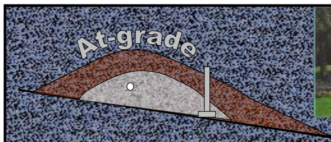
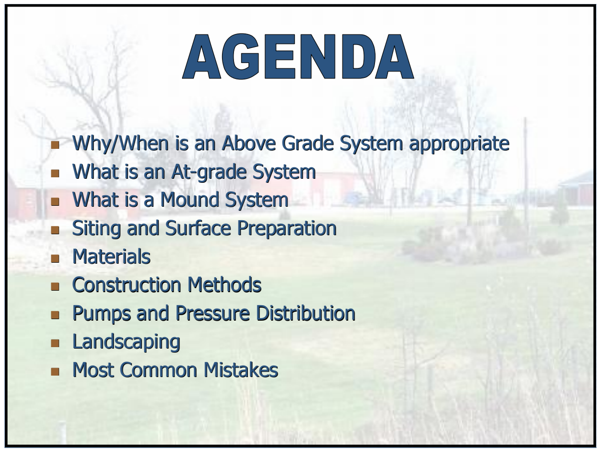


Mounds and At-Grades the Easy Way

Speakers: Bill Zimmerman
Hillary Maurer



AGENDA

- Why/When is an Above Grade System appropriate
- What is an At-grade System
- What is a Mound System
- Siting and Surface Preparation
- Materials
- Construction Methods
- Pumps and Pressure Distribution
- Landscaping
- Most Common Mistakes

Why do we need above grade systems?

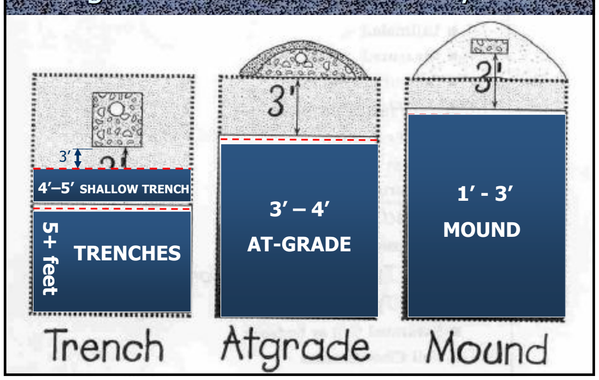
- Need 3 feet unsaturated soil to treat wastewater
  - Aerobic bacteria cannot survive without air
  - Saturated soil passes contaminants and disease-causing organisms untreated into groundwater
- Why 3 feet?

Soil Absorption Treatment

Parameters	Raw waste	Septic tank effluent	One-foot below trench bottom	3-feet below trench bottom
$CBOD_5$ (mg/l)	270-400	140-220	0	0
TSS (mg/l)	300-400	45-65	0	0
Fecal coliform (MPN/100ml)	1,000,000 – 100,000,000	1000 – 1,000,000,000	0-100	0
Viruses (CFU/ml)	unknown	1000 – 1,000,000.000	0-1000	0
Total Nitrogen (mg/l)	100-150	50-60	<30	*
Total Phosphorus (mg/l)	10-40	10-30	<10	*

* background

Design Choices – Soil Based Systems



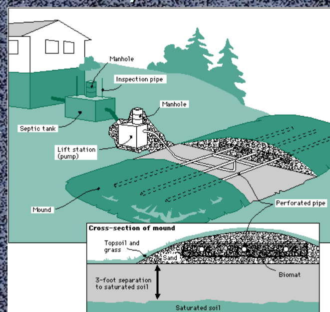
Above-Grade Systems

Mound

At-Grade

When to Use an Above-Grade System

- High water table
- Shallow bedrock
- Slowly permeable soils
- Rapidly permeable soils
- Limited space

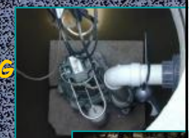


Above-grade systems

Both constructed on NATURAL SOIL

Both require PRESSURE DOSING using an EFFLUENT PUMP

Effluent Filter Strongly Recommended



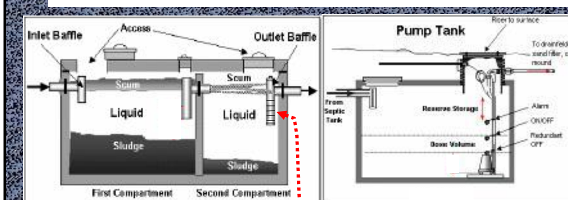
Basic Components

At-grades and Mounds

Tanks for At-Grades and Mounds

SEPTIC TANK CAPACITY MUST BE A MINIMUM OF _____ GALLONS.
Watertight tank, not subject to corrosion or decay. Withstands top and lateral earth pressure. Contractor shall verify that tank is watertight.

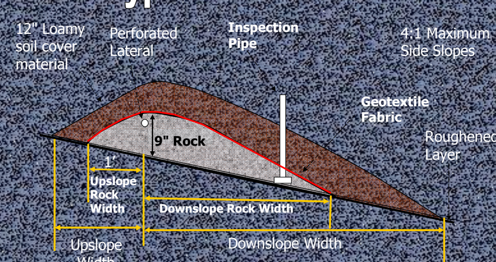
PUMP/DOSING TANK minimum capacity should be 1.5 times daily use.

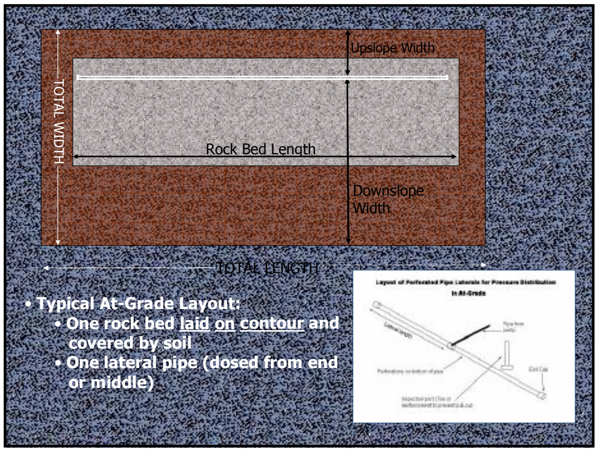


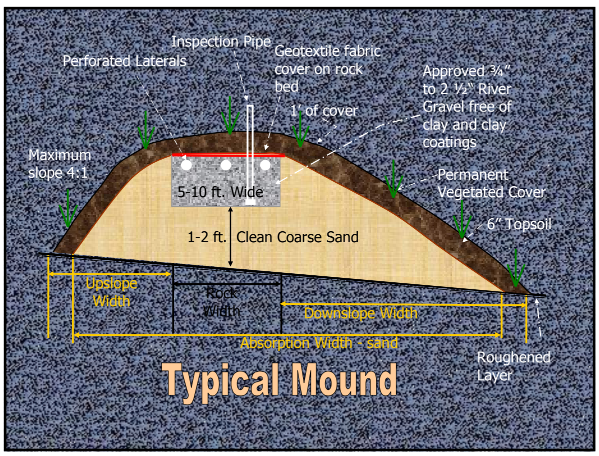
Effluent Filter!!!!

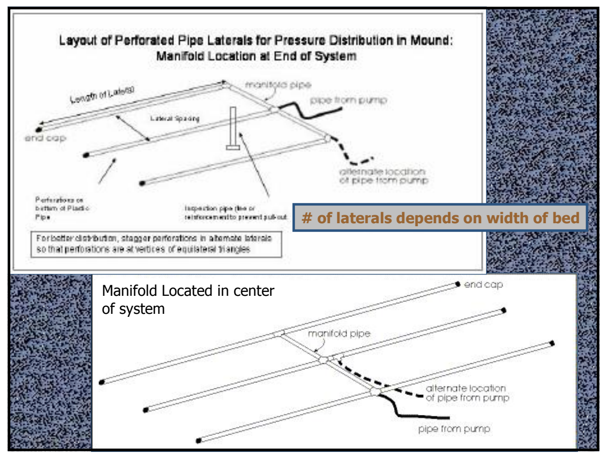
Keep Groundwater OUT

Typical At-Grade









Mound and At-Grade Design Considerations

- Depth to the Limiting Layer
- Flow Rate (gallons per day)
- Loading Rate/Absorption Rate (a.k.a. Soil Sizing Factor) (sq.ft./gpd)
- Linear or Landscape Loading Rate (gpd/ft)
- Rock Bed should be Narrow!
  - 5-10 feet for Mound
  - 6-15 feet for At-grades (based on LLR)
- Mound Sand Depth (1-2 feet based on limiting layer)

FIRST - Review Design

- Review designer's/engineer's assumptions
- Verify soil has not been disturbed
- Verify depth to Limiting Layer
- Pipe size & hole spacing
- Review pump specifications (Calculate total dynamic head (TDH) before buying)
- When in doubt, STOP AND ASK QUESTIONS

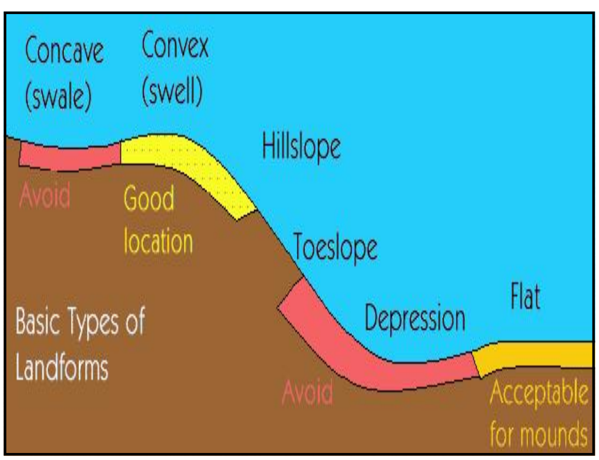
Double Check Depth to Limiting Layer

Look for Gray or Mottled Soils and Water

SITING, SURFACE PREPARATION, AND CONSTRUCTION

Site Adaptation

- Engineer/Designer Drawings – Concepts
- Property lines
- Underground utilities
- Site access
- Drainage
- Landscaping
- Elevations



Placing the Mound or At-Grade

Contour

Slope < 6%

Do not put in drainageway/swale

Divert surface water

- Swale above berm
- Interceptor drain when necessary

KIN- Keep it NATURAL

Beware of Compaction

From
- Existing conditions
- Equipment
- Materials – Do not store on treatment area

KIL – Keep it LEVEL

Meet with Owner to discuss final appearance!

Communication is the KEY

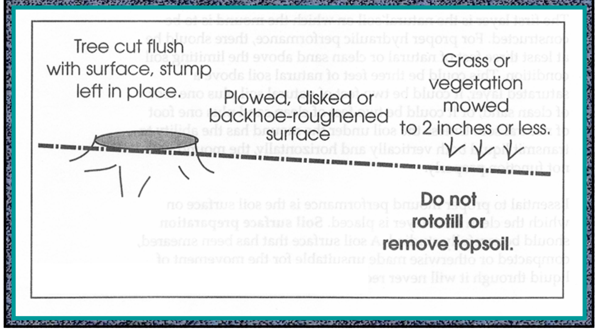
Where Are We?

- Reviewed design with designer and/or health department
- Verified Limiting Layer/Soil Conditions
- Staked out the site and **REVIEWED WITH PROPERTY OWNER**
- Asked questions & planned construction
- Now…Let's move dirt!

KID- Keep it DRY

- Check soil (ribbon)
- Do NOT begin when soil is wet
- Best construction months are June - September

Surface Preparation



Site preparation – If vegetation is dense, may be best to carefully remove it

Specialized equipment is used to prepare soil and make trenches to lay pipe

Can be "homemade" or adapted from other uses

- Prepare the surface by roughing topsoil
- Do NOT turn soil over

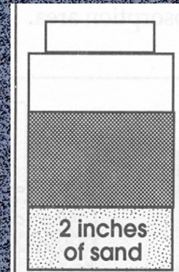
Bury force main

Deliver materials away from mound and downslope areas

Sand Quality – Coarse and No Fines

THE JAR TEST

- PLACE 2" OF SAND IN 1 QUART JAR
- FILL WITH WATER ¾ FULL
- SHAKE



If the fines that settle out in 1 hour accumulate to a depth of greater than **1/8 inch,** then the percentage of fines is too great and the sand **should not** be used for mound construction.

ROCK

- Clean
- ¾ inch – 2 inches
- Washed river gravel, free of clay and clay coatings
- Limestone or crushed rock is not recommended, but can be used if meets specs.

Rock Alternatives

EZflow replaces rock in a mound

Infiltrator chamber replaces rock in at-grade (Picture not taken in Iowa!)

Pressure Distribution

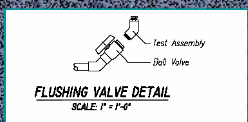
- Check designer's specs
- Most designs call for 1/8" - 1/4" perforations
- Perforation Preparation
  - impacts flow
  - impacts distribution
  - goal is even distribution
- Prep pipe in shop – holes evenly spaced, straight, burrs removed

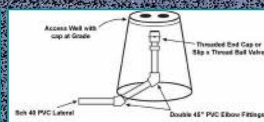
Pipe

- Schedule 40 PVC –pressure pipe
- Pipe Strength
- Joints
- Glue

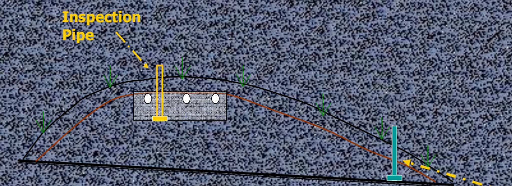
Plan for Easy Maintenance

- Flushing valves make line cleaning easier
- Can buy preassembled or make your own
- Lines with ¼" holes need to be cleaned every 2 – 3 years!





Inspection Pipe(s)



To check for signs of ponding water on gravel/sand interface

Can also install inspection pipe to check soil/sand interface

Install inspection pipe here on at-grade (rock/soil interface)

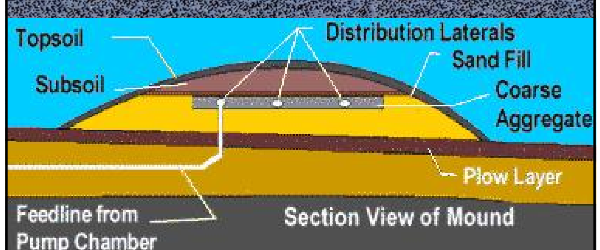
CHECK SQUIRT HEIGHT

COVER PIPE

Geotextile Fabric

Non-woven fabric allows air & water movement

Cover – Good Topsoil



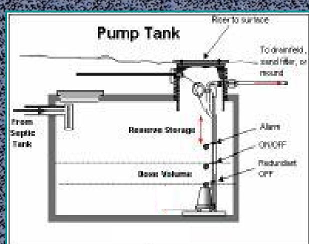
- Mounded
- Same texture as surrounding soil

Topsoil

Pumps Pressure Distribution & Controls

DOSING/PUMP CHAMBER

- CAPACITY
- WATERTIGHT
- VENTING
- ACCESS TO PUMP
- WIRING
- FLOAT CONTROLS

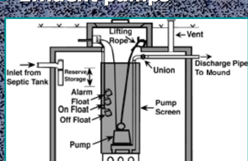


Above Grade Systems

- Need Even Pressure Distribution
  - Less than 10% discharge variation
- Need Adequate Head Pressure
  - At least 1 foot of head for typical single-family home
  - At least 2 feet of head for all other uses

Pump Types

- **Sump pumps**
  - **Not used for septic systems**
- **Sewage pumps**
  - **Ejector pumps**
  - **Grinder pumps**
- **Effluent pumps**



What's wrong with this setup?

Pump Specifications

- Parts
  - Motor
  - Casing
  - Impeller
- Sizing
  - Capacity (gpm)
  - Head
  - Pump Curve

Pump Selection

- Before buying pump
  - Need to know required gallons per minute
  - Need to know Total Dynamic Head
- Check pump curves to find best pump

Pump Selection

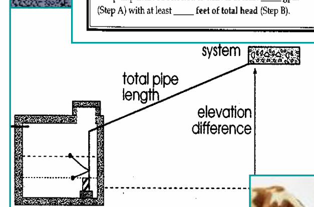
Use required gallons/minute and TDH to select pump

Friction Loss in Plastic Pipe
Per 100 feet

flow rate gpm	nominal pipe diameter 1.5"	2"	3"
20	2.47	0.73	0.11
25	3.73	1.11	0.16
30	5.23	1.55	0.23
35	6.96	2.06	0.30
40	8.91	2.64	0.39
45	11.07	3.28	0.48
50	13.46	3.99	0.58
55		4.76	0.70
60		5.60	0.82
65		6.48	0.95
70			

C. Pump selection

1. A pump must be selected to deliver at least ____ gpm (Step A) with at least ____ feet of total head (Step B).



Do NOT select pump by horsepower!

Calculate Required Pump Discharge Rate

Example

Pressure Distribution Information

Manifold Length	6	feet
Manifold Diameter	2	inches
Center or End Manifold	end	
# of Laterals	3	laterals
Lateral Length	43	feet
Lateral Diameter	2	inches
Spacing Between Laterals	3	feet
Hole Diameter	0.25	inch
Hole Spacing	30	inches
# of Holes/Lateral	18	holes/lateral
Square foot/Hole	7	(Recommended 6-10 sqft/hole)
Discharge Rate/Hole	0.74	gallons/minute/hole
Lateral Discharge	13	gallons/minute/lateral
Pipe Volume	24	gallons

Pump Information

Number of Pumps	1	pump
Flow per Pump	450	gallons/day/pump
Estimated Doses/day	4	doses/day
Estimated Dosing Volume	113	gallons/dose
Minimum Dosing Volume	121	gallons/dose (5X pipe volume)
Minimum Discharge Rate of Pump	**40**	**gallons/minute**

E-30 Volume of Liquid in Pipe

Pipe Diameter inches	Gallons per foot
1	0.045
1.25	0.078
1.5	0.11
2	0.17
2.5	0.25
3	0.38
4	0.66

E-9 Friction Loss in Plastic Pipe
Per 100 feet

flow rate gpm	nominal pipe diameter 1.5"	2"	3"
20	2.47	0.73	0.11
25	3.73	1.11	0.16
30	5.23	1.55	0.23
35	6.96	2.06	0.30
40	8.91	2.64	0.39
45	11.07	3.28	0.48
50	13.46	3.99	0.58
55		4.76	0.70
60		5.60	0.82
65		6.48	0.95
70		7.44	1.09

Calculate Required TDH

Example

Additional Pump Information
(to be completed by contractor at time of installation)

Friction Loss (in feet per 100 feet of main)	2.64	feet
	x	
Length of Force Main	125	
	=	
Friction Loss in Force Main	330	
	x	
Friction loss in Force Main X 1.25 (for fittings)	413	
	÷ 100	
Total Friction Loss =	4.13	
Amount of Head Required at Manifold +	5	feet
Elevation Difference Between Pump and Point of Discharge +	9	
Total Head Required =	19	

Friction Loss for 2" pipe @ 40 GPM = **2.64**

NOTICE TO CONTRACTOR
Do NOT purchase pump until you have completed this section.

Add
Total Friction Loss +
Amount of Head Required at Manifold +
Elevation Difference between Pump and Point of Discharge to find TOTAL HEAD REQUIRED

USE THIS INFORMATION TO SELECT CORRECT PUMP

The pump selected must deliver at least 40 gallons per minute with at least 19 feet of head

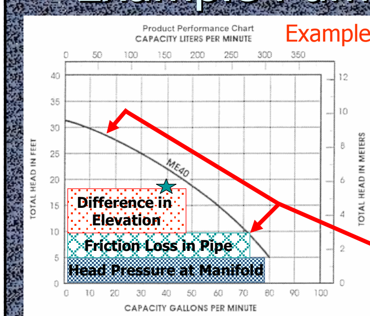
Reading Pump Curves

- Flow Rate in gallons/minute
- T.D.H. – Total Dynamic Head Pressure

NEED TO KNOW:

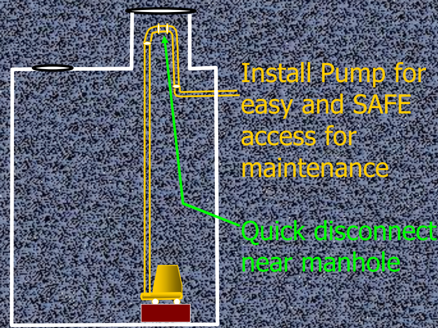
1. Change in elevation
2. Friction loss in force main
3. Pressure requirements for the system
   - For all pressure distribution add 3-5 feet of head pressure at manifold

Example Pump Curve



Choose the Right Pump:

1. Know what you need before buying
2. Stay close to – but under - the curve line. Do NOT oversize!
3. Stay in middle 2/3 of curve



Controls

- Floats
- Alarms
- Switches
- Timers
- Event counters
- Running time clocks

Electrical Connections

- **Follow National Electrical Code**
- **Use qualified (licensed) electrician**
- **Pump, alarm, and dosing requirements**

LANDSCAPING THE SYSTEM

What you need to tell your clients

Landscaping

- No trees on bed

Guidelines for Planting On and Near Mounds/At-Grades

- Plant vegetation to prevent erosion
- Topsoil on mound/at-grade should be 6-12 inches in depth
- Use minimal tilling when planting
- Around berm, use plants that do not like water or wet soils (type of root system important)
- On top, use plants that can withstand very dry conditions

Trees and Shrubs

- Do not plant on top of system
- Small shrubs ok on side slopes
- Trees - Minimum of 20 ft from the edge bed
- Minimum 50 ft for
  - poplar, maple, willow and elm
- Avoid irrigation and fertilization on mound/at-grade

Wildflowers

- Daylilies and peonies and other low maintenance plants can be grown
- Extra care must be taken to mulch or plant close together so soil will not be exposed and erode.
- Do not plant edible plants
- Remember minimal traffic on systems is best

Low maintenance lawn grasses

- *Fine fescues* can make a dense cover and only need to be mowed once or twice a year. Tolerate dry soils & shady sites.
- Kentucky bluegrass and perennial ryegrass can be planted on a mound if mowed regularly

Landscape!

Or Leave in Grass

Most Common Construction Mistakes

- Improper soil evaluation
  - Didn't identify Limiting Layer
  - Didn't identify soil texture and/or structure
- Alteration of site
  - Soil not "natural"
  - Compaction
  - Didn't prepare site correctly

Most Common Construction Mistakes

- Wrong equipment
  - Destroy structure of soil
  - Compact area
- Materials
  - Tanks not watertight
  - Sand not coarse
  - Wrong type or size of pump

Most Common Construction Mistakes

- Not designed & built for easy maintenance
  - Risers
  - Septic tank filter not accessible
  - Pump quick disconnect
  - Line cleaning – flushing valves
  - Inspection pipes
- Didn't consult homeowner

When You Have Questions …

- STOP work until you get them answered
- We're all on the learning curve, so work with each other
- Contact "experts" for advice

Worksheets for sizing pump chambers & selecting proper pump can be found at the University of Minnesota Extension Service Website:

http://septic.coafes.umn.edu/professional/worksheets/index.html

We would like to acknowledge that various drawings and photos were obtained from University of Minnesota Extension Service and Washington State pamphlets and worksheets. Both states offer very fine public information programs for general use by onsite professionals.

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  B & L Zimmerman Construction
  Parnell, IA

Chapter X
Pumps and Hydraulics

TABLES

FIGURES

INTRODUCTION

Pumps are often utilized in onsite 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 in an aerobic treatment unit (ATU)
- lift drainage water to lower a water tables

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

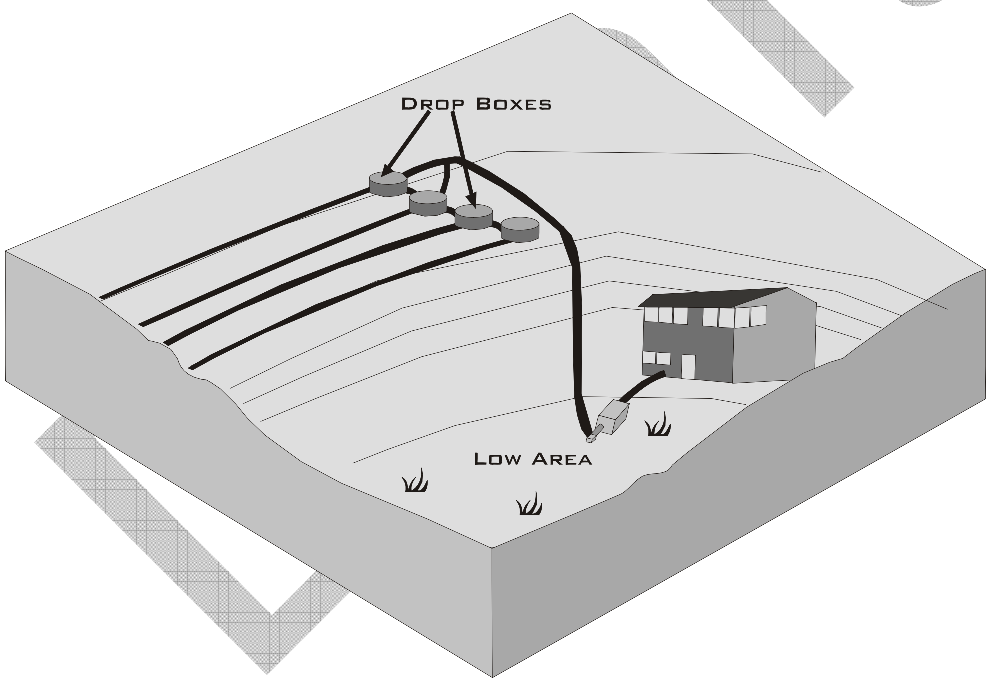


Figure X-1. Pump Used to Lift Wastewater to a Higher Elevation than the Source

PUMP APPLICATIONS

Pumping raw sewage requires that the pump be designed 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 grind up tough and stringy solids, such as a rag or wet strength towels to avoid clogging the pump. Common applications for pumping raw sewage are when a lagoon is at a higher elevation or plumbing fixtures in a basement must be lifted up 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 so 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 onsite 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 onsite 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 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 located following some type of 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 the screens are plugged and water cannot enter the intake, the pump may be damaged.

When an effluent pump is used to deliver to a conventional absorption field, flow may discharge to a drop box as shown in Figure X-2. On a sloping as shown in Figure X-1 water that upper laterals can not 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 from the pump is directed to the wall of the drop box, opposite from the inlet 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, then a series of drop boxes (see Figure X-3) as shown in Figure X-1 may be used to direct the flow through the absorption field.

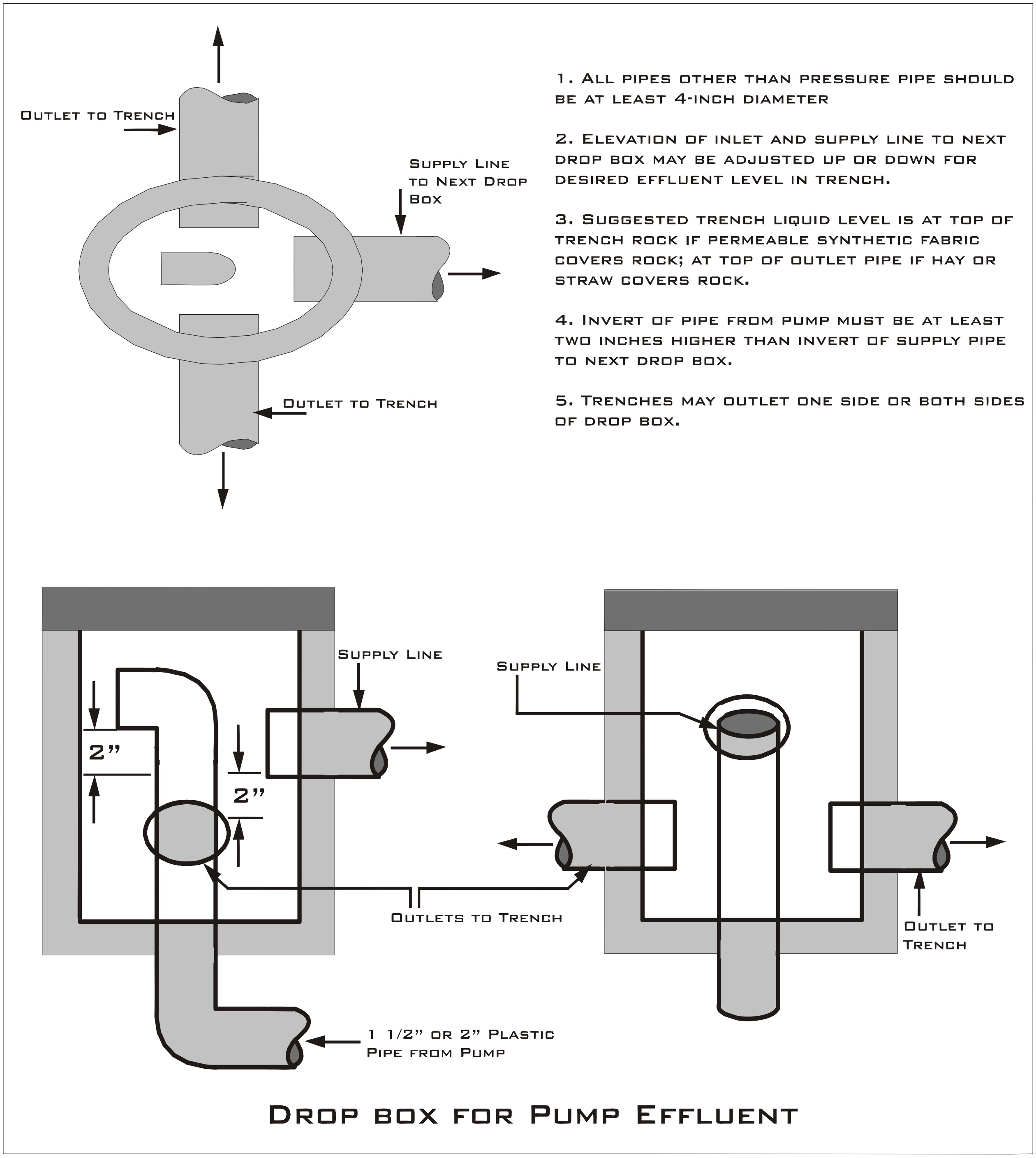


Figure X-2. Pump Discharge Delivery to a Gravity Distribution, Lateral Drop Box

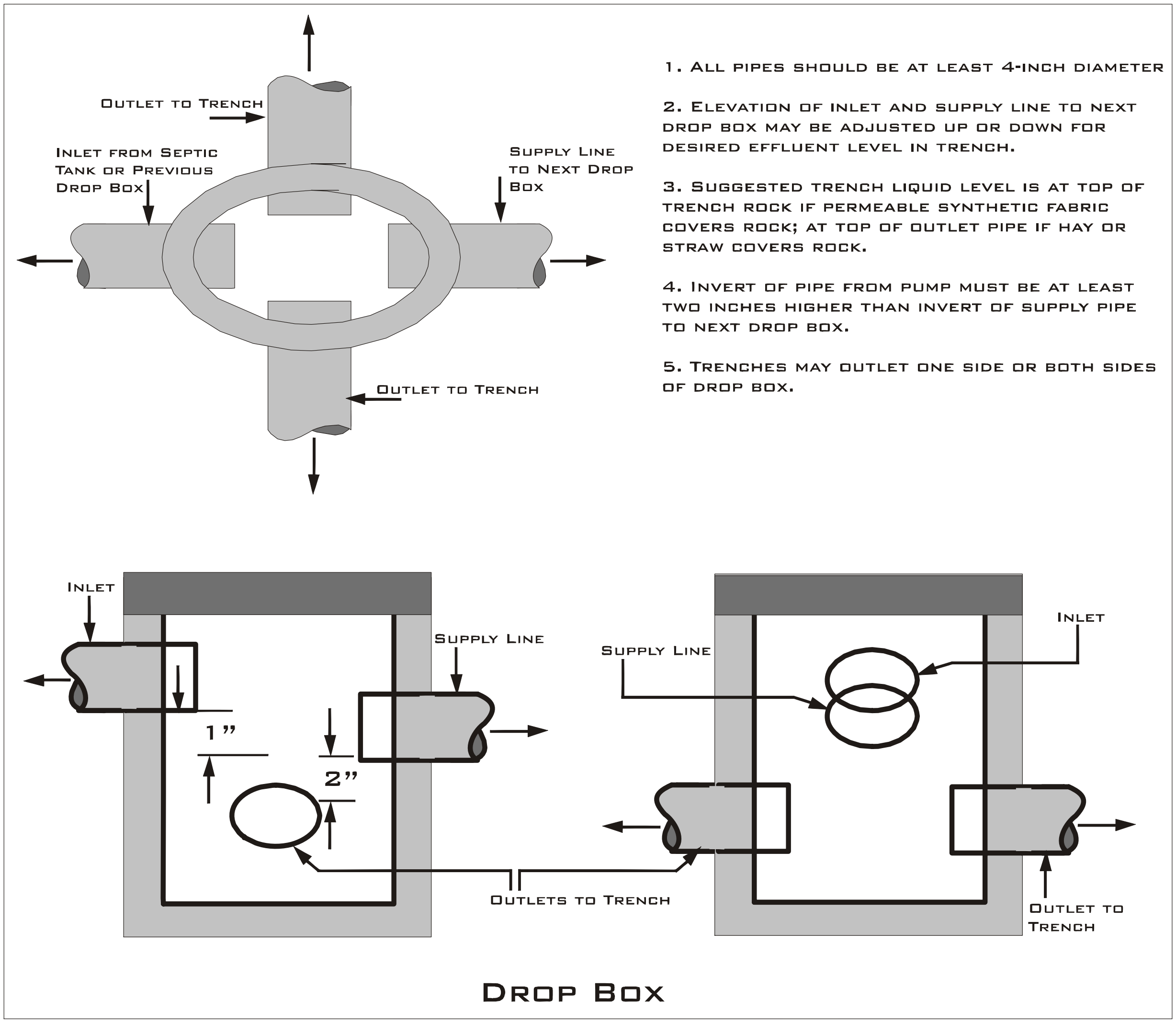


Figure X-3. Gravity Distribution Lateral Drop Box

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. 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.

