University Curriculum Development for Decentralized Wastewater Management

Effluent Conveyance

Module Text

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September 2004

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Citation of Materials

Trotta, P.D., and J.O. Ramsey. 2005. Effluent Conveyance - Text. *in* (M.A. Gross and N.E. Deal, eds.) University Curriculum Development for Decentralized Wastewater Management. National Decentralized Water Resources Capacity Development Project. University of Arkansas, Fayetteville, AR.

Effluent Conveyance in Onsite and Decentralized

Overview

Background - Conventional Sewer Systems

Traditionally, the evolution of a growing city's infrastructure included a multi branched, large, and often deep, gravity flow sewer system that takes untreated sewage to the municipal sewage treatment plant. Such systems, which generally were designed to serve the entire urban area, took advantage of a continuous downhill path to the lowest point in the city. This was often near a river or stream as it left the city. Post World War II neighborhoods were often built faster than the sewer system could reach them. Onsite systems (septic tanks or cesspools) were temporary solutions but were considered nuisances and property values in the neighborhoods inevitably rose when the sewer did in fact make it down the avenue to service the neighborhood.

The hydraulic design of large gravity flow sewers is a well developed engineering discipline and no attempt will be made to duplicate the systematic design procedures from individual homes, through collectors, interceptors and trunk lines down to the sewage treatment plant. A basic introduction to sewer hydraulics is presented in the Gravity Flow Dispersal Section of the curriculum. There are, however, many design considerations, which can be easily adapted from these classical municipal engineering procedures for the growing onsite and decentralized alternatives. For a detailed design procedure for a gravity flow collection system the reader should consult any one of many available hydraulic design textbooks orientated towards municipal engineering.

The function of the typical municipal sewage collection system is to transport sewage from its point of origin to where it will be treated and/or reintroduced back into the hydrologic cycle. Gravity flow systems are favored but lift stations and pressure (or vacuum) mains are sometimes needed where the topography is not suitable for a continuous downhill path.

All aspects of Onsite and Decentralized wastewater treatment and dispersal involve the movement of effluents of varying qualities. The movements of wastewaters, effluents and reclaimed waters include transfers from:

- Individual homes and other waste generating facilities to community collection systems.
- Individual homes and other waste generating facilities to onsite treatment facilities.
- Onsite treatment facilities to onsite disposal facilities.
- Community collection systems to community treatment facilities.
- Community treatment facilities to community dispersal facilities.
- Transfers between components of treatment facilities and/or dispersal.

Design Constraints of Large Scale Systems

Large-scale municipal gravity flow sewer systems are generally designed with the following constraints in mind:

- Flow velocities should be within given limits (generally greater than 2.5 ft/sec) to prevent grease and solids deposition at low velocities or pipe scour at high velocities.

- Average, low, or high flows should be within given percent of total depth limits to accommodate growth, infiltration and inflow, or velocity requirements.

- Inverted siphons or short sections with positive slopes that result in pressure flow are generally not permitted under any circumstance.

- Minimum sewer diameters (generally 6 inches or 8 inches) are often specified to facilitate system service.

- Minimum and maximum sewer depths are often specified to address, mechanical protection of the pipe, safety and practicality issues. (Excessive depths ranging up to 30 ft are not unheard of in large, flat urban areas where excavation is not hampered by rock).

- Access facilities (man-holes) are often specified at minimum intervals, changes in slope, direction, pipe size, or at vertical hydraulic drops used as energy dissipaters.

- Minimum horizontal and vertical separations to other utilities are critical due to the relatively inflexible horizontal and vertical alignments of the large sewers and their tendency to leak (infiltration and exfiltration).

Decentralized Sewer Systems

Decentralized and individual systems have the same transport requirements as large municipal systems, although some of the basic assumptions that influence the designs may be different. The terminology that is used in the onsite and decentralized literature, as in most fields, is not totally consistent and the same terms are sometimes used to describe somewhat varying applications. In this module we will emphasize the distinctions between pump powered systems and gravity-powered systems. The onsite counterparts or alternatives to municipal sewer system include:

Septic Tank Effluent Pump

Septic Tank Effluent Pump (STEP) systems which deliver the partially clarified septic tank effluent to a relatively small, often shallow pressure line which carries the effluent to a local site for additional treatment and/or dispersal. The Sump & Sewage Pump Manufacturers Association includes systems that pump effluent to distribution boxes or manifolds for subsequent gravity flow to an absorption field or sewer line as STEP systems as well. (SSPMA, 1998)

Enhanced-Flow STEP systems

This term tends to be used if the STEP system delivers effluents in predetermined volume increments to a distribution box or manifold for gravity flow to an absorption field. The enhancement is the use of predetermined volumes and possibly dosing intervals that may provide better subsequent treatment and dispersal when compared to systems which do not control the volume and/or timing

Low Pressure Pipe/Low Pressure Distribution

Low Pressure Pipe (LPP) and Low Pressure Distribution (LPD) systems are small-scale variants of STEP systems that are typically used to carry effluent from an onsite treatment system to an onsite dispersal system such as a mound or trench system located at a higher elevation or in undulating terrain. LPP/LPD systems pressure dose the dispersal lateral directly. (Recently many designers have found it prudent to use high head pumps to accomplish the same objective and drip irrigation tubing is becoming popular for effluent dispersal.)

Septic Tank Effluent Gravity

Septic Tank Effluent Gravity (STEG), or Variable Grade Sewer (VGS) systems which pre-treat residential wastewater in a septic tank for discharge into typically small diameter, gravity flow sewers which may be designed to allow for some undulation (areas with positive slope) of the downgrade path. (Thus the alternate name VGS).

Grinder Pump

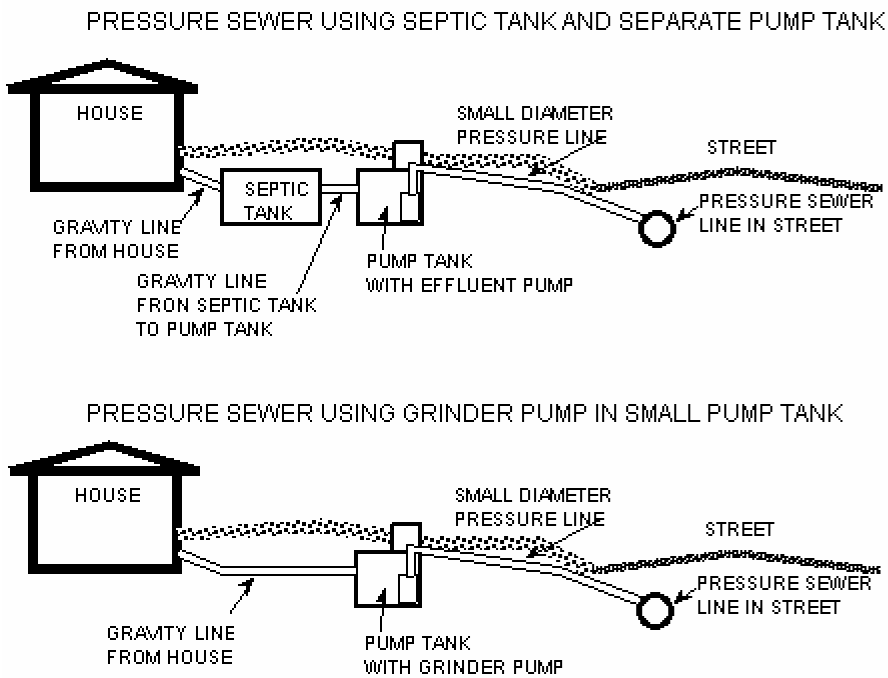
Grinder Pump (GP) systems which grind up residential sewage and inject it into a pressure line which like the STEP system carries the effluent to a local site for additional treatment and dispersal.

