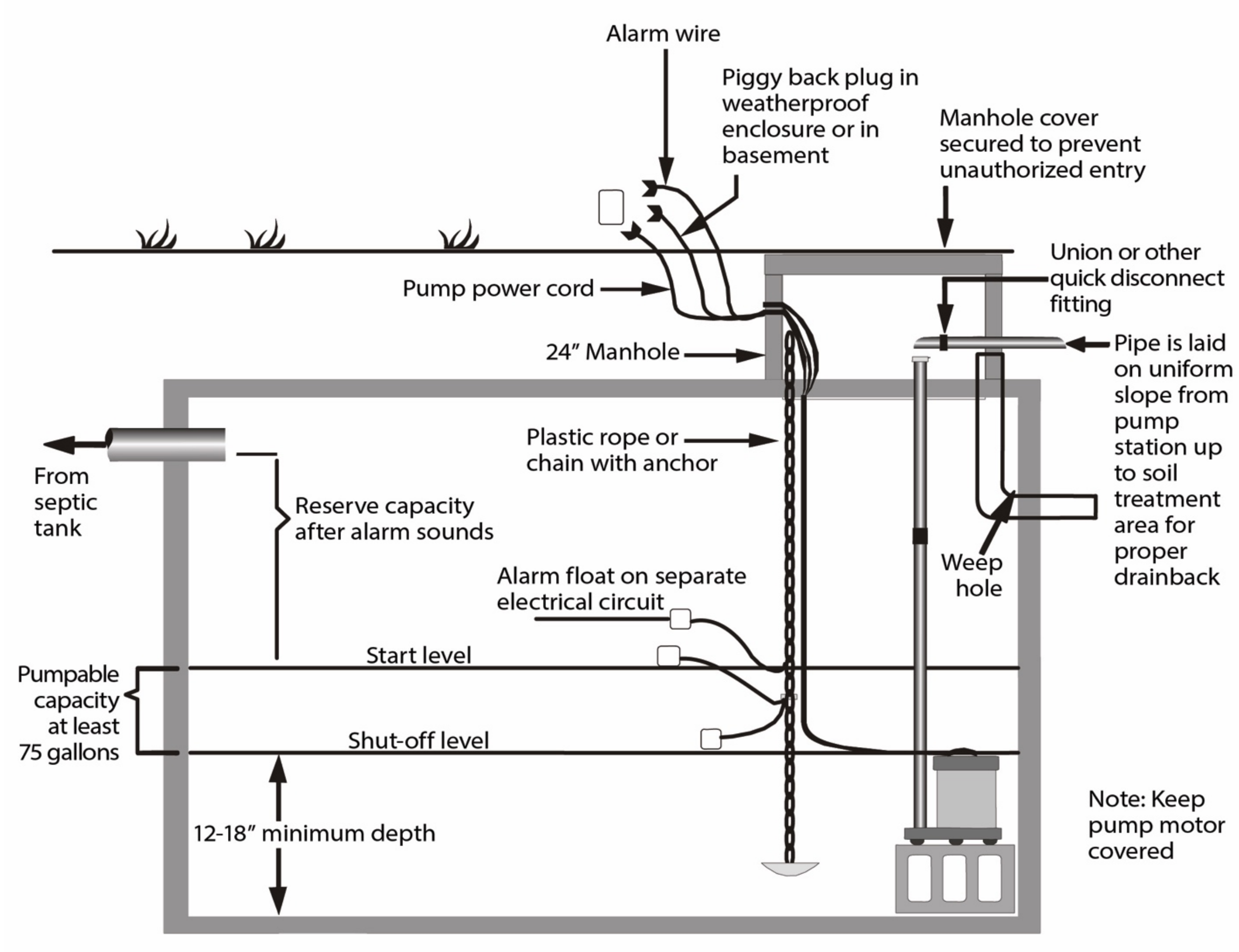
Remember the pump intake and body of the pump must be kept submerged to provide lubrication and cooling of the pump. The low-water alarm will indicate abnormal conditions and alert the homeowner to a problem in the pump tank. The on/off float is designed to turn the pump on when the water level gets high and to turn the pump off when the water level returns to the normal operating level. The high-water alarm is activated when the water level gets high. It should be set so that additional reserve capacity, in the event of pump failure, is still available in the tank.

Examples of the float configuration for a pump station are shown in Figure X-7 where the control box is attached to the house exterior, and Figure X-8 where the control box is remote.

Please note the floats should be tethered to a rigid float tree that is mounted in the tank. They should be on a short tether and must be carefully set so as not to become tangled up with each other or come to rest on the top of any surface (especially the pump casing) inside the tank. The most common problem encountered with these systems is that the floats are entangled and not able to work properly. Also, the vertical location of the pumps must be carefully set to provide the proper dose volume per cycle to protect the pump with the low-water alarm, and to notify the homeowner when high-water conditions exist. A qualified installer should be employed to set the floats at the proper height and configuration.

Some pumping stations may be equipped with a timer to operate the on/off cycle of the pump. These stations are designed to provide a specific dose at a given time interval. This type of configuration is commonly used for dosing media in a fixed-film treatment system or for a media filter such as a sand or textile filter. These systems also have low- and high-water alarms but the on/off cycle is controlled by the timer in the control box that may have to be adjusted to provide the proper dose. Adjusting the timer on these systems must be done by a qualified technician who understands how the timer is set. The control box for the pump station must be waterproof and corrosion-resistant. The control box should be mounted in a protected area near the pump tank. The control panel should be within sight of the pump tank to facilitate service on the unit. All electrical connections must be watertight and in accordance with local electrical codes. The control box should contain a wiring diagram mounted permanently in the box. The control box should contain an electrical shut-off to allow the service provider to cut the power to the system and should also be equipped with an audible and visual alarm to notify the homeowner when alarm conditions exist. The alarm circuit should be separate from the pump circuit so that if the pump trips the circuit breaker, the alarm circuit will continue to operate. Most control boxes have a silencer to shutoff the audible alarm after the homeowner is aware of the conditions. The wiring from the control box to the pump tank is usually buried and must be protected from traffic.

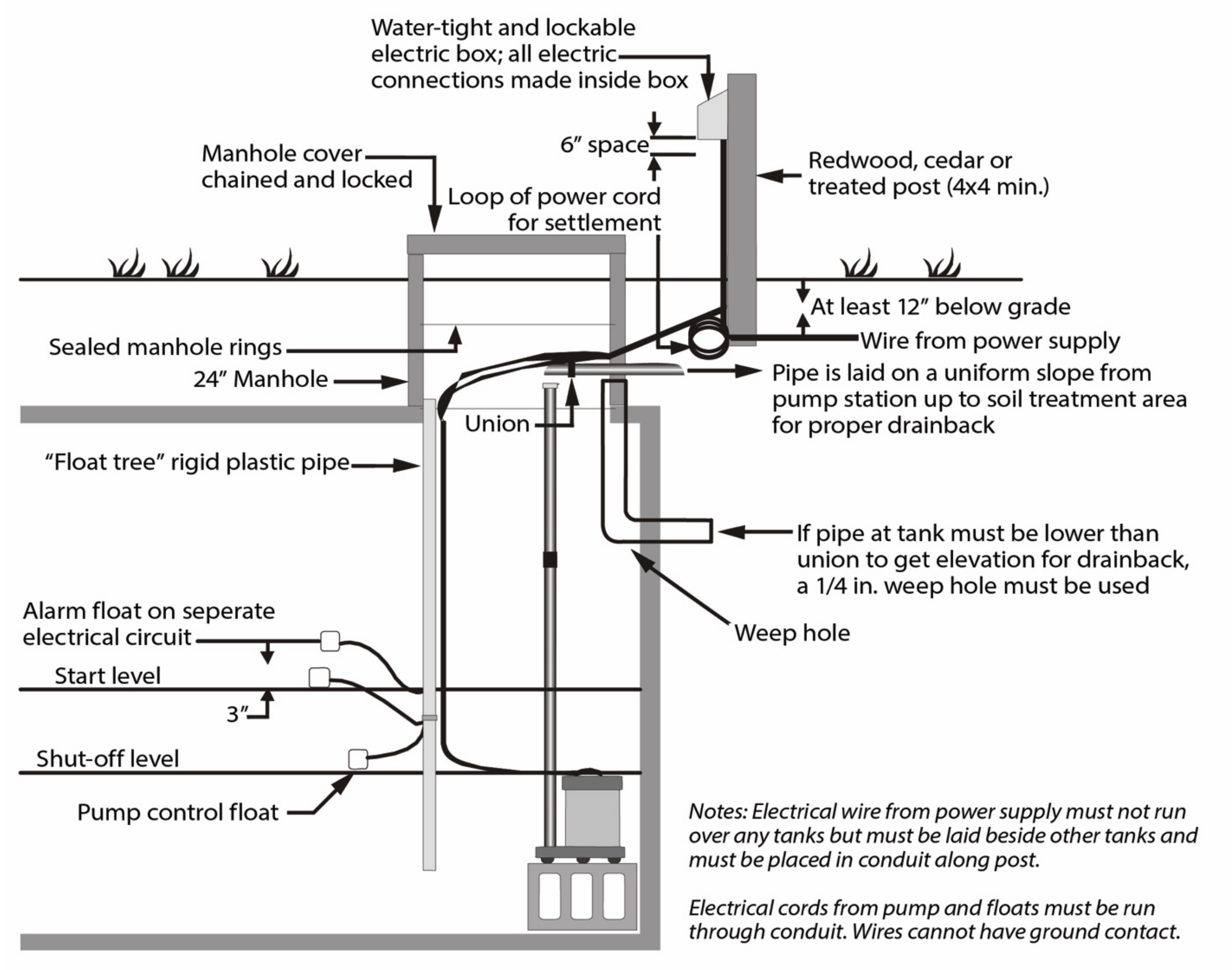
Figure X-7. Section of Pump Tank Showing Pump, Wiring, and Float Locations



Whenever two or more residences have a common soil treatment system, or if an establishment deals with the public (such as a restaurant, motel, or school), dual pumps should be installed as shown in Figure X-9. The dual pumps provide a back-up to keep the system in operation during mechanical problems with one pump. The dual pump, or duplex system, is similar to the single, or simplex pump, except that the control box must contain an alternator that alternates the pump being used.