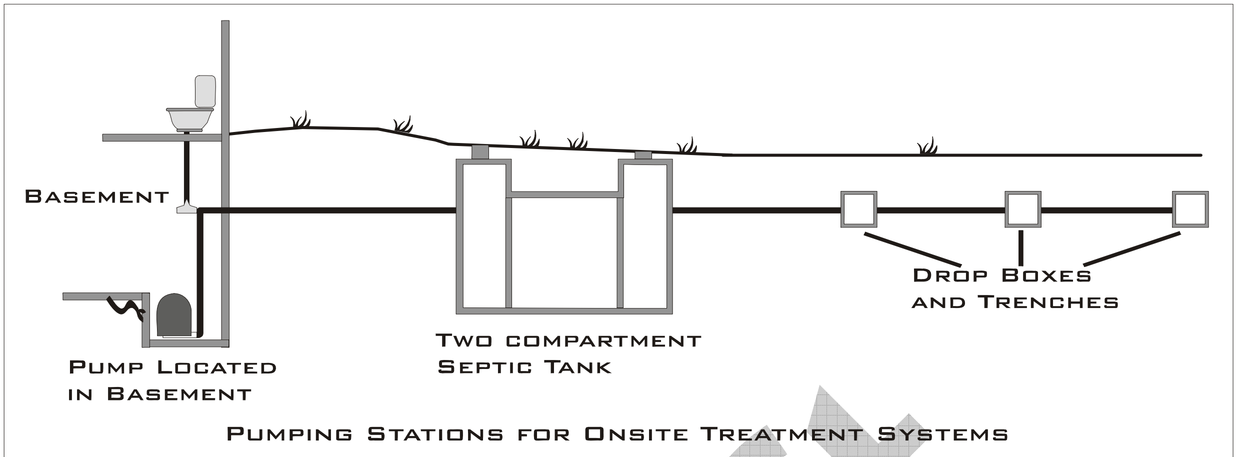


Figure X-4. Gravity Serves Ground Floor and a Pump Lifts Sewage from Basement

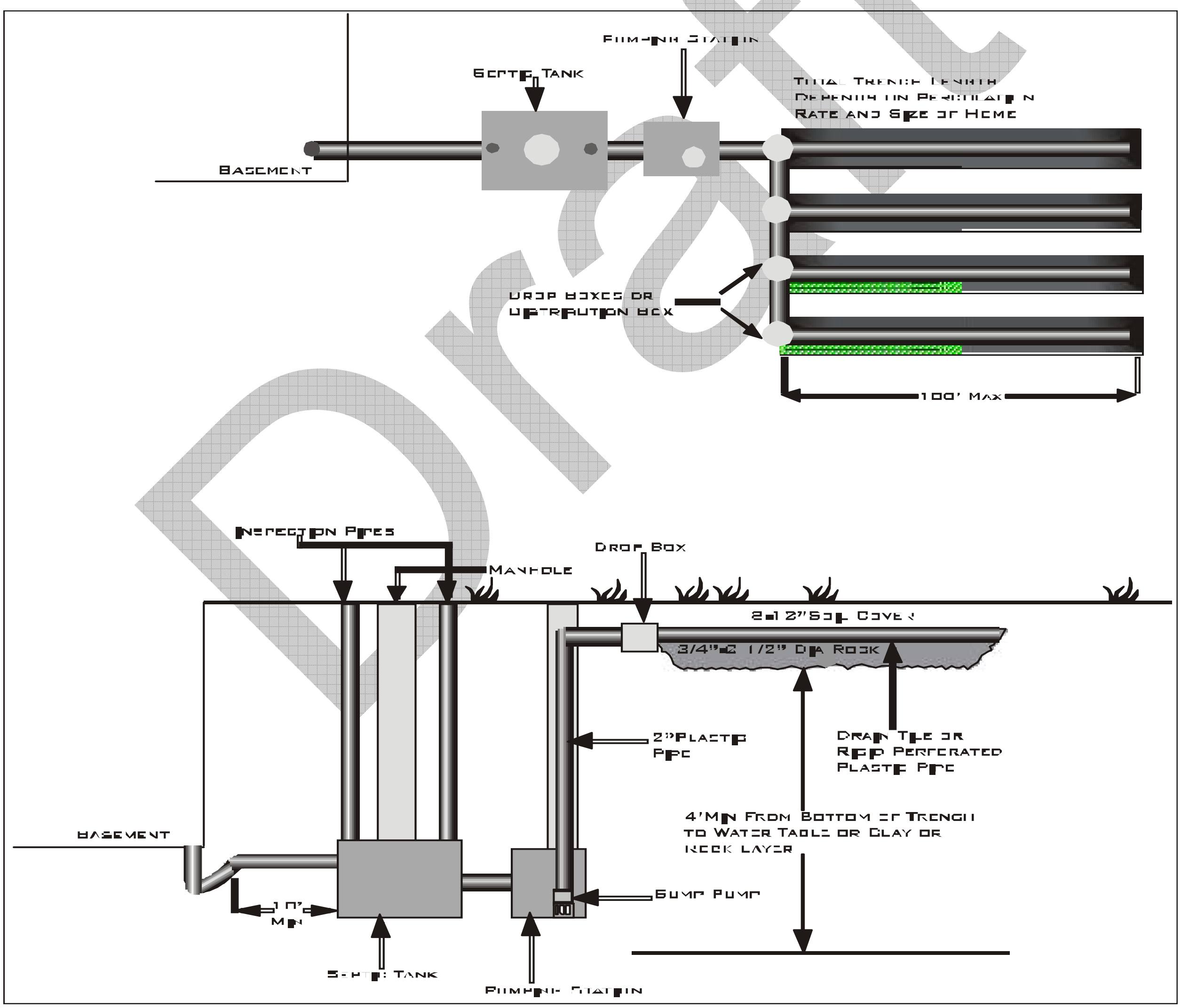


Figure X-5. Pump Lifts Tank Effluent to a Shallow Absorption Field

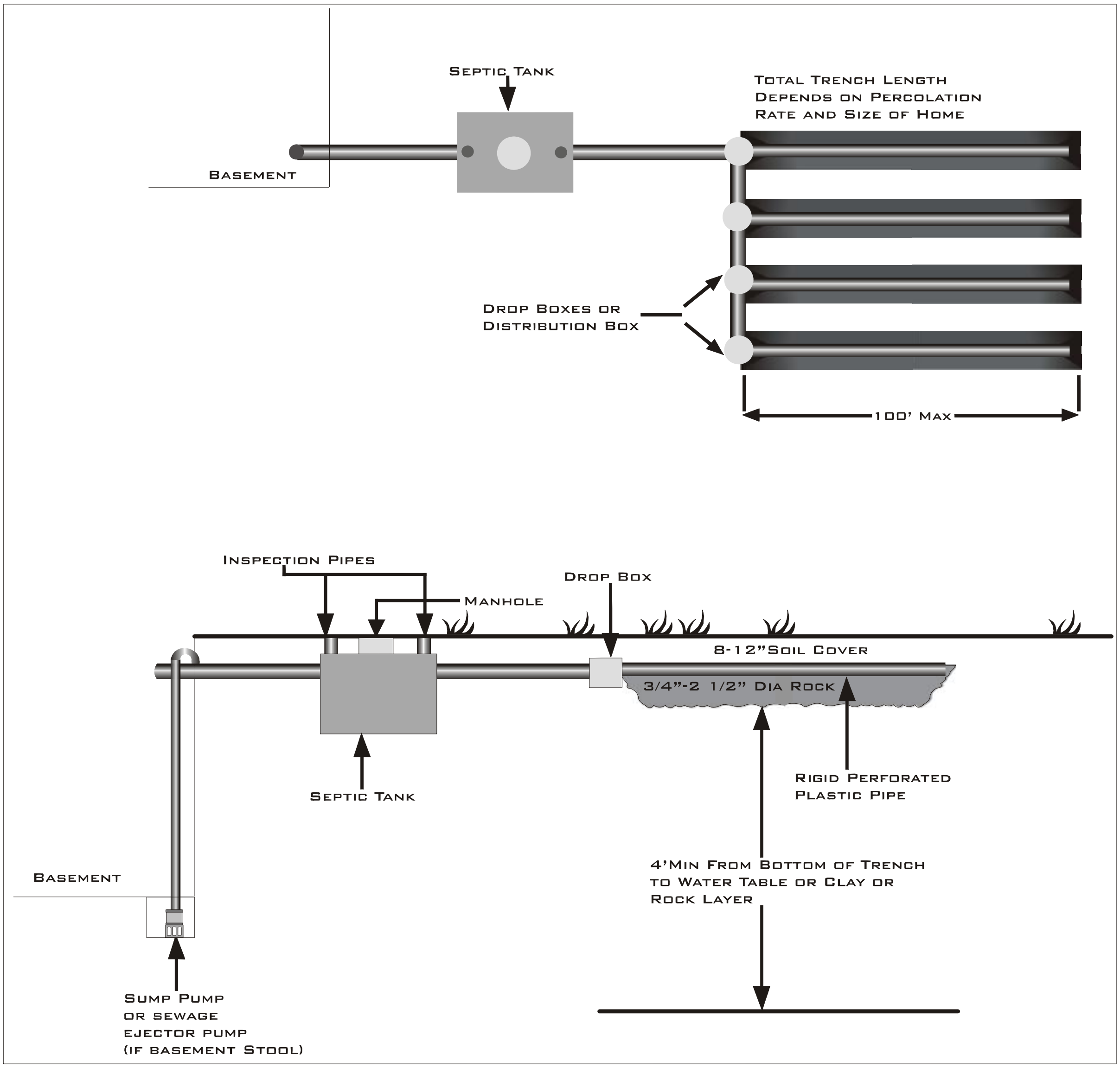


Figure X-6. Pump Lifts Sewage to Shallow Septic Tank Placement for Easier and Much Less Costly Maintenance

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 which 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 which is located outside the house and is accessible for any 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 that 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 be pumped 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 to 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 that 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. Filters should also be used in pump stations which are used to 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. The grinder pump is equipped with blades mounted on the impeller which literally grind up and shred solids before they enter the impeller. The grinder pump must grind up the solids to a size that will not clog the impeller. Two types of grinder pumps are currently available; a centrifugal grinder pump and a positive displacement grinder pump. The centrifugal grinder pump is more common, but this pump usually is 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 which serves some type of central wastewater treatment facility.

A pump handling solids, or 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 which are suspended in the pump tank. The floats may be mechanical or mercury floats and may be rated as “normally on” or “normally off.” In any 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 the float 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 that 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 then 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 and should be set to provide additional reserve capacity in the event of pump failure. 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 that the floats should be tethered to a rigid "float tree" which is mounted in the tank. The floats should be on a short tether and must be carefully set so that they do not 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 hung up and not able to work properly. Also, the vertical location of the pumps must be carefully set up 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. Setting the floats at the proper height and configuration should be done by a qualified installer.

Some pumping stations may be equipped with a timer to operate the on/off cycle of the pump. These pump 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 sand or textile filter. These systems also have low and high water alarms but the on/off cycle is controlled by the timer. The timer is located in the control box and 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

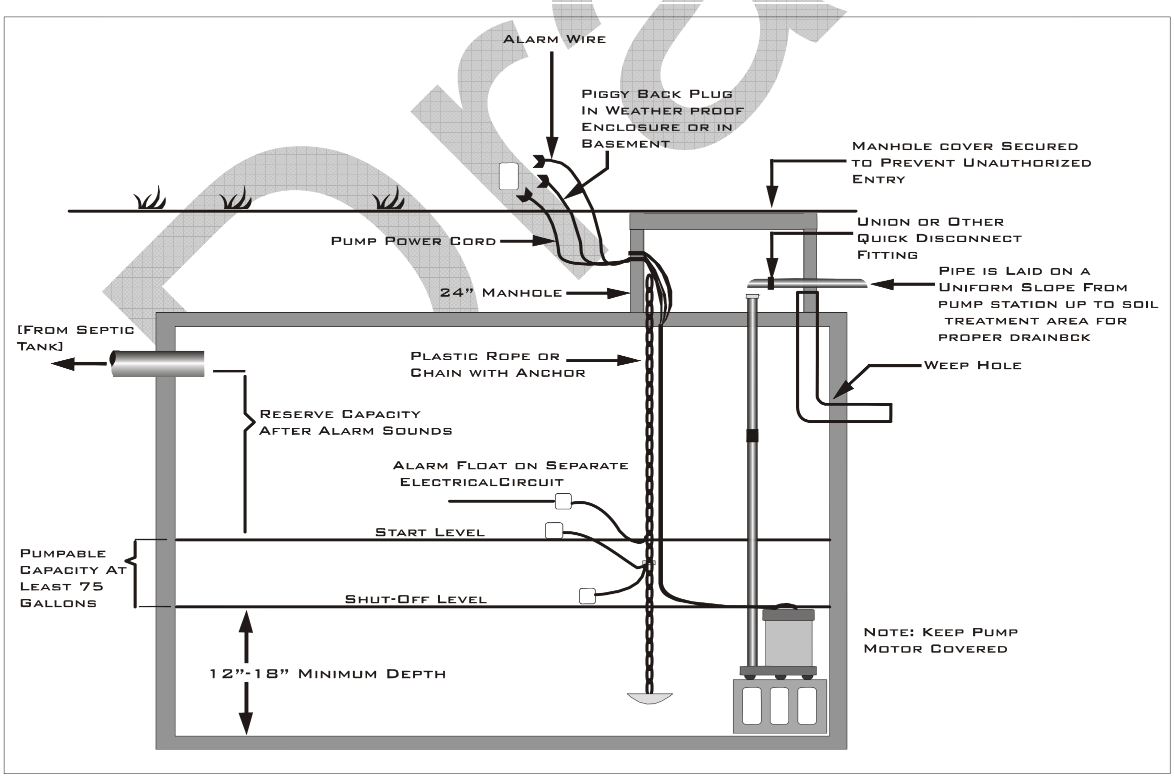


Figure X-7. Section of Pump Tank Showing Pump, Wiring, and Float Locations

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. The control box 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 of the control boxes have a silencer to shut-off 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.

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 which alternates which pump is 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

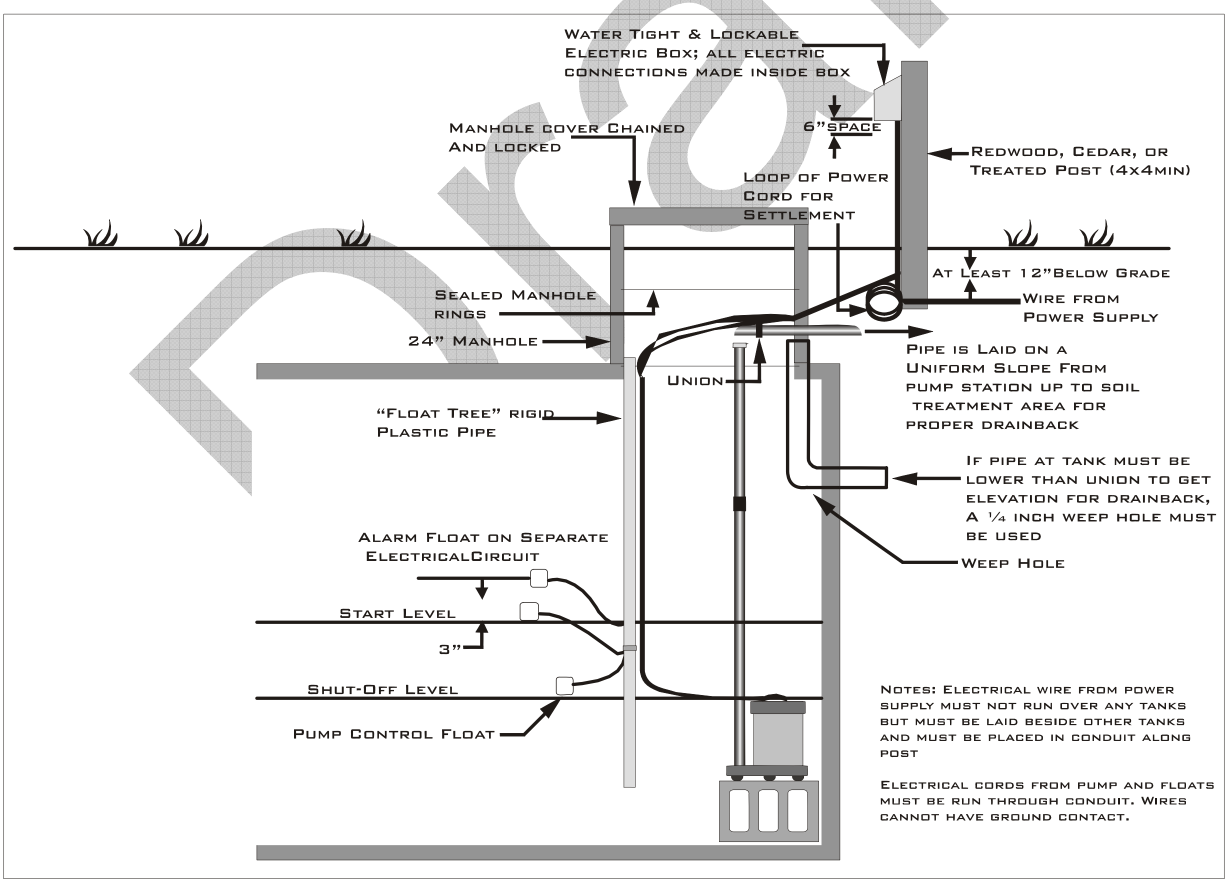


Figure X-8. Section of Pump Tank Showing Wiring and Electricity Away from House

configuration is called the lead pump and the lag pump. All duplex control panels should be wired to operate in this way.

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. 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.

The setting of the pump control depends on the surface area of the tank. For example, if a pumping 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 in this tank would contain 7.5 gal. x 20 sq.ft. = 150 gallons. Thus, to pump 150 gallons would require that the pump start level 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

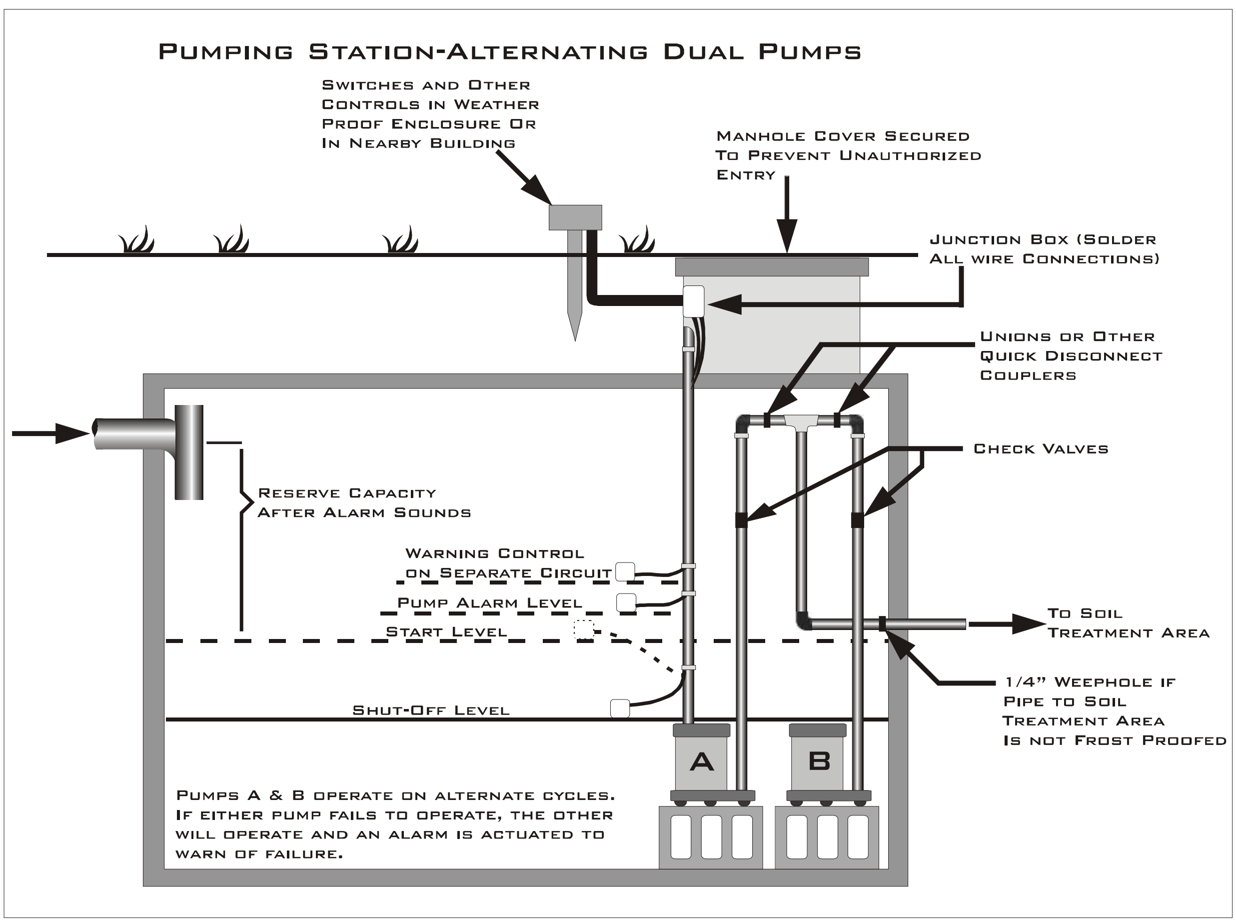


Figure X-9. Duplex Pump Station has Two Alternating Pumps

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 total pump dose of 150 gallons.

For a circular tank, which is four feet in diameter, the surface area is calculated as pi(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 l foot depth in the tank, there are 12.5 cubic feet = 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 must also be added into the flow to provide the proper dose.

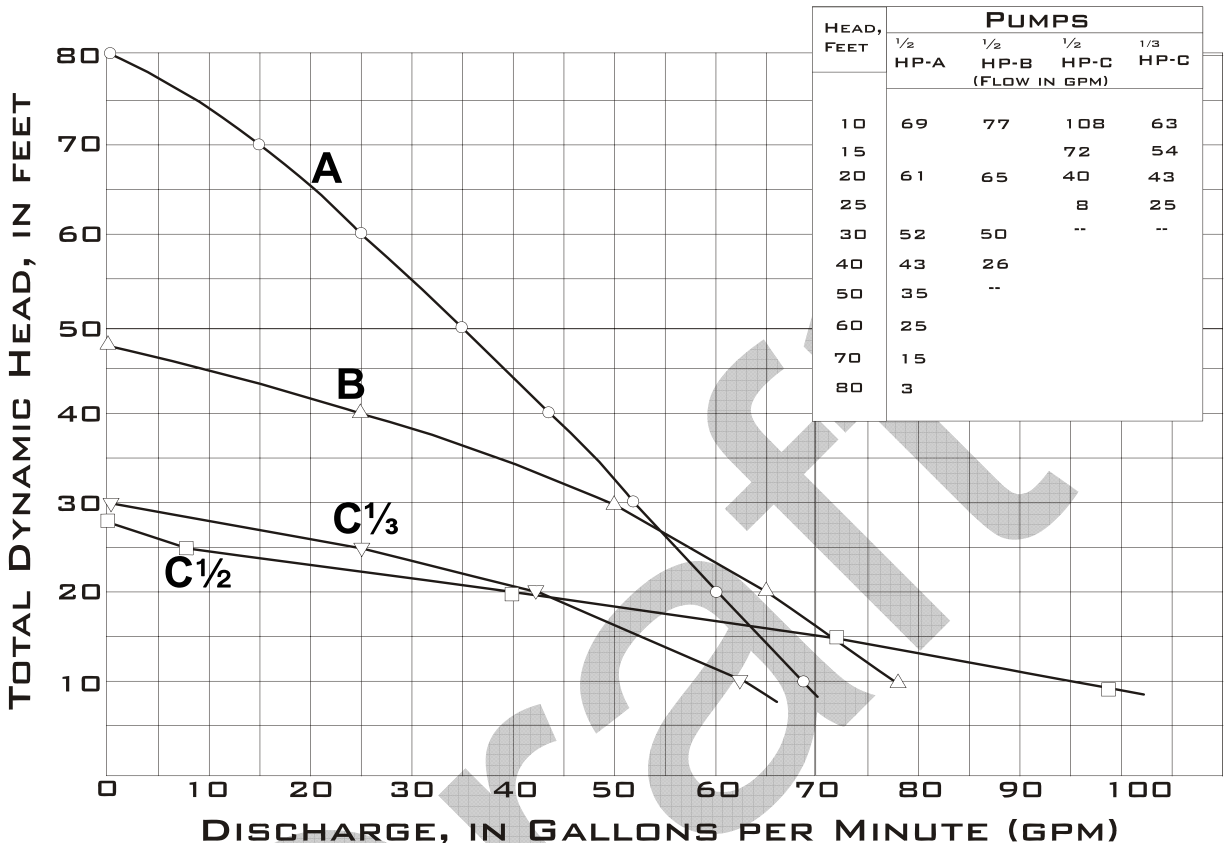
PUMP DISCHARGE CURVE

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 delivers will decline. The point at which the characteristic curve intersects the vertical axis is the maximum head that the pump develops and is often called the shut-off head. The maximum head for pump C with a 1/3 horsepower as shown in Figure X-10 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. This information can be read from the graph or the table in Figure X-10.

Referring to Figure X-10, the shutoff head of the ½ horsepower pump 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 the requirements for a system are a pump which will deliver 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.



Head, Feet	½ HP-A	½ HP-B	½ HP-C	1/3 HP-C
	(Flow in gpm)			
10	69	77	108	63
15			72	54
20	61	65	40	43
25			8	25
30	52	50	--	--
40	43	26		
50	35	--		
60	25			
70	15			
80	3			

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

Table X-1. Schedule 40 Plastic Pipe Friction Loss, Diameter, and Volume

Pipe Diameter	1"	1.25"	1.5"	2"	2.5"	3"	4"
Inside dia.	1.05"	1.38"	1.61"	2.067"	2.47"	3.07"	4.03"
Gals/100ft	4.4	7.8	10.6	17.4	24.9	38.4	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

Friction Loss in ft/100ft = $1042 \times [\_Q\_]^{1.85} / c \times d^{2.63}$

where Q = Flow (gpm)

d = Internal Pipe Diameter (inches)

c = Pipe Constant (150)

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 factor. 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. The three factors that determine the total dynamic head of a pump are:

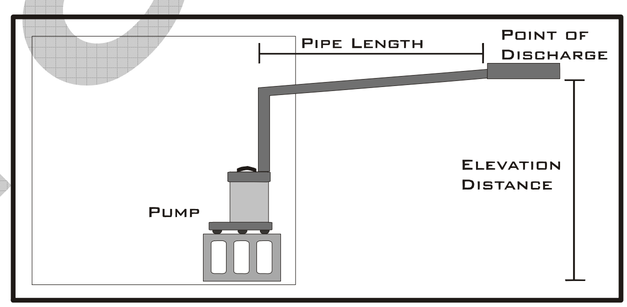
- elevation difference between the pump and point of discharge,
- the pressure requirements of the operating system, and
- the friction loss in the piping.

For the example in Figure X-11 (11A), assume that there is an elevation difference of 17 feet between the top of the pump and the manifold 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, then zero would be put in section B, item 2. 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-11A

	Head Perforation Diameter (inches)	
	7/32	¼
1.0a	0.56	0.74
1.5	0.69	0.90
2.0b	0.80	1.04
2.5	0.89	1.17
3.0	0.98	1.28
4.0	1.13	1.47
5.0	1.26	1.65

a Use 1.0 foot of head for residential systems
b Use 2.0 feet of head for other establishments



Friction loss depends on the flow rate, type of pipe, pipe diameter, and pipe length and fittings. The friction loss for 40 gallons per minute in 2 inch pipe is 2.64 feet per 100 feet. Note from the table that friction loss increases very rapidly as the pipe diameter decreases. For example, the friction loss for a 40 gpm in 1½ inch diameter pipe is 8.91 feet per 100 feet.

Another factor is the length of the pipe and the fittings through which the effluent flows from the pump to the point of discharge. The friction loss within the pressure distribution laterals is included in that design. But the friction loss from the pump to the manifold must be calculated and added to the pump head requirements.

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.

Assume that the distance from the pump to the manifold of the mound is 140 feet. Multiply the delivery line length by 1.25, the factor to allow for fitting losses, results in a total equivalent length of 175 feet. The total friction loss is 175 x 2.64 feet per 100 feet or 4.6 feet of friction loss. The total head requirement for this system is 17 + 5 + 4.6 or 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 1/3 horsepower and the ½ horsepower low head pumps. Thus, these pumps are not suitable for these pumping requirements. If these pumps were the only ones available, the flow would need to be reduced by using fewer or smaller perforations, 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 be equivalent to about 600 watts of power consumption. 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 in amperage by the voltage 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. Knowing the kilowatt hours, the 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.

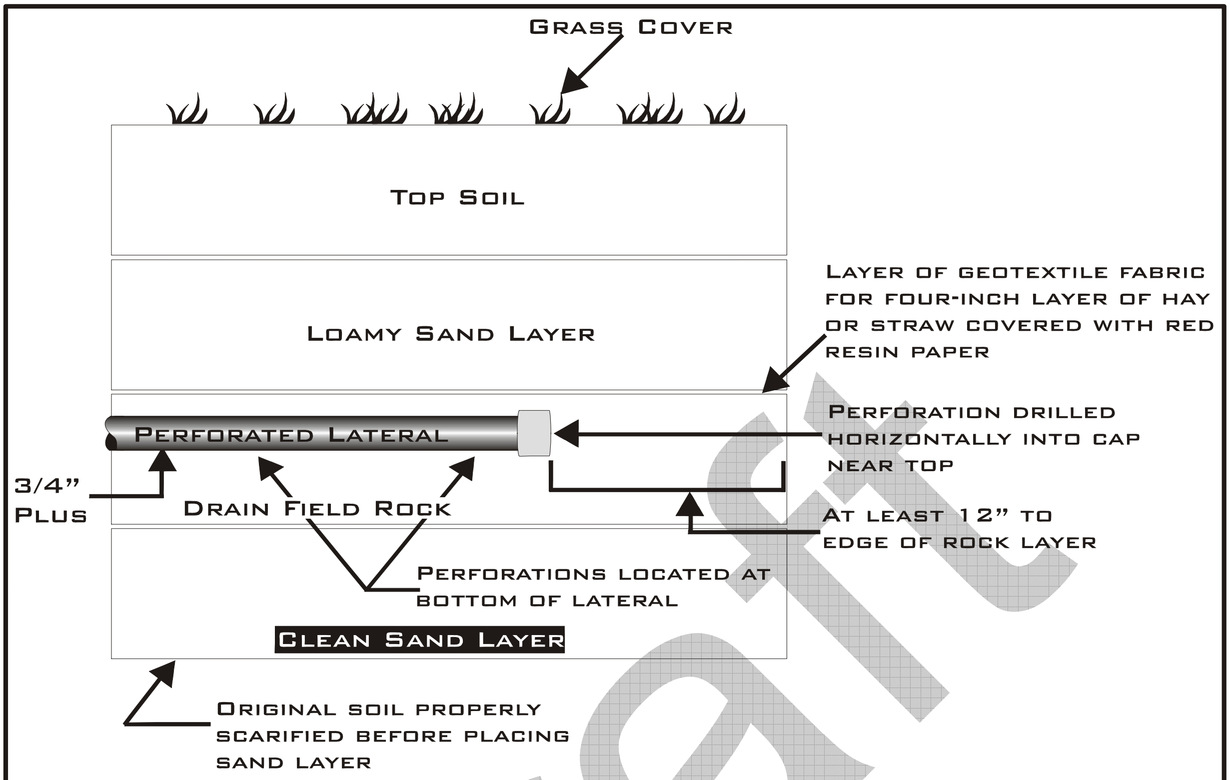


Figure X-12. Section of Low Pressure Pipe Lateral Indicating Holes for Drainage to Minimize Chances of Freezing

PRESSURE DISTRIBUTION NETWORK

Designing a pressure distribution network is a detailed procedure which involving flow rates, pipe diameters, number and size 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 which 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:

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 onsite 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-8 to VI-26. However, 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). 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 the number of orifices must be matched with the flow rate to the network. Typical orifice sizes are 1/4" and 3/16" with spacing of 30-36 inches. See Table X-2 for orifice discharge rates for a typical range of heads.
4. Determine the lateral pipe diameter. Select a diameter that is 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 the hole size, hole spacing, and lateral length. Charts have been developed to help in selecting suitable minimum lateral diameters. See Table X-1 or X-3 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 1/4" orifices, 3.5 ft for 3/16" orifices and if smaller holes, such as 1/8" is 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 lateral.