Vacuum Systems

Vacuum Systems use a centrally located vacuum source to lower the internal pressure of a collection system substantially below atmospheric pressure. Atmospheric pressure becomes the driving force to drive the sewage down the pipe toward the discharge point. The maximum lift that can be developed by such systems is limited by the vapor pressure of the sewage (water mostly) and will generally not exceed 25 feet.

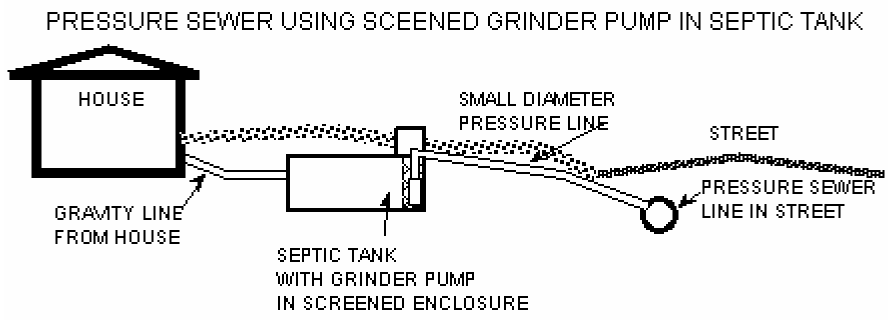
Figure 1 compares the basic features of STEP and GP systems. As can be seen the STEP system illustrated uses a septic tank for pretreatment. The septic tank removes most of the solids enabling the design of a pressure sewer line with less concern about solids, grease, fats or oils degrading the hydraulics of the pressure sewer lines. There is no inherent limitation to the amount of treatment that can be provided prior to it being pumped to a more centralized location for further treatment and/or dispersal. Communities have been developed with STEP like systems using individual aerobic treatment systems discharging to a community STEP system for delivery of the treated effluent to a community scaled constructed wetland or other treatment or dispersal component. Grinder pumps enable the design of GP systems with less concern about large solids but the blended effluent may be more difficult to process when it reaches the treatment facility.

Figure 1 Basic Features of STEP and GP Systems.



Recent innovations in both septic tank screens, pump vaults and reliable high head pumps have enabled a more economical design for STEP system making use of the septic tank itself as the pump chamber. Figure 2 below, illustrates a residential installation of a STEP system using a screened pump vault placed inside a single compartment septic tank.

Figure 2 Single Tank STEP System



Design Constraints for Decentralized Approaches

The alternatives to deep, conventional sewer systems used in onsite or decentralized systems may have many of the same type of constrains placed upon there design but the constraints will often be less restrictive.

- Smaller diameter pipes (in the range of 2 inches) are less subject to crushing from loads placed on the soils above and can therefore be buried at smaller depths in the range of 2 to 3 feet subject to local requirements and freezing considerations.

- Changes in vertical and horizontal alignments during design and construction present fewer redesign problems.

- With STEP, GP, STEG, and LPP systems the fluid being moved is more homogeneous than “raw” sewage and the problems of solids deposition is greatly reduced. This can reduce the need for requiring minimum velocities at all times in favor of occasional cleansing velocities, as well as reducing the concerns about solids plugging up the low points of inverted siphons or sections with positive slopes.

- Smaller diameter pipes are easier to handle during construction and the jointing opportunities are somewhat greater. Solvent welds of common PVC or ABS pipe may be acceptable.

- Appurtenances such as access facilities, clean outs, air relief valves and isolation valves can be placed in small shallow valve boxes rather than in large pre-built or site built access chambers.

- The incremental and intermittent nature of the individual discharges to the common interceptor line coupled with the inherent variation of home discharges results in a design flow value which can be somewhat less than that typically required for uncontrolled, passive, gravity flow municipal sewers.

Economic (and other) Considerations in Choosing Systems

Overall system cost will dictate which type of collection system is most appropriate for a given situation. Hybrid systems which may have elements of both STEG and STEP systems can take advantage of topography when available for gravity flow but can also make use of distributed lift stations and force mains when adverse ground slopes are encountered. An additional consideration that can influence the choice of a system is the pace of development expected in the community. Proving the adequacy of a hydraulic design both during the initial phases of development when the community may be sparsely settled and when it is fully constructed can be a challenge for the designer of the conventional gravity flow sewer who must address minimum and maximum design velocities under both conditions. The removal of most of the solids prior to discharge to a STEP or STEG system reduces the concern about minimum and maximum velocities and thus makes these systems more amenable to slowly growing communities. The significant reduction in upfront costs for the developer who uses a STEP or STEG approach and the incremental installation of the individual septic tanks, pump basins, pumps and controls may allow a more economical project overall. This approach may seem to run counter to the concept of “economies of scale” which would encourage a big, comprehensive system being built at the onset. But, the possibility of incremental development, even if some of the equipment is duplicated at every site, can delay a significant portion of the system costs to the point in time when the capacity is actually needed.

Pressure Flow vs. Gravity Flow

All the transfers mentioned in the previous section involve the movement of effluents either by:

- Pressure Flow
- Gravity Flow

Pressure Flow hydraulics most often makes use of a pump to provide the energy necessary to overcome friction, provide velocity, and/or change elevation. Gravity Flow hydraulics always make use of gravity as the source of the force necessary to overcome friction and provide velocity. There are, however, situations in which the distinction can be blurred. For example water may flow in a pipe under pressure due to its connection to a storage facility elevated above the pipe’s elevation. And, conversely, a pump can discharge to a large diameter pipe, channel or tank where subsequent transfers are a result of gravity flow.