The duplex system is set up so that if one pumps fails, the alarm will be activated, while the other pump will be operated to keep the system working. In addition, if the flow increases and one pump cannot keep the water level down, then the other pump will also operate. This configuration is called the lead pump and the lag pump. All duplex control panels should be wired to operate in this way.

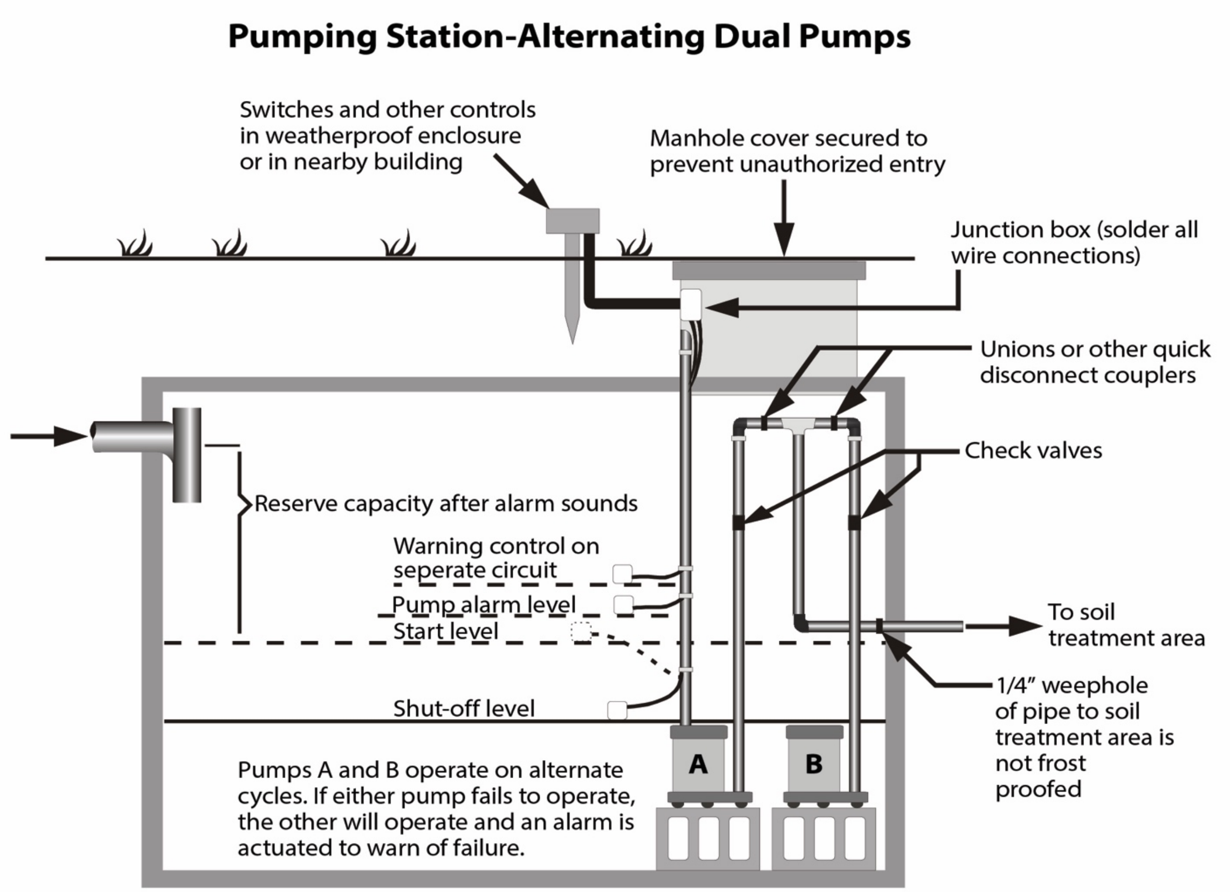
Figure X-8. Section of Pump Tank Showing Wiring and Electricity Away from House



PUMP PRESSURE AND FLOW

For dosing the absorption field, the pump station should be designed to provide a pump-out volume of 25% of the daily design flow per dose. The daily sewage flow from a four-bedroom home is 600 gpd. Thus, the start and stop levels should be set to pump 0.25 x 600 gpd = 150 gallons. Note that in the case of a large home with fewer occupants than designed for, the dose rate may need to be recalculated.

Figure X-9. Duplex Pump Station with Two Alternating Pumps



Setting the pump's start and stop controls depends on the configuration of the tank. For example, if the pump tank is rectangular with inside dimensions of 4 by 5 feet, the surface area is 20 square feet. Since each cubic foot of water contains 7.5 gallons, a l-foot depth of liquid (20 cu. ft.) in this tank would contain 7.5 gal./sq. ft. x 20 sq. ft. = 150 gallons. Thus, to pump 150 gallons, the pump start level should be 1 foot above the pump stop level.

In most domestic applications, the pipe from the pumping station is buried only deep enough to prevent physical damage and on enough of a slope to drain back to the tank after each pump operation. If exactly 150 gallons is pumped, then with the drain-back, less than 150 gallons will be pumped to the absorption field. The volume of drain-back must be calculated based on the capacity and length of the pipe to the distribution box. This volume must be added to the volume to be pumped to provide a net pump dose of 150 gallons.

For a circular tank, which is four feet in diameter, the surface area is calculated as pi or π (3.14) times the radius squared. In this case the surface area would be 3.14 x 2^{2} = 12.57 square feet. For a foot depth in the tank, there are 12.5 cubic feet. 12.5 ft^3 x 7.5 gal/ft^3 = 94 gallons. If 150 gallons are to be pumped and the tank contains 94 gallons per foot of depth, then 150 gal/ 94 gal/ft = 1.6 feet or 19 inches. The start control must be 19 inches higher than the stop control in order to pump out the 150 gallons per cycle. Again, the drain-back liquid must also be added into the flow to provide the proper net dose. The amount of liquid (gallons/100 linear feet) for various pipe sizes is included in Table X-1.

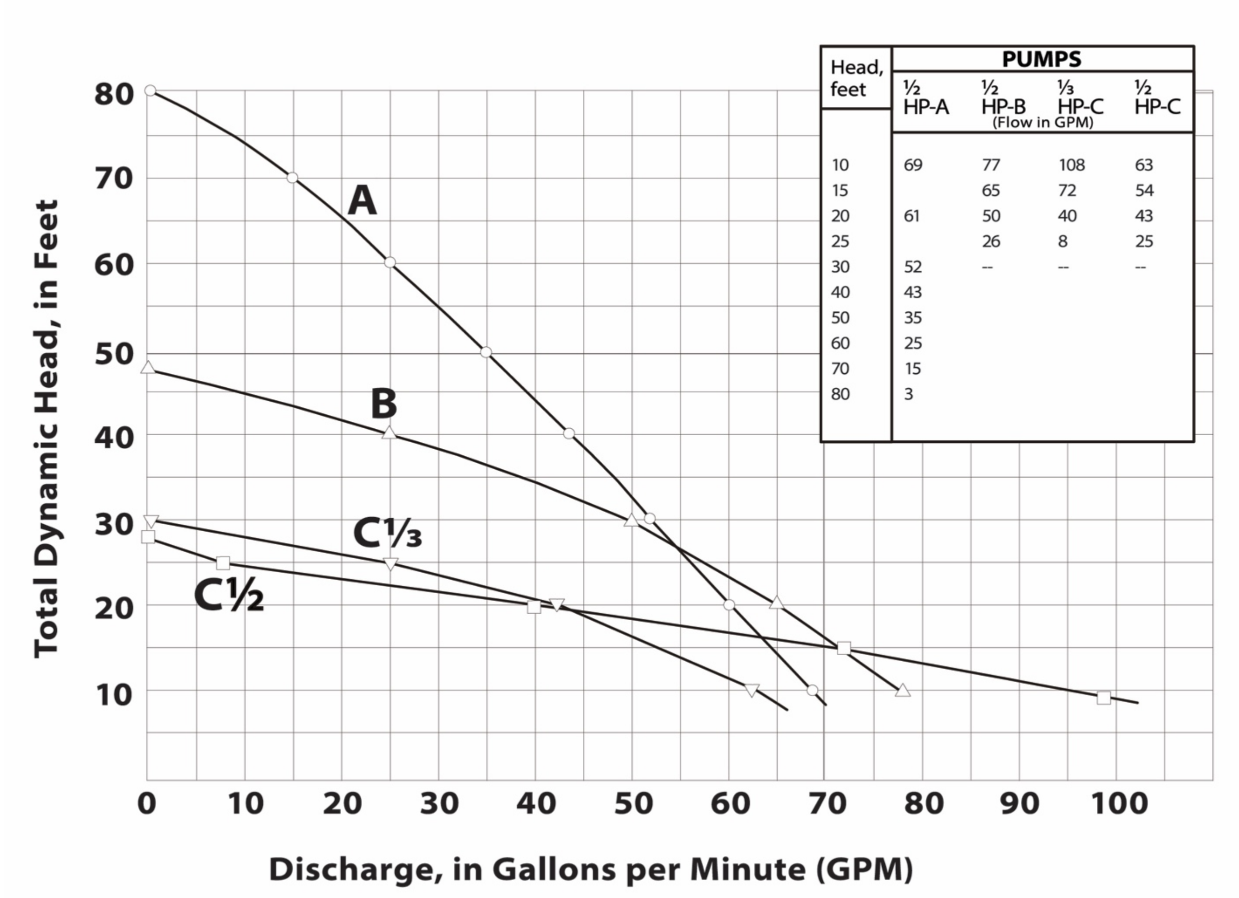
PUMP DISCHARGE CURVE

The common submersible sump pump operates under conditions described by the characteristic pump curve. The pump curve is unique for each pump at a specific operating speed and describes the head-discharge relationship for the pump. Four pump curves are shown in Figure X-10 as examples. The total dynamic head is given on the vertical axis and the pump discharge in gallons per minute is shown on the horizontal axis.