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.

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	1.00	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

Table X-3. Friction Loss in PVC Plastic Pipe, feet/100 ft of pipe

	Nominal Pipe Size, inch						
Flow, gpm	3/4	1	1-1/4	1-1/2	2	3	4
1							
2							
3	3.24						
4	5.52						
5	8.34						
6	11.68	2.88					
7	15.53	3.83					
8	19.89	4.91					
9	24.73	6.10					
10	30.05	7.41	2.50				
11	35.84	8.84	2.99				
12	42.10	10.39	3.51				
13	48.82	12.04	4.07				
14	56.00	13.81	4.66	1.92			
15	63.63	15.69	5.30	2.18			
16	71.69	17.68	5.97	2.46			
17	80.20	19.78	6.68	2.75			
18		21.99	7.42	3.06			
19		24.30	8.21	3.38			
20		26.72	9.02	3.72			
25		40.38	13.63	5.62	1.39		
30		56.57	19.10	7.87	1.94		
35			25.41	10.46	2.58		
40			32.35	13.40	3.30		
45			40.45	16.66	4.11		
50			49.15	20.24	4.99		
60				28.36	7.00	0.97	
70				37.72	9.31	1.29	
80					11.91	1.66	
90					14.81	2.06	
100					18.00	2.50	0.62
125					27.20	3.78	0.93
150						5.30	1.31
175						7.05	1.74

Note: This is confusing, these values do not agree with those in Table X-1. Which is correct?

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. This performance curve is 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 that 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.

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 onsite 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 that 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 that 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 that the system be dosed 4 times daily, based on the design flow. The 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 5 times the lateral pipe volume.

2. Size the dose chamber. The dose chamber must be large enough to provide the following:
    - The dead space resulting from positioning the pump above the tank bottom
    - The 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 times dosed systems.

DESIGN EXAMPLE

Design a pressure distribution network for a mound whose absorption area is 113 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) - 0.5 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.
   Perforation spacing - Each Perforation covers a maximum area of 6 ft^2. The absorption area is 3 ft wide.
    a. With one lateral down the center on each side of the center feed.
       Spacing = area per orifice / width of absorption area
       = 6 ft^2/ 3 ft = 2.0 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)
       = (6ft^2 X 2) / (3 ft) = 3 ft.

   The designer may stagger orifice spacings with laterals 1.5 ft apart.
   Perforation size - Select from 1/8, 3/16 or ¼ inch. Use 3/16 inch as per earlier discussion.

4. Determine the lateral diameter.
   Using Figure X-13 (3/16 inch) with a perforation spacing of 2 ft. Use one lateral on each side of the center feed with lateral length of 74 ft and 2 ft orifice spacing requies a lateral 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 diameters to be reduced to 1.5 in.

5. Determine number of perforations per lateral.
   Using 2 ft spacing in 74 ft yields:
   Number of perforations = (p/x) + 1 = (74 / 2) + 1 = 38 perforations/lateral
   For two laterals (one on each side), the total number of perforations = 76
   Check - maximum of 6 ft2 / perforation = 150 ft x 3 ft / 6 sq ft = 75 so ok

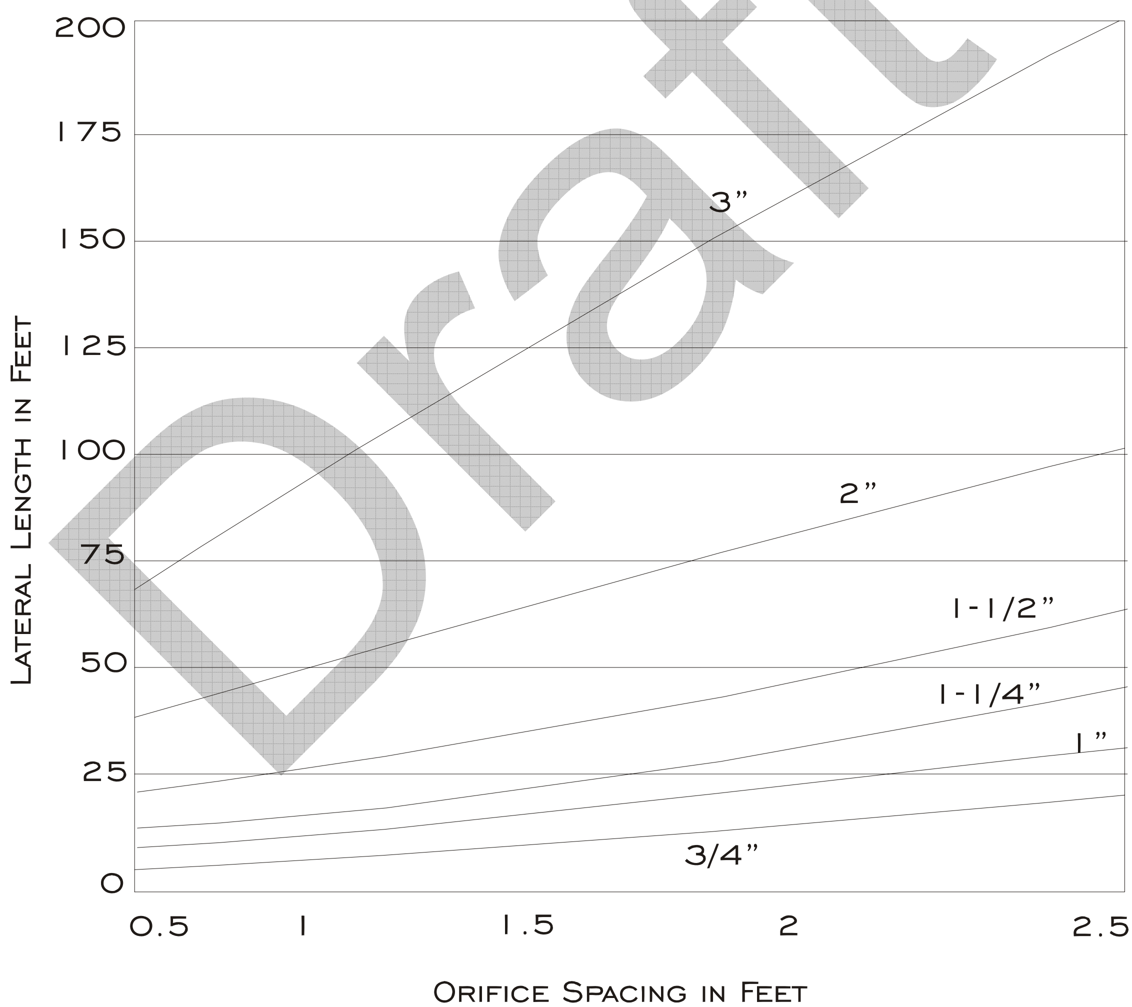


Figure X-13. Graph of Lateral Length, Orifice Spacing, and Pipe Size for 3/16 inch Orifices.

6. Determine lateral discharge rate (LDR).
   Using network pressure (distal) pressure of 3.5 ft and 3/16” diameter perforations. Table X-2 gives a discharge rate of 0.78 gpm.
   LDR = 0.78 gpm/ perforation times 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 2 laterals are required. (If two laterals were used on each side of 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 A-2 for 60 gallons and 125 ft.
    = 8.75 ft for 2” diameter Use 3” diameter unless pump can handle
    = 1.21 ft for 3” diameter added friction loss of 2 in. diameter pipe.
    Total Dynamic Head = 14.7 ft (3”force main) or 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.

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 5 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 that the effluent is lifted.

d. The force main head is the head loss in the force main for the given flow rate. Need to select a force main diameter. For this example, use 3" force main. The first three flow rates are not on the chart 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 (0.97 ft) x 1.25 for distance = 1.21 ft.

Table X-4. Head and Calculations for Various Operating Flow and Pressure

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

e. The network head is calculated by $H = 1.3 \times (Q/(11.79x\ d^2))^2$. 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 3/16" diameter orifice the equation is $H = 1.3 \times (Q/0.4145)^2$. The total head is the sum of the elevation, force main and network heads.

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

Note: Orifice is synonymous with perforation.

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.

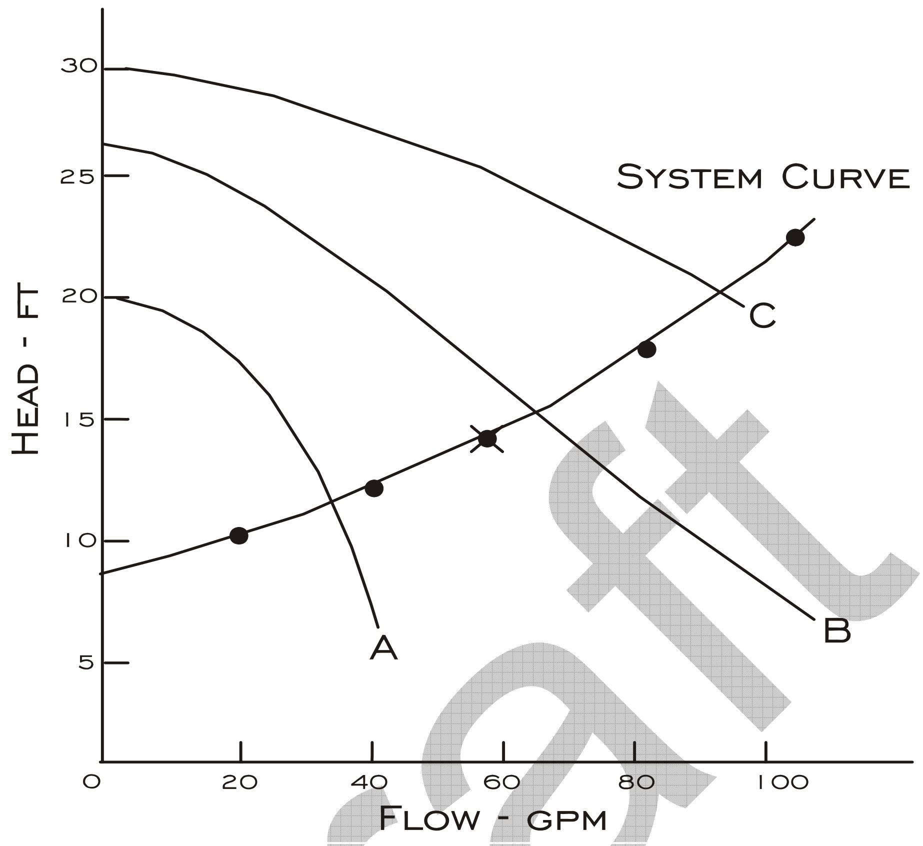


Figure X-14. Plotted System Operating Curve and Pump Curves to Select Pump

4. Determine the dose volume.
   More recent thinking is that the dose volume should be reduced from the larger doses recommended earlier. Use 5 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 = 56 ft	Force main length = 125 ft
Void volume = 0.163 gal/ft	Void volume = 0.367 gal/ft

   Net dose volume = 5 x 56 ft x 0.163 gal/ft = 46 gallons per dose
   Flow Back from force main = 125 ft x 0.367 gal/ft = 46 gallons.
   Set the floats so that a total of 92 gallons will be dosed with 46 gallons flowing back into 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 2/3 daily design flow for reserve capacity.

6. Select controls and alarm from products available from suppliers.

Table X-5. Void Volume for Various Diameters of Schedule 40 Pipe

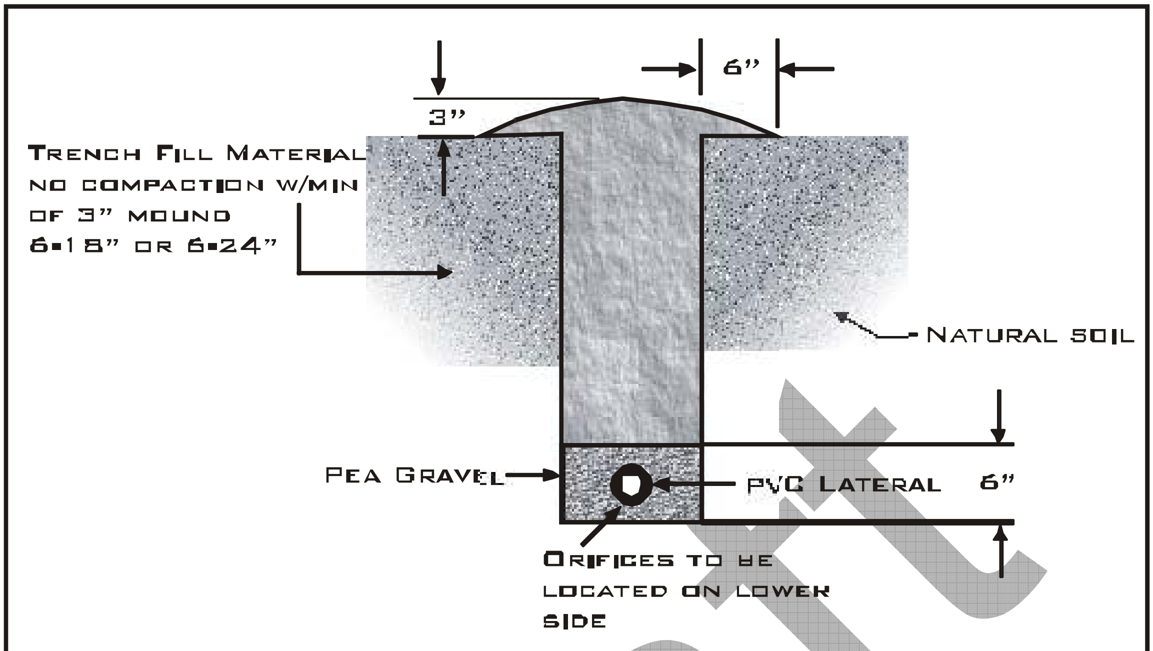
Normal Pipe Size (inches)	Void Volume (gal/ft)
3/4	0.023
1	0.041
1-1/4	0.064
1-1/2	0.092
2	0.163
3	0.367
4	0.650
6	1.469

LOW PRESSURE PIPE SYSTEM

Some sites may have severe limiting factors which 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 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, a septic tank effluent filter, a 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 that the system remains operational. A low pressure pipe system is dependant upon the operation of several mechanical devices, including the pump and



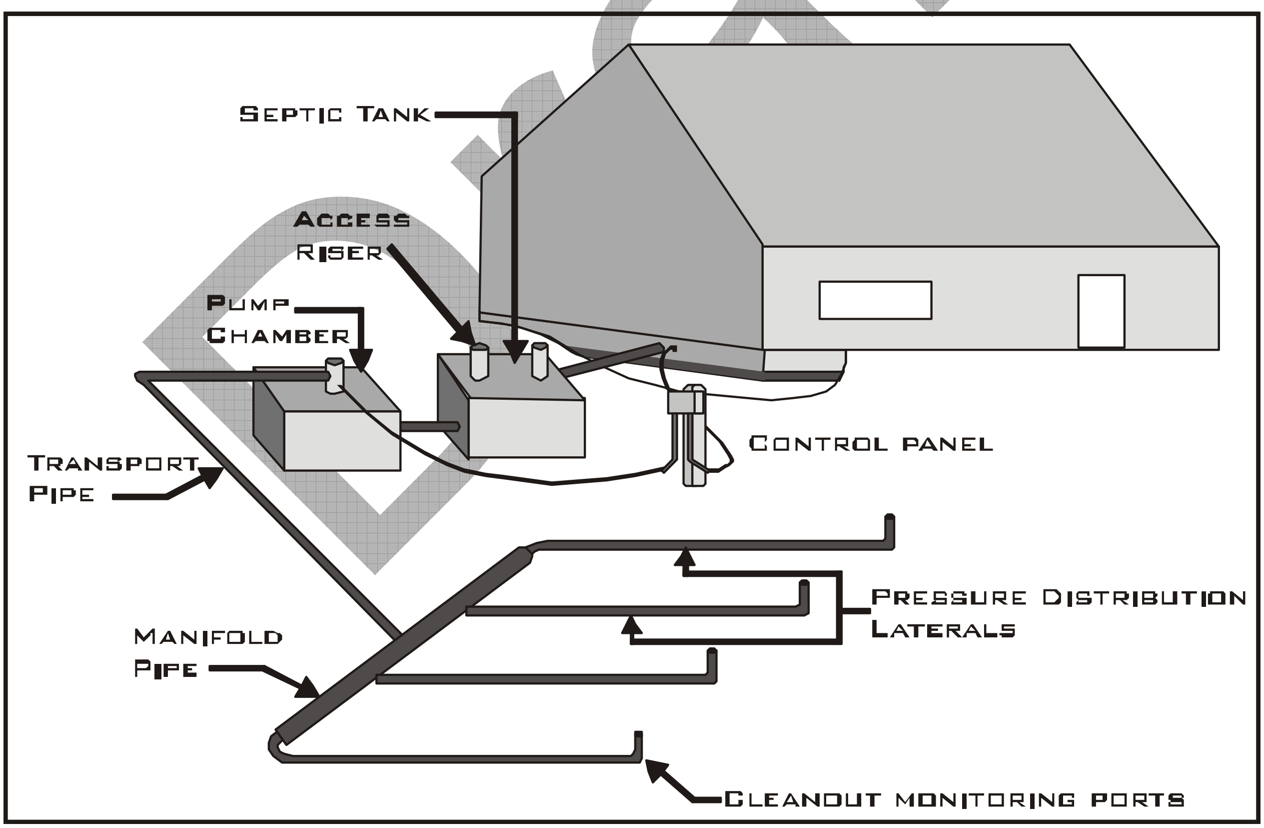


Figure X-15. Typical Pump Tank, Controls, and Pipe Network Components

must be supplied with a reliable power source. The pump tank configuration is shown in Figure VII-14. The system should have a visual and audible alarm to notify the homeowner if the system is malfunctioning.

The laterals are usually 1-2 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-16 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 which 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, which is located 6 inches above the lateral line as shown in Figure X-15. 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 the laterals from draining back into the pump tank. This configuration is commonly used on steep slopes and 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 the even flow distribution. Other configurations which 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 from the supply end should be turned up and be 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, the dosing volume, the 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, the drain back volume, the dosing cycles, and the 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 that the system is properly functioning. The most common way to test the system is commonly called 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 see that the actual volume of discharge does not vary over 15%. This is especially important in systems on a steep slope where the system is designed to drain back between dosing cycles. In this type of system, the lower elevation lateral

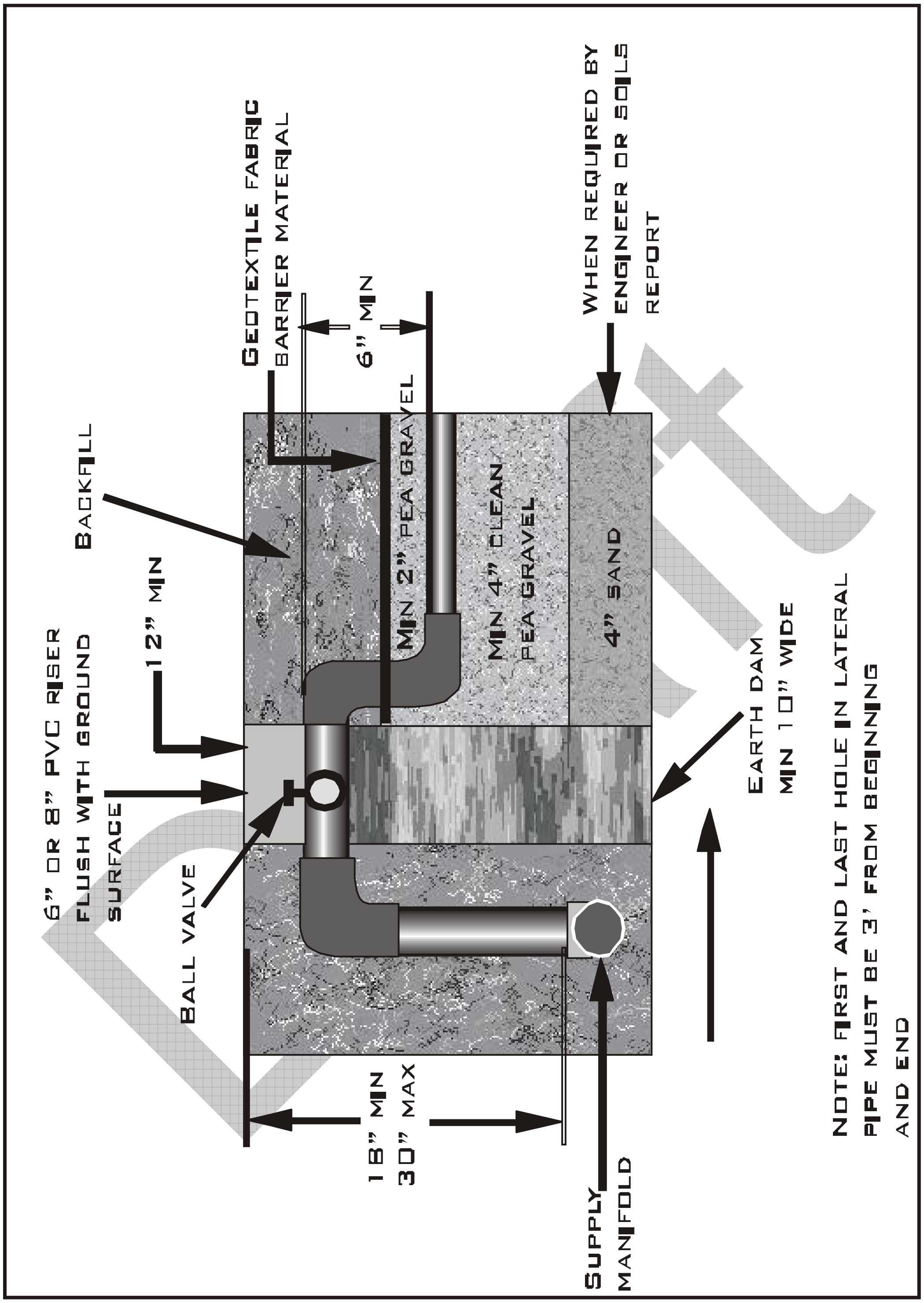


Figure X-16. Supply Manifold and Connection to Low Pressure Lateral

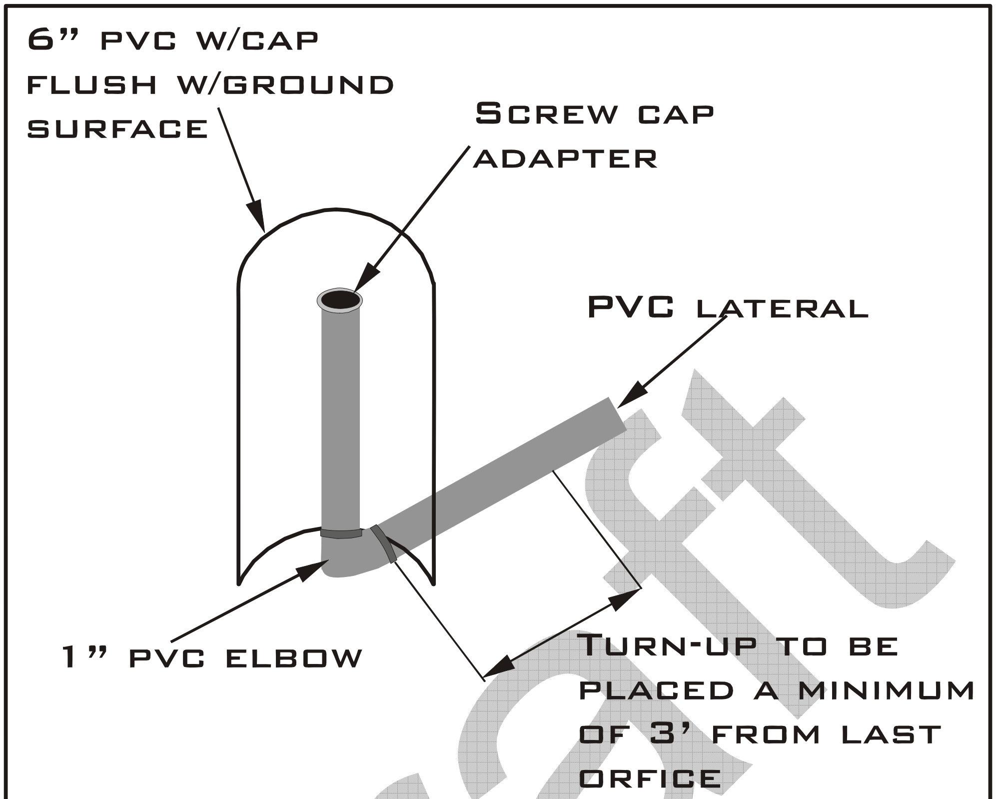


Figure X-17. Detail of Turn-up and Cleanout for Low Pressure Distribution for Sand Filter, Mound or Low-Pressure Pipe Lateral

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

Maintenance

As discussed earlier, these systems will require maintenance. The equipment must be checked on a regular schedule to be sure that 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 which develops in the pipes. The laterals should be tested for residual pressure and for equal flow. The pump run time and the 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 1000 gallons and equipped with an effluent filter having a maximum particle size of 1/8 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 VII-5) = 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-2 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)
= 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 VII-7) x 360 ft of lateral pipe/100 ft of pipe
= 14.76 gallons

c.) Volume in supply line = 16.2 gal/100 ft of 2 inch pipe x 35 ft supply line/100 ft of pipe
= 5.67 gallons

d.) Total volume in system = (volume of laterals) 14.67 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 7.48 gal/ft^3
= 90 ft^3 x 7.48 gal/ft^3
= 673.2 gal

Trench should have approximately 30% void space
= 673.2 gal x 30% = 201 gal

As long as the void space in 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-6. **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

Combine with Table X-2 to Make One Table

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. The selection of the right pump based on the pump type and the pump characteristics can be time consuming, but is critical to the proper operation of the system. Pumping stations must be accessible to allow for service and maintenance on the pumps.

REFERENCES AND READING MATERIALS

Iowa DNR. *Onsite Sewage Design and Reference Manual, Section E: Pressure Distribution and Pumping Systems*, Iowa Department of Natural Resources, 2003. www.iowadnr.com/water/septic/manual.html www.iowadnr.com/water/septic/manual.html

Jones, D.D., J.E. Yahner, and E.R. Miller. *Steps in Constructing a Mound (Bed-Type) Septic System*, ID-163, Purdue University, Cooperative Extension Service, April 1990. www.ces.purdue.edu/extmedia/ID/ID-163.html

Kang, Y.W. K. Mancl and R. Gustafson. *Pressure Distribution of Wastewater Design and Construction in Ohio*, Bulletin 829, The Ohio State University, Cooperative Extension http://ohioline.osu.edu/b829/index.html

Miller, E.R. D.D.Jones, and J.E. Yahner,. *Steps in Constructing a Pressure Distribution Septic System*, ID-163, Purdue University, Cooperative Extension Service, January 1985.

Missouri Farm•A•Syst. *Reducing the Risk of Groundwater Contamination by Improving Household Wastewater Treatment*, WQ680, University of Missouri, Farmstead Assessment System Fact Sheet #6, October 1995. http://extension.missouri.edu/explore/envqual/wq0680.htm#system

Mote, C.R. *Pressurized Distribution for On-Site Domestic Wastewater-Renovation Systems*, Bulletin 870, University of Arkansas, Agricultural Experiment Station, April 1984.

Sievers, D.M. *Design and Construction of On-Site Systems*, Special Report 477, University of Missouri, Department of Agricultural Engineering, Spring 1995.

Solomon, Clement, Peter Casey, Colleen Mackne, and Andrew Lake. *Low-Pressure Pipe Systems*, WWFSOM29, National Small Flows Clearinghouse.

US EPA. *Decentralized Systems Technology Fact Sheet: Low Pressure Pipe Systems*, EPA 832-F-99-076, EPA, Office of Water, September 1999. www.epa.gov/owm/septic/pubs/finallpp.pdf

Washington. *Recommended Standards and Guidance for Performance, Application, Design, and Operation & Maintenance: Pressure Distribution Systems*, WA DOH Publication #337-022, Department of Health, Division of Environmental Health, Office of Shellfish and Water Protection, July 2007. http://www.doh.wa.gov/ehp/ts/ww/pres-dist-rsg-7-1-2007.pdf

Wisconsin, DOC. *Pressure Distribution Component Manual for Private Onsite Wastewater Treatment Systems*, SBD-10706-P (N 01/01), Department of Commerce, Division of Safety and Buildings, January 2001.
http://commerce.wi.gov/SBdocs/SB-PowtsManualPressureDistributionShowEdits.pdf

WSU. *Properly Managing Your Pressure Distribution System*, EB1673, Washington State University, Cooperative Extension, June 1992.

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July 2009

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Contents **Page**

Preface

The recommended standards contained in this document have been developed for statewide application. Regional differences may, however, result in application of this technology in a manner different than it is presented here. In some localities, greater allowances than those described here may reasonably be granted. In other localities, allowances that are provided for in this document may be restricted. In either setting, the local health officer has full authority in the application of this technology, consistent with Chapter 246-272A WAC and local jurisdictional rules. If any provision of these recommended standards is inconsistent with local jurisdictional rules, regulations, ordinances, policies, procedures, or practices, the local standards take precedence. Application of the recommended standards presented here is at the full discretion of the local health officer.

Local jurisdictional application of these recommended standards may be:

1) **Adopted as part of local rules, regulations or ordinances -** When the recommended standards, either as they are written or modified to more accurately reflect local conditions, are adopted as part of the local rules, their application is governed by local rule authority.

2) **Referred to as technical guidance in the application of the technology -** The recommended standards, either as they are written or modified to more accurately reflect local conditions, may be used locally as technical guidance.

Application of these recommended standards may occur in a manner that combines these two approaches. How these recommended standards are applied at the local jurisdictional level remains at the discretion of the local health officer and the local board of health.

The recommended standards presented here are provided in typical rule language to assist those local jurisdictions where adoption in local rules is the preferred option. Other information and guidance is presented in text boxes with a modified font style to easily distinguish it from the recommended standards.

Glossary of Terms: A glossary of common terms for all RS&Gs can be found on the DOH Web site at http://www.doh.wa.gov/ehp/ts/WW/RSG_Glossary_8-29-07.pdf.

Typical RS&G Organization:

Standards Section	**Explanation**
Performance	How this technology is expected to perform (treatment level and function)
Application	How this technology is to be applied. This section includes conditions that must be met prior to proceeding with design. Topics in this section describe the “approved” status of the technology, component listing requirements, permitting, installation, testing and inspection requirements, etc.
Design	How this technology is to be designed and constructed (includes minimum standards that must be met to obtain a permit).
Operation and Maintenance	How this technology is to be operated and maintained (includes responsibilities of various parties, recommended maintenance tasks and frequency, assurance measures, etc)
Appendices	Design examples, figures and tables, specific applications, and design and installation issues.