An alternative approach that could be used to distinguish the types of fluid transfers which are important to an onsite or decentralized system designer is the distinction between flows contained within a vessel where there is no free water surface at atmospheric pressure versus flow in which a free water surface at atmospheric pressure exists. This concept may be somewhat less intuitive so we will continue with the distinction of pressure flow versus gravity flow based upon the use of pumps.

A second distinction which could be used when considering onsite or decentralized effluent conveyance is the distinction between the collection of wastewater prior to treatment and the distribution of effluent subsequent to treatment. This distinction is also encumbered by exceptions. Consider a system which pumps partially treated effluent from an individual onsite system into a common community pressure collection system. Such a system is partially collection and partially distribution. (Such systems are commonly called STEP systems, i.e., Septic Tank Effluent Pumping, even though the treatment might be more elaborate than septic tank treatment.

For the purposes of this module we will stick with the titles of Pressure Distribution versus Gravity Distribution although the discussions will range beyond the strict limitations of these words.

Wastewater Design Flows

One of the most crucial elements in the design of any hydraulic system is the design flow. Once carefully established, the design of the hydraulic elements can proceed rationally. Onsite systems are subject to greater uncertainty in the development of their design flow than are larger municipal systems that have larger populations that tend to smooth out local variations and also have longer histories of flow data gathering for analysis. A large amount of effort has been expended over the years to develop rational design flows for both large-scale gravity flow systems and smaller decentralized STEP systems. The conclusions from these different studies vary somewhat but there is general agreement that the design flow STEP systems should be at least 4 times the average daily flow rate (often taken to be 150 gpd/bedroom or 0.1 gpm/bedroom) with even greater peaking factors for very small systems. This is in contrast to the traditional approach for large gravity based community systems where peaking factors of 4 are assigned for very small systems with peaking factors descending to approximately 2 for larger systems.

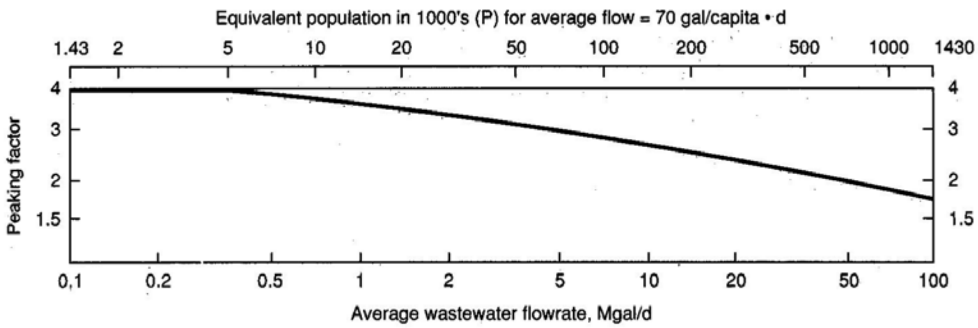
Community Sewer System Design Flows and Peaking Factors

The design of all wastewater collection systems, whether they are centralized, decentralized or individual onsite systems, are affected by the daily variation of the wastewater or treated effluent that they are designed to carry. The literature developed for centralized sewer systems contains many approaches for determining reasonable design flows considering the variability of flows. Most approaches to computing appropriate wastewater collection system design flows make use of *Peaking Factors* or *Peak to Average Ratios* to enable the designer to develop a reasonable design flow based upon averaged household or per capita flows.

Municipal design requirements for sewers often stipulate that the peak flow and/or the average flow must be contained within given percentages of the sewer pipe’s capacity. As is expected by simple applications of probability concepts, the ratio of peak flow to average flow increases as the population served gets smaller and the individual high and low flows are not balanced out by other homes.

Peaking Factors are available in many engineering books and are often stipulated by the municipality. Figure 3 (Metcalf and Eddy, 1991) shows peak to average ratio data that has been widely used, and shows the relationship between the hourly peak flows to the long term average flow. Although this graph assumes 70 gpd/person for its development the peaking factor trend remains valid for other flows as well. It can be seen in this graph that for smaller systems a peaking factor of 4 is recommended. This value is to be multiplied by the appropriate average daily flow rate to determine the design flow rate to be used for the system.

Figure 3 Hourly Peaking Factors For Domestic Wastewater Flow rates (Mgal/d x 0.0438 = m^3/s.) (Metcalf and Eddy, 1991)



An alternative analytical relationship which models the above relationship is offered by Cooper Consultants (Thrasher, 1987) and takes the form:

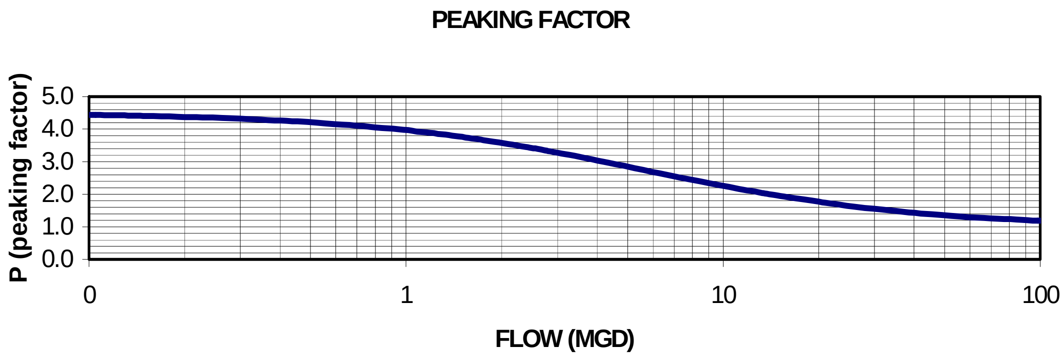
$$P = 1 + 14/\ (4+p^{0.5}) \text{ where:}$$

P = Peaking factor

p = population

This equation results in the peaking factor relationship illustrated in Figure 4. This illustration graphs the relationship between the average flow in MGD and the peaking factor. This relationship is quite close to the Metcalf and Eddy peaking factor values provided above.

Figure 4 Plot of Peaking Factors Developed by Cooper Consultants (Thrasher, 1987)



Design Flows for STEP & STEG Systems

The design flows for STEP & STEG systems are influenced by a variety of factors. These factors can be divided into four sets:

- Factors within the house. Life style variation
- Plumbing features of the house
- Pretreatment systems present
- Characteristics of the pump and pump tank systems used within the STEP system.

STEP and STEG systems are community systems (as are conventional and as such they need to have sufficient capacity for all the homes (and businesses) that will ultimately be connected to them. It may be economically inefficient to design these systems for the combined peak flow of all the homes because the fluctuations of the discharges from the contributing homes are generally out of phase with each other. Work schedules, meal times, bath times, and laundry times for different families occur at different times. This results in the peak discharges from one home typically being at different times than the peak discharges from the others. As the number of homes connected increases averaging affects set in and the ratio of the anticipated peak discharge to the average decreases.