As the discharge rate increases, the total dynamic head a centrifugal pump can deliver will decline. The point at which the characteristic curve intersects the vertical axis is the maximum head the pump develops and is often called the shut-off head. The maximum head for pump C with a ⅓ horsepower as shown in Figure X-10 (labeled C ⅓) is 30 feet. This can be visualized by thinking of a standpipe just over 30 feet tall. The pump can raise the liquid level to a height of 30 feet but the flow at or above that head is zero. At any head less than 30 feet, some flow will occur. For example, at 25 feet of total head, the discharge will be approximately 25 gallons per minute. Referring to Figure X-10, the shutoff head of the ½-horsepower pump "A" is 80 feet (intersection of the pump curve with the vertical axis). At 40 feet of head, the pump can discharge 43 gallons per minute. The pump supplier can provide a pump curve to be used to select the right pump for each application. Note that even though pumps A, B, and C ½ all are half horsepower, each has a very different pump curve.

If requirements for a system are that a pump delivers 20 gallons per minute at 20 feet of total dynamic head, none of the pumps presented in Figure X-10 will deliver precisely this specification. A gate valve will need to be installed to dissipate a small amount of head so that the actual head delivered by the pump will be approximately 21 feet. If the ½-horsepower high-head pump A is used, and exactly 20 gallons per minute are desired, then the pump will actually deliver about 65 feet of total head, 45 feet of which will be dissipated in the gate valve. If the pump application is to deliver flow to a pressure distribution system in a mound, for example, this is a self-balancing system. As the flow tends to increase, the pressure at the perforations also increases and the pump simply operates at a particular point on its own particular characteristic curve. A gate valve is not needed with a pressure distribution system.

Figure X-10. Example Pump Curves for Four Pumps: Two Low-Head with Different Size Motors, a Medium-Head, and a Higher-Head Pump



Head, feet	PUMPS ½ HP-A	½ HP-B (Flow in GPM)	⅓ HP-C	½ HP-C
10	69	77	108	63
15		65	72	54
20	61	50	40	43
25		26	8	25
30	52	--	--	--
40	43			
50	35			
60	25			
70	15			
80	3			

Table X-1. Plastic Pipe Friction Loss, Diameter, and Volume

Pipe Diameter	1"	1.25"	1.5"	2"	2.5"	3"	4"
Inside dia. Gals/100ft	1.05" 4.4	1.38" 7.8	1.61" 10.6	2.067" 17.4	2.47" 24.9	3.07" 38.4	4.03" 66.1
Flow,gal/min							
1	0.08						
2	0.28						
3	0.59	0.16					
4	1.01	0.27					
5	1.53	0.40	0.19				
6	2.14	0.56	0.27				
7	2.85	0.75	0.35	0.11			
8	3.65	0.96	0.45	0.13			
9	4.53	1.19	0.56	0.17			
10	5.51	1.45	0.69	0.20	0.09		
12	7.72	2.03	0.96	0.28	0.12		
14	10.27	2.70	1.28	0.38	0.16		
16	13.14	3.46	1.63	0.48	0.20		
18		4.30	2.03	0.60	0.25		
20		5.23	2.47	0.73	0.31	0.11	
25		7.90	3.73	1.11	0.47	0.16	
30		11.07	5.23	1.55	0.65	0.23	
35		14.73	6.96	2.06	0.87	0.30	
40			8.91	2.64	1.11	0.39	0.10
45			11.07	3.28	1.38	0.48	0.13
50			13.46	3.99	1.68	0.58	0.13
55				4.76	2.00	0.70	0.19
60				5.60	2.35	0.82	0.22
65				6.48	2.73	0.95	0.25
70				7.44	3.13	1.09	0.29
80				9.52	4.01	1.39	0.37
90				11.84	4.98	1.73	0.46
100				14.38	6.06	2.11	0.56
125					9.15	3.18	0.85
150					12.83	4.46	1.19
175					17.06	5.93	1.58
200						7.59	2.02

SELECT THE PUMP FOR THE APPLICATION

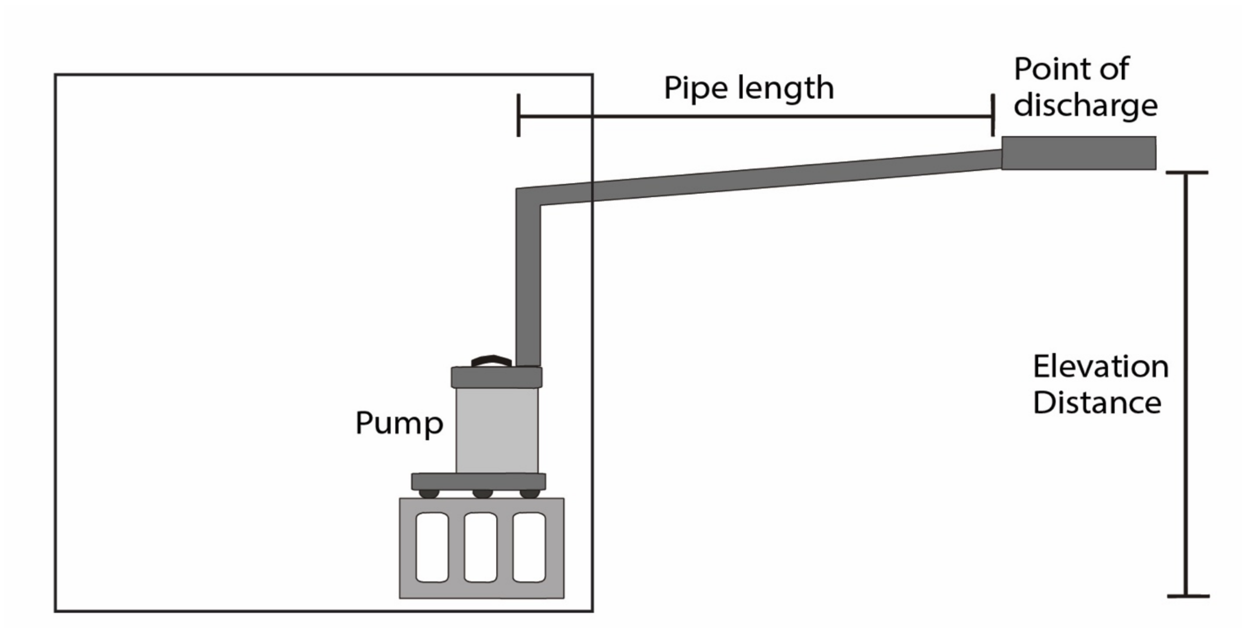
The pumps must be selected for the specific flow and head (or pressure) requirements; not just on the basis of horsepower, voltage, or other factors. Just because a pump worked well in one application does not mean it will work well in a different one. With wastewater, the presence of solids must be considered. Three factors determine the total dynamic head of a pump:

- elevation difference between the pump and point of discharge,
- pressure requirements of the operating system, and
- friction loss in the piping.

Example: Size a system to deliver 40 gallons/min to a manifold in a mound-style wastewater treatment system that is 140 feet from the pump.

This example is illustrated in Figure X-11, where an elevation difference of 17 feet exists between the top of the pump and the manifold, which is the point of discharge in the pressure-distribution system. When pumping to a pressure distribution system, as in this example, add five feet for pressure required at the manifold. If pumping to a drop-box gravity system, no additional friction loss needs to be added. Table X-1 shows the friction loss for Schedule-40 PVC plastic pipe. Friction loss calculations are based on the Hazen-Williams equation. The table also includes the amount of water contained in 100 feet of the various pipe diameters.

Figure X-11 Example Problem



Friction loss depends on the flow rate, type of pipe, pipe diameter and length, and fittings. Friction loss for 40 gallons per minute (gpm) in 2-inch Schedule-40 plastic pipe is 2.64 feet per 100 feet. (Note from Table X-1 that friction loss increases very rapidly as the pipe diameter decreases. For example, friction loss for a 40 gpm in 1 ½ -inch diameter pipe is 8.91 feet per 100 feet.)

Total friction loss from the pump to the manifold due to the pipe, fittings, and manifold itself must be calculated and added to the pump head requirements. Friction losses within pressure-distribution laterals are already included in their design.

In addition to straight pipe, the piping system has valves, elbows, tees, and other fittings. Each of these fittings can be expressed in equivalent lengths of straight pipe. A simplified way to account for these fittings is to multiply the length of the straight pipe by a factor of 1.25.