Introduction

Pressure distribution applies effluent uniformly over the entire absorption area such that each square foot of bottom area receives approximately the same amount per dose at a rate less than the saturated hydraulic conductivity of the soil. This process promotes soil treatment performance by maintaining vertical unsaturated flow and also reduces the degree of clogging in finer textured soils. Pressure distribution closely approaches uniform distribution. (See guidance section below)

A pressure distribution system consists of a pretreatment component to separate the major solid materials from the liquid, a screening device to protect the pump and distribution lateral orifices from solids, and a means to deliver specified doses of effluent, under pressure, to the distribution system (Converse, 1974; Converse, et al., 1975; Otis, et al., 1978). The distribution system consists of small 1 to 2 inch diameter laterals with small discharge orifices. A pressure head is created within the laterals, usually by means of a pump or siphon.

Pressure distribution is applicable to any system which uses soil as a treatment medium and may improve long term performance of those systems. It is required by WAC 246-272A for certain site and soil conditions, and for high daily design flows. Pressure distribution is also a required component for mounds, sand filters and sand lined trenches and beds.

Research evidence indicates that wastewater traveling vertically through 2-4 feet of suitable, unsaturated soil provides adequate treatment of wastewater. Research also indicates that the method of distribution of septic tank effluent within the soil absorption field can affect the system's treatment performance.

A frequently used, and the simplest method for distributing effluent is gravity flow. Gravity flow allows wastewater to flow by gravity through large diameter pipes into the subsurface soil absorption system. Distribution is usually localized in a few areas within the field, which results in overloading of the infiltrative surface in those areas until a mature biomat develops. This overloading can lead to groundwater contamination in coarse granular soils due to insufficient treatment, or more rapid clogging in finer textured soils.

A second method of distribution, dosing, can overcome some of these problems. It is dealt with in a separate publication entitled, Dosing Gravity Drainfield Systems, revised .July 2007. Because effluent is distributed over a larger portion of the absorption area and the period between doses is maximized, the degree of soil clogging is reduced. However, localized overloading may still occur.

A third method is pressure distribution, which comes closest in achieving uniform distribution.

Pressure distribution is usually used in locations where it is either desirable or required to:

1) *achieve uniform application of wastewater throughout the drainfield area;*
2) *treat and dispose of effluent higher in the soil profile;*
3) *avoid potential contamination of ground water beneath excessively permeable soils;*
4) *improve the treatment performance and extend the life expectancy of a drainfield or other component;*
5) *reduce the potential for breakout or seepage on slopes;*
6) *distribute effluent to all sand filters, mounds, all Type 1 soils, and all other soils with less than 24 inches of vertical separation.*

Pressure distribution is also appropriate for sites in aquifer sensitive areas and for larger drainfield systems. Finally, in certain conditions where pumping is necessary due to elevation problems, pressure distribution can be incorporated with only a little additional effort.

System Components / Process Summary

Pressure distribution systems require the following basic components: septic tank (or other pretreatment to the same quality as domestic septic tank effluent), pump or siphon chamber or equivalent, transport line, manifold, laterals, and drainfield.

Figure 1 illustrates the major components of a typical pressure distribution system that are described below:

Component	*Primary Function*
Septic tank (or other pre-treatment device)	*Solids separation and storage*
Screen	*Protect pump and distribution network orifices from solids*
Pump chamber (surge tank)	*Transport a specific volume of effluent from the Surge tank pump chamber to the distribution network. Accumulate effluent between pump cycles and during malfunction.*
Transport line	*Pipeline that connects the pump to the Manifold.*
Manifold	*Piping network connecting the transport line to the various laterals.*
Control Panel	*NEMA-rated box containing all the controls for the pumping system, dose cycle counter, pump run time meter, and alarm controls.*
Laterals	*Small diameter pipes with orifices which distribute effluent within a trench or bed.*
Drainfield	*Allows the septic tank effluent to pass into the native soil or other receiving media where various biological and physical processes provide additional treatment.*

1. Performance Standards

1.1. Intent

The intent of pressure distribution is uniform distribution of effluent throughout the receiving component.

1.2. Measure of Performance

The variation in orifice discharge rates within any one lateral must not be more than 10%,

The variation in orifice discharge rates over the entire distribution system must not be more than 15%,

The squirt height difference must not exceed 21% (10% flow difference) between orifices on any one lateral. The squirt height difference over the entire system must not exceed 32% (15% flow difference). ***Remember to use a new drill bit during construction****. The following table gives the actual distances.*

Maximum Flow Difference Allowed (Inches)

Nominal Residual Squirt Height	*10% Flow Difference*	*15% Flow Difference*
2 Feet	*5 Inches*	*7.5 Inches*
5 Feet	*12.5 Inches*	*19 Inches*

1.2.1. A minimum residual pressure of 0.87 psi (2 feet of head) is required for systems with 3/16 inch diameter orifices and larger, and 2.18 psi (5 feet of head) is required for systems with orifices smaller than 3/16 inch.

Generally, the testing should verify that distribution is uniform with the required minimum residual pressure, that the system is dosed at the proper volume and frequency, and that the alarms are functioning properly. Suggested methods are provided below. If problems are encountered during testing, the installer should notify the designer or engineer. Wiring problems should be referred to the electrician. Described below are the steps for conducting a pressure test.

- ❑ *Measure squirt height.*

  Minimum squirt height for orifice size:
  3/16" orifice size = 2' or 24" squirt height
  1/8" orifice size = 5' or 60" squirt height
  5/32" orifice size = 5' or 60" squirt height

- ❑ *Check uniformity of squirt height.*

- *An alternate method to the squirt height is to attach a clear PVC stand pipe to the end of the lateral. The true residual head is measured from the top of the lateral pipe to the top of the water column.*

- *Check float placement. High water alarm, "on" level, "off" level, and "redundant off" alarm must activate or deactivate at the elevation called out on the plan. It is recommended that, for simplicity and accuracy, these adjustments be made with the float tree out of the water.*

- *Ensure that the pump delivers the correct dose to the drainfield.*

- *Demand dose systems: Verify that "dry" float settings (completed above) send the correct dose to the drainfield when floats are in water. This may require minor adjustments of float placement.**

- *Timed dose systems:*

(1) Determine the time required to send a full dose to the drainfield. This can be done by running the system in manual. Be sure there is plenty of water in the pump chamber. Timer run times provided by designers or engineers must be field tested.

*(2) Using the time obtained above, verify that when the system runs automatically it runs the time required to send the proper dose to the drainfield. This is important because timers are difficult to set, i.e., setting a timer to 2.2 minutes may not ensure a run time of 2.2 minutes. Two steps to speed this process are to start testing with the pump chamber mostly full and to set "off" time temporarily to minutes or seconds.**

(3) Verify that the timer off time is the same as that specified in the plan or will dose the system the correct number of times a day. Check this number in minutes and note the off time. One can verify activation levels by use of lights on timer. For instance, if the drainfield is to receive 4 doses per day, the off time should be approximately 6 hours.

(4) Verify that high water alarm does not turn the pump on. If high water alarm turns the pump on, the system will not be approved.

Timed dose systems only: Verify that the system will dose the correct number of times per day and that no float in the system turns the pump on independent of the timer. A system with a timer override float will not be approved.

If problems are discovered during the functional testing, first contact the designer or engineer. If the wiring needs adjustment, the electrician should be contacted.

In preparation for Health District final inspection, fill the pump chamber.

An additional test for equal distribution, which takes into consideration draindown after the pressure cycle, is described here. However, it is somewhat tedious. For systems with laterals *having more than 18 inches difference in elevation, the volume of liquid from an orifice (same size as the others in the laterals) placed in a plug or cap in the end of each lateral can be collected from a complete cycle and measured. The variation between the largest volume and the least volume collected must not be more than 15%. Use of manifold designs shown in Figures 6A and 6B will eliminate significant drainback.*

** Determination of float activation level in water may take several tries. For both system types, note pump run time that delivers proper dose. Record the results.*

2. Application Standards

2.1. Listing

Pressure distribution is a public domain distribution technology, but it does not appear on the department's List of Registered On-site Treatment and Distribution Products. It may be permitted by local health officers as a public domain pressure distribution system (WAC 246-272A-0010) as long as there is departmental RS&G for this technology.

2.2. Permitting

Installation permits, and if required, operational permits must be obtained from the appropriate local health officer prior to installation and use.

2.3. Pretreatment

2.3.1. A pressure distribution system must be preceded by a properly sized two-compartment septic tank with effluent baffle screen (see 2.3.3). Exception, see 2.4.2.

2.3.2. Septic Tank - The septic tank must be designed in compliance with Washington State On-Site Sewage System Regulations (WAC 246-272A-0232) and with the Washington State Recommended Standards and Guidance for On-site Sewage System Tanks. Until sewage tank rules are available, all septic tanks must also:

2.3.2.1. be watertight to a level above any possible seasonal ground water. The local health officer may require leak testing.

2.3.2.2. include screening of the effluent, unless the screening is around the pump.

2.3.2.3. have service access manholes and monitoring ports for the inlet and outlet.

Septic Tank - (See Figure 2)

Watertightness - Some or all of the following materials can be used to aid in achieving water tightness of the tanks:

- *Use caste-in-place flexible rubber gaskets in the inlet and outlet openings, using stainless steel clamps to seal the rubber to the pipes.*
- *Use of flexible rubber gaskets sealed to the inlet and outlet openings with a ratcheted expansion seal and using stainless steel clamps to seal the rubber to the pipes.*
- *Use of expanding grout material as a means of sealing tanks and risers. Some grouts will shrink and crack over time, and thus allow tanks to leak well after the tank is backfilled. Bentonite backfill around the tank seams and pipe entrances may help provide a water tight tank.*
- *Epoxy is another effective method of sealing some kinds of joints, but the weather conditions must be ideal and there is no capacity for flex.*
- *Rubber grommets around smaller inlet and discharge pipes, conduit and junction box penetrations can also be effective in controlling leaks.*
- *Risers – A 24-inch riser is practical when installing it over a 20-inch hatch of the septic tank because a solid foundation is needed to attach the riser to the tank. If a riser is integral to the top of the tank, a 20-inch riser will suffice.*

2.3.3. Outlet Baffle Screen / Filter - An outlet baffle screen or filter must meet the following performance criteria:

2.3.3.1. protect the pressure distribution drainfield discharge orifices from plugging by particles larger than the orifices.

2.3.3.2. protect the effluent pump from damage due to particles which exceed the pump’s capacity to pass (may be an issue with some types of pumps).

2.3.3.3. perform these functions without loss of performance between routine service events.

2.3.3.4. perform these functions with routine service no more frequent than that required for other system components or the system as a whole.

2.3.3.5. is constructed of durable, non-corroding materials.

2.3.3.6. draws liquid from the “clear zone” of the septic tank.

2.3.3.7. be designed, constructed and installed for easy and thorough cleaning.

Outlet Baffle Screen / Filter -- Effluent flowing through a screen at the outlet of the septic tank is at very low head and therefore particles cannot be forced through the openings. In addition, servicing the screen does not involve pump, control floats, wiring and discharge pipes. Below are listed some specific criteria for baffle screens that meet the standards.

Maximum mesh opening of 1/8 inch (protects discharge orifices of 3/16 inches or larger, and pumps with the capacity to pass up to a 1/8 inch sphere. For orifices smaller than 3/16 inches diameter, the screen should have a mesh size of 1/16 inch smaller than the orifice it is designed to protect.)

Non-corrosive material (durability leads to improved product life-span and performance).

Provide an open area flow capacity at least equal to the flow capacity provided by a 4 inch diameter PVC pipe. The minimum area will very likely require a high frequency of cleaning and therefore not meet the standard of performance between service intervals. In standard practice a much larger flow area is used. The larger flow areas will result in longer intervals between services for the same hydraulic and organic strength loadings.

The screen must be securely fastened to prevent dislodging or misalignment (this relates to long-term performance and servicing).

Be easily removable and/or designed, constructed and installed for easy and thorough cleaning (this relates to long-term performance and servicing).

Draw liquid from the "clear zone" of the septic tank, the zone between 40% down from the top of the liquid and 40% up from the bottom of the tank (this relates to performance and service interval, as well as general septic tank performance).

Be capped, covered or otherwise constructed to prevent scum or other floatable solids from discharging from the tank by bypassing the screen or filter (this relates to product performance).

Other specifications may be used to meet the outlet baffle screen / filter performance.

2.4. Pump Chamber

2.4.1. Pump Chamber Requirements - All pump chambers must be structurally sound and conform to Washington State On-Site Sewage System Regulations (WAC 246-272A) and with the Washington State Recommended Standards and Guidance for On-site Sewage System Tanks. Until sewage tank rules are available, all pump chambers must also:

2.4.1.1. be water tight to a level above any possible seasonal ground water. Leak testing may be required.

2.4.1.2. all pump chambers must be equipped with a twenty-four (24) inch minimum diameter, water tight riser with a secured lid that extends to the ground surface. Lids must be equipped with an airtight gasket to eliminate nuisance odors and be secured from accidental or intention removal by unauthorized persons, especially children.

2.4.1.3. the internal volume of the pump chamber must be sufficient to provide the daily design flow volume, dead space below the pump inlet for sludge accumulation, and sufficient depth to provide full time pump submergence, when required. An additional emergency storage volume of at least 75% of the daily design flow is also required (may include volume to flood capacity in both the pump tank and the septic tank).

Pump Chamber Volume - *For most applications, an 18 inch minimum space for sludge accumulation in the pump chamber is prudent. Pump chambers receiving septic tank effluent will accumulate sludge and scum, and in some new systems it will form quite rapidly. The sludge level will never be above the intake of the pump. Emergency storage is required for periods of power outages or equipment malfunctions.*

For systems where continuous operation and maintenance are provided by a management entity acceptable to the local health department, a reduction in the volume required for reserve storage may be considered.

Reductions in pump chamber volume may also be considered when "Duplex" or redundant pumps are used.

2.4.1.4. include a screen if one is not provided at the outlet of the septic tank. (See 2.3.3 for performance criteria of the screen.) The local health officer may allow this option only if O&M is assured through contract with third party entity.

Screening at the septic tank outlet may result in a higher quality effluent than screening around the pump, as the flow rate through a septic tank baffle screen is much lower than through a screen around the pump. However, large pump chambers can continue to accumulate screenable solids, as it is still a biologically active fluid. Therefore, it should be assumed that septic tank effluent, once screened, can still produce sludge and scum in the pump chamber. A pump screen designed to fulfill the performance requirements and to prevent collapsing between service intervals may be a wise choice either by itself or in conjunction with a septic tank effluent filter. However, some pumpers insist that screens in pump chambers are a bad idea and cause many maintenance problems.

2.4.2. Pump Vault System in a Single Compartment Septic Tank - Septic tanks in Washington must have two compartments. However, an exception to this is where a pump vault is used in a single compartment septic tank. In addition to meeting all the requirements for pressure distribution systems with a separate pump chamber, there are additional criteria and limitations when using this combination of septic tank and pump vault. They are listed below:

2.4.2.1. The minimum storage and pump working volumes in the septic tank must be equivalent to a septic tank with a separate pump chamber. The minimums are a) sufficient volume to handle the functions of a septic tank, and to keep the pump submerged, when required, b) surge volume to hold one day's design flow, and c) additional storage for emergency situations equal to 75% of the surge volume.

2.4.2.2. The pump vault must:

2.4.2.2.1. extract liquid from the middle of the clear zone of the septic tank,

2.4.2.2.2. have integral screening or other methods to prevent solids greater than 1/8" to pass into the pump,

2.4.2.2.3. have screening with a minimum wetted open area of 12 ft^2,

2.4.2.2.4. be able to supply liquid to the pump as rapidly as it is discharged from the vault while keeping the pump submerged,

2.4.2.2.5. perform to these specifications between normal service intervals established for the rest of the system (minimum time – 6 months).

2.4.2.3. The pump vault must be designed and constructed to facilitate removal and maintenance of the vault screen, pumps, and floats.

2.4.2.4. The flow rate from the pump must not exceed 30 gpm. The fluctuation of the liquid level in the tank must not exceed 10 inches. Larger fluctuations are allowed for emergency storage to accommodate power outages or pump failure.

2.4.2.5. The minimum hydraulic detention time in the tank must be 24 hours. The clarified zone must be at least 10 1/2 inches, with a minimum clearance of 3 inches between the bottom of the scum layer and the entrance to the screening device. The minimum distance between the top of the sludge and the entrance to the screening device must be 6 inches.

2.4.2.6. The effluent quality discharged from a pump vault in a single compartment tank must be equal to the expectation for a separate pump chamber that receives screened effluent from a two-compartment septic tank.

2.4.2.7. Materials and construction must assure a watertight vessel, which is resistant to corrosive attack by chemicals and conditions typical for a sewage environment.

2.4.2.8. The minimum size of septic tank must be 1500 gallons as measured at the invert of the outlet. In addition, the lowest liquid level (pump off) must have a minimum of 1000 gallons, and thereafter coincide with the requirements of WAC 246-272A-0232.

2.5. Pumps, Fittings and Controls

2.5.1. Pumps must be selected to pump effluent and be capable of meeting the minimum hydraulic flow and head requirements of the proposed on-site system. Additional requirements that pumps and pump installations must meet are:

2.5.1.1. Pumps

2.5.1.1.1. All pumps must be installed so that they can be easily removed and/or replaced from the ground surface. (Under no circumstances shall pump replacement and/or repair require service personnel to enter the pump tank).

2.5.1.1.2. All pumps must be fitted with unions, valves and electrical connections necessary for easy pump removal and repair. All pumps must be protected by approved outlet baffle screens in the chamber preceding the pump chamber or by pump screens, as described in previous sections.

In addition, pumps and controls should have gas-tight junction boxes or splices and have electrical disconnects (as per National Electric Code) appropriate for the installation. The boxes should be placed so that they do not interfere with the servicing of other components.

2.5.1.1.3. Pumps and electrical hook-ups must conform to all state and local electrical codes.

2.5.1.1.4. If any portion of the pump fittings or transport line is at a higher elevation than the drainfield, the system must be equipped with an air vacuum release valve or other suitable device to avoid siphoning.

If a check valve is used in the system, a vent hole should be installed upstream from the check valve so the pump volute (impeller chamber) is kept filled with effluent. Some pumps may cavitate if the impeller is not kept submerged. Under most circumstances this hole will be needed only when the chamber is first filled with liquid or after it has been cleaned.

2.5.1.2. Pump Controls

2.5.1.2.1. Timed dosing is recommended whenever pressure distribution is used. WAC 246-272A requires timed dosing in the soil dispersal component for all sites where technologies meeting Treatment Levels A or B are mandated, soil dispersal components having daily design flows between 1000 and 3500 gallons of sewage per day, and in the soil dispersal component following all repairs using the allowances of Table IX in WAC 246-272A-0280. Technologies, such as sand filters, recirculating gravel filters, sand lined trenches, mounds, and other treatment components also require timed dosing in order to assure those technologies meet treatment requirements or expectations. For systems requiring time-dosed pressure distribution, accessible controls and warning devices are required, and must:

2.5.1.2.1.1. meet the functional requirements for pressure distribution,

2.5.1.2.1.2. deliver prescribed dose sizes uniformly to the orifices in the distribution network,

2.5.1.2.1.3. deliver the effluent to the distribution network in evenly spaced doses over a 24 hour period,

2.5.1.2.1.4. provide prescribed resting periods between doses,

2.5.1.2.1.5. assures no more than the design volume for each 24 hour period is delivered to the receiving component,

2.5.1.2.1.6. record and store the pump run time and number of dose cycles,

2.5.1.2.1.7. have controls and components listed by Underwriter’s Laboratory or equivalent, and

2.5.1.2.1.8. alarm circuit independent of the pump circuit.

Timed dosing is not required for pressure distribution soil dispersal components following treatment components that are timed dosed. The flow is already time-dosed to the treatment component, and therefore the pump chamber out of the treatment component may be demand-dosed. To detect extraneous flows, an elapsed time meter can be used to monitor the integrity of the liner of a packed bed filter, such a sand filter, by comparing the volume of liquid pumped from the sand filter with the volume pumped into it.

Timed dosing is strongly recommended on all pressure distribution systems. This type of system enhances performance, reliability, and protection from abuse. The requirements in WAC 246-272A and the recommendations in this document are based on the need to control the size of doses to the coarser and single grained soils and treatment media. Timed dosing also prevents hydraulic overload of the receiving component. Usual sources of hydraulic overload are excessive water use in the facility or groundwater infiltration into the septic tank or pump chamber.

Timed dosing means that both the length of each dose (produces gallons per dose) and the interval between doses (which determines the number of doses per day) is controlled by a timing device whenever a dose volume is in the pump chamber. The number of pump cycles should be adjustable and in sufficient number to meet the design needs of the system.

As the number of dose cycles increases, the amount of effluent delivered per dose must decrease (in order to prevent more than daily design dose from being delivered to the drainfield). Delivering more than 6 or 8 doses per 24 hours will require one or more of the following features to be designed into the system:

- *orifices at 12:00 o'clock to keep the piping network full or mostly full of effluent between doses (to reduce the volume per dose)*
- *transport, manifold and lateral pipe diameters are reduced (to reduce the volume per dose)*
- *orifice size is reduced (to help reduce the volume per dose)*
- *fluid velocity in pipes is increased (to help scour the pipe and as a consequence of the reduced pipe size)*
- *residual hydraulic head at the orifices is increased (to help clear the smaller orifices)*
- *check valves are placed into the system to prevent flowback (to reduce the volume per dose)*
- *a performance test of the check valves, as many do not perform as intended.*

2.5.1.2.2. When the treatment component is timed dosed prior to a soil dispersal component, the soil dispersal component does not need separate timed dosing. In this case, only a demand dosing system with a dose cycle counter and hour meter (or a water meter on the water supply or sewage stream) is required for the soil dispersal component.

2.5.1.2.3. Demand controlled pressure distribution systems must include an electrical control system that:

2.5.1.2.3.1. has the alarm circuit independent of the pump circuit.

2.5.1.2.3.2. will meet the functional and reliability requirements for pressure distribution.

2.5.1.2.3.3. has controls and components that are listed by UL or equivalent.

2.5.1.2.3.4. is secure from tampering and resistant to weather (minimum of NEMA 4).

2.5.1.2.3.5. located outside, within line of sight of the pump chamber.

2.5.1.2.4. All control panels must have cycle counters and elapsed time meters for all pumps. Alternatively, a water meter on either the water supply or sewage streams will satisfy this requirement.

2.5.1.2.5. All control panels must be equipped with both audible and visual high liquid level alarms and the alarms must be placed in a conspicuous location.

2.5.1.2.6. Float switches must be mounted independent of the pump and transport line so that they can be easily replaced and/or adjusted without removing the pump.

The minimum requirements for timed pump cycle controls are a timer actuator float for the pump and a high liquid level alarm. In addition, a low liquid level off float is highly recommended. [See next section, Floats, for a discussion.]

2.5.1.2.7. Electrical control and other electrical components must be approved by Underwriters Laboratories (UL) or equivalent.

2.5.1.2.8. Other standards that engineers, designers and installers need to be aware of and comply with are electrical standards for pump and control systems established by Washington State Department of Labor and Industries.

A control box or panel installed on a treated 4” X 4” post is acceptable practice and does not produce irritating resonations for the building occupants as occurs when the control panel is mounted on buildings.

2.5.1.2.9. Minimum Dose Frequency - The minimum dosing frequency must be according to the following:

Soil Type 1 and 2	4 times per day
Soil Type 3	4 times per day
Soil Types 4-6	1 to 2 times per day

Dose Frequency - Although this standard lists the minimum frequency for various soil types, more frequent doses than the minimum recommendation may be desirable in some designs. Dosing of drainfields provides intermittent aeration to the infiltrative surface. With this method, periods of loading are followed by periods of resting, with cycle intervals ranging from hours, to a day or more. The resting phase should be sufficiently long to allow the system to drain and expose the infiltrative surface to air, which encourages degradation of the clogging materials by aerobic bacteria. In sands, however, the rapid infiltration rates can lead to bacterial and viral contamination of shallow ground water, especially when first put into use. Therefore, systems constructed in these soils should be dosed with small volumes of wastewater four or more times a day to prevent saturated conditions from occurring and hence, inadequate treatment. In finer textured soils, saturated flow is much less likely, so frequent doses do not add to the performance. Large, less frequent doses are more suitable in these soils to provide longer aeration times between doses (EPA, et al).

2.5.2. Floats (or other types of liquid level sensors)

2.5.2.1. For pump chambers serving single family residences, the necessary floats or liquid level sensors are to actuate and turn off the pump control system, and a high water alarm float. “Redundant off” controls are optional, but highly recommended, and may be required by the local health officer.

2.5.2.2. Commercial and multi-family applications are required to meet Washington State Department of Labor and Industries requirements for Class I, Division I locations. These locations include redundant off and special ratings on installed motors and equipment.

2.5.3. Siphons - Siphons may be used for charging a pressure distribution system. However, they are flow-dependent and cannot provide evenly spaced doses, nor limit the daily volume (See Appendix D). Therefore, siphons cannot be used where standard 2.5.2.1. is required unless specific design elements cause the siphon to produce the performance of 2.5.2.1. Where siphons are used the following requirements apply:

2.5.3.1. The area to be dosed must be downhill from the siphon chamber and according to manufacturer’s instructions for minimum elevation differential.

2.5.3.2. The effluent must be screened before entering the siphon chamber.

2.5.3.3. The siphon must be installed to allow access for maintenance and cleaning.

2.5.3.4. The dose counter(s) must be incorporated into the design and installation.

2.5.3.5. Siphons can only be used where timed dosing is not required, or where some system or arrangement delivers effluent to the siphon chamber evenly over a 24 hour period and no more than the maximum design flow for the system.

These criteria can be met by the use of a small electric pump, which delivers effluent to the siphon chamber evenly over a 24-hour period. This pump is sized to deliver no more than the daily design flow to the siphon chamber in a 24 hour period. The siphon then doses a sand filter, mound or sand-lined drainfield.

2.5.3.6. Siphons may only be used where they will be monitored and managed to the satisfaction of the local health officer.

Other important considerations:

- *Proper siphon size must be selected, as they are available in many sizes.*
- *Air leaks in the siphon or fittings will prevent the siphon from functioning.*
- *If the siphon chamber fills too rapidly, the bell and siphon will not receive a full dose of air and will enter a trickling mode.*
- *Adjustment to the "trip" level of the liquid in the siphon chamber is limited; dose volume is better handled by careful sizing of the siphon chamber.*
- *Blockage of the snifter tube, even momentarily, at the end of the discharge cycle, will cause the siphon to enter a trickling mode.*
- *Transport pipe must be vented just outside the siphon chamber and other venting must be placed in the system as needed.*
- *It is advisable not to bury the transport pipe until the system is tested and proper operation is verified; additional venting may be needed for unanticipated air locks (see Figures 9 and 10).*

2.6. Piping Materials

The pipe materials must meet the following minimum specifications:

2.6.1. At a minimum, the material must meet ASTM D2241 Class 160 or equivalent.

2.6.2. For schedule 40 and schedule 80 PVC, use ASTM D1785.

2.7. Manifold

(See Appendix A-4)

The primary function of the manifold is to deliver equal flow to all lateral orifices while minimizing system friction losses. While manifold patterns may take many forms, the most common are the center and the end manifolds. End manifolds suffice for short laterals but center manifolds allow for use of smaller lateral pipe sizes.

Manifold / Lateral Connection:

The laterals can be connected to the manifold in several ways. The manifold to lateral connection must be appropriate for the site conditions and the specific use. Several types are described below:

- *A header manifold is positioned at an elevation below the laterals (Figure 3A), with check valves, flow control valves and feeder lines to each lateral. This configuration will maintain the manifold, feeder lines and laterals full between doses, will not allow drain back, and can be adjusted at one location to equalize residual head in all laterals. This arrangement can deliver small volumes per dose, allowing many doses per day, if desired. Caution should be taken to minimize the potential for effluent freezing in the laterals and manifold.*
- *A header manifold is placed at an elevation above the laterals (Figure 3B) without check valves, with flow control valves and feeder lines to each lateral. The measured flows from an orifice in each lateral are nearly equal without the use of check valves and without maintaining the system full between doses.*
- *Tee-to-Tee with manifold below (See Figure 4) - When freezing and sloping site conditions are not a concern, this method of construction can be used to allow a very rapid pressurization of the system, especially if the transport line remains full between doses. When check valves are used in the manifold just downstream of each lateral, the manifold (and laterals too, when orifices are in the 12 o'clock position) stays full of effluent between doses. With this style, (1) there is no drainback from the upper laterals and manifold into the lower lateral, (2) the system is completely charged within just a second or two after the pump is turned on, and (3) the system can be dosed with very small volumes per dose.*
  [Note caution about check valves in section 2.7.1. of this section.]
- *Cross construction (See Figure 5) - If the lateral orifices are drilled in the 6 o'clock position, this design will allow the laterals and a portion of the manifold to drain between doses, assuming the transport line remains full between doses.*
- *Tee-to-Tee with the manifold above - If the lateral orifices are drilled in the 6 o'clock position, the entire distribution network will drain after each dose. This may be desirable on a sloping site (where check valves are not installed in the manifold), to prevent upper laterals from draining back through the manifold to the lowermost laterals, thereby overloading them. If the orifices are drilled in the 12 o'clock position, the laterals will remain full between doses. This may be desirable when the objective is to pressurize the distribution network quickly without the use of check valves. Caution should be taken to minimize the potential for effluent freezing in the laterals.*

Sloping Sites: *Manifold designs for sloping sites are particularly critical. Laterals at different* *elevations will have different residual pressures, with the lowest lateral having the highest residual. In addition, if the manifold is not designed correctly the lowest lateral will receive pressure before the top lateral and system backflow will continue to the lower laterals after the pumping cycle has ended. In this instance, the lowest trench will receive more flow than the others, with the potential for overload. While there may be several solutions to these problems, Figures 3A & 3B illustrate two methods for resolving them. The check valves and flow control valves shown in Figure 6A and 6B are assumed to be an integral part of the manifold.*

2.7.1. Check Valves

2.7.1.1. When check valves are used, they must be installed so that they can be removed for servicing or replacement. This means that unions or some other fitting need to be included in the installation of check valves.

2.7.1.2. The location of check valves must be well documented and marked. Preferably they are located in a structure that is accessible from the surface.

Check valves occasionally require maintenance, and therefore should be installed so that they can be removed for servicing or replacement. Unions placed at the check valve are a common means to allow servicing of the check valves while avoiding destruction or severe excavation of the manifold. Some brass check valves can be disassembled without removing them from the line.

2.8. Laterals

(See Appendix A-1, A-2, A-3)

The laterals in a pressure distribution system are perhaps the most important design aspect. All design considerations to this point are essentially to serve the delivery of equal flows to each square foot of drainfield bottom area.

Orifice Design:
The actual flow rate from each orifice is best represented by the equation:

$$Q_o = 11.79\, d^2\, h^{0.5}$$

where:

Q_o	*is the orifice flow in gallons per minute*
d	*is the orifice diameter in inches*
h	*is the discharge head in feet (also called residual head)*
	(see Appendix A-2 for a derivation of this equation)

There are other factors complicating accurate calculation of the orifice flow rate such as accurate drilling of holes, class of pipe, size of pipe, and slight variations in the friction coefficients used for fittings. Proper technique and practice in drilling holes includes use of *proper drill size and a sharp bit. Accurate holes also may require jigs or other drill stabilizing tools to prevent wobble and to drill the hole perpendicular to the pipe. Proper layout and control will ensure that the design number of orifices are actually placed in each lateral.*

The above formula for calculating orifice discharge rates is recommended. However, the choice of coefficient to use in a design can vary from 11.79 to 16, depending on the experience of the designer in being able to predict accurately and control for the friction losses and other variables of construction and manufacture. For many designers, experience has shown that use of a slightly higher coefficient in the equation more accurately predicts the actual flow. For whichever coefficient is selected, it is critically important that the same coefficient be used throughout the design. Other ways to handle the inaccuracies are to add 10% to the total flow after the calculations, or to design to more than minimum residual head. All of these are acceptable.

2.8.1. Residual Pressure Requirements - For systems with orifice diameters of 3/16 inch or larger, the minimum residual head at the orifice is 2 feet (.87 psi). For systems with orifices less than 3/16 inch diameter, the minimum residual head is 5 feet (2.18 psi).

2.8.2. Orifice Size and Orientation

2.8.2.1. Orifices must be no smaller than 1/8 inches in diameter.

2.8.2.2. When using gravelless chambers with pressure distribution, the orifices must be oriented in the 12 o'clock position. If one or two orifices are placed in the 6 o'clock position to facilitate draining after each pump cycle (to prevent freezing in areas of the state where that may occur or to prevent build-up of microbial growth inside the laterals), they must have some mechanism to break the flow (an orifice shield that drains, a pad of gravel, etc.).