Although the in-house, life style factors and interior plumbing features affect conventional systems as much as onsite or decentralized systems the presence of pretreatment systems and the pumps and tanks of STEP systems are unique to onsite and decentralized systems and can have additional impacts on the rational development of a design flow for the STEP system.

It is important to note that the peaking factors developed for large gravity flow centralized systems are based upon the average flow during the peak hour and will likely be less than the peak flow rate or averaged flow during a peak interval less than one hour. Typical design recommendations for onsite and decentralized systems have adapted data from conventional system design standards that may not address all the factors that affect STEP system design. STEP and STEG systems modify the pattern of delivery of the effluent to the community system. STEP systems, in particular, are susceptible to flow spikes when relatively large discharges are possible from each house for short durations.

Figure 5 shows a multiplicity of houses connected to a single pressure line, a STEP system.

Figure 5 STEP System

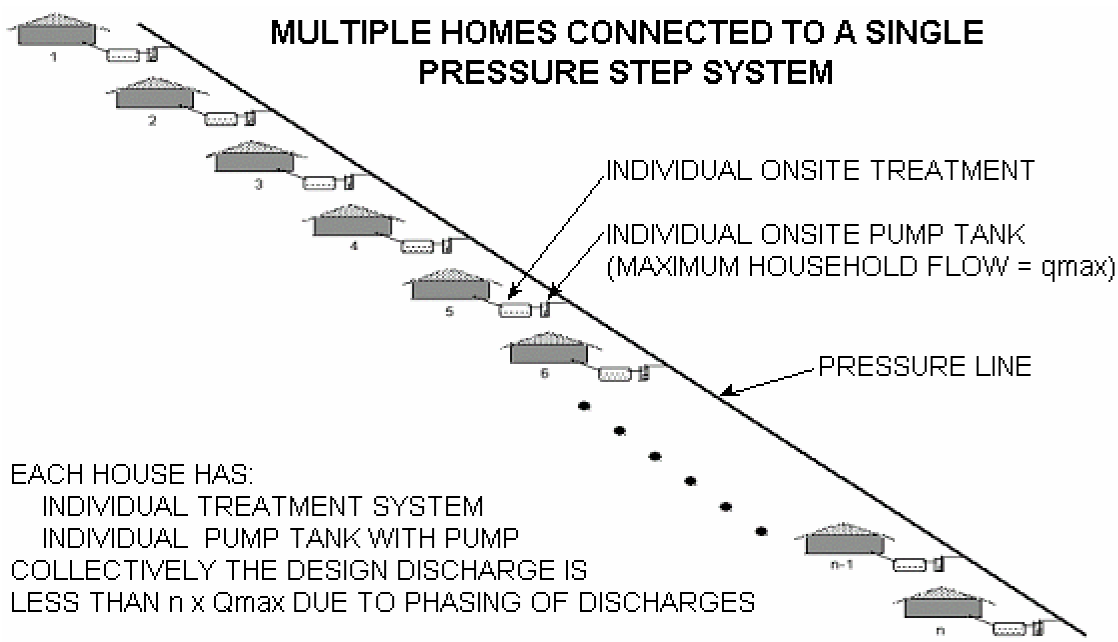
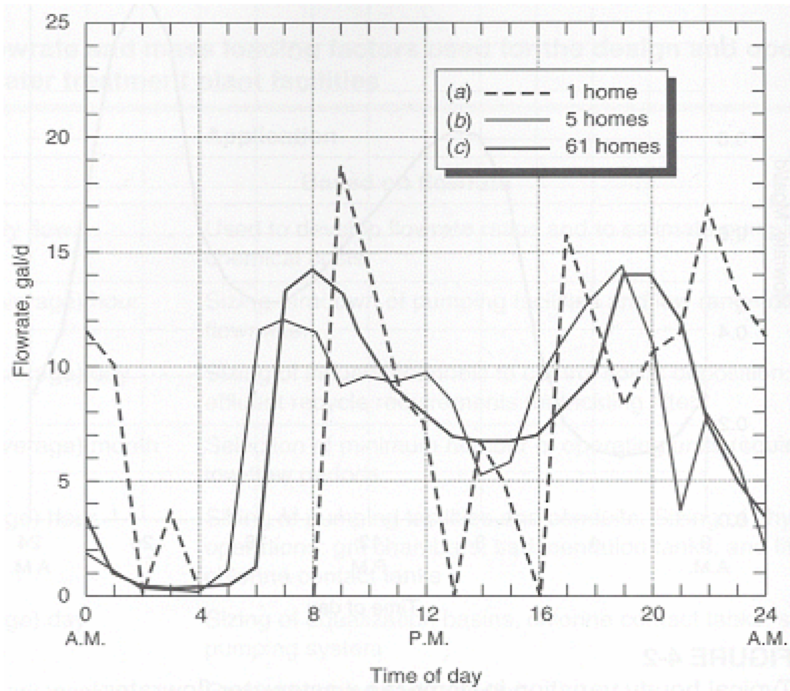


Figure 6 (Crites and Tchobanoglous, 1998) shows a typical diurnal flow pattern for a single residence with gravity flow discharges, the average per house discharge from five homes and the average per house discharge from 61 homes. Note how the instantaneous peaks are reduced as the flows from multiple homes are averaged. (*It is likely that the graphic is mislabeled with gal/d when gal/hour was intended. This, however, does not change the ratios between peaks and averages.*)

Figure 6 Diurnal Flow From 1, 5 and 61 Houses connected to gravity line. (Crites and Tchobanoglous, 1998, from Baker, 1990)



STEP and STEG systems often serve small clusters of homes and the method used by the designer and accepted by the system regulator for addressing these issues should be worked out before a detailed design is initiated. A STEP design that uses approaches based upon hourly data from conventional gravity flow systems might not have the built in capacity to handle the peak flow from all the homes served by a STEP system if, for example, a power interruption resulted in all systems coming on simultaneously when the power is restored.

The peak flows and their duration from a home serviced by a STEP or STEG system is influenced by the usage of the home occupants and a variety of hydraulic factors which change the delivery of effluent to the STEP or STEG system. A flushing toilet (assume 1.5 gallons/flush and a 15 second flush) has a momentary discharge equivalent to 240 gallons per day. A shower, tub, washing machine or dishwasher discharging an estimate of 3 gallons per minute results in 180 gallons per hour. Obviously if a toilet is flushed while the dishwasher is draining while vegetables are being rinsed in the kitchen can produce significant short duration peak flows.

In addition to the attenuation of flows caused by multiple homes, there are two hydraulic features of the home's wastewater system which decrease the peak flow to a collection system (STEP, STEG or conventional):

- Flow characteristics of the gravity lines that carry contaminated water to the treatment system. The homes drain lines will attenuate the instantaneous peak flows due to their lengths, shallow slopes and internal friction.