Multiply the delivery-line length (140 ft.) by 1.25, the factor to allow for fitting losses, resulting in a total equivalent length of 175 feet. Total friction loss for the fittings is 175 x 2.64 ft/100 ft = 4.6 feet. Total head requirement for this system is the sum of the friction losses due to the straight pipe, plus manifold, plus fittings: 17 + 5 + 4.6 = 26.6 feet of head.

The pump must be selected to deliver at least 40 gallons per minute at a total dynamic head of 26.6 feet. This point located on the curves in Figure X-10 falls above the characteristic curves of the ⅓ -horsepower and the ½-horsepower low-head pumps. Thus, these pumps are not suitable for these requirements. If these pumps were the only ones available, the flow would need to be reduced by using fewer or smaller perforations in the mound, or in some cases the friction loss could be reduced by using a larger diameter pipe.

Since a requirement is to have slightly excess capacity to deliver flow at the specified head, select the pump curve just above the plotted head-flow point (26.6-40) on Figure X-10. This point is above the low-head curves, but below the ½-horsepower medium head (B). Because discharge into a pressure-distribution system is self-balancing, no valve is needed to reduce pressure.

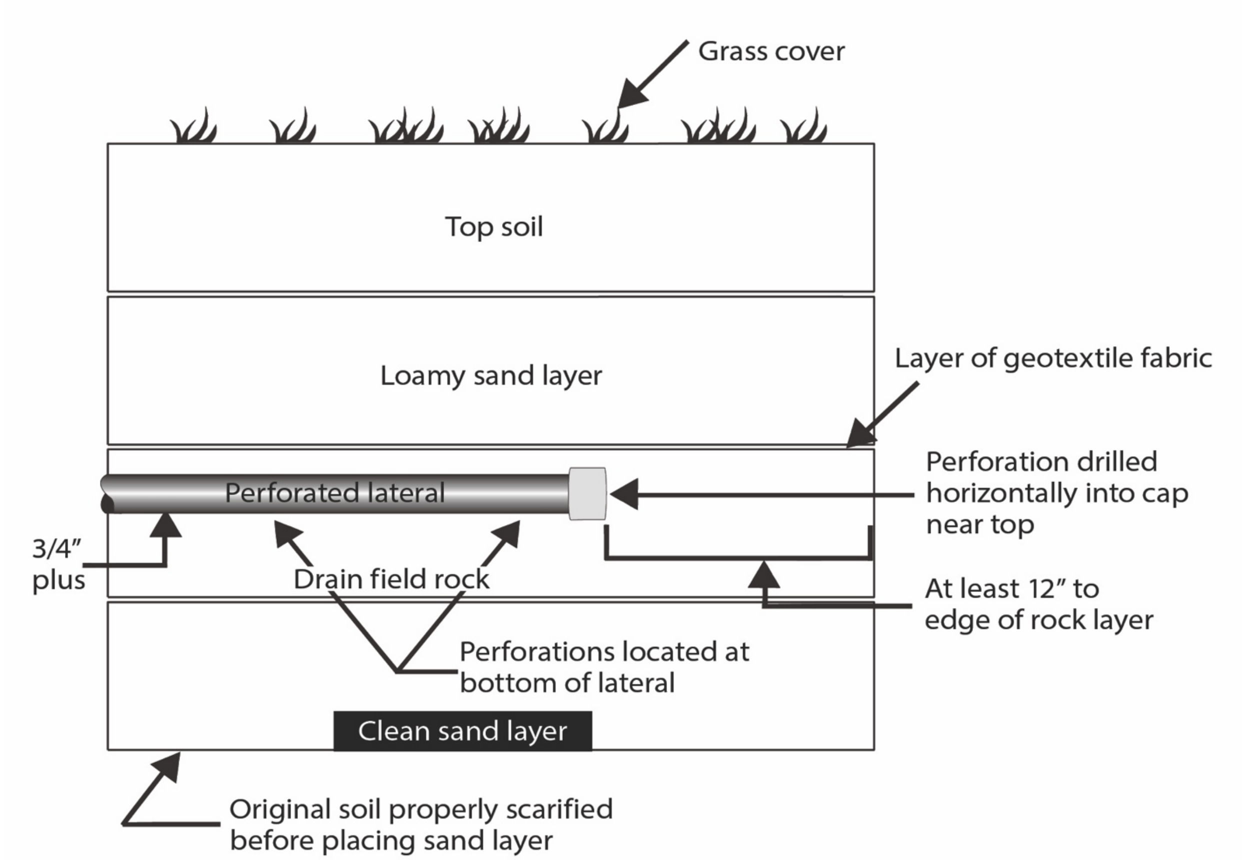
While it can be exactly calculated by a trial-and-error solution, the pump will likely deliver somewhat in excess of 45 gallons per minute at a total dynamic head of slightly more than 30 feet. Again, it is necessary to point out that the B ½-horsepower pump operates exactly on its own characteristic curve.

ENERGY REQUIREMENTS

The amount of energy required for pumping sewage is relatively small. If the pump delivers 40 gallons per minute and 174 gallons are to be pumped per dose, then pump operating time is 4.35 minutes per cycle with 4 cycles per day for a total time of 17.4 minutes per day. The ½-horsepower pump will likely use about 600 watts of power. (Wattage may be estimated. The nameplate amperage on a motor is typically the maximum current draw during startup, which occurs very rapidly. Continuous running current is often ½ to ⅓ of the startup amperage. An estimate of the pump energy use can be calculated by multiplying the current (amps) by the voltage (V) to obtain the wattage, if the running amperage is not known use ½ of the startup amperage.) In this example, the pump will use 600 watts x 0.29 hours = 174 watt-hours or 0.18 kilowatt hours per day. Given the kilowatt hours, energy costs may be calculated using the current price per kilowatt hour.

Although freezing of lines has not been identified as a problem for continuous use-systems, Figure X-12 shows how to frost-proof the lateral line in the soil absorption system. The discharge piping should be sloped to drain back into the pump station as described earlier.

Figure X-12. Section of Low-Pressure Pipe Lateral Indicating Holes for Drainage to Minimize Chances of Freezing



PRESSURE DISTRIBUTION NETWORK

Designing a pressure distribution network is a detailed procedure involving flow rates, pipe diameters, number and size of orifices, lateral pressure, and pressure or head delivered by the pump. Interrelationships are involved such as pipe size versus friction head loss. The process may require trying various combinations to come up with a design that produces an efficient system. However, tables and charts have been developed to help determine appropriate combinations for common designs. If the first design combination isn't satisfactory for some reason, such as requiring an unusually expensive pump, other combinations can be tried to see if improvements can be made.

A pressure distribution network must be designed to ensure uniform distribution of the wastewater. The pressure distribution system consists of the following:

1) lateral pipes with equally spaced holes drilled into the invert of the pipe;

2) manifold and main connected to the laterals;

3) dosing or pump tank to collect septic tank effluent to be pumped to the mound;

4) pump to pressurize the system; and

5) controls, alarm and power supply to operate the pump.

To avoid requiring a very large pump, small-diameter pipes, usually 1- to 1½-inch but occasionally larger, are used to distribute the wastewater. The 4-inch lateral pipe used for gravity-fed soil absorption systems is not suitable because it is too large, and the holes are not appropriately sized or spaced to provide even effluent distribution.

Schedule-40 PVC pipe and fittings are typically used in low-pressure on-site distribution systems. Orifices (holes) are drilled perpendicular to the pipe and are placed on the pipe invert (underside) or top. Any burrs or rough edges must be removed from the holes so they do not collect debris and clog. Holes should be drilled carefully. If the holes are not very close to the size specified in the design, the discharge will be different and will alter the performance of the system.

PRESSURE DISTRIBUTION NETWORK DESIGN

The following design is for the mound system described in Chapter VI pages VI-11 to VI-29. A very similar design procedure is used for any pressure distribution network including sand filter, media filter, low-pressure laterals, or drip dispersal.

Pipe Network Design Steps

1) Refer to the mound design to determine the absorption bed area (A x B). Mound design is covered in Chapter VI. The network configuration and length of the laterals will be based on the absorption bed area. The absorption area width will determine how many parallel lines will be used for the distribution network. The lateral length is measured from the distribution manifold to the end of the lateral. A center manifold is preferred because it minimizes pipe sizes. Remember: all lateral lines are to be on the same elevation or the operating head must be adjusted so they are equal.

2) The spacing between lateral lines should always be less than 5 feet. Spacing can also be based on 6 sq ft per orifice, as is used in sand-filter systems.

3) Determine the perforation spacing and size. The size of the perforations or orifices, spacing of the orifices, and number of orifices must be matched with the flow rate to the network. Typical orifice sizes are ¼” and $^{3}/_{16}$” with spacing of 30-36 inches. See Table X-2 for orifice discharge rates (gpm) for a typical range of pressure heads (ft).

4) Determine the lateral pipe diameter. Select a diameter large enough to keep pressure losses low (less than 15 percent of the operating pressure), but small enough to keep costs low. The lateral diameter selection is based on hole size, hole spacing, and lateral length. Charts have been developed to help in selecting suitable minimum lateral diameters. See Table X-1 for friction loss of Schedule-40 PVC pipe.

5) Determine the number of perforations per lateral.

6) Determine the lateral discharge rate, discharge per orifice times number of orifices. Select the pressure head to be maintained at the end of each lateral. Typical distal pressure is 2.5 ft for ¼" orifices, 3.5 ft for $^{3}/_{16}$" orifices and if smaller holes, such as ⅛" are used, consider using 5 ft of head. The lower the operating head, the more critical the friction head loss becomes to keep discharge uniform along laterals.