See sections on orifice size, orientation and shields, and Figure 7.

2.8.3. Orifice Spacing

To prevent excessive variations in discharge rates and possible subsequent localized hydraulic overload, the maximum acceptable flow deviations stated in the Performance Testing section (Section 1.2.) of this document must be heeded.

2.8.3.1. Sand filters (including sand lined trenches), mounds and pressure distribution in soil types 1, and 2 and in medium sands, must have a minimum of one orifice per 6 ft^2 of infiltrative surface area, evenly distributed.

2.8.3.2. In other soil types, there must be a minimum of one orifice every six feet on center along the lateral.

While these are minimum requirements, orifices spaced at closer intervals may be prudent. Closer orifice spacing should be considered when small doses are specified and where the infiltrative surface is in highly structured soils or has large macropores.

2.8.3.3. The maximum spacing between the outside laterals and the edge of the trench or bed must be 1/2 of the selected orifice spacing, ±0.5 feet.

2.8.4. Orifice Shields

Orifice shields may be the half pipe design, the local cap type, or another design which accomplishes the same end result. See Figure 7.

2.8.4.1. When orifices are oriented in the 12 o'clock position, orifice shields or gravelless chambers must be provided.

2.8.4.2. The shields must be strong enough to withstand the weight of the backfill and large enough to protect the orifice from being plugged by pieces of gravel.

2.8.5. Cleanouts and Monitoring Ports

2.8.5.1. All pressure distribution laterals must be equipped with cleanouts and monitoring ports at the distal ends (see Figures 8A and 8B). These cleanouts and monitoring ports must:

2.8.5.1.1. have threaded removable caps or plugs on the ends of the laterals to allow for cleaning the laterals and for monitoring the lateral pressure,

2.8.5.1.2. be large enough to allow access to caps or plugs with hands, tools, etc.

2.8.5.1.3. be accessible from the ground surface,

2.8.5.1.4. be open and slotted at the bottom, and

2.8.5.1.5. be void of gravel to the infiltrative surface to allow visual monitoring of standing water in the trench or bed.

2.8.5.2. All designs must show them in detail and explain how they accomplish the respective tasks.

The functions of monitoring and cleanout can be separated and also be accomplished in other ways.

2.8.6. Trenches

2.8.6.1. In a pressure drainfield, as in any drainfield, the bottom of the trench must be level, ± 0.5 inches.

2.8.6.2. The bottom and sides of the trench must not be smeared.

2.8.6.3. In gravel-filled trenches and beds, an acceptable geotextile must be used on top of the gravel before backfilling.

2.8.6.4. On sloping sites, the trenches and laterals must run parallel to the natural ground contours.

2.9. Minimum Design Submittal

A completed design must include the following as a minimum:

2.9.1. meeting all requirements of WAC 246-272A-0200,

2.9.2. daily design flow,

2.9.3. septic tank size, location and outlet invert elevation,

2.9.4. pump pickup elevation and location, or siphon invert elevation and location,

2.9.5. size of pump or siphon chamber,

2.9.6. transport line length, location, highest elevation, and diameter,

2.9.7. all valves or other such components in the system,

2.9.8. manifold diameter, location, length, and orientation,

2.9.9. lateral diameter, location, length, orientation, and elevations,

2.9.10. orifice diameter, spacing, and orientation,

2.9.11. dose volume, pumping rate (gpm), dose frequency, and design residual pressure,

2.9.12. location and detail of access ports on the laterals,

2.9.13. detail of pump controls, floats, and the position of the floats,

2.9.14. an electrical wiring diagram specific to the project, 2.9.15. system parameters and calculations used by the designer to arrive at the component sizing and flow distribution shown in the design, and

2.9.16. a user's manual for the pressure distribution system must be developed and provided to the homeowner and the local health department. This document may be developed in conjunction with the installer and submitted with the as-built information, but will be the responsibility of the designer.

2.10. Construction Record Information

A completed construction record submission must contain, at a minimum, the following items:

2.10.1. all the items contained in the design submittal listed above, as installed, identifying any changes from the approved plan,

2.10.2. the measured drawdown per dose cycle,

2.10.3. timer functions,

2.10.4. residual pressure and/or squirt height at the end of each lateral, as inspected, and

2.10.5. pump run time and pump time off.

2.11. User’s Manual

The user’s manual that is a part of the design submittal must contain, at a minimum, the following:

2.11.1. diagrams of the system components,

2.11.2. explanation of general system function, operational expectations, owner responsibility, etc.,

2.11.3. specifications of all electrical and mechanical components installed (occasionally components other than those specified on the plans are used),

2.11.4. names and telephone numbers of the system designer, local health jurisdiction, component manufacturers, supplier/installer, and/or the management entity to be contacted in the event of a failure,

2.11.5. information on the periodic maintenance requirements of the various components of the sewage system, and

2.11.6. information on "trouble-shooting" common operational problems that might occur. This information should be as detailed and complete as needed to assist the system owner to make accurate decisions about when and how to attempt corrections of operational problems, and when to call for professional assistance.

3. Operation and Maintenance

The systems must be monitored and maintained at a frequency commensurate with the site, soil, system complexity and use patterns. As a minimum, it is strongly recommended that the items in 3.1 - 3.5 be inspected at six months and then yearly, after the system is put into use. The local health department permit should clearly delineate who must perform the inspections. Refer to the system construction record for initial readings and settings. The owners of pressure distribution systems should be notified that their systems should be inspected and / or serviced on a yearly basis.

3.1. Evaluate Drainfield

3.1.1. for indications of surfacing effluent.

3.1.2. for appropriate vegetation, landscaping impacts, ponds, etc.

3.1.3. for absence of heavy traffic.

3.1.4. for inappropriate building.

3.1.5. for impervious materials or surfaces.

3.1.6. for abnormal settling or erosion.

3.2. Evaluate Laterals

3.2.1. for residual pressure at the distal ends. Confirm that it is the same as those recorded on the construction record. If not the same, laterals and orifices need to be cleaned.

3.2.2. for equal flows in each lateral.

3.2.3. for need for cleaning. Clean laterals and orifices as necessary.

3.3. Measure Pump Run Time per Cycle and Drawdown

Compare these values with those recorded in the construction record. If not the same, evaluate the system for improperly set timer control, float switches, clogged laterals, plugged orifices.

3.4. Test Alarms

Test alarms for proper functioning (high and low liquid level).

3.5. Evaluate Septic Tank and Pump Chamber

3.5.1. for sludge and scum accumulations; pump when the sludge and scum thickness total 1/3 of the depth of the tank.

3.5.2. for clogging, damage, and proper placement of outlet baffle screen. Clean each time it is inspected or as needed to avoid clogging.

3.5.3. for signs of leaking in tanks and risers. Repair or replace if necessary.

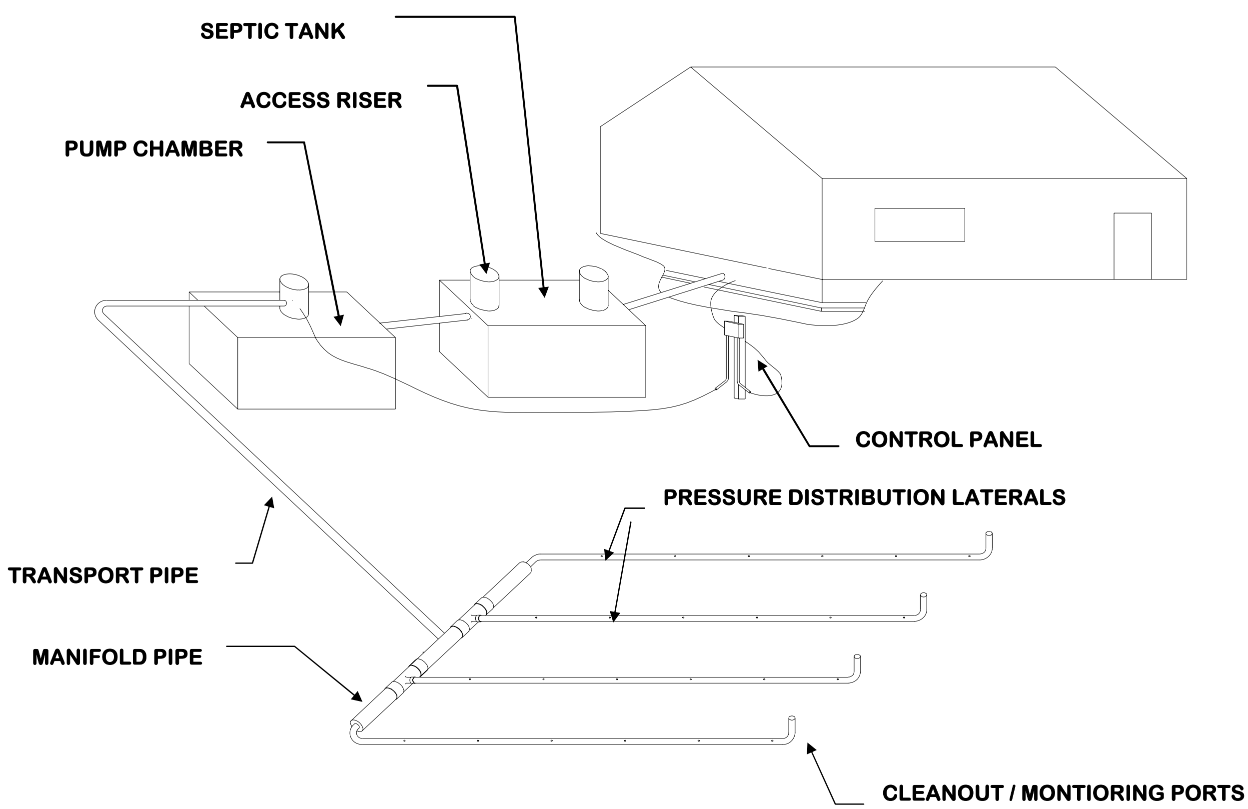
3.5.4. for risers and lids being above grade and having lids that are secure.

3.5.5. for properly functioning of floats. Movement should not be restricted. Floats should be positioned correctly and provide positive instrumentation signals. Adjust and repair as necessary.

3.6. Findings and Repairs

All findings and repairs are to be recorded, records filed for ready access, and reports sent to local health department.

Figures



PRESSURE DISTRIBUTION DRAINFIELD

FIGURE 1

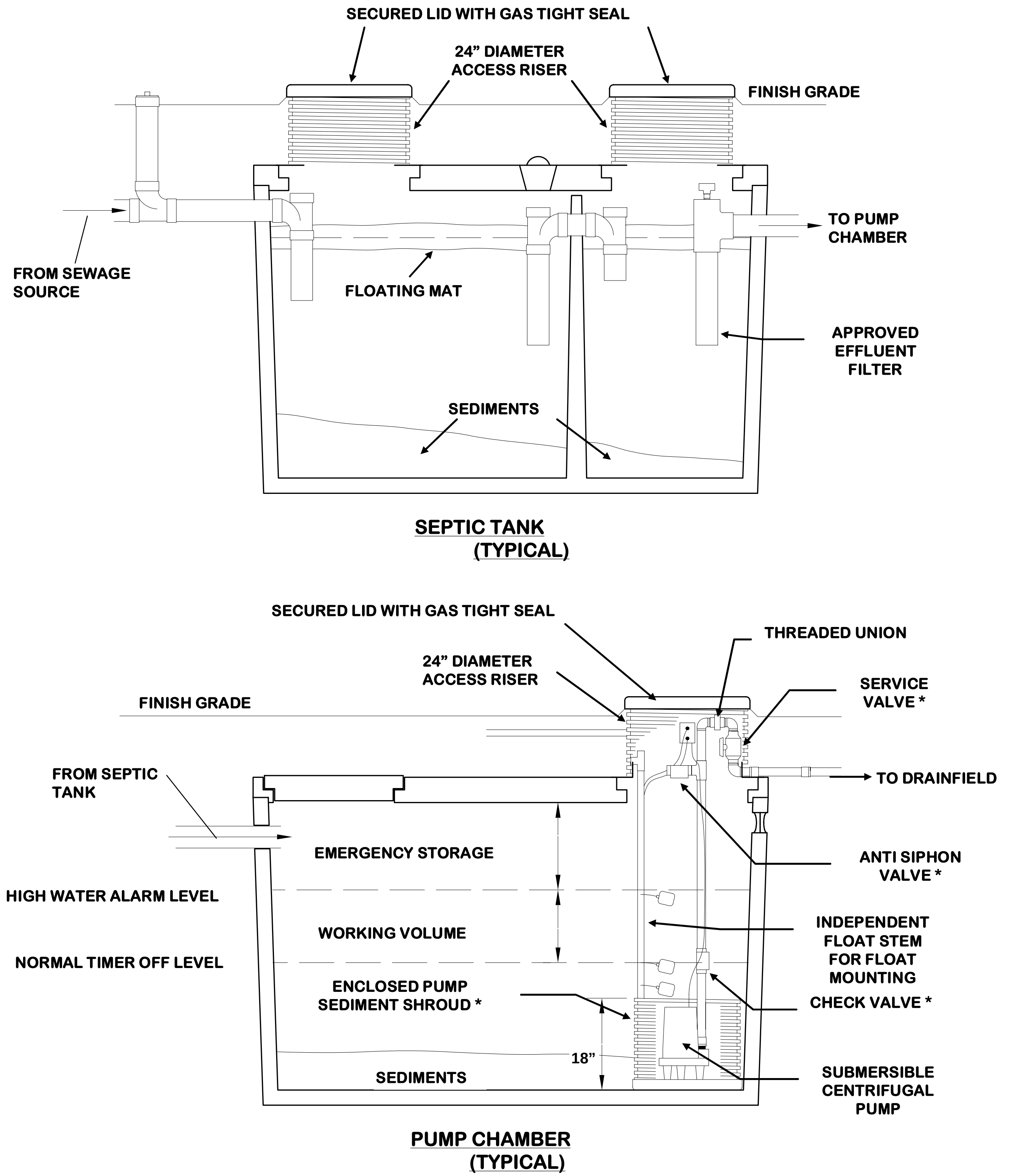
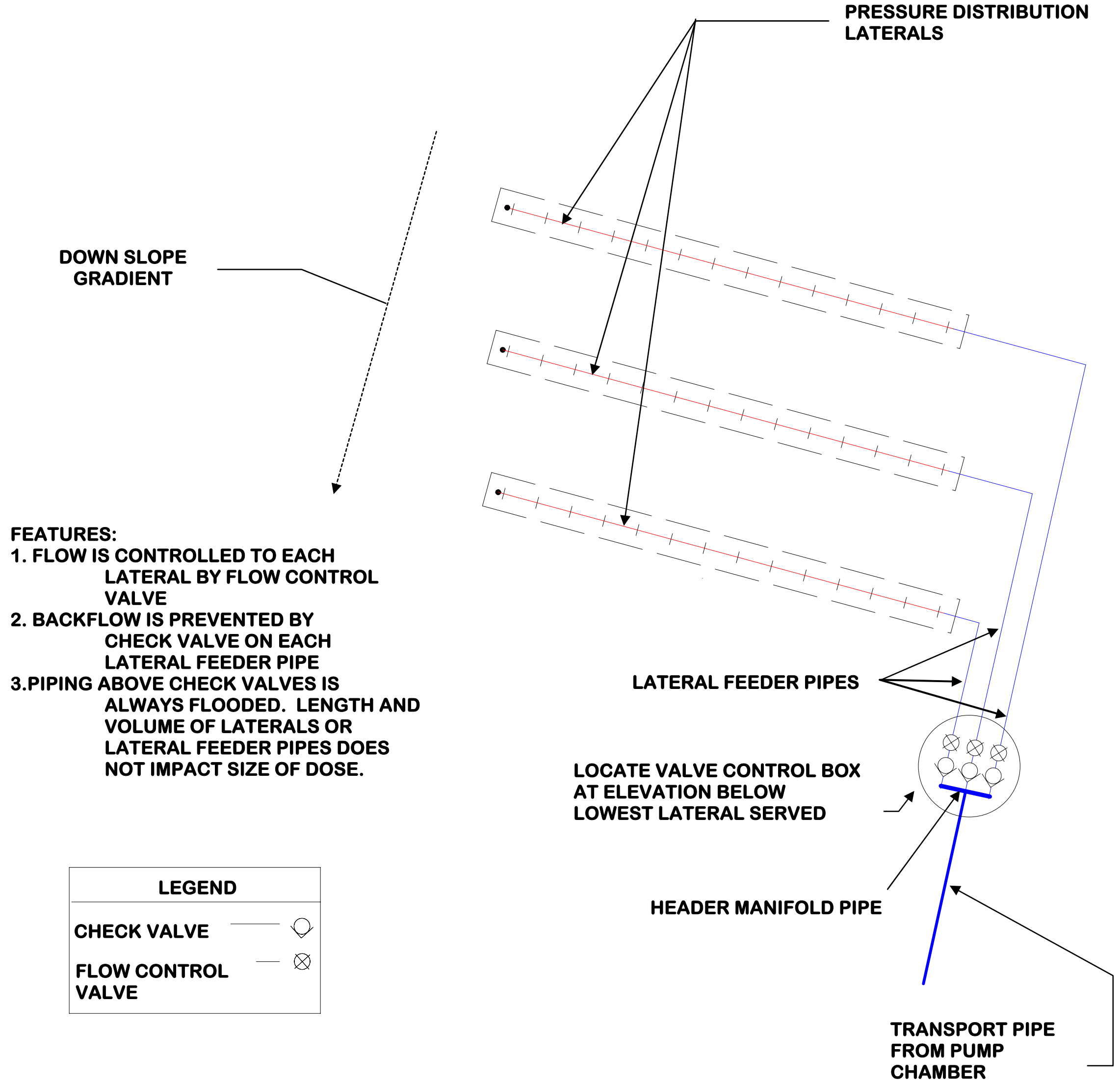
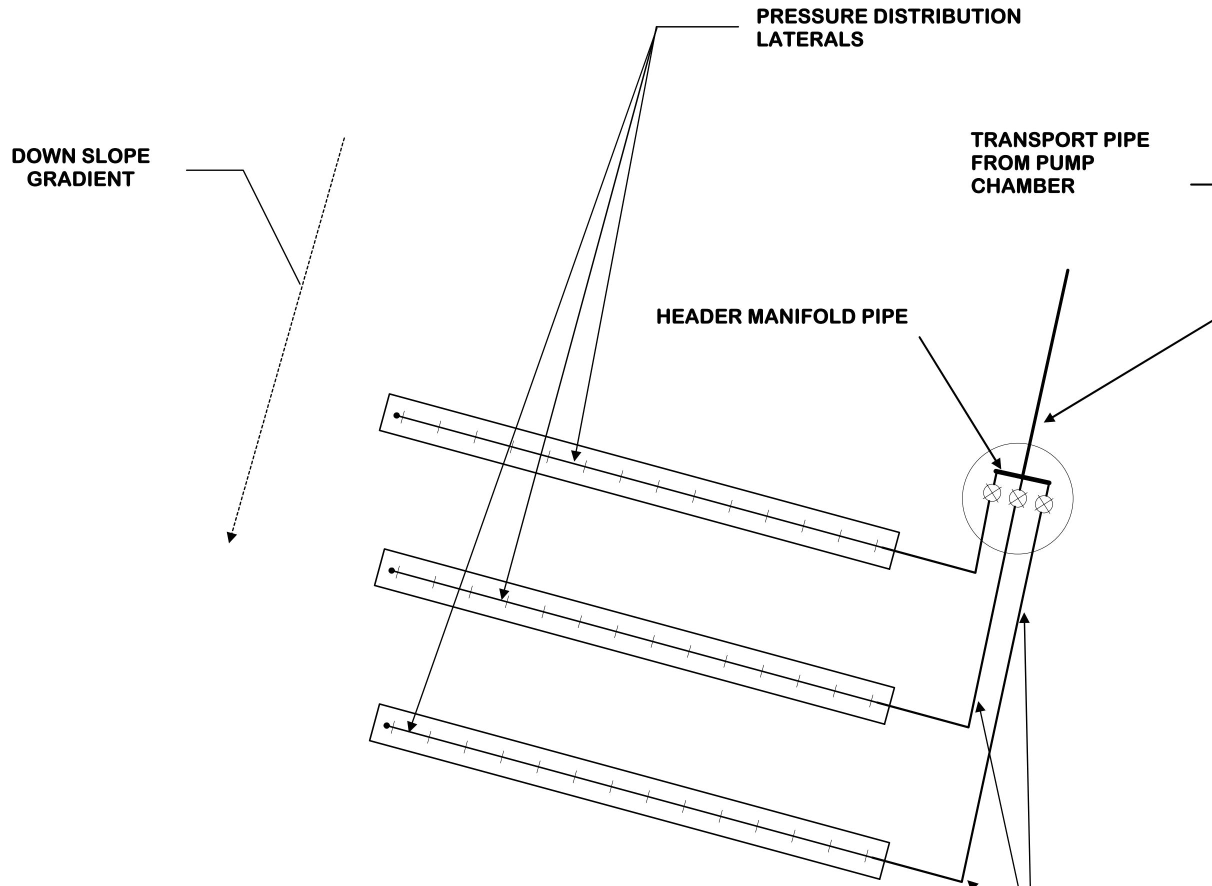

SECURED LID WITH GAS TIGHT SEAL
24" DIAMETER ACCESS RISER
FINISH GRADE
TO PUMP CHAMBER
FROM SEWAGE SOURCE
FLOATING MAT
APPROVED EFFLUENT FILTER
SEDIMENTS
SEPTIC TANK (TYPICAL)
SECURED LID WITH GAS TIGHT SEAL
THREADED UNION
24" DIAMETER ACCESS RISER
SERVICE VALVE *
FINISH GRADE
FROM SEPTIC TANK
TO DRAINFIELD
EMERGENCY STORAGE
ANTI SIPHON VALVE *
HIGH WATER ALARM LEVEL
WORKING VOLUME
INDEPENDENT FLOAT STEM FOR FLOAT MOUNTING
NORMAL TIMER OFF LEVEL
ENCLOSED PUMP SEDIMENT SHROUD *
CHECK VALVE *
18"
SEDIMENTS
SUBMERSIBLE CENTRIFUGAL PUMP
PUMP CHAMBER (TYPICAL)
* AS NEEDED

FIGURE 2



PRESSURE DISTRIBUTION DRAINFIELD (SLOPING GROUND)

FIGURE 3A



FEATURES:
1. FLOW IS CONTROLLED TO EACH LATERAL BY FLOW CONTROL VALVE
2. BACKFLOW IS PREVENTED BY MANIFOLD POSITION ABOVE LATERALS
3. ACCEPTABLE VARIANCE OF DOSE TO EACH LATERAL MAY BE IMPOSSIBLE DUE TO DIFFERING VOLUMES WITHIN EACH OF THE LATERAL FEEDER PIPES, ALL OF WHICH DRAIN AFTER PUMP CYCLE

LEGEND	
CHECK VALVE	
FLOW CONTROL VALVE	

<u>PRESSURE DISTRIBUTION DRAINFIELD</u>
<u>(SLOPING GROUND)</u>

FIGURE 3B

Manifold Below

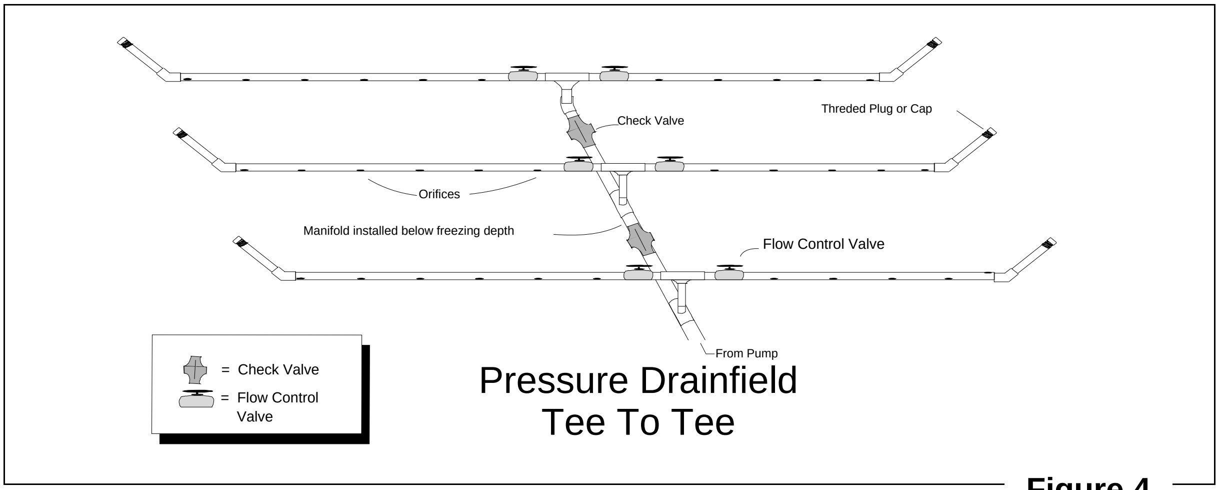


Figure 4

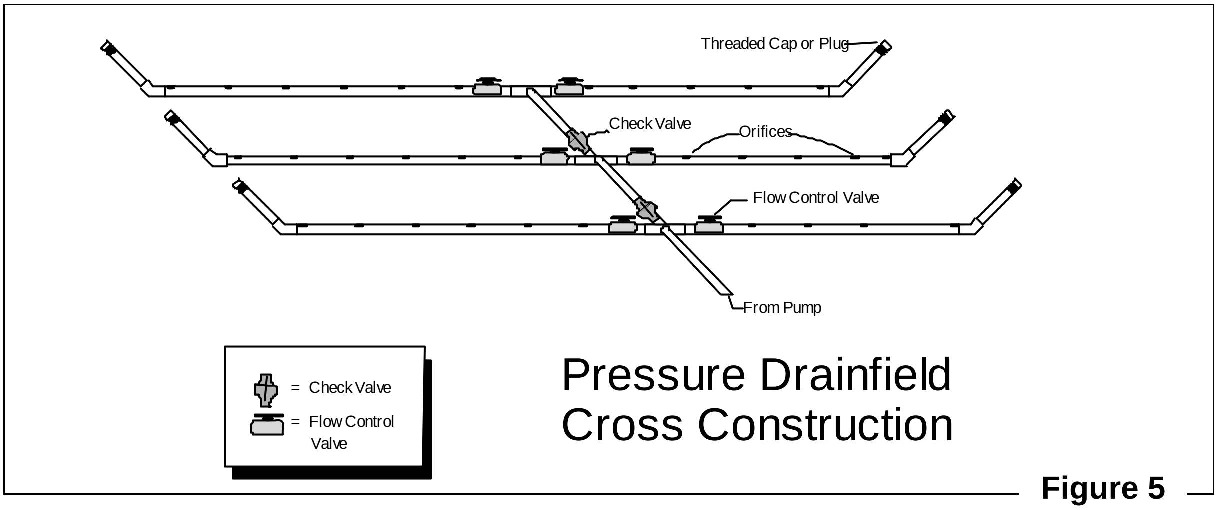
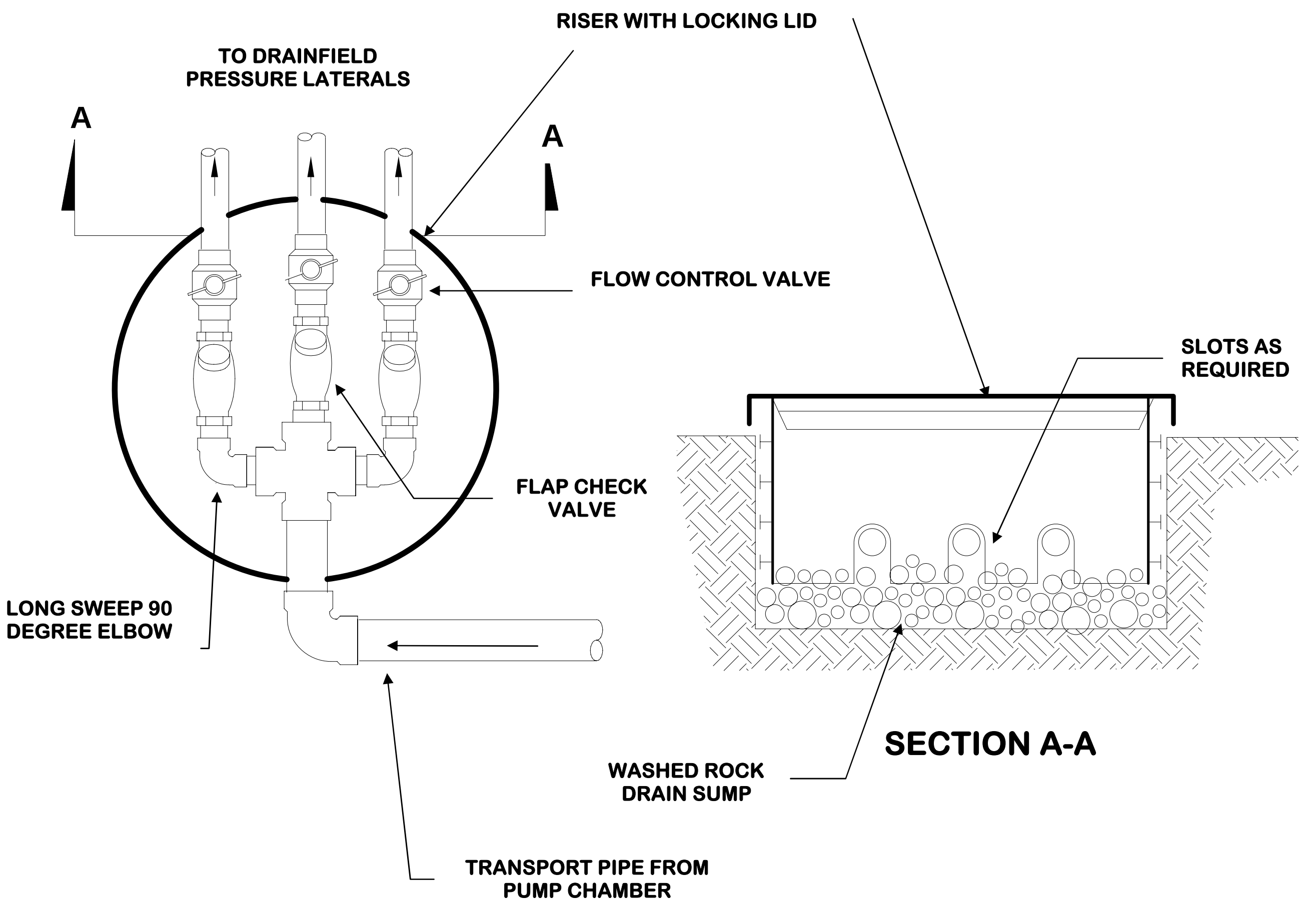
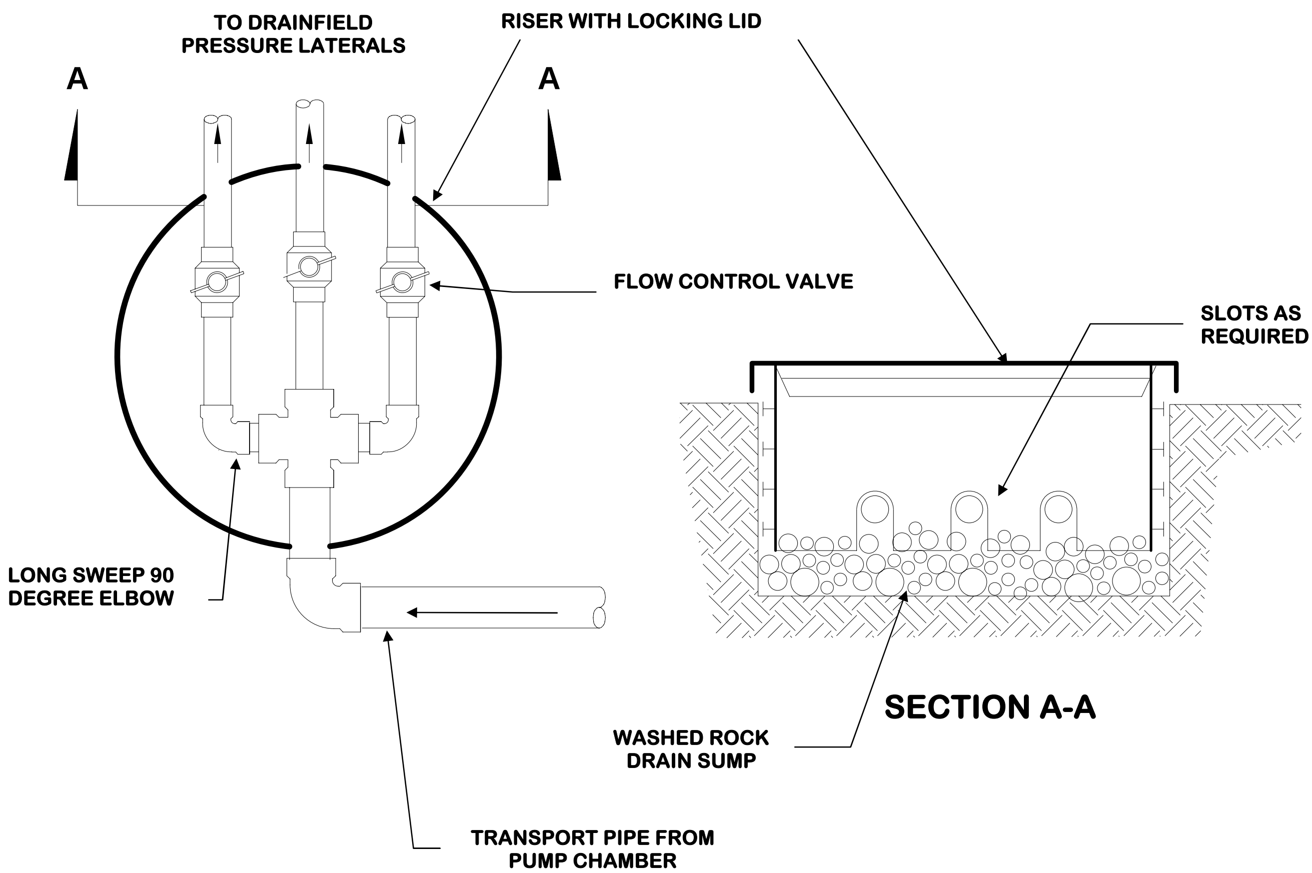