- Pre-treatment tanks if used will discharge by flow over effluent weirs that will only discharge if the water level in the entire tank increases. Even a relatively small septic tank will reduce the discharge to downstream features of the system considerably.

Flow Modulation from Multiple Contributors to a STEP system

Overview

There are several additional hydraulic features of the pump systems used within STEP systems that can either increase or decrease the peak flows that reach it. The factors to be considered include:

- The discharge characteristics of the pump chosen.
- The tank dimensions.
- The control floats set points.
- The use of timers.

Within practical limits, the flows into the pump basin and the flows out of the pump basin will overtime equalize but the pump, pump basin and set points will have a dramatic effect upon the discharge characteristics from the individual houses into the common line.

Figure 7 illustrates a simulated diurnal flow stream from a home. The graph represents a total daily flow of 500 gallons. The graph shows a series of 15 minute averaged flow rates over a 24-hour day. The values range from zero to a peak 15-minute average flow of 1.4 gallons per minute. The average inflow into the discharge tank over the entire 24-hour period is 0.35 gpm. Superimposed upon this graph are two possible pump discharges and durations. A pump with a discharge averaging 8 gpm cycles on and off between predetermined tank storage limits (100 gallons and 30 gallons) while a pump with a discharge averaging 5 gpm cycles on and off between different predetermined tank storage limits (60 gallons and 30 gallons). At the end of the typical 24-hour period the total flow volumes are all equal but the peak discharges and their durations are significantly different. The 8 gpm pump produces a peak discharge which is almost 25 times the average 0.35 gpm daily average flow while the 5 gpm pump produces a peak discharge almost 15 times the average flow.

It is obvious that the peak discharge rate from an individual house is determined by the pump and can certainly exceed a peaking factor of 4 multiplied by the average daily flow rate of 0.35 gpm (4 * 0.35 = 1.4 gpm)

Figure 7 Illustrative Flows Into and Out of a Pump Tank

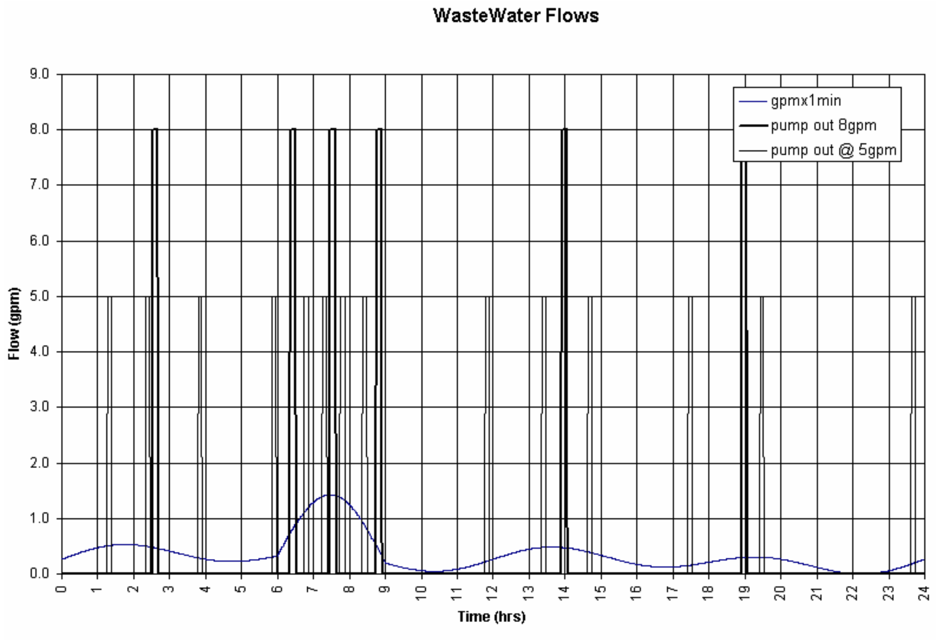
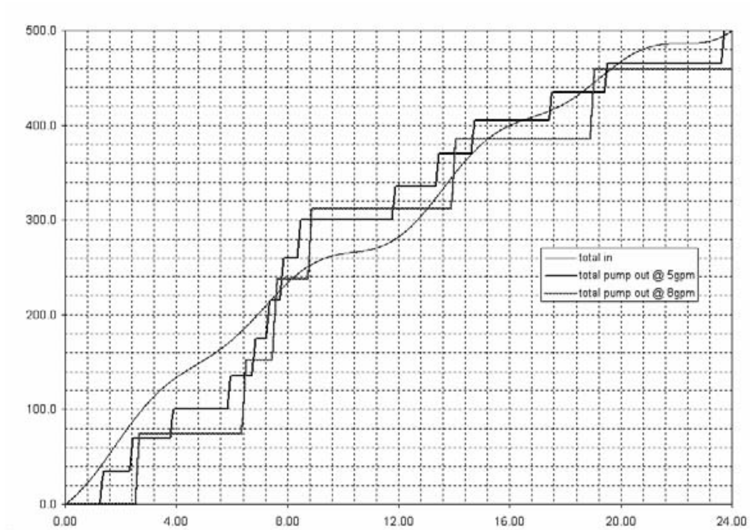


Figure 8 Illustrates the associated total cumulative flows both into (by gravity) and out (via the pump) of the tank.

Figure 8 Cumulative Inflows and Outflow with a Pump Discharge



Design Flow and Peak Flow Estimation for STEP Systems

It should be apparent that joining the discharges from several homes and developing a reasonable design flow presents a challenging problem. The flow rate, and flow duration from any individual pump system can vary considerably but can be predicted within limits. However, the phasing of the discharges from one contributing home relative to another is certainly random.

Various alternative approaches have been used over the years to rationally balance the risk and small probabilities of all the homes discharging at the same time and the costs associated with the increased hydraulic capacity otherwise needed.

In the book *Design and Use of Pressure Sewer Systems* David Thrasher, P.E. (Thrasher, 1987) acknowledges an engineering report by Cooper Consultants, a division of Cooper Communities, Inc. for a community in Arkansas that identified two fundamental approaches to developing a design flow:

Probability Method - Positive Displacement Pumps

The "probability method" is based on the probability of simultaneous pump operation and is recommended for systems with semi-positive displacement pumps. These pumps have very steep operating curves and can deliver essentially the same flow to the common line regardless of the pressure in that line. This simplifies matters considerably since once a reasonable estimate can be made about how many pumps might be on simultaneously, the combined discharge can easily be determined by multiplying the number of houses by the somewhat constant flow from any one of the similar pumps. If centrifugal pumps (all the same or different) are used in which the operating curves are flatter the problem becomes far more complicated since each pump will discharge a different amount depending upon the pressure at the connection point between the pump's discharge line and the common pressure (STEP) line in the street.