7) Determine the number of laterals and the spacing between laterals. If the 6 sq ft per orifice guideline is used, the orifice spacing and lateral spacing are interrelated. For absorption area widths of 3 feet, one distribution pipe along the length requires an orifice spacing of 2 feet. For a 6-ft-wide absorption area with the same configuration, it would require orifice spacing of 1 foot along the pipe.

8) Calculate the manifold size and length. Determine the main connection to the manifold, center, or end. The point of the main/manifold connection determines the length of the manifold. The manifold length is measured from the main/manifold connection to the end of the manifold. The minimum manifold diameter can be determined from a chart and is based on the lateral flow rate, lateral spacing, and manifold length.

   Determine the network discharge rate. This value is used to size the pump or siphon. Take the lateral discharge rate and multiply it by the number of laterals, or take the perforation discharge rate and multiply it by the number of perforations.

9) Develop a system performance curve as a way to predict how the distribution system performs under various flow rates and heads. The flow rate is a function of the total head that the pump works against. As the head becomes larger, the flow rate decreases, but the flow rate determines the network pressure and thus the relative uniformity of discharge throughout the distribution network. The easiest way to select the correctly sized pump is to evaluate the system performance curve and the pump performance curve. Where the two curves cross is where the system operates relative to flow rate and head. The total dynamic head the pump must operate at is the sum of —

   a) System network head (1.3 x distal pressure with minimum 2.5 feet),

   b) Elevation difference, and

   c) Friction loss in the pipe network.

Table X-2. Orifice Discharge Rates in Gallons Per Minute (gpm)

Pressure head (Feet)	Orifice Diameter (in)				
	1/8	3/16	1/4	5/16	3/8
2.5	0.29	0.66	1.17	1.82	2.62
3.0	0.32	0.72	1.28	1.99	2.87
3.5	0.34	0.78	1.38	2.15	3.10
4.0	0.37	0.83	1.47	2.30	3.32
4.5	0.39	0.88	1.56	2.44	3.52
5.0	0.41	0.93	1.65	2.57	3.71
5.5	0.43	0.97	1.73	2.70	3.89
6.0	0.45	1.02	1.80	2.82	4.06
6.5	0.47	1.06	1.88	2.94	4.23
7.0	0.49	1.10	1.95	3.05	4.39
7.5	0.50	1.14	2.02	3.15	4.54
8.0	0.52	1.17	2.08	3.26	4.83
8.5	0.54	1.21	2.15	3.36	4.97
9.0	0.55	1.24	2.21	3.45	4.97
9.5	0.57	1.28	2.27	3.55	5.11
10.0	0.58	1.31	2.33	3.64	5.24

Sizing the Pump or Siphon

The effluent pumps used for pressurizing the distribution networks are either centrifugal effluent pumps or turbine effluent pumps. The turbine effluent pump, which is a slightly modified well pump, is relatively new to the on-site domestic wastewater industry. In comparison, the centrifugal pump is a higher capacity/lower head pump with a relatively flat performance curve. The turbine pump is a lower-capacity/higher-head pump with a relatively steep performance curve. Turbine pumps probably have a longer life and may be the preferred choice for timed dosing because of their longevity relative to stop/starts.

Using pump performance curves, select the pump that best matches the required flow rate at the operating head. Plot the pump performance curve on the system curve. Then determine if the pump will produce the flow rate at the required head. Do not undersize the pump.

Care must be taken in sizing siphons. The head the network operates against has to be developed in the force main by backing up effluent in the pipe. If the discharge rate out of the perforations is greater than the siphon flow rate, the distal pressure in the network will not be sufficient. Some manufacturers recommend the force main be one size larger than the siphon diameter to allow the air in the force main to escape. However, this will reduce the distal pressure in the network, and it may drop below the design distal pressure.

1) Determine the dose volume required. The lateral pipe volume determines the minimum dose volume. The recommended dose volume has been 5-10 times the lateral volume. It has also been recommended the system be dosed four times daily, based on the design flow. Residents do not always use the design flow and so some mounds are only dosed once a day. When timed dosing is used, effluent is applied a number of times per day, with smaller doses. However, sufficient volume needs to be applied to distribute the effluent uniformly across the network. Thus, net dose volume is five times the lateral pipe volume.

2) Size the dose chamber. The dose chamber must be large enough to provide the following:

   - Dead space resulting from positioning the pump above the tank bottom
   - Dose volume
   - A few inches of head space for the alarm-warning float
   - Reserve capacity based on 100 gallons per bedroom (recommended)

   If timed dosing is selected, the pump chamber or septic tank/pump chamber must have sufficient surge capacity. If a turbine pump is used and must be submerged, there may not be enough surge capacity provided by the reserve capacity because turbine pumps are relatively tall.

3) Select quality controls and alarms. Follow electrical code for electrical connections. Some may have to be made outside the dose tank. There are excellent user-friendly control panels for timed-dose systems.

DESIGN EXAMPLE

Design a pressure distribution network for a mound whose absorption area is 150 ft long by 4 feet wide. The force main is 125 feet long and the elevation difference is 9 ft from the lowest wastewater level in the dosing tank to the highest point in the main or manifold.

Distribution Network Design Steps

1) Configuration of the network. This is a narrow absorption unit on a sloping site, so use 1 or 2 lines with a center feed creating two laterals.

2) Determine the lateral length using a center feed, the lateral length is

   lateral length = (B / 2) - 1 ft where: B = absorption length

   = (150 / 2) – 1 ft (The 1 ft = the distance from the end of the lateral pipe to the end of the gravel bed.)

   = 74 ft

3) Determine the perforation spacing and size. Two examples, A and B, are included. Each perforation covers a maximum area of 6 ft^2. The absorption area is 4 ft wide.

   a) With one lateral down the center on each side of the center feed, orifice spacing = area per orifice / width of absorption area

      = 6 ft^2/ 4 ft = 1.5 ft.

   b) With two laterals down the center on each side of the center feed, spacing = (area/orifice x no. of laterals) / (absorption area width)

      = (6 ft^2 X 2) / (4 ft) = 3 ft.

      The designer may stagger orifice spacings with laterals 1.5 ft apart. Perforation size —Select from ⅛, $^3/_{16}$ or ¼ inch. Use $^3/_{16}$ inch as per earlier discussion.

4) Determine the lateral diameter.

   Use Figure X-13 ($^3/_{16}$ inch) with a perforation spacing of 2 ft. Using one lateral on each side of the center feed with lateral length of 74 ft and 2-ft orifice spacing requires a lateral pipe diameter of 2 in (see Figure X-13).

   For 2 laterals on each side of the center feed and lateral length of 74 ft, with a

   3.5 ft orifice spacing allows the lateral pipe diameters to be reduced to 1.5 in. (Figure X-13: length = 74/2=37; orifice spacing = 3.5/2=1.75)

5) Determine number of perforations per lateral.

Using 2 ft spacing in 74 ft yields —

Number of perforations = (pipe length/orifice spacing) + 1 = (74 / 2) + 1 = 38 perforations/lateral. For two laterals (one on each side), the total number of perforations = 76 Check - maximum of 6 ft^2 / perforation = 150 ft x 3 ft / 6 sq ft = 75, so ok.

6) Determine lateral discharge rate (LDR).

Using network pressure (distal) pressure of 3.5 ft typical of the $^3/_{16}$" diameter perforations, table X-2 gives a discharge rate of 0.78 gpm.

LDR = 0.78 gpm/ perforation x 38 perforations = 29.6 gpm

7) Determine the number of laterals.

This was determined in Steps 3 and 4. Use one lateral on each side of center feed so two laterals are required. (If two laterals were used on each side of the center feed, they would be spaced 1.5 ft apart.)

8) Calculate the manifold size.

Since there is only one lateral per each side of center feed, there is no manifold. (Had two laterals been used, the manifold could be the same size as the force main as it is an extension of the force main.)

9) Determine the network discharge rate (NDR).

NDR = Number of laterals x lateral discharge rate (LDR)

= 2 laterals x 29.6 gpm

= 59.2 or 60 gpm

Pump has to discharge a minimum of 60 gpm against a total dynamic head yet to be determined.

10) Total dynamic head is the sum of the following:

System head = 1.3 x distal head (ft)