Figure 5



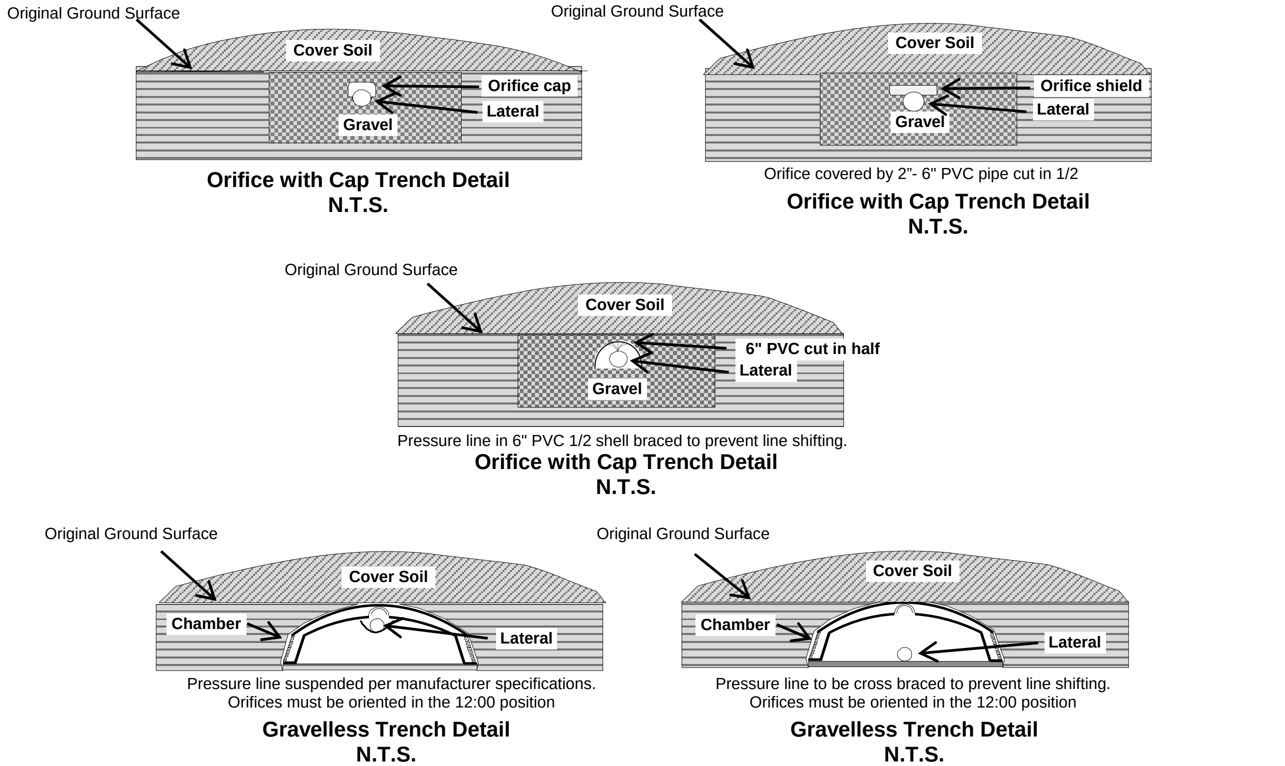
DRAINFIELD CONTROL BOX
(SLOPING GROUND; MANIFOLD BELOW LATERALS)

FIGURE 6A



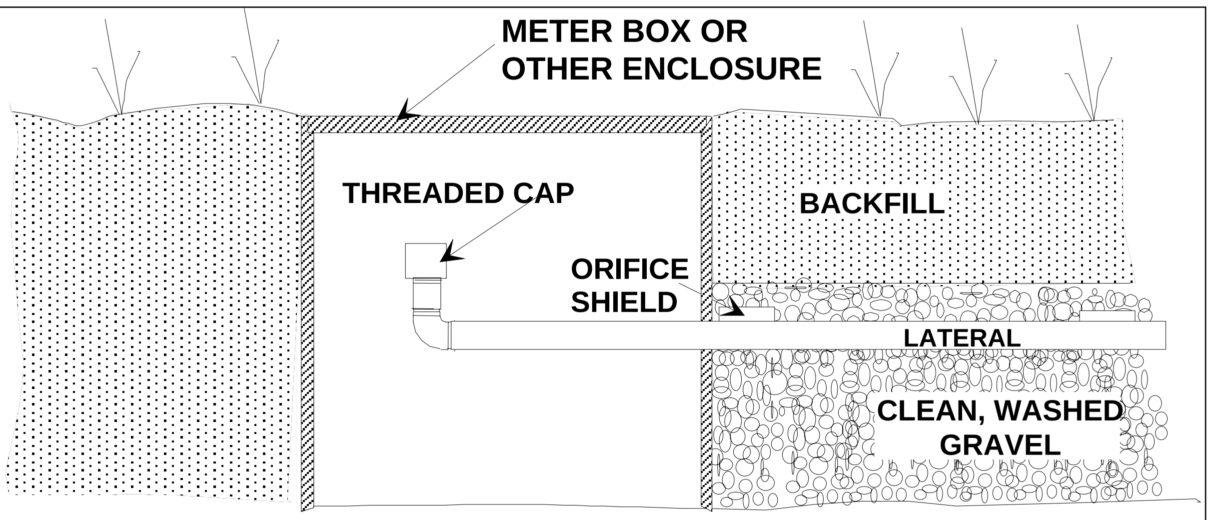
DRAINFIELD CONTROL BOX
(SLOPING GROUND; MANIFOLD ABOVE LATERALS)

FIGURE 6B



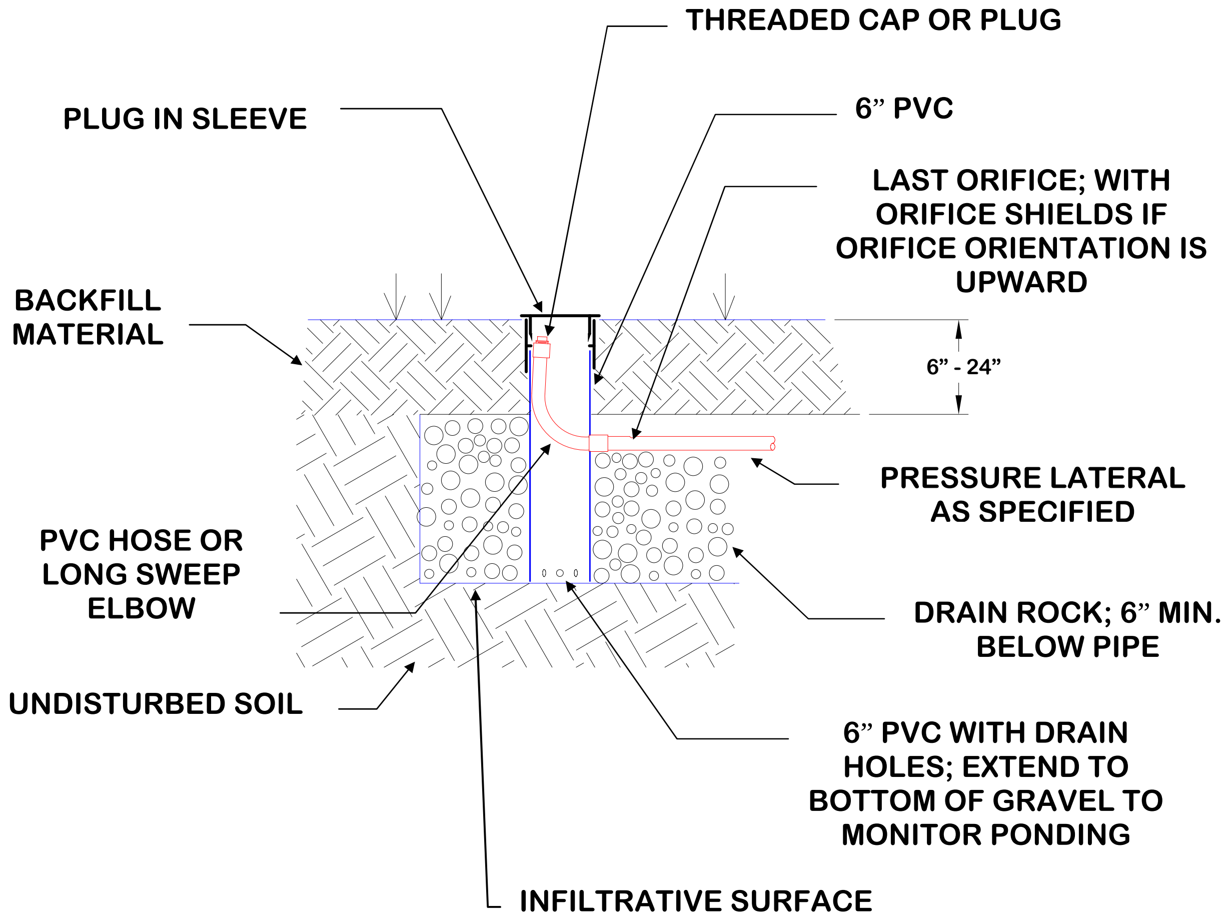
Orifice Caps and Shields

FIGURE 7



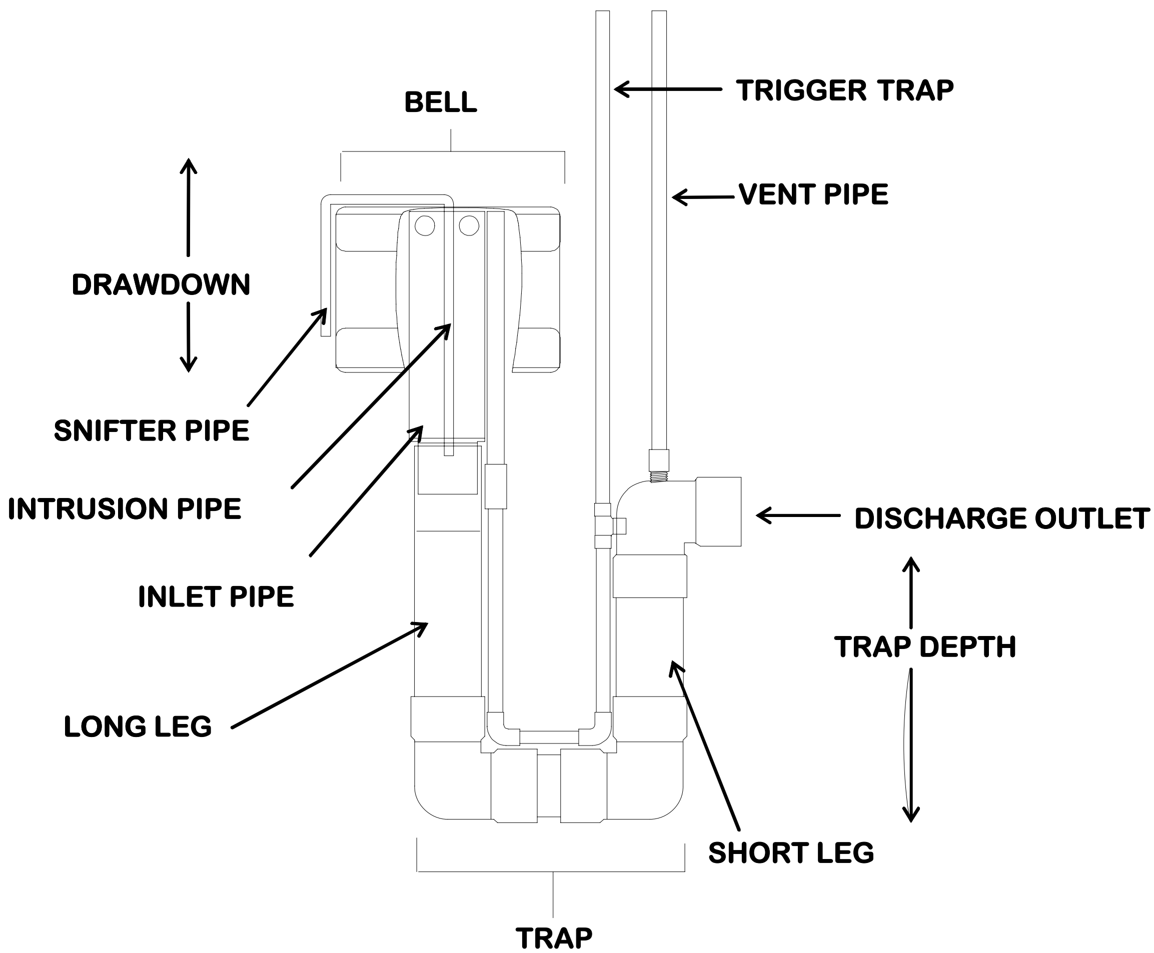
CLEANOUT AND MONITORING PORT

FIGURE 8A



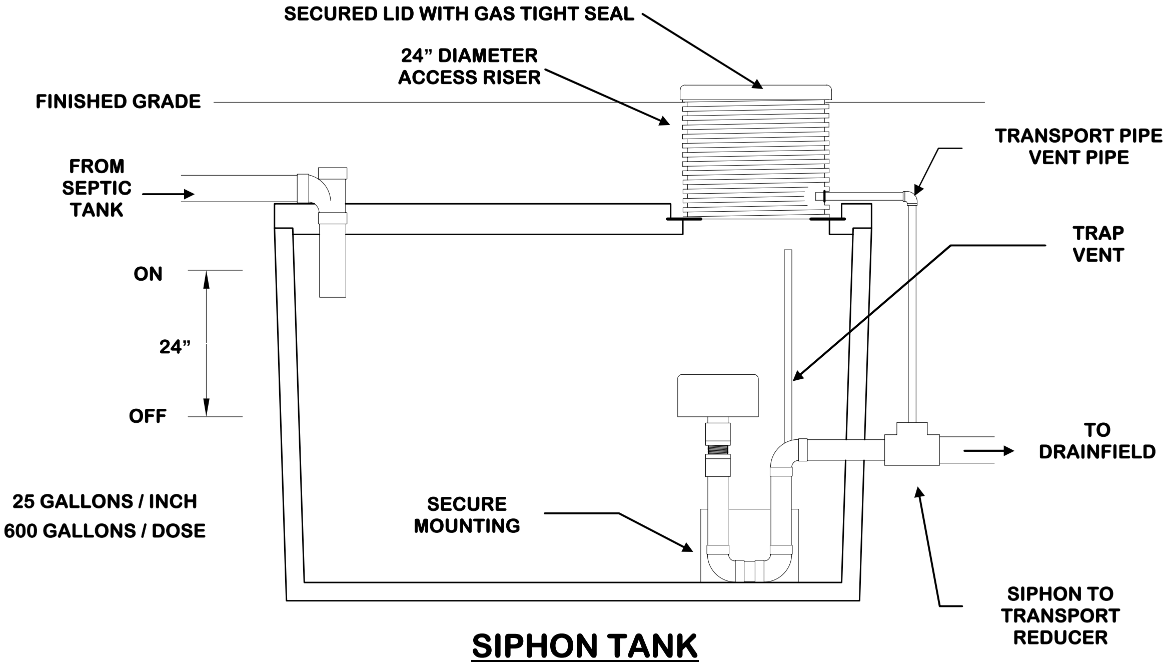
MONITORING/CLEANOUT PORT (EXAMPLE)

FIGURE 8B

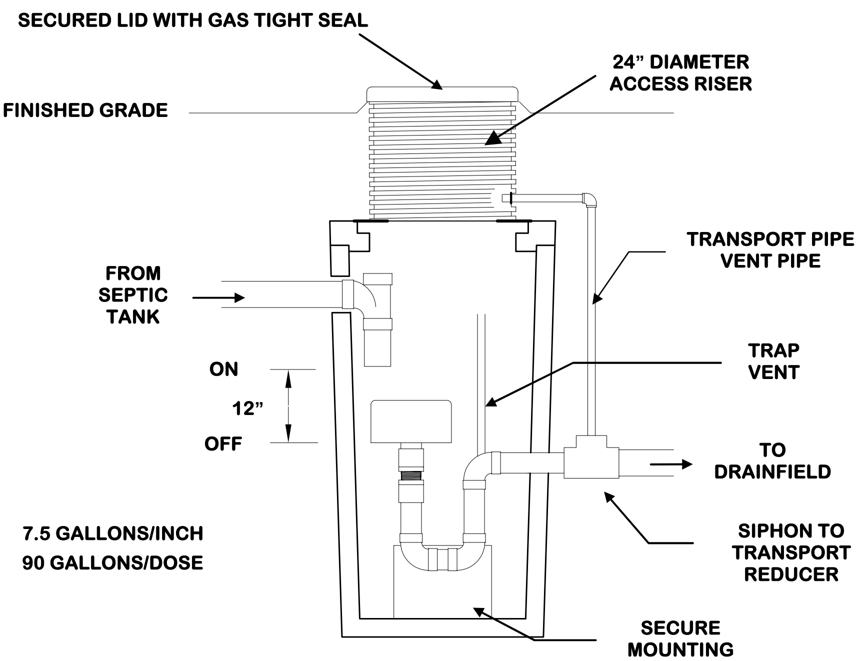


EXAMPLE OF SIPHON

FIGURE 9



SIPHON TANK
HIGH DOSE VOLUME EXAMPLE



SIPHON TANK
LOW DOSE VOLUME EXAMPLE

FIGURE 10

Appendix A – Useful Tables for Pressure Distribution

The design tables in the four sections of this appendix have been developed in order to allow the designer to evaluate alternative lateral configurations.

Appendix A-1, LATERAL DESIGN TABLE, has a table of maximum lateral lengths for various lateral diameters, orifice diameters and orifice spacings, and includes design criteria used to calculate maximum lateral lengths.

Appendix A-2, ORIFICE DISCHARGE RATE DESIGN AID, contains a derivation of an equation used to calculate orifice discharge rates and includes a table of discharge rates for various residual heads and orifice diameters.

Appendix A-3, FRICTION LOSS DESIGN AID, includes a derivation of an equation that can be used to calculate friction losses and a table of constants to simplify the calculation. Also included is a table of friction loss for PVC pipe fittings.

Appendix A-4, MAXIMUM MANIFOLD LENGTHS, lists the assumptions used to calculate the enclosed tables for maximum manifold length, one for 1/8 inch and 5/32 inch orifices (where the minimum residual head at the distal orifice must be 5 feet) and one for orifices of 3/16 inch and up (where the minimum residual head at the distal orifice must be 2 feet).

Throughout Appendix A, it is assumed that laterals and manifolds will be constructed using only PVC pipe materials conforming to ASTM standards D-2241 or D-1785.

A-1: LATERAL DESIGN TABLES

The maximum allowable length for any lateral is determined by allowable differences in discharge rates between the proximal and distal orifices. These tables assume that $Q_p/Q_d \leq 1.1$

Where Q_p = the proximal orifice discharge rate
Q_d = the distal orifice discharge rate

The maximum allowable difference in discharge rates is 10%. The maximum allowable lateral length is a function of lateral diameter and orifice diameter and is independent of the residual pressure.

Orifice discharge rates are a function of orifice diameter and residual pressure (see Appendix A-2 for a discussion). Table A-1 gives the maximum lateral length for each orifice diameter, lateral diameter, and orifice spacing.

Table A-1
Lateral Design Table

			Maximum Lateral Length (ft)		
Orifice	**Lateral**	**Orifice Spacing**	Pipe Material		
(inches)	(inches)	(feet)	**Schedule 40**	**Class 200**	**Class 160**
1/8	1	1.5	42	51	
1/8	1	2	50	62	
1/8	1	2.5	57.5	72.5	
1/8	1	3	66	81	
1/8	1	4	80	96	
1/8	1	5	90	110	
1/8	1	6	102	126	
1/8	1.25	1.5	66	76.5	79.5
1/8	1.25	2	80	92	96
1/8	1.25	2.5	92.5	107.5	110
1/8	1.25	3	105	120	123
1/8	1.25	4	124	144	148
1/8	1.25	5	145	165	175
1/8	1.25	6	162	186	192
1/8	1.5	1.5	85.5	96	100.5
1/8	1.5	2	104	116	120
1/8	1.5	2.5	120	135	140
1/8	1.5	3	135	150	156
1/8	1.5	4	164	184	188
1/8	1.5	5	190	210	220
1/8	1.5	6	210	240	246
1/8	2	1.5	132	141	145.5
1/8	2	2	160	170	176
1/8	2	2.5	185	197.5	202.5
1/8	2	3	207	222	228
1/8	2	4	248	268	276
1/8	2	5	290	310	320
1/8	2	6	324	348	360
5/32	1	1.5	31.5	39	39
5/32	1	2	36	46	46
5/32	1	2.5	42.5	52.5	52.5
5/32	1	3	48	60	60

Table A-1
Lateral Design Table (continued)

			Maximum Lateral Length (ft)		
Orifice	**Lateral**	**Orifice Spacing**	Pipe Material		
(inches)	(inches)	(feet)	**Schedule 40**	**Class 200**	**Class 160**
5/32	1	4	56	72	72
5/32	1	5	65	80	85
5/32	1	6	72	90	96
5/32	1 1/4	1.5	48	55.5	58.5
5/32	1 1/4	2	58	68	70
5/32	1 1/4	2.5	67.5	77.5	80
5/32	1 1/4	3	75	87	90
5/32	1 1/4	4	92	104	108
5/32	1 1/4	5	105	120	125
5/32	1 1/4	6	120	138	144
5/32	1 1/2	1.5	63	70.5	73.5
5/32	1 1/2	2	76	84	88
5/32	1 1/2	2.5	87.5	97.5	102.5
5/32	1 1/2	3	99	111	114
5/32	1 1/2	4	120	132	136
5/32	1 1/2	5	140	155	160
5/32	1 1/2	6	156	174	180
5/32	2	1.5	96	103.5	106.5
5/32	2	2	116	124	128
5/32	2	2.5	135	142.5	147.5
5/32	2	3	150	162	168
5/32	2	4	184	196	200
5/32	2	5	210	225	235
5/32	2	6	240	252	264
3/16	1	1.5	24	30	
3/16	1	2	28	36	
3/16	1	2.5	32.5	42.5	
3/16	1	3	39	45	
3/16	1	4	44	56	
3/16	1	5	50	65	
3/16	1	6	60	72	
3/16	1.25	1.5	37.5	43.5	45

Table A-1
Lateral Design Table (continued)

			Maximum Lateral Length (ft)		
Orifice	**Lateral**	**Orifice Spacing**	Pipe Material		
(inches)	(inches)	(feet)	**Schedule 40**	**Class 200**	**Class 160**
3/16	1.25	2	46	54	56
3/16	1.25	2.5	52.5	62.5	62.5
3/16	1.25	3	60	69	72
3/16	1.25	4	72	84	88
3/16	1.25	5	85	95	100
3/16	1.25	6	96	108	114
3/16	1.5	1.5	49.5	55.5	57
3/16	1.5	2	60	68	70
3/16	1.5	2.5	70	77.5	80
3/16	1.5	3	78	87	90
3/16	1.5	4	92	104	108
3/16	1.5	5	110	120	125
3/16	1.5	6	120	138	144
3/16	2	1.5	76.5	81	84
3/16	2	2	92	98	102
3/16	2	2.5	105	112.5	117.5
3/16	2	3	120	129	132
3/16	2	4	144	152	160
3/16	2	5	165	180	185
3/16	2	6	186	198	210
7/32	1	1.5	19.5	24	
7/32	1	2	24	30	
7/32	1	2.5	27.5	35	
7/32	1	3	30	39	
7/32	1	4	36	44	
7/32	1	5	45	55	
7/32	1	6	48	60	
7/32	1.25	1.5	31.5	36	37.5
7/32	1.25	2	38	44	46
7/32	1.25	2.5	42.5	50	52.5
7/32	1.25	3	48	57	60
7/32	1.25	4	60	68	72
7/32	1.25	5	70	80	80

Table A-1
Lateral Design Table (continued)

			Maximum Lateral Length (ft)		
Orifice	**Lateral**	**Orifice Spacing**	Pipe Material		
(inches)	(inches)	(feet)	**Schedule 40**	**Class 200**	**Class 160**
7/32	1.25	6	78	90	90
7/32	1.5	1.5	40.5	45	46.5
7/32	1.5	2	50	54	56
7/32	1.5	2.5	57.5	62.5	65
7/32	1.5	3	63	72	75
7/32	1.5	4	76	88	88
7/32	1.5	5	90	100	105
7/32	1.5	6	102	114	114
7/32	2	1.5	63	66	69
7/32	2	2	76	80	84
7/32	2	2.5	87.5	92.5	95
7/32	2	3	99	105	108
7/32	2	4	116	124	132
7/32	2	5	135	145	150
7/32	2	6	156	162	168
1/4	1	1.5	16.5	21	
1/4	1	2	20	24	
1/4	1	2.5	22.5	27.5	
1/4	1	3	27	33	
1/4	1	4	32	40	
1/4	1	5	35	45	
1/4	1	6	42	48	
1/4	1.25	1.5	27	30	31.5
1/4	1.25	2	32	36	38
1/4	1.25	2.5	37.5	42.5	45
1/4	1.25	3	42	48	48
1/4	1.25	4	48	56	60
1/4	1.25	5	55	65	70
1/4	1.25	6	66	72	78
1/4	1.5	1.5	34.5	39	39
1/4	1.5	2	42	46	48
1/4	1.5	2.5	47.5	52.5	55

Table A-1
Lateral Design Table (continued)

			Maximum Lateral Length (ft)		
Orifice	**Lateral**	**Orifice Spacing**	Pipe Material		
(inches)	(inches)	(feet)	**Schedule 40**	**Class 200**	**Class 160**
1/4	1.5	3	54	60	63
1/4	1.5	4	64	72	76
1/4	1.5	5	75	85	85
1/4	1.5	6	84	96	96
1/4	2	1.5	52.5	55.5	58.5
1/4	2	2	64	68	70
1/4	2	2.5	72.5	77.5	80
1/4	2	3	81	87	90
1/4	2	4	100	104	108
1/4	2	5	115	120	125
1/4	2	6	126	138	144

A-2: ORIFICE DISCHARGE RATE DESIGN AID

Orifice discharge rates can be calculated using Toricelli's equation:

$$Q = C_d\, A_o \sqrt{2gh}$$

Where: Q = the discharge rate in ft^3/sec
C_d = the discharge coefficient (unitless)
A_o= the cross sectional area of the orifice in ft^2
g = the acceleration due to gravity (32.2 ft/sec^2)
h = the residual pressure head at the orifice in ft

The formula shown above can be simplified for design purposes by incorporating the discharge coefficient and using conversion factors so that the discharge is given in gallons per minute and the orifice diameter is given in inches. The discharge coefficient depends on the characteristics of the orifice and is usually determined empirically. This value can range from 0.6 to 0.8 but a value of 0.6 was assumed for the purpose of this design aid. The formula therefore simplifies to:

$$Q = 11.79\, d^2 \sqrt{h}$$

Where: Q = the orifice discharge rate in gpm
d = the orifice diameter in inches
h = the residual pressure head at the orifice in feet

On the next page Table A-2 gives orifice discharge rates (in gpm) generated using the above formula for various residual pressures (head) and orifice diameters.

Table A-2

Orifice Discharge Rates (gpm)					
Head (ft)	Orifice Diameter (in)				
	1/8	5/32	3/16	7/32	1/4
2			0.59	0.80	1.04
3			0.72	0.98	1.28
4			0.83	1.13	1.47
5	0.41	0.64	0.93	1.26	1.65
6	0.45	0.71	1.02	1.38	1.80
7	0.49	0.76	1.10	1.49	1.95
8	0.52	0.81	1.17	1.60	2.08
9	0.55	0.86	1.24	1.69	2.21
10	0.58	0.91	1.31	1.78	2.33

For residuals greater than 10 feet or for orifice diameters greater than 1/4 inch, the equation <u>must</u> be used. This is also true if the residual pressure is not a whole number. For large systems use the equation and verify with Table A-2.

<u>Note</u>: Table A-2 was generated assuming that the minimum residual head at the distal orifice is 5 feet when orifices are 1/8 and 5/32 inch in diameter, and 2 feet for larger orifice diameters.

A-3: FRICTION LOSS DESIGN AID

Friction losses in pipes can be calculated using the Hazen-Williams formula:

Original form: $V = 1.318 * C * R^{0.63} * S^{0.54}$

Where: V = velocity (ft/sec)
C = Hazen-Williams flow coefficient (unitless)
R = hydraulic radius (ft^2/ft)[1]
S = slope of energy grade line (ft/1000 ft)

This equation can be modified through algebraic substitutions and using unit conversions to yield a formula that directly calculates friction loss[2]:

$$f = \frac{10.46 L Q^{1.85}}{C^{1.85} D^{4.87}}$$

Where: f = friction loss (ft)
D = actual inside pipe diameter (in)
L = length of pipe (ft)
Q = flow (gpm)
C = Hazen-Williams flow coefficient (unitless)

The Hazen-Williams flow coefficient (C) depends on the roughness of the piping material. Flow coefficients for PVC pipe have been established by various researchers in a range of values from 155 to 165 for both new and used PVC pipe. A coefficient of C = 150 generally is considered to yield conservative results in the design of PVC piping systems.[3]
The equation shown above can be further simplified by assuming that only PVC pipe conforming to ASTM standard D-2241 (or D-1785 for Schedule 40 and Schedule 80 pipe) is used. With this assumption, the inside diameters ("D") for the various nominal pipe sizes can be determined and combined with all other constants to yield the following equation:

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[1] hydraulic radius = cross sectional area of the conduit divided by the inner perimeter of the conduit.

[2] Analysis of Pipe Flow Networks, Jeppson, Ann Arbor Science Publications, 1983 (p. 41).

[3] Handbook of PVC Pipe Design and Construction, 2nd Edition, Uni-Bell Plastic Pipe Association, 1982.

$$f = L\,(Q/K)^{1.85}$$

Where:

f = friction loss through pipe (ft)

L = length of pipe (ft)

Q = flow (gpm)

K = Constant from Table C-3-1

(K can be determined for any PVC pipe conforming to the above ASTM standards using the equation $K = 42.17 * D^{2.63}$)

Table A-3-1

Table for Constant "K"			
Nominal Pipe Diameter	Schedule 40	Class 200	Class 160
1	47.8	66.5	
1.25	98.3	122.9	129.4
1.5	147.5	175.5	184.8
2	284.5	315.2	332.5
2.5	454.1	520.7	551.1
3	803.9	873.3	920.5
4	1642.9	1692.7	1783.9
6	4826.6	4677.4	4932

Friction loss for some PVC pipe fittings, given in terms of equivalent length of pipe, are provided in Table A-3-2.