EnviroOne a manufacturer of positive displacement effluent pumps conducted an EPA authorized study in Rochester N.Y. (EPA, 1972) and recommend the results of the study in their design manual. (Environment/One, no date). The Environment/One approach shows a significant increase in the peak to average ratio, as the number of dwelling units served gets small. The agreement between this "probabilistic" method and the rational methods described below is striking. In both cases a value of 0.5 gpm/residence per day is recommended for larger communities and a dramatic rise in design flow is suggested for smaller communities. 0.5 gpm corresponds to a daily flow rate of 720 gpd addresses both a peaking factor and the assumed number of people per household. Table 1 below develops the peak ratios and design flow rates per house recommended for the Environment/One system. (Note: The Environment/One system has been referred to as a Low Pressure Sewer System. It could be classified with LPP systems due to the use of low-pressure positive displacement pumps or a GP system due to its use of grinders in the pump assembly. It is included here, however, as a STEP system variant for this discussion of design flows and peaking factors.)

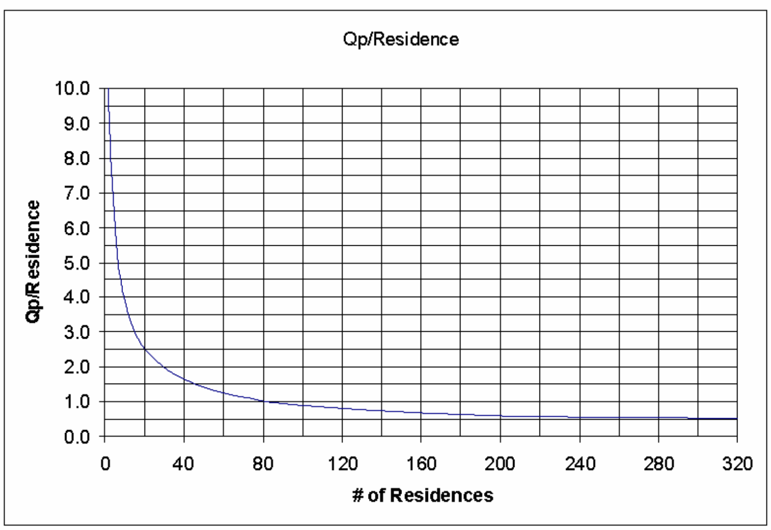
Table 1 Peak Ratios and Design Flows from Envirnment/One Design Guidelines

Number of Grinder Pumps or Houses Connected (*)	Midpoint of Range	Likely Maximum Number Operating Simultaneously (*)	Ratio	(Peak) Design Discharge Using 12 gpm pumps	Peak Flow/House (gpm)
1	1.0	1.00	1.00	12	12.00
2-3	2.5	2.00	0.80	24	9.60
4-9	6.5	3.00	0.46	36	5.54
10-18	14.0	4.00	0.29	48	3.43
19-30	24.5	5.00	0.20	60	2.45
31-50	40.5	6.00	0.15	72	1.78
51-80	65.5	7.00	0.11	84	1.28
81-113	97.0	8.00	0.08	96	0.99
114-146	130.0	9.00	0.07	108	0.83
147-179	163.0	10.00	0.06	120	0.74
180-212	196.0	11.00	0.06	132	0.67
213-245	229.0	12.00	0.05	144	0.63
246-278	262.0	13.00	0.05	156	0.60
279-311	295.0	14.00	0.05	168	0.57
312-344	328.0	15.00	0.05	180	0.55

* Environment/One Published Data

The dramatic increase in flow recommended by this design procedure becomes quite evident when the data is plotted. Figure 9 shows how the design flow per residence rises from about 0.5 gpm for more than 200 houses to in the range of 10 gpm for less than 10 houses.

Figure 9 Environment/One Design Flows/Residence

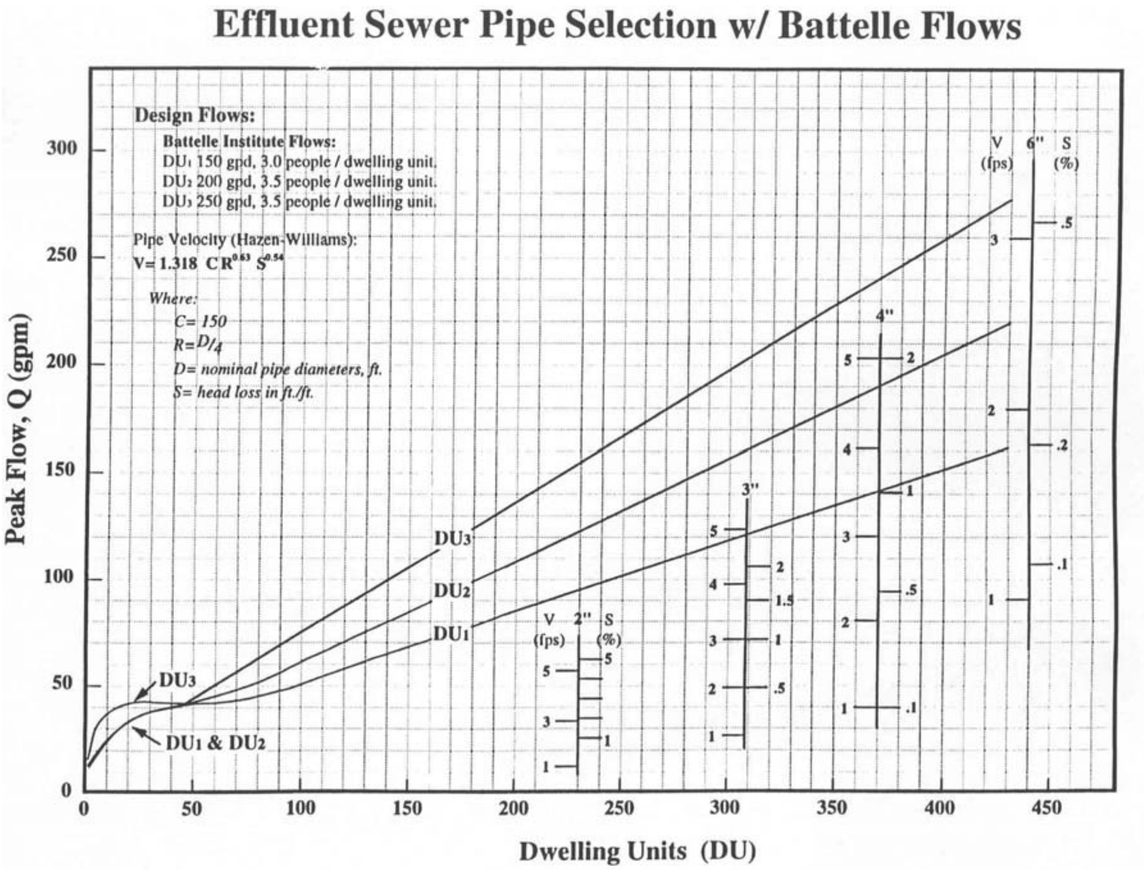


Rational Method – Centrifugal Pumps

The "rational method" acknowledges the possibility of multiple and possibly different centrifugal pumps all discharging to the same common line. Centrifugal pumps have arching operating curves and deliver markedly different flows depending upon the head they must overcome to discharge into the common line.