= 1.3 x 3.5 ft

= 4.5 ft

Elevation head = 9.0 ft

Friction Loss = Table X-1 for 60 gpm and 125 linear feet

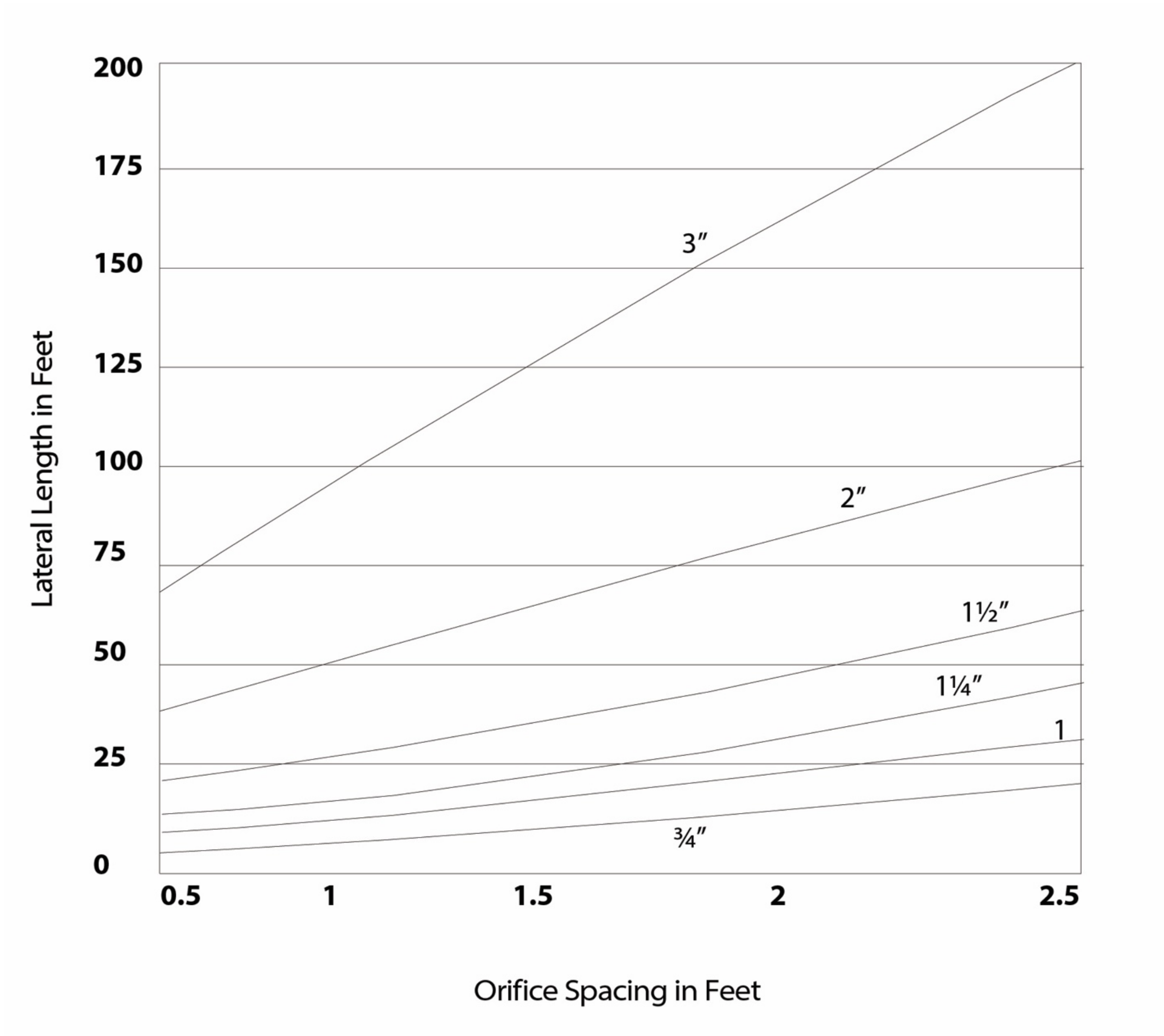
= 5.6 x 1.25 = 7 ft for 2" diameter. (Use 3" diameter (0.80 x 1.25 = 1) unless pump can't handle)

Total Dynamic Head = System network head + Elevation difference + Friction loss

4.5 + 9 + 1.21 = 14.7 ft (3” force main) or 4.5 + 9 + 8.75 = 22.3 ft (2” force main)

Pump must discharge 60 gpm against a head of 14.7 ft with 3” force main. These are the calculated flow and head values. The actual flow and head will be determined by the pump selected. A system performance curve plotted against the pump-performance curve will give a better estimate of the flow rate and total dynamic head the system will operate under. The next section gives an example.

Figure X-13. Graph of Lateral Length, Orifice Spacing, and Pipe Size for 3/16-inch Orifices.



Force Main, Pressurization Unit, Dose Tank, and Controls Design Steps

1) Calculate the system performance curve. Use Table X-4 to develop a system performance curve. Follow the procedures:

   a) Select five flow rates with 2 points above and below the network discharge rate of 60 gpm.

   b) Calculate the orifice (perforation) flow rate for each of the flows. This is done by dividing the flow rate by the number of orifices in the network. For the 30 gpm and 76 orifices, the orifice flow rate is 0.395 gpm.

   c) The elevation head is the height the effluent is lifted.

Table X-3. Heads and Calculations for Various Operating Flows and Pressures

Total Flow, gpm	Orifice Flow, gpm	Operating Head, ft	Elevation Difference, ft	Force Main Loss, ft	Total Head, ft
30	0.395	1.18	9	0.33	10.51
40	0.526	2.09	9	0.46	11.55
50	0.658	3.27	9	0.86	13.13
60	0.789	4.71	9	1.21	14.92
70	0.921	6.42	9	1.61	17.03
80	1.053	8.39	9	2.07	19.46

   d) The force main head is the head loss in the force main for the given flow rate. Select a force main diameter. For this example, use a 3-inch force main. The first three flow rates are not on the chart (Table X-1) and heads were calculated. Normally, the system is not operated in this range because velocities are less than 2 ft/sec. For the 60 gpm, the friction loss is (0.8 ft) x 1.25 (125 linear feet) = 1 ft. of head.

   e) The network head is calculated by $H = 1.3 \times (Q/(11.79 \times d^2))^2$. H is head in ft, Q is orifice flow rate in gpm, and d is orifice diameter in inches. The 1.3 is an adjustment factor for friction loss in laterals. For a $^3/_{16}$-inch diameter orifice, the equation is $H = 1.3 \times (Q/0.4145)^2$.

   f) The total head is the sum of the elevation, force main, and network heads.

2) Determine the force main diameter.

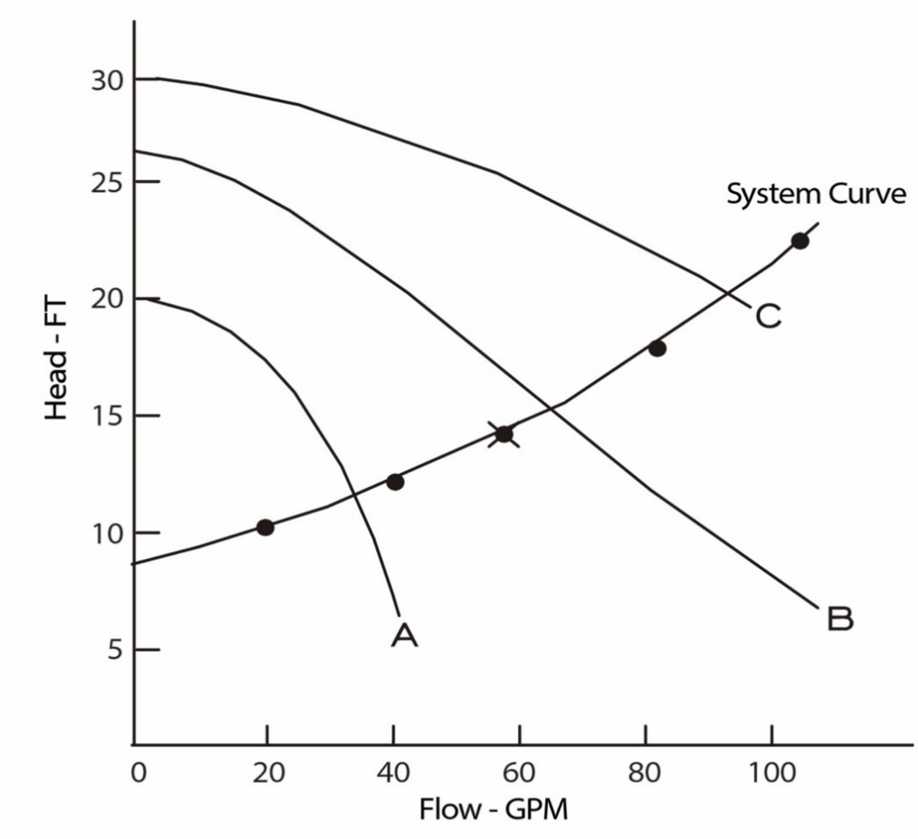
   Force main diameter = 3” (determined in Step 10 of Distribution Network Design).

3) Select the pressurization unit.

   Plot the performance curves of several effluent pumps and the system performance curve. For the system curve, plot the flow rates versus the total head. On the system curve, Figure X-14, place an X on the curve at the desired flow rate (in this case 60 gpm).

   Select the pump, represented by the pump performance curve, located next along the system performance curve just after 60 gpm (Pump B), as that is where the pump will operate. Pump C could be used but it is over sized for the conditions.

Figure X-14. Plotted System Operating Curve and Pump Curves to Select Pump



4) Determine the dose volume.

   More recent thinking is that the dose volume should be reduced from the larger doses recommended in the past. Use five times the total pipe void volume. Calculate void volume from the length of pipe and the volume per foot from Table X-5 for the pipe sizes.

Lateral diameter = 2”   Force main diameter = 3”

Lateral length = 74 ft   Force main length = 125 ft

Void volume = 0.163 gal/ft   Void volume = 0.367 gal/ft

Net dose volume = 5 x 74 ft x 0.163 gal/ft = 60 gallons per dose

Flow back from the force main = 125 ft x 0.367 gal/ft = 46 gallons.

Set the floats so that a total of 106 gallons will be dosed, with 46 gallons flowing back into the pump chamber to drain the pipe when the pump stops.

5) Size the dose chamber.

Based on the dose volume, storage volume, and room for a block beneath the pump and control space, a 500- to 750-gallon chamber will suffice. If timed dosing is implemented, then a larger tank will be required to provide surge storage. Use $^{3}/_{16}$ daily design flow for reserve capacity.