TABLE A-3-2
Friction Loss for PVC Fittings[1]

Equivalent Length of Pipe (feet) PVC Pipe Fittings				
Pipe Size (in)	90^o Elbow	45^o Elbow	Through Tee Run	Through Tee Branch
.5	1.5	0.8	1.0	4.0
.75	2.0	1.0	1.4	5.0
1	2.25	1.4	1.7	6.0
1.25	4.0	1.8	2.3	7.0
1.5	4.0	2.0	2.7	8.0
2	6.0	2.5	4.3	12.0
2.5	8.0	3.0	5.1	15.0
3	8.0	4.0	6.3	16.0
4	12.0	5.0	8.3	22.0
6	18.0	8.0	12.5	32.0
8	22.0	10.0	16.5	38.0

[1] From SPEC-DATA, Sheet 15, Plastic Pipe and Fitting Association, November 1994

A-4: MAXIMUM MANIFOLD LENGTHS

Tables A-4-1 and A-4-2 can be used to determine maximum manifold lengths for various manifold diameters, lateral discharge rates and lateral spacings. The method used to determine the table values is described below.

Pressurized distribution systems are designed to assure even distribution of effluent throughout the drainfield area. Even distribution maximizes the treatment capabilities and useful life of the absorption area. Completely uniform distribution is difficult or impossible to obtain because of friction losses that occur in all piping networks so we settle for a standard or acceptable variance in orifice discharges throughout the network. The maximum lateral lengths in Table A-1 were developed to assure there will be no more than a 10% variance (drop) in the discharge rates between the proximal and distal orifices in any given lateral. The maximum manifold lengths in the tables below were developed to assure there will be no more than a 15% variance in discharge rates between any two orifices in a given distribution system.

Two assumptions used to develop these tables are: (1) the maximum variance in orifice discharge rates within a network occurs between the proximal orifice in the first lateral connected to a manifold and the distal orifice on the last lateral connected to the manifold and (2) the friction loss that occurs between the proximal orifice of a lateral and the point where the lateral connects to the manifold is negligible.

Using the assumptions mentioned above a computer program was developed to calculate maximum manifold lengths for various manifold diameters, lateral discharge rates and lateral spacings. The program assumes that the discharge rate at the distal orifice of the last lateral in a distribution system is as listed in Table A-2 for a given orifice size at the required minimum residual head. That value is multiplied by 1.1 and 1.15 to determine the maximum allowable discharge rates at the proximal orifices of the last and first laterals in the network, respectively. The residual head (h) that corresponds to those discharges was calculated by manipulating the orifice discharge equation in Appendix A-2 and solving for "h".

Using the simplified equation in Appendix A-3, the friction loss that occurs across the manifold was calculated for various materials and pipe diameters ("K"), lateral discharge rates ("Q") and lateral spacings ("L"). The program adds the friction loss calculated for successive pipe segments to the residual pressure, which corresponds to the proximal orifice discharge at the last lateral. The combined value is compared to the residual pressure at the proximal orifice of the first lateral until it is equal to or greater than this value.

Maximum manifold lengths were calculated as described above for various pipe materials and orifice diameters. Slightly greater manifold lengths were obtained when 1/8 and 5/32 inch orifices were assumed using 5 feet residual pressure at the distal orifice (see Table A-4-2). These tables were generated using Schedule 40 as the pipe material, which yields the most conservative results (shorter manifold lengths).

Table A-4-1
(for orifice diameters of 3/16 in. and up with minimum 2 feet of residual head)

Maximum Manifold Length (ft)																																					
Lateral Discharge Rate (gpm/lateral)		Manifold Diameter (inches)																																			
		1 1/4						1 1/2						2						3						4						6					
Central Manifold	End Manifold	Lateral Spacing (ft)																																			
		2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10
5	10	4	6	4	6	8	10	6	6	8	12	8	10	10	12	16	18	24	20	22	27	32	42	48	60	34	45	52	72	80	90	72	93	112	144	176	200
10	20	2	3	4				2	3	4	6	8		6	6	8	12	8	10	12	15	20	24	32	30	22	27	32	42	48	60	46	57	72	90	112	120
15	30	2						2	3	4				4	6	4	6	8	10	10	12	12	18	24	20	16	21	24	30	40	40	34	45	52	66	80	90
20	40							2						2	3	4	6	8		8	9	12	12	16	20	12	18	20	24	32	30	28	36	44	54	64	80
25	50													2	3	4				6	9	8	12	16	10	10	15	16	18	24	30	24	30	36	48	56	60
30	60													2	3	4				6	6	8	6	8	10	10	12	16	18	24	20	22	27	32	42	48	60
35	70													2	3					4	6	8	6	8	10	8	12	12	18	16	20	18	24	28	36	40	50
40	80													2						4	6	4	6	8	10	8	9	12	12	16	20	18	21	28	36	40	40
45	90																			4	3	4	6	8	10	6	9	8	12	16	20	16	21	24	30	32	40
50	100																			4	3	4	6	8	10	6	9	8	12	16	10	14	18	24	30	32	40
55	110																			2	3	4	6	8		6	6	8	12	8	10	14	18	20	24	32	30
60	120																			2	3	4	6			6	6	8	12	8	10	12	15	20	24	32	30
65	130																			2	3	4	6			6	6	8	6	8	10	12	15	20	24	24	30
70	140																			2	3	4				4	6	8	6	8	10	12	15	16	24	24	30
75	150																			2	3	4				4	6	8	6	8	10	10	15	16	18	24	30
80	160																			2	3	4				4	6	4	6	8	10	10	12	16	18	24	30
85	170																			2	3					4	6	4	6	8	10	10	12	16	18	24	20
90	180																			2	3					4	3	4	6	8	10	10	12	12	18	24	20
95	190																			2	3					4	3	4	6	8	10	8	12	12	18	16	20
100	200																			2						4	3	4	6	8	10	8	12	12	18	16	20

Instructions: This Table can be used to determine maximum length of a given diameter manifold *or* to determine required minimum diameter for a given manifold length. Known values must include:

1) Manifold - lateral configuration (end or central)
2) Lateral discharge rate "Q" in gallons per minute
3) Lateral spacing in feet

Example A: Central manifold configuration, lateral discharge "Q" = 40 gpm, lateral spacing = 6 ft., manifold diameter = 4 inch; Maximum length = 12 ft.
Example B: End manifold configuration, lateral discharge "Q" = 30 gpm, lateral spacing = 6 ft., manifold length = 18 ft.; Minimum diameter = 3 inch

TABLE A-4-2
(for orifice diameters of 1/8 in. and 5/32 in. with minimum 5 feet of residual head)

Maximum Manifold Length (ft)																																					
Lateral Discharge Rate (gpm/lateral)		Manifold Diameter (inches)																																			
		1 1/4						1 1/2						2						3						4						6					
Central	End	Lateral Spacing (ft)																																			
Manifold	Manifold	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10	2	3	4	6	8	10
5	10	6	9	8	12	16	10	8	12	12	18	16	20	14	18	20	30	32	40	30	39	48	60	72	80	48	63	76	96	120	130	100	129	156	204	240	280
10	20	4	3	4	6	8	10	4	6	8	6	8	10	8	12	12	18	16	20	18	24	28	36	40	50	30	39	48	60	72	80	64	81	100	126	152	180
15	30	2	3	4				4	3	4	6	8	10	6	6	8	12	8	10	14	18	20	24	32	30	22	30	36	42	56	60	48	63	76	96	112	130
20	40	2						2	3	4	6			4	6	8	6	8	10	12	15	16	18	24	30	18	24	28	36	40	50	40	51	60	78	96	110
25	50							2	3	4				4	6	4	6	8	10	10	12	12	18	16	20	16	21	24	30	40	40	34	45	52	66	80	90
30	60							2						4	3	4	6	8	10	8	9	12	12	16	20	14	18	20	24	32	40	30	39	48	60	72	80
35	70							2						2	3	4	6			8	9	12	12	16	20	12	15	20	24	24	30	26	36	40	54	64	70
40	80													2	3	4				6	9	8	12	16	10	12	15	16	18	24	30	24	30	36	48	56	70
45	90													2	3	4				6	6	8	12	8	10	10	12	16	18	24	20	22	30	36	42	56	60
50	100													2	3					6	6	8	6	8	10	10	12	12	18	24	20	20	27	32	42	48	60
55	110													2	3					4	6	8	6	8	10	8	12	12	18	16	20	20	24	28	36	48	50
60	120													2						4	6	8	6	8	10	8	9	12	12	16	20	18	24	28	36	40	50
65	130													2						4	6	4	6	8	10	8	9	12	12	16	20	18	21	28	36	40	50
70	140													2						4	6	4	6	8	10	8	9	12	12	16	20	16	21	24	30	40	40
75	150																			4	3	4	6	8	10	6	9	8	12	16	20	16	21	24	30	32	40
80	160																			4	3	4	6	8	10	6	9	8	12	16	10	14	18	24	30	32	40
85	170																			4	3	4	6	8		6	9	8	12	16	10	14	18	20	30	32	40
90	180																			2	3	4	6	8		6	6	8	12	8	10	14	18	20	24	32	30
95	190																			2	3	4	6	8		6	6	8	12	8	10	14	18	20	24	32	30
100	200																			2	3	4	6			6	6	8	12	8	10	12	15	20	24	32	30

Instructions: This Table can be used to determine maximum length of a given diameter manifold *or* to determine required minimum diameter for a given manifold length. Known values must include:

1) Manifold - lateral configuration (end or central)
2) Lateral discharge rate "Q" in gallons per minute
3) Lateral spacing in feet

Example A: Central manifold configuration, lateral discharge "Q" = 40 gpm, lateral spacing = 6 ft., manifold diameter = 4 inch; Maximum length = 18 ft.
Example B: End manifold configuration, lateral discharge "Q" = 30 gpm, lateral spacing = 6 ft., manifold length = 24 ft.; Minimum diameter = 3 inch

Appendix B – Volume of Pipe
(gallons per foot)

	Type of Pipe		
Nominal Diameter (in)	PR 160	PR 200	Schedule 40
0.75		0.035	0.028
1	0.058	0.058	0.045
1.25	0.098	0.092	0.078
1.5	0.126	0.121	0.106
2	0.196	0.188	0.174
2.5	0.288	0.276	0.249
3	0.428	0.409	0.384
4	0.704	0.677	0.661
5	1.076	1.034	1.039
6	1.526	1.465	1.501
8	2.586	2.485	
10	4.018	3.861	
12	5.652	5.432	

Appendix C - Advantages / Disadvantages of Dosing Systems

Demand Dosing, Timed Dosing, Reduced Dose Volumes, Orifices in 12:00 o'clock Position, Orifices in 6:00 o'clock Position, Network Remaining Full or Partially Full between doses

1. Demand Dosing

    a. Least complex of control systems and therefore least costly to install and easiest to understand.
    b. Not sensitive to heavy use days and therefore will not activate the alarm circuit with weekend guests, large laundry days or parties.
    c. <u>Does not protect</u> the drainfield, mound or sand filter from hydraulic surges and overload.
    d. Does not meter the effluent to the receiving component throughout a 24 hour period; instead delivers the dose whenever a dose volume accumulates in the pump chamber. Household water use patterns are usually in morning, evening and weekend surges.

2. Timed Dosing

    a. Meters the effluent to the receiving component in discrete, evenly spaced doses.
    b. Allows more frequent, smaller doses to be pumped to the receiving component, thereby promoting unsaturated flow through the soil or filter media.
    c. Protects the receiving component from hydraulic overload.
    d. Sensitive to heavy use days and therefore may often activate the alarm circuit when the volume of wastewater exceeds the design flow. Some causes are: weekend guests, large laundry days, parties, and leaking fixtures.
    e. More costly and complicated installation and maintenance.
    f. Can be used to help detect groundwater leaking into the septic tank or pump chamber.

3. Reduced Dose Volumes

    a. More frequent, smaller doses with intervening resting and aeration periods, are pumped to the receiving component, thereby assuring unsaturated flow through the soil or filter media.
    b. May require smaller orifices, smaller transport and lateral pipes, check valves and orifices in the 12 o'clock position in order to reduce the flow rate and to maintain the system full of effluent between doses. The smaller orifices will increase the frequency of maintenance due to clogging. Likewise, maintaining the pipes full of effluent between doses will promote more rapid biological growth on the inside of the pipes and thereby promote clogging of the orifices.

4. Orifices in the 12 o'clock Position

a. As mentioned above, orifices in this position will maintain the laterals full or partially full and therefore reduce the amount of effluent needed to pressurize the system. This feature is important when designing a system with reduced dose volumes.
b. Orifices in the "up" position require the use of orifice shields or chambers, to prevent blocking of some orifices with gravel pieces. Shields also deflect the squirt over a wider surface area and spread the effluent over more of the infiltrative surface. Shields have the greatest importance in systems with medium to coarse sand soils or with imported media providing the treatment.
c. Maintaining effluent in the lines will promote biological growth, which will accelerate clogging of the orifices and buildup of sludge and slime in the lines. It also makes the laterals subject to freezing in areas where this is a concern.
d. May be drained by putting a few orifices in the 6:00 o’clock position, or by draining laterals and transport line back to the surge tank. However, these practices will increase the dose volume required.

5. Orifices in the 6 o'clock Position

a. When some or all of the orifices are in the "down" position, the laterals will drain between dose cycles retarding the biological growth in them and reducing freeze up potential. When the system drains, a good rule of thumb for equal distribution is to design the dose volume to be at least 7 times the volume of the liquid that drains after a dose.
b. When the orifice at the distal end (farthest from the manifold) is in the down position, sludge in the lines tends to be driven to the distal end of the lateral and out the last orifice. As that orifice clogs, the next in line will clog, and so on.
c. Although systems with some or all of the orifices in the down position may be less prone to clogging, they also will require a larger dose volume to pressurize the system, due to laterals draining between pump cycles.
d. Orifices in the down position cannot be directed to gravelless chambers, and therefore will not have as wide a distribution pattern. However there are special orifice shields available for orifices oriented in this position.

6. Network Remaining Full, or Partially Full, Between Doses
(laterals can rarely be maintained at a level grade, therefore some orifices will be lower than others, so some of the effluent will drain out the lowest 12:00 o’clock orifice)

a. Allows smaller, more frequent doses with intervening resting and aeration periods, to be pumped to the receiving component, thereby promoting unsaturated flow through the soil or filter media.
b. Maintaining effluent in the lines will promote biological growth, which will accelerate clogging of the orifices and buildup of sludge in the lines. It also makes the laterals subject to freezing in areas where this is a concern.

Appendix D - Advantages / Disadvantages of Siphon Dosed Systems

1. Some advantages of siphons are:

    a. they do not require electricity;

    b. there are no moving parts;

    c. they can be constructed entirely of corrosion resistant material;

    d. they require very little maintenance;

    e. they do not require external controls as cycling is automatic;

    f. duplex installations can be made to alternate automatically;

    g. they can dose a remote drainfield without a large transport line to the siphon chamber;

    h. they allow the use of small pumps with low energy consumption, to dose a system with high velocity requirements.

2. Some drawbacks of siphons are:

    a. they cannot, by themselves, limit the total volume discharged to the drainfield in a day and therefore cannot protect the pressure distribution component from hydraulic overload;

    b. they can go into a trickling mode and will remain there until manually recharged with air; this condition does not achieve equal distribution and may destroy the receiving component;

    c. they are slower to enter the fully pressurized phase which can result in somewhat unequal distribution on a sloped site; and

    d. the available head to pressurize the system is fixed and therefore design and installation errors cannot be overcome by increasing the pressure head.

Appendix E - References

Converse, J.C., 1974. Distribution of Domestic Waste Effluent in Soil Absorption Beds, Transactions of the America Society of Agricultural Engineers, Vol. 17, No2, pp. 299-309.

Converse, J.C., J.L. Anderson, W.A. Ziebell, and ZJ. Bouma, 1975. Pressure Distribution to Improve Soil Absorption systems, Home Sewage Disposal, Proceedings National Home Sewage Disposal Symposium, American Society of Agricultural Engineers, St. Joseph, MI pp. 104-115.

Otis, R.J., J.C. Converse, B.L. Carlisle, and J.E. Witty, 1978. Effluent Distribution, Home Sewage Treatment, Proceedings of the 2nd National Home Sewage Treatment Symposium, American Society of Agricultural Engineers, St. Joseph, MI pp. 61-85.

APPENDIX O-VII

PRESSURE DISTRIBUTION NETWORK DESIGN

Septic tank effluent or other pretreated effluent can be distributed in a soil treatment/dispersal unit either by trickle, dosing or uniform distribution. Trickle flow, known as gravity flow, occurs each time wastewater enters the system through 4" perforated pipe. The pipe does not distribute the effluent uniformly but concentrates it in several areas of the absorption unit. Dosing is defined as pumping or siphoning a large quantity of effluent into the 4" inch perforated pipe for distribution within the soil absorption area. It does not give uniform distribution but does spread the effluent over a larger area than does gravity flow. Uniform distribution, known as pressure distribution, distributes the effluent somewhat uniformly throughout the absorption area. This is accomplished by pressurizing relatively small diameter pipes containing small diameter perforations spaced uniformly throughout the network and matching a pump to the network.

This material has been extracted and modified from a paper entitled “Design of Pressure Distribution Networks for Septic Tank- Soil Absorption Systems” by Otis, 1981. It also includes material from the “Pressure Distribution Component Manual for Private Onsite Wastewater Treatment Systems” by the State of Wisconsin, Department of Commerce, 1999.

Design Procedure

The design procedure is divided into two sections. The first part consists of sizing the distribution network which distributes the effluent in the aggregate and consists of the laterals, perforations, and manifold. The second part consists of sizing the force main, pump, dose chamber, and suitable controls.

A. Design of the Distribution Network: Steps:

1. Configuration of the network.

The configuration and size of the absorption field must meet all soil and site criteria. Once any limitations have been established, the distribution network can be designed.

2. Determine the length of the laterals.

Lateral lengths are defined as the distance length from the manifold to the end of the lateral. For a center manifold it is approximately one half the length of the absorption area. For end manifolds it is approximately the length of the absorption area. The lateral should end about 6" to 12" from the end of the absorption bed.

3. Determine the perforation size, spacing, and position.

The size of the perforation or orifices, spacing of the orifices and the number of orifices must be matched with the flow rate to the network.

Size: The typical perforation diameter has been 1/4", but with the requirement of Class I effluent, carry-over particles have been greatly reduced allowing smaller diameter orifices to be used. Orifices as small as 1/8" are commonly used in sand filter design, however orifice shields are generally used to protect the orifice from being compromised by the aggregate. Smaller diameter perforations are also at risk from burrs when drilling. Shop drilling the orifices under tight specifications reduces the concern. A sharp drill bit will drill a much more uniform orifice than a dull drill. Replace drills often. Remove all burrs and filing from pipe before assembling it. As a compromise, one might consider using 5/32" or 3/16" diameter orifices which will allow for more orifices than if 1/4" orifices were used.

Spacing: It is important to distribute the effluent as uniformly as possible over the system to increase effluent/soil contact time and maximize treatment efficiency. Typical spacing has been 30-36" but some designers have set spacing further apart to reduce pipe and pump sizes. Typical spacing for beds has been 6 ft^2/orifice (J.C.Converse; 2000).

Positioning: In cold climates, it is essential that the laterals drain after each dose event to prevent freezing. Because of the longer laterals normally encountered in mounds, the orifices are typically placed downward for draining as it is much more difficult to slope the lateral toward the manifold/force main because of their greater length.

4. Determine the lateral pipe diameter.

Based on the selected perforation size and spacing, Fig. A-1a through A-3b should be used to select the lateral diameter. Lateral diameter is also used to determine dose volume. (Fig. A-5).

5. Determine the number of perforations per lateral.

Use: $N = (p/x) + 0.5$ for center feed/center manifold
$N = (p/x) + 1$ for end fed/end manifold

Where:
N = number of perforations,
p = lateral length in feet and
x = perforation spacing in feet.
Round number off to the nearest whole number.

6. Determine the lateral discharge rate.

Based on the distal pressure selected, Table A-1 gives the perforation discharge rate. The designer must choose an operational pressure (in units of feet) at a distal point. This is the starting point of selecting a pump and determining if the system has equal distribution.

7. Determine the number of laterals and the spacing between laterals.

Since the criteria of 6 ft^2/orifice is the guideline, the orifice spacing and laterals spacing are interrelated. For absorption area widths of 3 ft, one distribution pipe along the length of trench requires an orifice spacing of 2 ft. For a 6 ft wide absorption area with the same configuration it would require orifice spacing of 1 ft. or the system could utilize a manifold with several laterals and have better coverage. **Ideally, the best option is to position the perforations to serve a square such as a 2.5' by 2.5' area** but that may be difficult to do but a 2' by 3' is much better than a 6' by 1' area.

8. Calculate the manifold size and length.

The manifold length is the length pipe between the outer laterals. For smaller systems assume the manifold size is the same as the force main diameter since the manifold is an extension of the force main. There are procedures for determining the manifold size for larger systems (Table A-2) from Otis, 1981.

9. Determine the network discharge rate.

This value is used to size the pump. 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.

B. Design and Selection of the Force Main, Pump, Dose Chamber and Controls.

1. Develop a system performance curve.

The system performance curve predicts 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 best way to select the pump is to evaluate the system performance curve and the pump performance curve. Where the two curves cross, is the point where the system operates relative to flow rate and head.

The total dynamic head that the pump must work against is the:

1. System network head (1.3 x distal pressure)

2. Elevation difference between the pump and the highest point in the system.

3. Friction loss in the force main.

The system network head is the pressure maintained in the system during operation to assure relatively uniform flow through the orifices. The 1.3 multiplier relates to the friction loss in the manifold and laterals which assumes that the laterals and manifold are sized correctly.

The elevation difference is between the pump and the highest point in the system in feet (the pump industry uses the bottom of the pump tank).

The friction loss in the force main between the pump tank and the inlet to the network is determined by using Table A-3. Equivalent length for fittings should be included. Equivalent lengths are found in Table A-4.

2. Determine the force main diameter.

The force main diameter is determined from Table A-2. The number of laterals and/or length of manifold should not exceed these maximums.

3. Select the pressurization unit.

Pumps

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. It can be oversized but will be more costly.

4. Determine the dose volume required.

The lateral pipe void volume determines the minimum dose volume. The recommended dose volume is 10 times the lateral volume. It is required that the system be timed dosed daily based on the design flow. Small doses need to be applied; however, sufficient volume is needed to distribute the effluent uniformly across the network. Table A-5 gives the void volume for various size pipes.

5. Size the dose tank.

For residential applications, the dose tank must be large enough to provide for:

a. The dose volume.
b. The dead space resulting from placement of the pump on a concrete block.
c. A few inches of head space for floats
d. 24 hour reserve capacity based on 150 gallons per bedroom.

The pump tank must have sufficient surge capacity to allow for timed dosing. See Section E of the manual for additional information and requirements for dosing other applications.

6. Select controls and alarms.

Select quality controls and alarms. Follow electrical code for electrical connections.

DESIGN EXAMPLE

This example will follow these steps to design a pressure distribution network for a bed system. All requirements found in Section F; *Absorption Field Methods* and *Guidelines for Class I Effluent* of the manual must be followed.

The bed absorption area is 452 ft^2 (113 ft long by 4 ft wide). The force main is 125 ft long and the elevation difference is 9 ft with three 90° elbows. Central manifold distribution system will be used.

A. Design of the distribution network. Steps:

1. Configuration of the network.

This is a narrow absorption bed on a sloping site. (4' x 113' = 452 ft^2)

2. Determine the lateral length.

Use a center feed, the lateral length is:

Lateral Length = (B / 2) - 0.5 ft Where: B = bed absorption length.
= (113 / 2) - 0.5 ft
= **56 ft**

3. Determine the perforation spacing and size.

Perforation spacing:

It is recommended that each perforation covers a maximum area of 6 ft^2. The absorption area is 4 ft wide.

Two laterals on each side of the center.

Spacing = (area/orifice x no. of laterals / (absorption area width)

= (6 ft^2 x 2) / (4 ft)
= **3 ft.**

Best option: Ideally, the best option is to position the perforations to serve a square but that may be difficult to do. In this example, each perforation serves a 2' by 3' rectangular area. With an absorption area of 6 ft wide with one lateral down the center, perforation spacing would be 1 ft apart and the perforation would serve an area of 6 by 1 ft which would be undesirable.

Perforation size:

Smaller diameter perforations may reduce system discharge flow rate, reduced pump requirements, at the same time increasing the number of orifices benefitting equal distribution through out the system. This example uses **3/16" perforations.**

4. Determine the lateral diameter.

Using **Fig. A-2a (3/16")** to determine the minimum lateral diameter:

The laterals on each side of the center manifold each has the length of 56 ft with 3 ft spacing between orifices, these point to a **lateral diameter of 1.5".**

5. Determine number of perforations per lateral and number of perforations.

Using 3.0 ft spacing in 56 ft a lateral yields 19 perforations each:

N = (p/x) +0.5 = (56 / 3.0) +0.5 = 19 perforations/lateral

Number of perforations = 4 lateral x 19 perforations/lateral = 76

Check - Maximum of 6 ft_2 / perforation =

Number of perforations = 412 sqft/6 ft^2 = 75; (76 > 75**, is okay**)

6. Determine lateral discharge rate (LDR).

Using network pressure (distal) pressure of 3.5 ft and 3/16" diameter perforations,
Table A-1 gives a discharge rate of 0.78 gpm, regardless of the number of laterals.

LDR = 0.78 gpm/perforation x 19 perforations = **14.8 gpm/lateral**

7. Determine the number of laterals.

This was determined in Step 3 and 4.

Two laterals on each side of center feed = **4 laterals spaced 2 ft apart.**

8. Calculate the manifold size.

The force main diameter is determined from Table A-2 on the manual. The manifold is generally the same size as force main as it is an extension of the force main or it could be one size smaller. This example will use a **2” manifold**.

9. Determine network discharge rate (NDR)

NDR = 4 laterals x 14.8 gpm/lateral = 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.

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** (Pump shut off to network elevation)

Head Loss in Force Main = Table A-3 and A-4 for 60 gallons and 125 ft of force main and 3 elbows.

Equivalent length of pipe for fittings can be found in Table A-3

3- 2" 90° elbows @ 9.0 ft each = **27 ft** of pipe equivalent.

Head Loss through 100’ of PVC pipe can be found in Table A-2

125’ of 2" force main plus the head loss in the fittings equals

= 7.0 (125 ft + 27 ft)/100 = **10.6 ft**

Total Dynamic Head (TDH) = Sum of the three

TDH = System head + Elevation head + Head Loss in Force Main

4.5 + 9 + 10.6 = 24.1 ft (2" force main) = **24 ft of head**

11. Pump Summary

Pump must discharge **60 gpm** against a head of **24 ft** with 2" 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.

12. Select the Pump

Using a performance curve from the pump manufacture, the point where the flow rate intersects (60 gpm) the total dynamic head (24 ft) should fall under the pump curve. A pump can be over sized, but undersized pumps will lead to failure in performance and/or longevity.

4. Determine the dose volume.

Determine the pipe void volume from Table A-5. Use 10 times the lateral void volume.

Dose Volume = 10 x length of lateral x number of laterals x Void volume

Lateral diameter =	1.5"
Lateral Length =	56'
No. of laterals =	4
Void volume =	0.092 gal/ft

10 x 56 x 4 x 0.092 = **206 gal./dose**

5. Size the dose tank.

The pump tank size should be based on the dose volume, 24 hour storage volume, and room for a block beneath the pump and control space. This example is for a residential application, additional information on dosing requirements can be found in Section E of the manual.

6. Select controls and alarm.

Time Dosing: The advantage of time dosing provides more frequent doses and levels out peak flows to the bed.

CONSTRUCTION AND MAINTENANCE

Good common sense should prevail when constructing and maintaining these systems. Water tight construction practices must be employed for all tanks. Surface runoff must be diverted away from the system. Any settling around the tanks must be filled with the soil brought to grade or slightly above to divert surface waters.

Table A-1 **Perforation Discharge Rates (GPM)**

Distal Pressure (ft)	Perforation Diameter (in) 1/8	5/32	3/16	1/4	5/16	3/8
	GPM					
1.0	0.18	0.29	0.41	0.74	1.15	1.66
1.5	0.23	0.35	0.50	0.90	1.41	2.03
2.0	0.26	0.41	0.58	1.04	1.63	2.34
2.5	0.29	0.45	0.66	1.17	1.82	2.62
3.0	0.32	0.50	0.72	1.28	1.99	2.87
3.5	0.34	0.54	0.78	1.38	2.15	3.10
4.0	0.37	0.57	0.83	1.47	2.30	3.32
4.5	0.39	0.61	0.88	1.56	2.44	3.52
5.0	0.41	0.64	0.93	1.65	2.57	3.71

Values were calculated as: gpm = $(11.79 \times d^2 \times \sqrt{h})$
Where: d = orifice dia. in inches and h = head feet.