Some designers make use of linear or semi linear relationship between the number of homes and the recommended peak design flow. A study conducted by the Battelle Institute resulted in recommended design charts for several hypothetical flow assumptions. Figure 10 below, provides design flows recommendations for up to 450 dwelling units, 3 or 3.5 people per household and 150, 200, or 250 gallons per household per day. Considering per capita average flows of 50 gpd and 3 people per dwelling unit, the Battelle recommendation results in peak to average flow ratio between 4 and 5 even for relatively large numbers of dwelling units. This conclusion can be verified by the reader by considering the recommended peak flows that are provided in gpm and multiplying by 60 minutes/hour and 24 hours per day and dividing by the number of dwelling units considered and the number of people assumed per household. For example, 50 gallons per minute is recommended for 100 dwelling units (DU_1 graph). This results in a peak flow per person of 240 gpd when an average flow of 50 gpd per person was initially assumed.

Figure 10 Battelle Recommended Design Flows



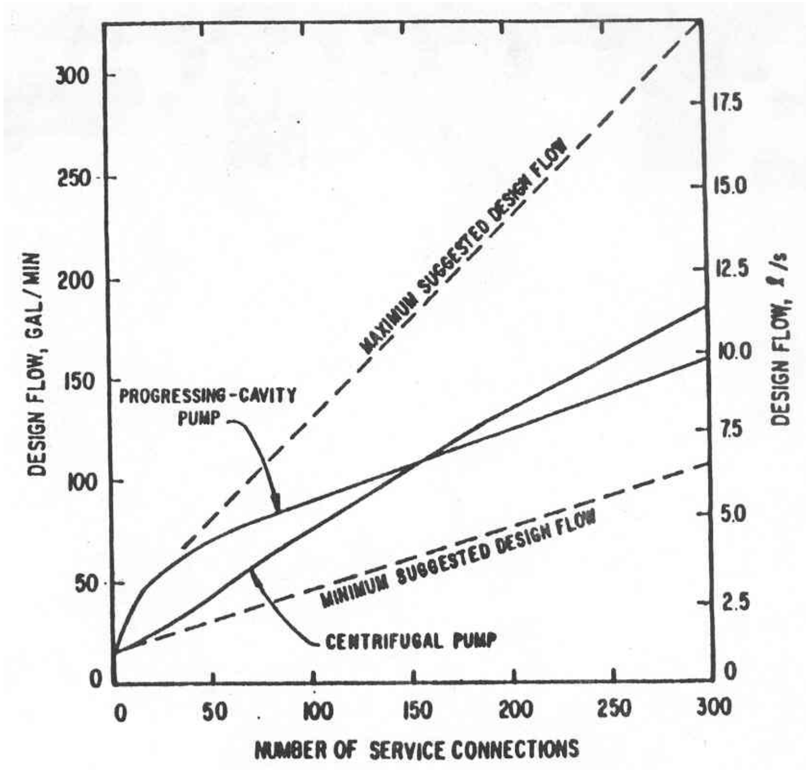
Similarly the design chart for Tellico Village, Tenn. (developed by Cooper Communities) shown below in Table 2 assumes an average flow of 50 gpd and 2.3 people per dwelling. It recommends somewhat lower flows for larger communities. The peak to average ratio used in this study ranges from 4.46 for one connection to 3.36 for large (>199 connections).

Table 2Cooper Communities Design Recommendations For Tellico Village, Tennessee (Thrasher, 1987)

No. of Lots	Avg. Flow (GPD)	Peak to Average Ratio	Peak Flow (GPM)	Suggested Design Flow (GPM)	Suggested Pipe Size (In.)	Headloss (Ft/1000Ft) C = 140
1	98	4.46	0.30	15	1.5	23.3
2	196	4.45	0.61	15	1.5	23.3
3	294	4.43	0.91	15	1.5	23.3
4	392	4.42	1.20	20	2	9.8
5	490	4.42	1.50	20	2	9.8
10	980	4.38	2.98	30	2	20.7
15	1,470	4.36	4.45	30	2	20.7
20	1,960	4.34	5.90	30	2	20.7
25	2,450	4.32	7.34	40	3	4.9
30	2,940	4.30	8.78	40	3	4.9
40	3,920	4.27	11.63	40	3	4.9
50	4,900	4.25	14.45	40	3	4.9
60	5,880	4.22	17.25	40	3	4.9
70	6,860	4.20	20.03	40	3	4.9
80	7,840	4.19	22.79	45	3	6.1
90	8,820	4.17	25.53	45	3	6.1
100	9,800	4.15	28.25	50	3	7.4
125	12,250	4.12	35.01	50	3	7.4
150	14,700	4.08	41.68	55	3	8.8
175	17,150	4.05	48.28	55	3	8.8
200	19,600	4.03	54.81	60	3	13.8
250	24,500	3.98	67.70	70	3	17.7
300	29,400	3.94	80.39	90	4	5.4
400	39,200	3.87	105.25	110	4	7.9
500	49,000	3.81	129.52	130	4	10.7
600	58,800	3.75	153.30	160	4	15.7
700	68,600	3.71	176.65	180	4	19.5
800	78,400	3.67	199.61	200	6	3.3
900	88,200	3.63	222.24	230	6	4.3
1000	98,000	3.59	244.55	240	6	4.6
1100	107,800	3.56	266.59	270	6	5.8
1200	117,600	3.53	288.36	290	6	6.6
1300	127,400	3.50	309.88	310	6	7.4
1400	137,200	3.48	331.18	330	6	8.3
1500	147,000	3.45	352.27	360	6	9.8
1600	156,800	3.43	373.15	380	6	10.8
1700	166,600	3.40	393.85	400	6	11.9
1800	176,600	3.38	414.37	420	6	13.0
1900	186,200	3.36	434.72	440	6	14.2

When considering the use of progressive cavity pumps similar to the Environment/One pumps or centrifugal pumps considerations must be made for the differing hydraulics of the pumps. Progressive cavity pumps will discharge very consistent flows over wide ranges of pressures while centrifugal pumps will discharge considerably less if they are discharging against a high pressure. This has been addressed in the recommendations made for the State of Florida by its Department of Environmental Regulations (Thrasher, 1987) that are summarized in Figure 11.