6) Select controls and alarm from products available from suppliers

Table X-4. Void Volume for Various Diameters of Schedule-40 Pipe

Normal Pipe Size (inches)	Void Volume (gal/ft)
3/4	0.023
1	0.041
1-1/4	0.064
1-1/2	0.092
2	0.163
3	0.367
4	0.650
6	1.469

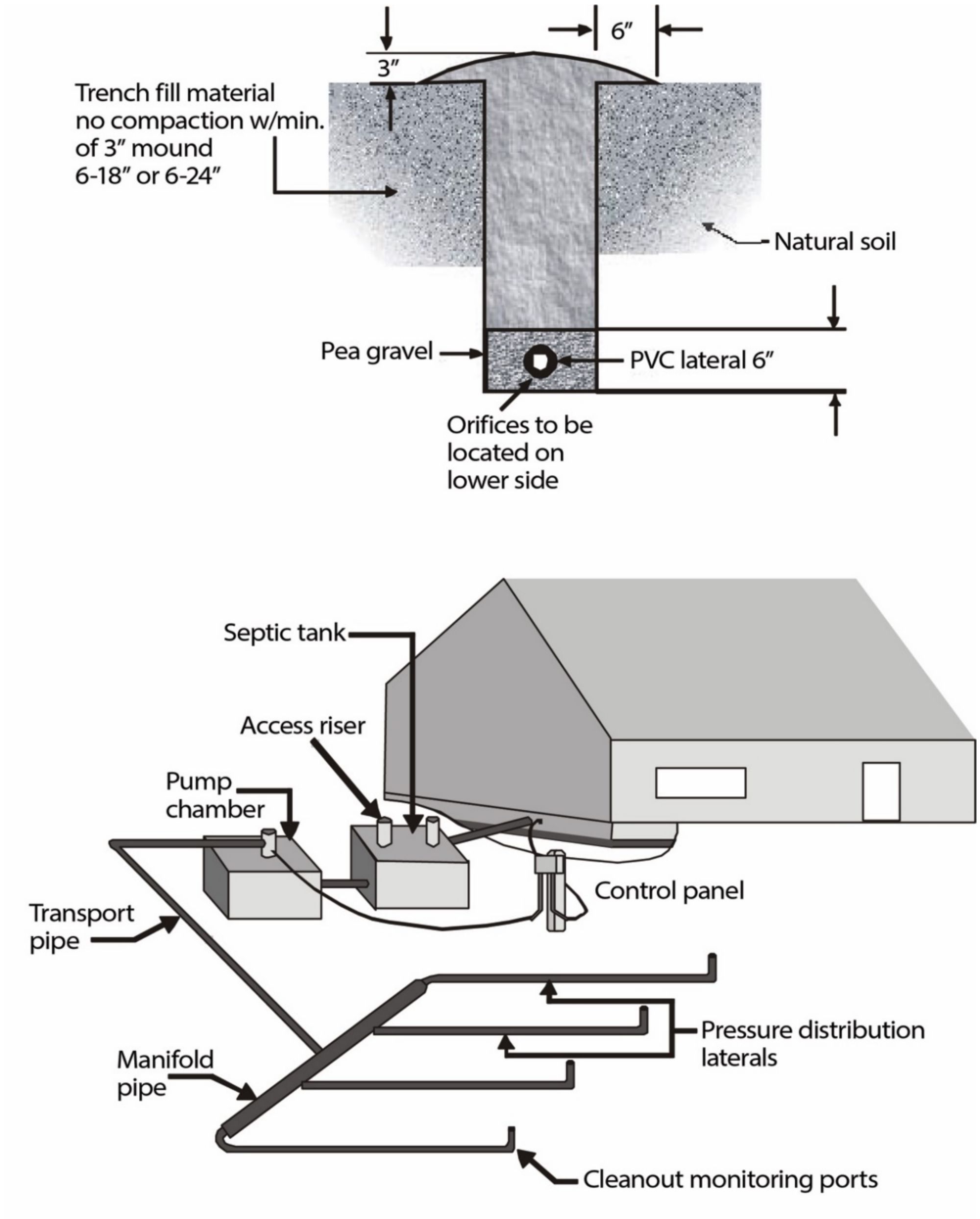
LOW-PRESSURE PIPE SYSTEM

Wastewater distribution in lateral fields by gravity depends on careful elevation control. Some sites may have severe limiting factors that must be addressed by using a system such as the low-pressure pipe system. Common applications for low-pressure pipe systems include small lot sizes, shallow soils, soils with a slow permeability rate, or steep slopes. The low-pressure pipe system can overcome these limitations by providing uniform flow distribution, alternating dosing and resting cycles, and shallow trenches. The effluent is distributed into the absorption field with a low-pressure pump system which can distribute the flow more evenly over the soil infiltrative surface. The low-pressure pipe system functions to dose the soil with wastewater, then allow a resting period, and then dose the soil again, repeating this process throughout the day on a timed basis. Dosing helps maintain aerobic conditions in the soil, which improves treatment of the wastewater and maintains soil permeability. The shallow placement of the trenches increases the vertical distance between the trench and any restrictive layer. In addition, the shallow depth keeps the wastewater effluent in the active soil layers where microbiological and plant activity is maximized. A typical application for a low-pressure pipe system is shown in Figure X-15 with a detail of the trench cross-section.

Components

The low-pressure pipe system includes pretreatment with a septic tank or alternative system, septic tank effluent filter, pump tank (including controls and pump), transport line to connect the pump to the absorption field manifold, and small-diameter lateral pipes. The pretreatment system must include an effluent filter to prevent clogging of the small-diameter pipe and small holes used to distribute the flow in the absorption field. The effluent filter must be cleaned every three months to assure the system remains operational. A low-pressure pipe system is dependent upon the operation of several mechanical devices, including the pump, and must be supplied with a reliable power source. The system configuration is shown in Figure X-15. The system should have a visual and audible alarm to notify the homeowner if the system is malfunctioning.

Figure X-15. Typical Pump Tank, Controls, and Pipe Network Components



The laterals are usually 1- to 2-inch diameter, Schedule-40 PVC with $^{3}/_{16}$-inch holes drilled at specified intervals. The wastewater effluent travels in the transport line to the manifold, which is used to feed into the laterals. The best configuration for the system is shown in Figure X-15, which shows the pump tank located at the lowest elevation in the system. This configuration will allow the transport pipe and the manifold to drain back into the pump tank during the rest cycle. Using this configuration, the discharge line from the pump should have a small weep hole that empties into the pump tank to allow the lines to drain back into the pump tank. With this configuration, a check valve is not needed on the transport line; however, the supply manifold should have a ball valve between the manifold and the lateral located 6 inches above the lateral line, as illustrated in Figure X-16. The 6-inch elevation and holes drilled below the 12 o’clock position will help the laterals drain out into the drainfield and will prevent them from draining back into the pump tank. This configuration is commonly used on steep slopes when freezing conditions may develop if the lines are not self-draining. The ball valve is used to adjust the flow so that all lines operate at the same head pressure, which is needed to keep an even flow distribution. Other configurations that use a check-valve with a header manifold pipe or a tee-to-tee connection will trap the wastewater in the lateral lines, which may create a problem during freezing conditions.

The ends of the laterals opposite the supply end should be turned up and equipped with a cleanout as shown in Figure X-17. The cleanouts should have a threaded or removable cap to allow for flushing the lateral lines and to monitor the head pressure in the laterals. The yard box covering the cleanout should be at least 6 inches in diameter and must be readily accessible for maintenance. If the orifices are placed in the 12 o'clock position, then orifice shields must be used to distribute the flow and keep small particles out of the orifice.

The design and construction of a low-pressure pipe system includes determining the orifice size and spacing, dosing volume and lateral pipe size, and sizing the pump. All of these factors must be incorporated into the design and included in the calculations of the friction loss in the pipe, drain-back volume, dosing cycles, and operating pressure for the system. A sample calculation for a low-pressure pipe system is included to demonstrate how the tables shown are used to determine the various system components.

Performance Testing

Since the low-pressure pipe system is dependent upon equal-flow distribution, the system should be tested prior to covering the trenches to be sure it is properly functioning. The most common way to test the system is the squirt test. The difference in orifice discharge rate must not exceed 10% within in one lateral and may not exceed 15% over the whole system. Once the pressure is adjusted so that the minimum residual pressure is equal in all the lines, then the system should be tested to insure the actual volume of discharge does not vary more than 15%. This is especially important in systems on a steep slope designed to drain back between dosing cycles. In this type of system, the lower elevation lateral will pressurize first and receive effluent for a longer period of time. In addition, the lower lateral may receive additional flow from the manifold or from the upper laterals.

The system should also be tested to determine pump drawdown, pump run time, timer function, and squirt height or residual head. The pump drawdown is calculated by determining the number of inches the liquid level drops in the pump tank during one cycle. From this information and the dimensions of the tank, total volume pumped during each cycle can be determined and recorded. The pump run time should be determined with a stopwatch, also be used to measure the time interval between pump cycles.

This data should be recorded at the time the system is placed into service and should be kept available at the site to aid in long-term monitoring and maintenance. The last test of the system is to determine the squirt height or residual head. The minimum residual head should be between 2-5 feet. The easiest method for measuring the residual head is to attach a clear pipe onto the end of the lateral and measure the static head, which is the vertical distance between the lateral and the top of the liquid standing in the clear pipe.