Table A-2 Maximum Manifold Length (ft) For Various Manifold Diameters Given the Lateral Discharge Rate and Lateral Spacing (from: Otis, 1981)

Lateral Discharge Rate	Manifold Diameter = 1¼”					Manifold Diameter = 1½”					Manifold Diameter = 2”					Manifold Diameter = 3”					Manifold Diameter = 4”					Manifold Diameter = 5”				
End Manifold / Center Manifold	Lateral Spacing (ft) 2	4	6	8	10	Lateral Spacing (ft) 2	4	6	8	10	Lateral Spacing (ft) 2	4	6	8	10	Lateral Spacing (ft) 2	4	6	8	10	Lateral Spacing (ft) 2	4	6	8	10	Lateral Spacing (ft) 2	4	6	8	10
10 / 5	4	8	6	8	10	10	8	12	16	20	12	16	24	24	30	26	40	48	56	70	42	64	84	96	110	84	134	174	200	240
20 / 10	4	4	6			4	4	6	8	10	6	8	12	16	20	16	24	30	32	40	26	40	54	64	70	54	84	106	128	150
30 / 15	2					2	4	6			4	8	6	8	10	12	16	24	24	30	20	26	36	48	60	42	64	84	96	110
40 / 20											4	4	6	8	10	10	12	18	16	20	16	24	30	32	40	34	52	66	80	90
50 / 25											2	4	6	8		8	12	12	16	20	14	20	24	32	40	30	44	60	72	80
60 / 30											2	4				8	12	18	16	20	12	16	24	24	30	26	40	48	64	70
70 / 35											2					6	8	12	8	10	10	16	18	24	30	24	36	48	56	60
80 / 40											2					6	8	6	8	10	10	12	18	16	20	22	32	42	46	60
90 / 45											2					4	8	6	8	10	8	12	18	16	20	20	28	42	46	50
100 / 50																4	4	6	8	10	8	12	12	16	20	18	28	36	40	50
110 / 55																4	4	6	8	10	8	12	12	16	20	16	24	36	40	40
120 / 60																4	4	6	8	10	6	8	12	16	10	16	24	30	32	40
130 / 65																4	4	6	8	10	6	8	12	16	10	14	24	30	32	40
140 / 70																2	4	6	8		6	8	12	8	10	14	20	24	32	40
150 / 75																2	4	6			6	8	12	8	10	14	20	24	32	30
160 / 80																2	4	6			6	8	6	8	10	12	20	24	32	30
170 / 85																2	4	6			4	8	6	8	10	12	20	24	24	30
180 / 90																2	4				4	8	6	8	10	12	16	24	24	30
190 / 95																2	4				4	8	6	8	10	12	16	18	24	30
200 / 100																2	4				4	4	6	8	10	10	16	18	24	30

Table A-3 Friction Loss in Schedule 40 Plastic Pipe
(ft/100 ft), Based on Hazan-Williams; C = 150

Pipe Diameter (Inches)

Flow (GPM)	1	1 1/4	1 1/2	2	3	4	6	8	10
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		**Velocities in this area are below 2 ft/sec.**			
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					
13		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					1.06	0.28			
80					1.38	0.37			
90					1.73	0.46			
100					2.09	0.55	0.07		
150						1.17	0.16		
200							0.28	0.07	
250							0.41	0.11	
300							0.58	0.16	
350							0.78	0.20	0.07
400							0.99	0.26	0.09
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

Table A-4 Friction losses through plastic fittings in terms of equivalent lengths of pipe
(Sump and Sewage Pump Manufacturers, 1998)

	--------------Nominal size fitting and pipe ------------------					
Type of Fitting	1¼	1½	2	2½	3	4
90° Elbow	7.0	8.0	9.0	10.0	12.0	14.0
45° Elbow	3.0	3.0	4.0	4.0	6.0	8.0
STD. Tee (Diversion)	7.0	9.0	11.0	14.0	17.0	22.0
Check Valve	11.0	13.0	17.0	21.0	26.0	33.0
Coupling/ Quick Disconnect	1.0	1.0	2.0	3.0	4.0	5.0
Gate Valve	0.9	1.1	1.4	1.7	2.0	2.3

Table A-5 **Void volume for various diameter pipes.**

Nominal Pipe Size (In.)	Void Volume (gal./ft)
3/4	0.023
1	0.041
1¼	0.064
1½	0.092
2	0.163
3	0.367
4	0.650
6	1.469

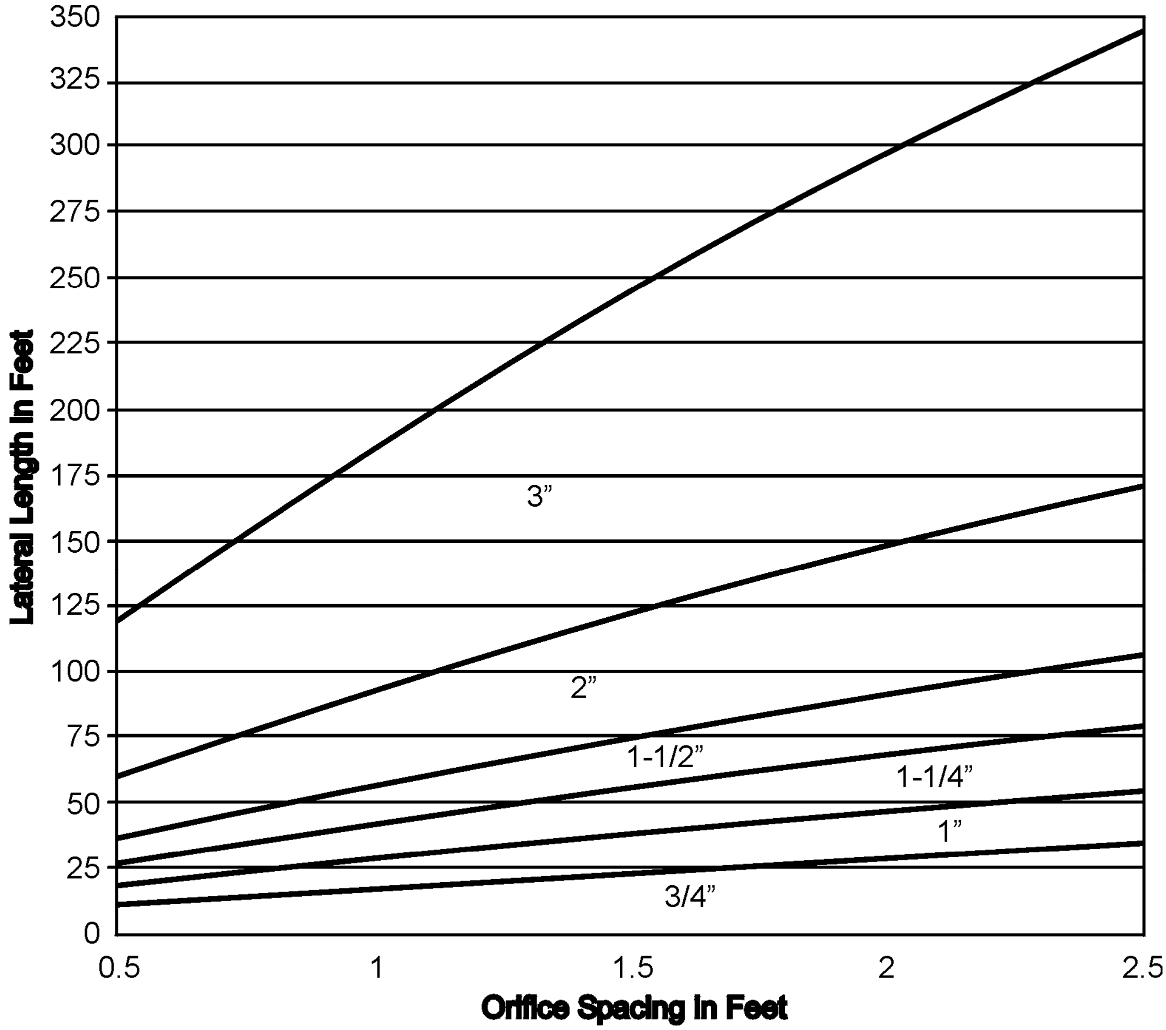

Fig. A-1a. Minimum lateral diameter based on orifice spacing for 1/8 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

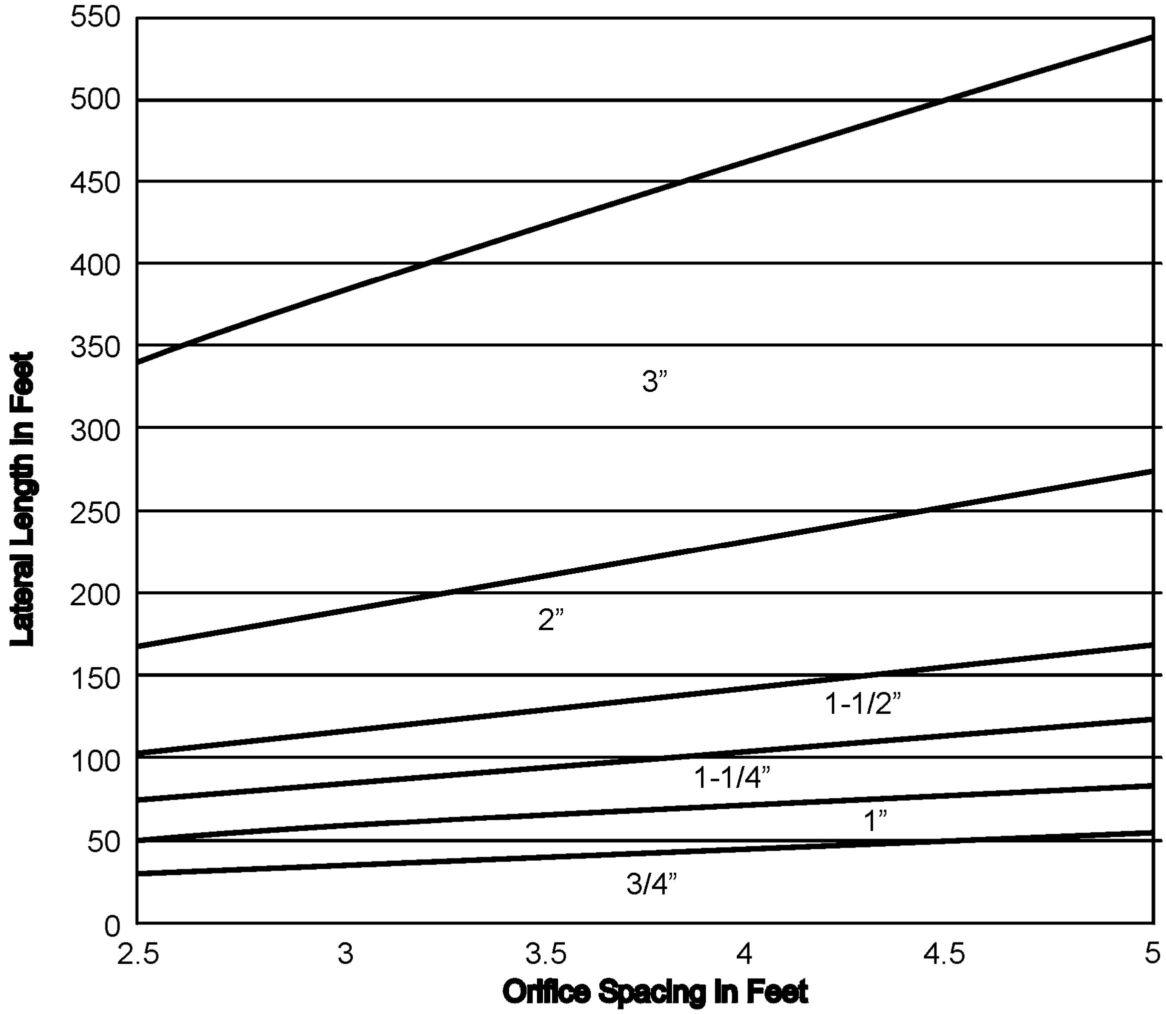

Fig. A-1b. Minimum lateral diameter based on orifice spacing for 1/8 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

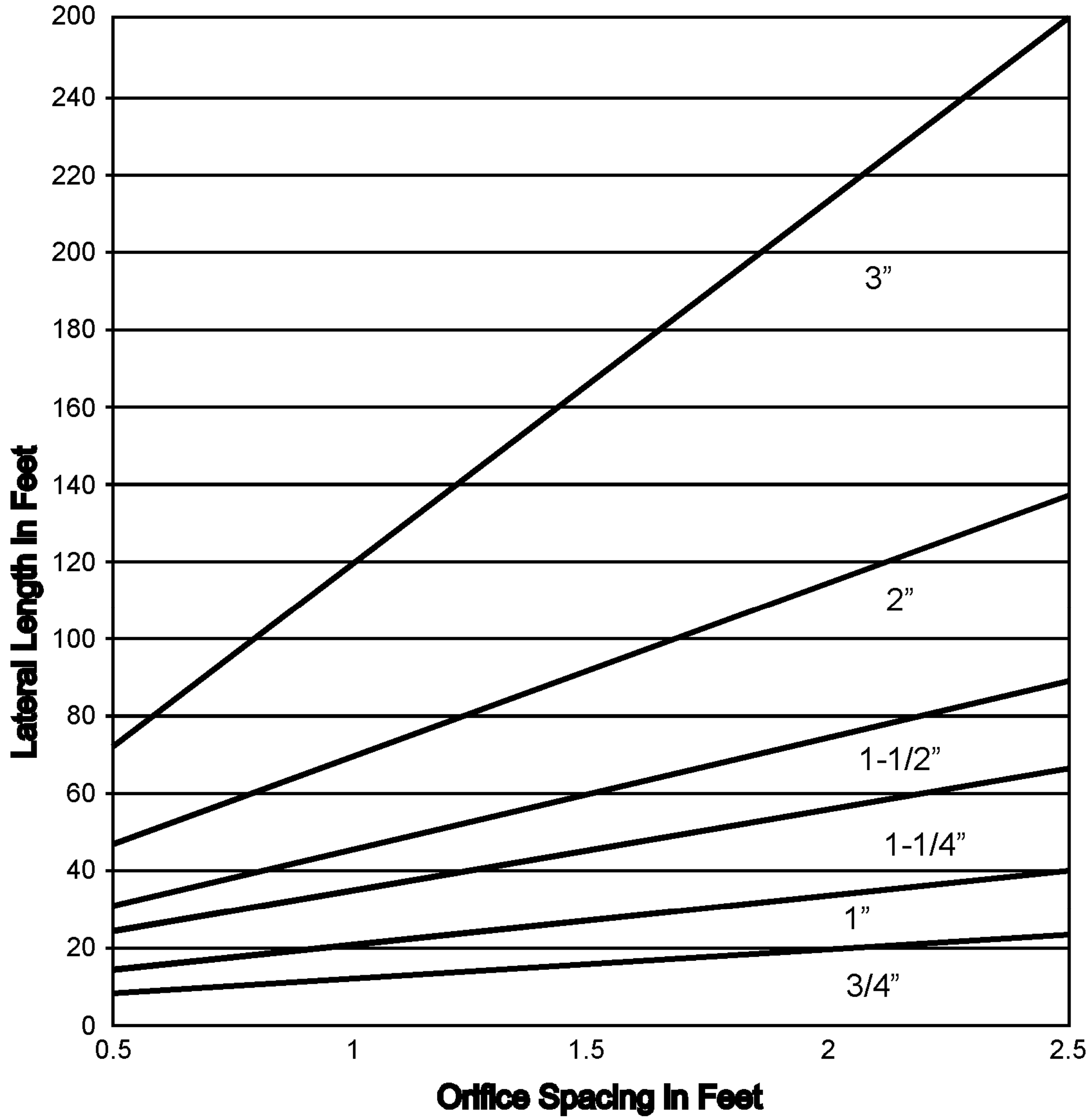

Fig. A-2a. Minimum lateral diameter based on orifice spacing for 5/32 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

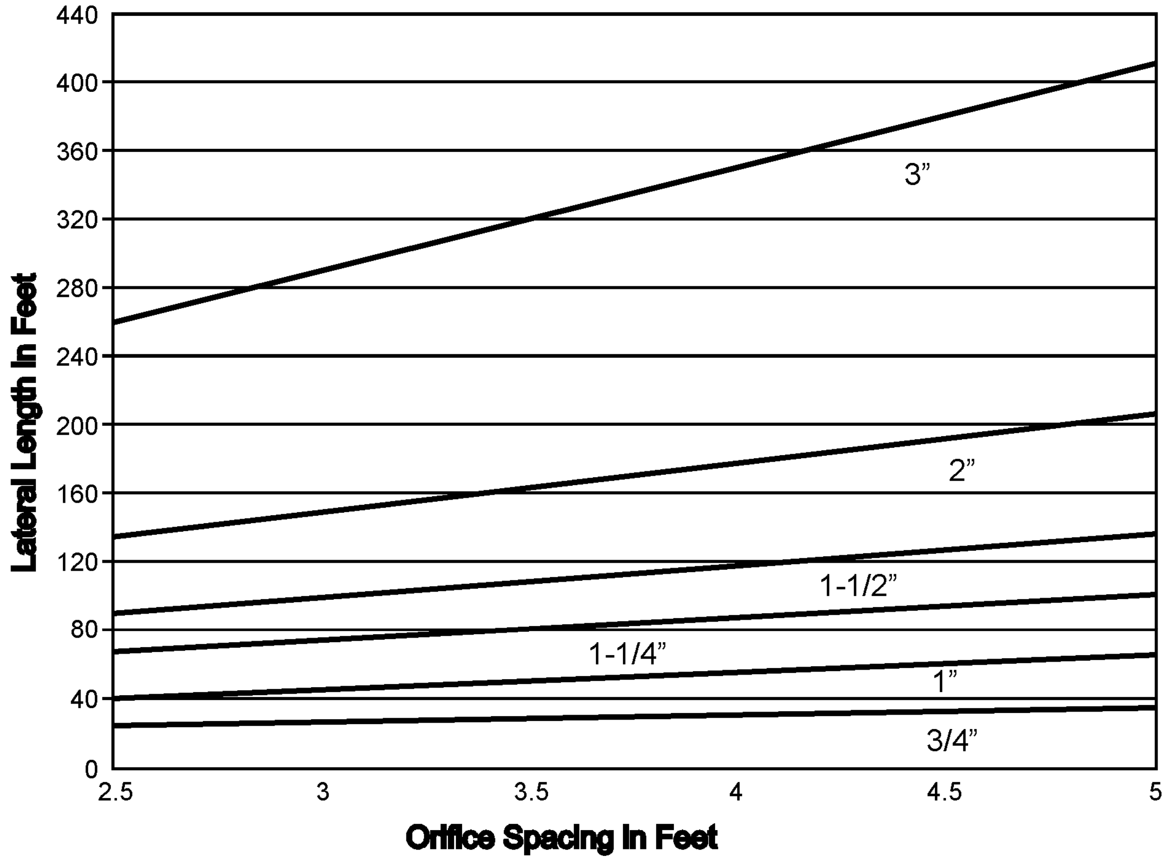

Fig. A-2b. Minimum lateral diameter based on orifice spacing for 5/32 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

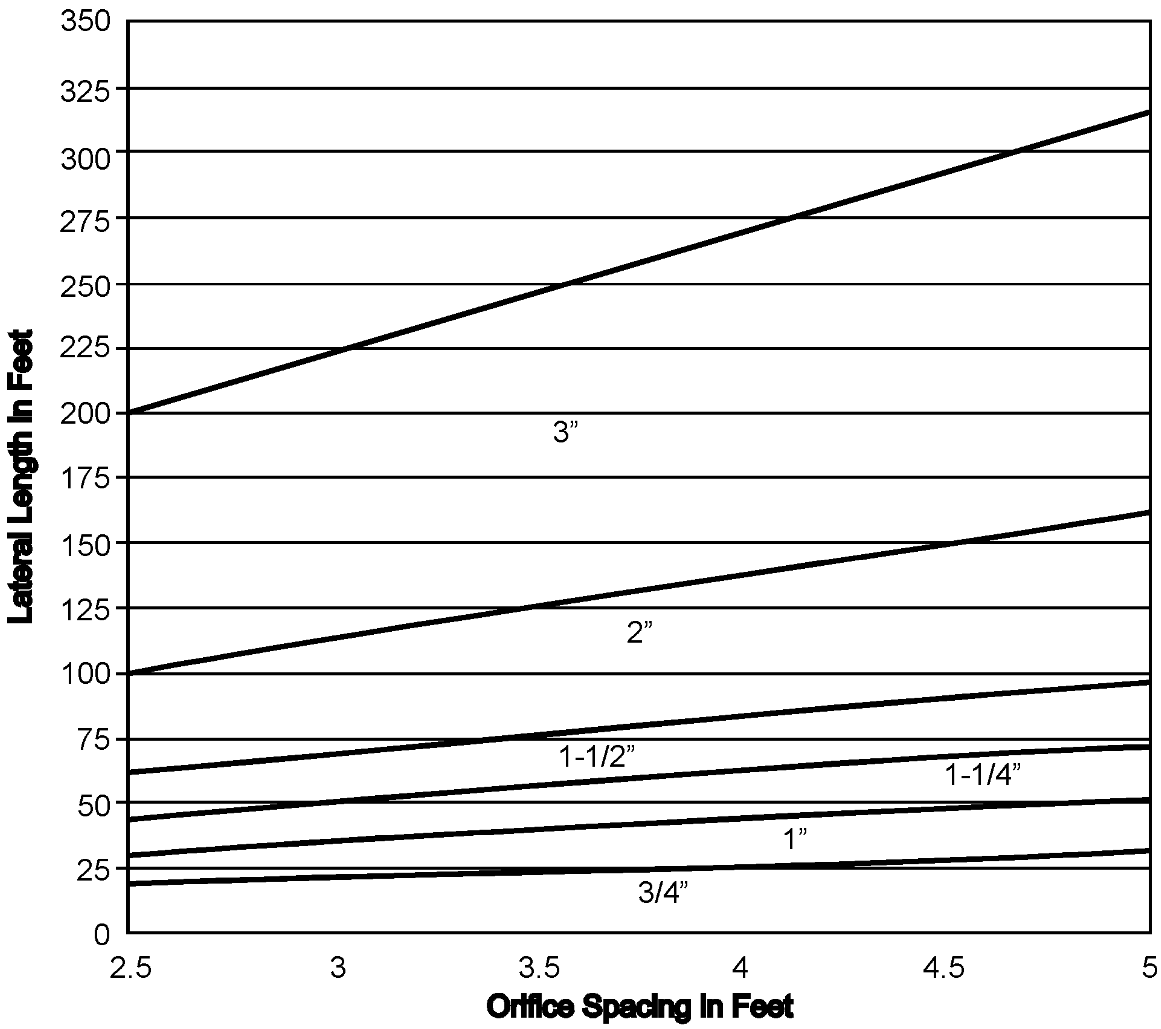

Fig. A-3a. Minimum lateral diameter based on orifice spacing for 3/16 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

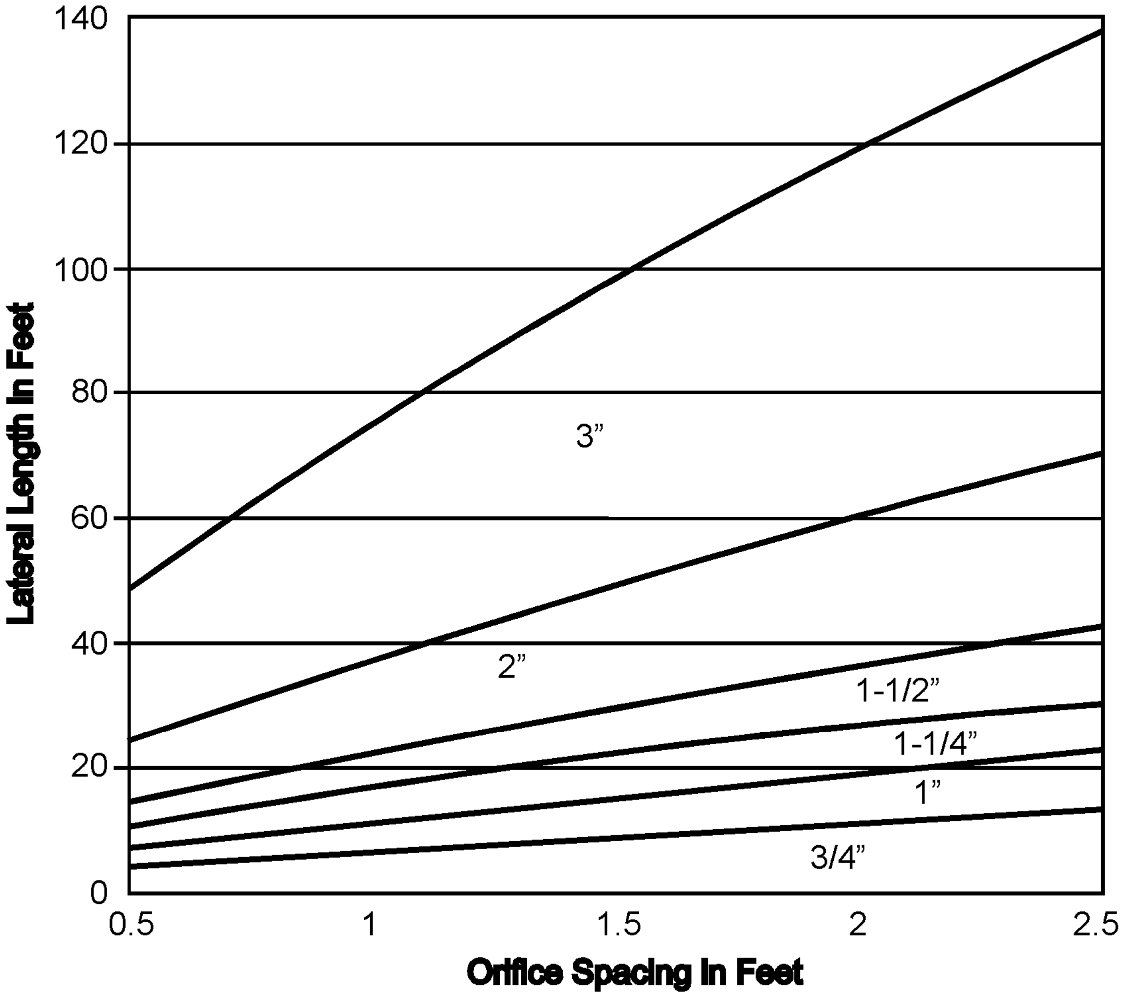

Fig. A-3b. Minimum lateral diameter based on orifice spacing for 3/16 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

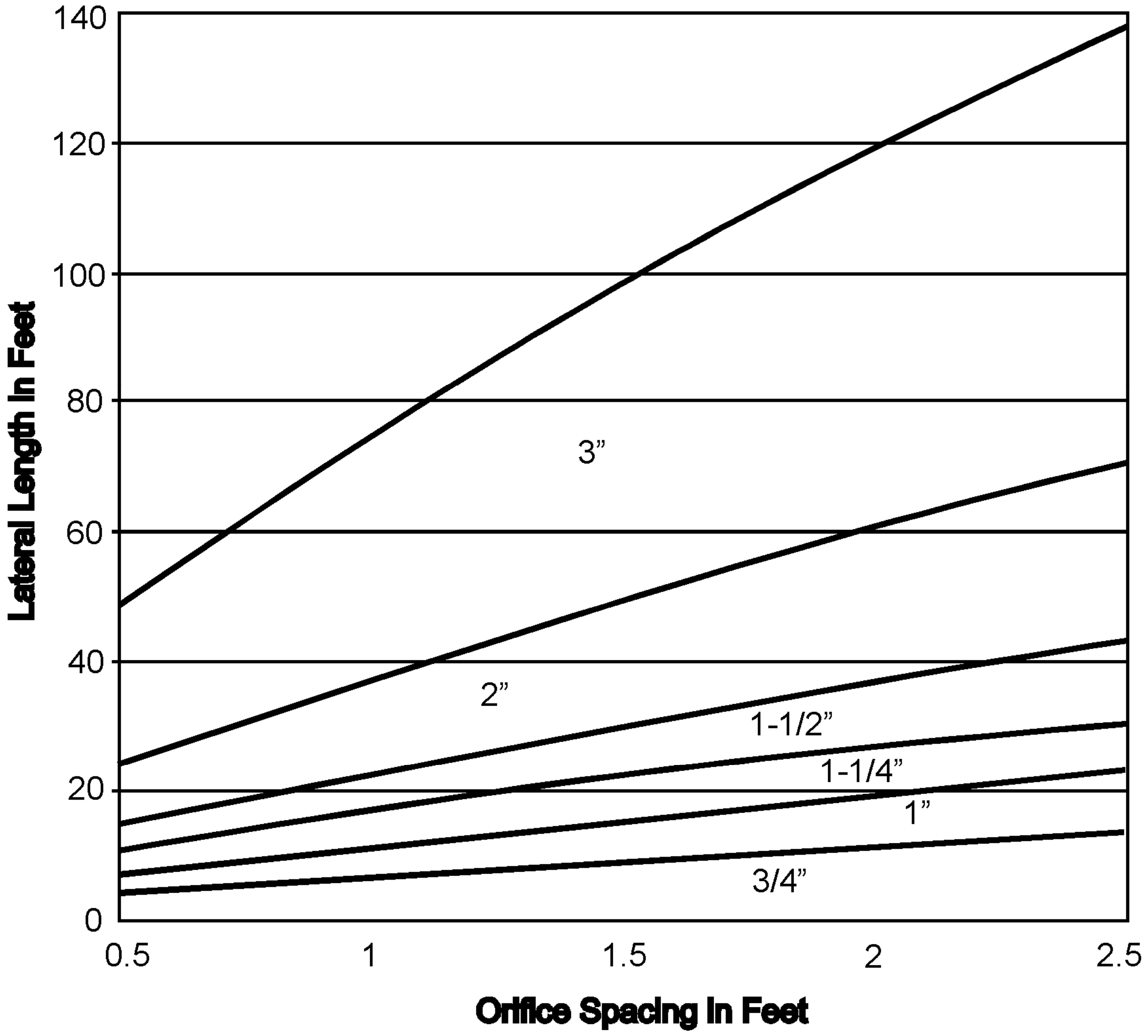

Fig. A-4a. Minimum lateral diameter based on orifice spacing for 1/4 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

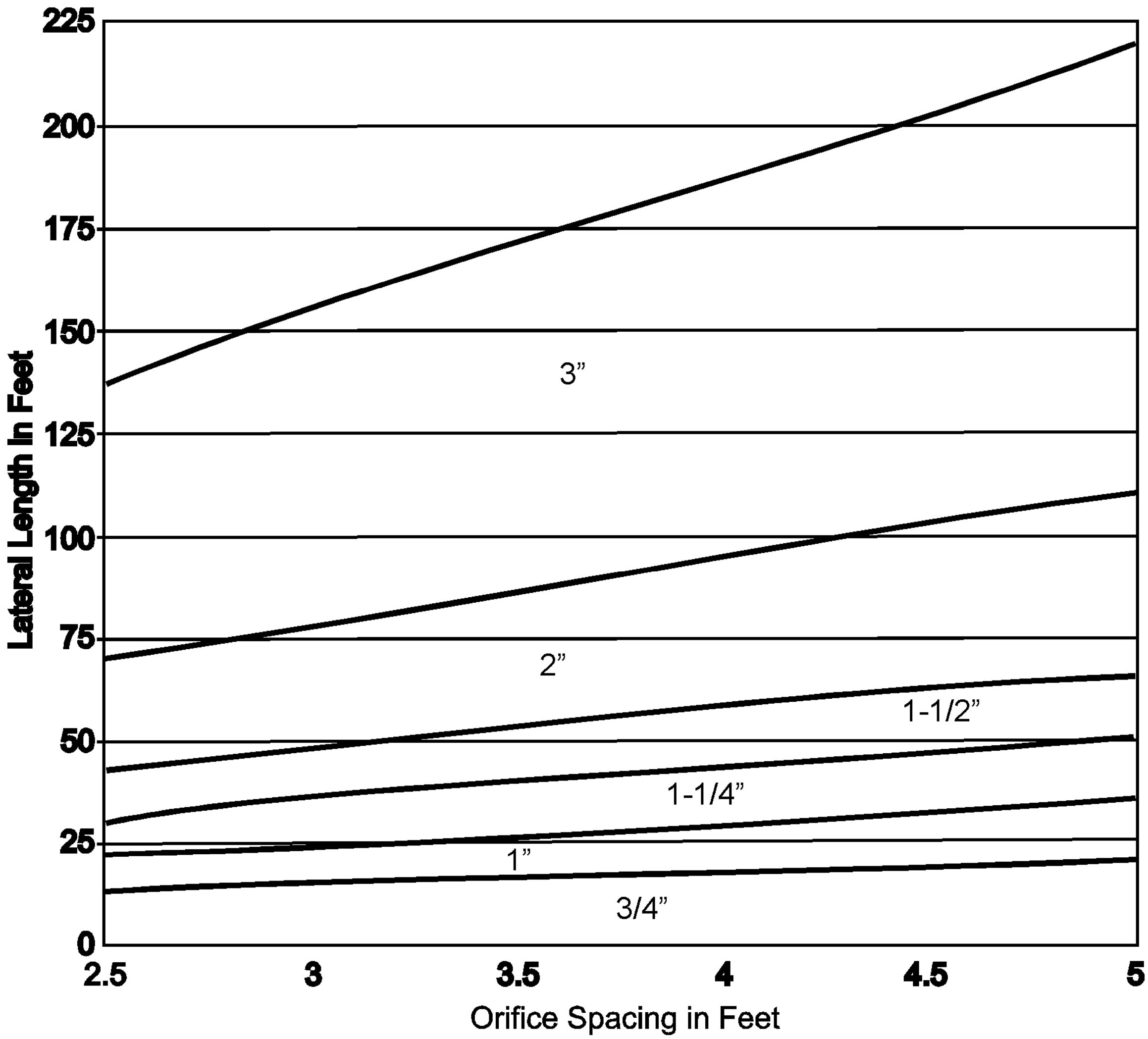

Fig. A-4b. Minimum lateral diameter based on orifice spacing for 1/4 in. diameter orifices (Wisc. Dept. Of Commerce, 1999)

References:

Converse, J.C. 1974 Distribution of domestic waste effluent in soil absorption beds. Trans. Of the ASAE. 17:299-304.

Converse, J. C. 1999. Septic tanks and pump chambers with emphasis on filters, risers, pumps surge capacity and time dosing. Small Scale Waste Management Project. 345 King Hall, University of Wisconsin-Madison, 1525 Linden Drive, Madison, WI 53706.

Converse, J.C. and E.J. Tyler. 2000. Wisconsin Mound Soil Absorption Systems, Siting, Design and Construction. Small Scale Waste Management Project. 345 King Hall, University of Wisconsin-Madison, 1525 Linden Drive, Madison, WI 53706. Publication No. 15.22.

Falkowski, G.M. and J.C. Converse. 1998. Siphon performance and pressure distribution for on-site systems. In. On-site Wastewater Treatment. Proceedings of the Fifth National Symposium on Individual and Small Community Sewage Systems. ASAE. Joseph, MI 49085.

Machmeier, R.E. and J.L Anderson. 1988. Flow distribution by gravity flow in perforated pipe. In. On-site Wastewater Treatment. Proceedings of the Fifth National Symposium on Individual and Small Community Sewage Systems. ASE. Joseph, MI 49085.

Otis, R.J. 1981. Design of pressure distribution networks for septic tank- soil absorption systems. Small Scale Waste Management Project. 345 King Hall, University of Wisconsin- Madison, 1525 Linden Drive, Madison, WI 53706. Publication No. 9.6.

Sump and Sewage Pump Manufacturers Association. 1998. Recommended guidelines for sizing effluent pumps. P.O. Box 647 Northbrook, Il 600065-0647.

Wisconsin Administrative Code. 1999. Pressure distribution component manual for private onsite wastewater treatment systems. Department of Commerce, Safety and Building Division, Madison, WI.