Figure X-16. Supply Manifold and Connection to Low-Pressure Lateral

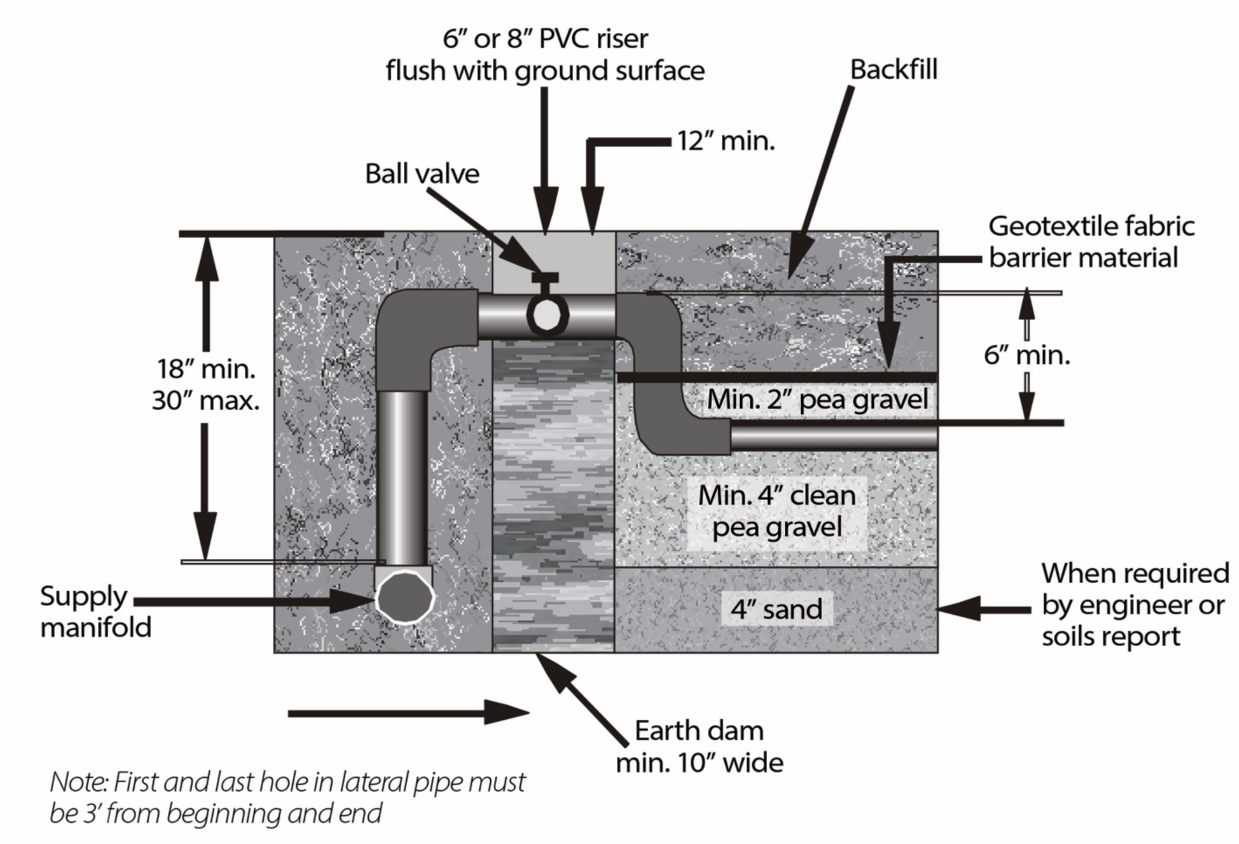
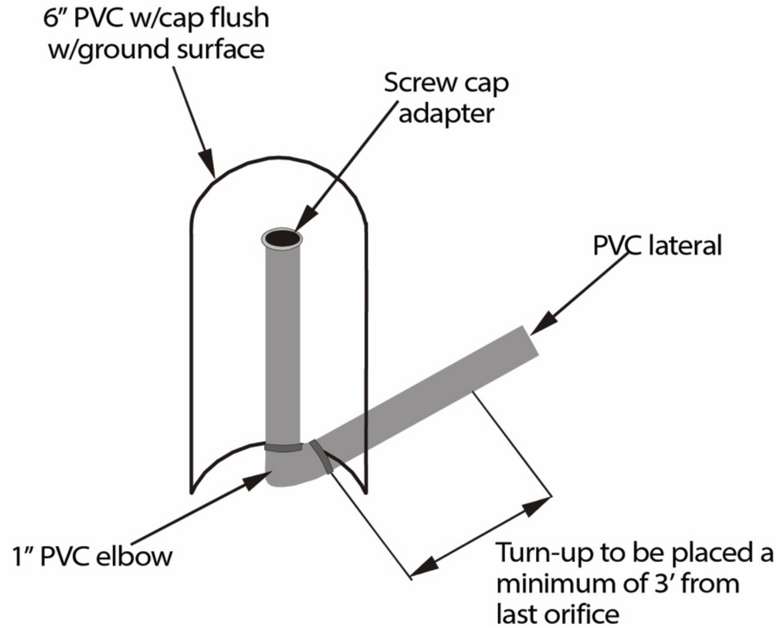


Figure X-17. Detail of Turn-Up and Cleanout for Low-Pressure Distribution for Sand Filter, Mound, or Low-Pressure Pipe Lateral



Maintenance

As discussed earlier, these systems will require maintenance. The equipment must be checked on a regular schedule to be sure screens in the tank or around the pump intake and the effluent filter are not clogged. In addition, the lateral lines should be flushed to dislodge any solids and remove the biological slime that develops in the pipes. The laterals should be tested for residual pressure and for equal flow. The pump run time and number of cycles should be checked against the original design of the system. The pump drawdown volume should also be checked periodically and may need to be adjusted with the timer in the control box. Additional items to be checked in the septic tank or pump tank include checking for sludge accumulation, checking the operation of floats, checking for signs of leaking in tank or the risers, and checking the operation of alarms present in the system.

Low-Pressure Pipe Design Example

Single-Family Residence: Design Flow = 360 gallons per day, (gpd); tables used in this section are from the EPA Design Manual for Onsite Wastewater Treatment and Disposal.

***Septic Tank*:**

a) Two-compartment septic tank with a minimum liquid volume of 1,000 gallons and equipped with an effluent filter having a maximum particle size of ⅛ inch.

b) Dosing tank with minimum of 500-gallon volume above pump intake.

***Absorption Field*:**

a) Elevation head = 15 ft from tank to highest line

b) Application rate = 0.2 gpd/ft^2

c) Minimum area = 360 gpd / 0.2 gpd/ft^2 = 1800 ft^2

d) Pipe 1-inch lines, 5 ft on center with $^3/_{16}$-inch holes drilled 6 ft on center

***Dosing Rate*:**

a) Based on $^3/_{16}$-inch holes at 4 ft of head

b) Number of holes per line = $\dfrac{\text{60ft (length) - 2 holes (3 ft from each end)}}{\text{Spacing - 6 ft on center}}$

= 10 holes per line

c) For 6 lines = 60 holes total

d) Lateral lines are 1-inch, Schedule-40 PVC

e) Flow rate per hole (according to Table X-6) = 0.83 gpm/hole

$\dfrac{\text{4 ft head}}{\text{2.31 psi/ft head}}$ = 1.73 psi

f) Total dose rate = (0.83 gpm/hole x 60 holes) + 0.83 gpm for air-vent hole

= 50.6 gpm

g) Elevation head = 5 ft (out of tank) + 15 ft (to top of absorption field)

= 20 ft

***Fitting Adjustment*:**

a) Supply line (from pump to last lateral) = 35 ft length, for a 2-inch diameter manifold at 50 gpm. See Table X-1 to find the friction loss per 100 ft of the supply line.

b) Friction head = $\dfrac{\text{35 ft (supply line length) x 3.98 ft head x 1.25 fitting adj.}}{\text{100 ft}}$

= 1.68 ft head

c) Total head = 20 ft (elevation head) + 1.68 ft (friction head) + 4 ft (pressure head)

= 25.68 ft

***Dosing Volume*:**

a) 2-4 doses per day based upon use

b) Volume in lateral lines = 4.1 gal/100 ft of 1-inch pipe (Table X-5) x 360 ft of lateral pipe

= 14.76 gallons

c) Volume in supply line = 16.2 gal/100 ft of 2-inch pipe x 35-ft supply line

= 5.67 gallons

d) Total volume in system = (volume of laterals) 14.76 gal + (volume of supply line) 5.67 gal

= 20.43 gallons

e) Volume of void space in trench = (cross sectional area) (0.5 ft x 0.5 ft) x 360 ft (length) x 7.5 gal/ft^3 (conversion factor)

= 90 ft^3 x 7.5 gal/ft^3

= 675 gal

Trench should have approximately 30% void space

= 675 gal x 30% = 203 gal

As long as the void space in the trench is smaller than the gallons per dose, the system will function.

Based upon these calculations, the pump must be sized to supply 50.6 gpm at 26 ft TDH (Total Dynamic Head).

Table X-5. Orifice Flow Rate in Gallons Per Minute (gpm) for Different Sizes and Operating Pressure Head

Pressure, psi	Head, ft	5/32 inch	3/16 inch	7/32 inch	1/4 inch
0.43	1.0	0.29	0.42	0.56	0.74
	1.5	0.35			
0.87	2.0	0.41	0.59	0.80	1.04
	2.5	0.45			
1.30	3.0	0.50	0.72	0.98	1.28
	3.5	0.54			
1.73	4.0	0.58	0.83	1.13	1.48
	4.5	0.61			
2.16	5.0	0.64	0.94	1.26	1.65
	5.5	0.66			
2.58	6.0	0.69	1.04	1.37	1.81

SUMMARY

Pumping stations may be required to overcome site obstacles. When pumping stations are needed, they must be carefully selected and designed to create a good working system. The pumping stations must be watertight and not subject to corrosion. Selection of the right pump based on pump type and pump characteristics can be time consuming but is critical to proper operation of the system. Pumping stations must be accessible to allow for service and maintenance on the pumps.

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