Figure 11 Comparison of Design Flow Recommendations for Progressive Cavity and Centrifugal Pump STEP systems. (Thrasher, 1987)



In "Effluent Sewer Technology STEP & STEG Systems" by Terry R. Bounds, 1996 a straightforward linear approach to estimating design flows is taken. Equivalent Dwelling Units or EDUs are used to adjust for communities with differing average habitation per house. One EDU is related to 3 people per home discharging 50 gallons of effluent per person or 150 gallons/home for each EDU. This approach minimizes the statistical issues for simplicity, possibly in light of the acknowledged high variability of per capita water consumption and the highly variable occupancy rates for homes. These systems are designed to have the built in capacity to handle the peak flow from all the homes simultaneously. The peak flow used for hydraulic design (Qp) is computed by an empirical relationship that equates the peak flow to the number of EDU's divided by two with a constant minimum capacity (D) added. The effect of the minimum capacity added (ranging from 0 to 20) implicitly provides for an increase in per capita or per home capacity as the system size decreases.

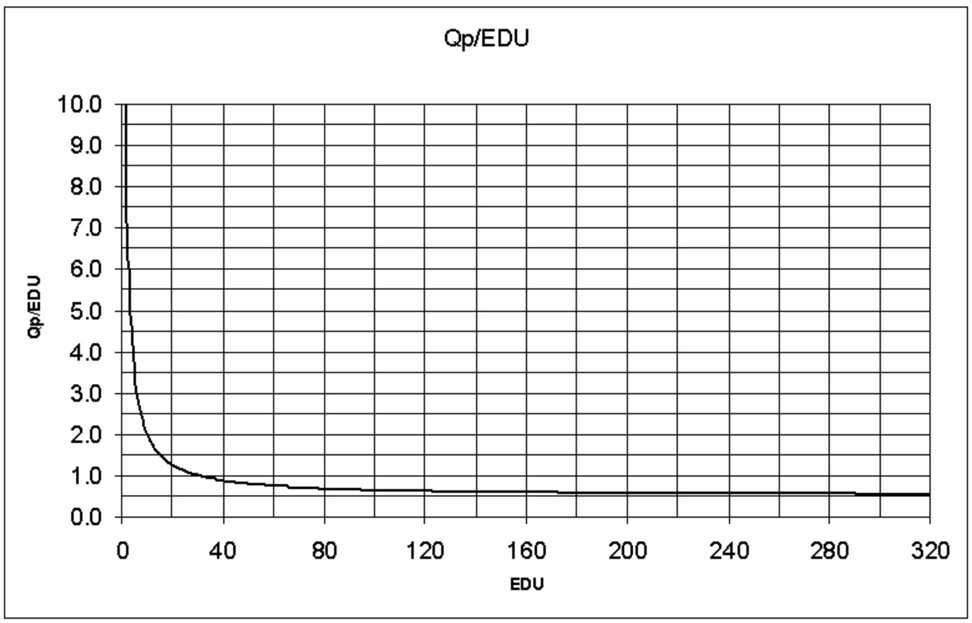
$$Qp = EDU/2 \quad + D$$

This simple approach can be adjusted for situations in which the per capita wastewater production differs from the assumed value of 50 gpd/person or the population density differs from 3 people per home. An alternative approach offered makes use of a similar relationship:

$$Qp = Qa/300 \quad + D \text{ where}$$

Qa is developed using specific data for the community. The approaches are equivalent when 50 gpd/person and 3 persons/dwelling are used. Figure 12 illustrates how the linear relationship effectively increases the design flow per dwelling for small system.

Figure 12 Qp/EDU (EDU=Equivalent Domestic Unit = 3 people. (Based upon; Terry Bounds, 1996)



Similarly, in “Design Module Number 13 – Small Diameter Sewers” (*Design Handbook for Small-Diameter, Variable Grade, Gravity Sewers*) (Simmons and Newman, USDA, 1982), an average design flow of 0.1 gal/min per residential connection is suggested for homes discharging 150 gallons per day. (150gpd/(24hr/d x 60 min/hr) = 0.104 gal/min). With the assumption that 150 gallons of storage (one day’s flow) is available in the pump tank and some implied acknowledgement of peaking factors the hydraulic design value suggested is revised to 0.4 gal/min per residential connection. This corresponds well to the one hour peaking factor of 4 discussed above for centralized gravity flow systems based upon the one hour peak flow average.

A statistical refinement is, however, incorporated into the estimation of flows for small communities by Flanigan and Cudnik in a Battelle Institute report (“Review and Considerations for the Design of Pressure Sewer Systems”). This approach is more conservative for smaller communities acknowledging a higher peak to average ratio for small communities.

In addition, there is always the possibility of all pumps going on at the same time such as might occur if a localized power outage occurred to a neighborhood while municipal water was still available. Effluent could build up in all the tanks in the neighborhood placing all the floats in a normally on position. When the power comes back on all (or many) of the pumps could also come on simultaneously. (Many timer-controlled systems have the time off interval before the time on interval that would delay some of the systems from starting immediately when the power comes back on.) The prudent designer will evaluate the alternatives and make a decision based upon rational risk assessment influenced by the requirements of the community, if available.

Pressure Distribution

Types of Pressure Distribution

There are several applications in onsite that can be considered Pressure Distribution. Several of the most common are:

- Pressure delivery to a distribution box for subsequent gravity flow to individual disposal trenches
- Pressure delivery to the laterals within the individual disposal trenches, which is often referred to as Low Pressure Pipe or LPP.
- Pressure delivery to the common community pressure line of a STEP system
- Pressure delivery to the common community gravity line of a STEG system

Common Considerations for all Pressure Distribution Systems

The design and/or analysis of any pressure distribution system, regardless of its placement in an onsite or decentralized facility, must address several common issues. Among these issues are:

- Friction Losses
- Changes of Elevation
- Available Energy

Each of these issues is considered in the following material. Friction losses are often the most misunderstood and will be considered first. The “Lift” or required change of elevation between a pump and the final discharge point as well as the energy required to do the necessary work will be considered thereafter.

Friction Losses

We need to consider carefully the head losses or friction losses that are subtracted from the system’s total energy and the pump’s energy that is added to the system’s total energy.

There are two kinds of friction loss;

- Friction loss along the length of pipes and
- Friction loss for; turns and bends, valves, expansions and contractions fittings, couplings, and transitions.
- Friction loss along an evenly perforated pipe.

The losses at bends, transitions, etc., are considered minor but if the pipe velocities are large and there are many of these elements in the system the minor losses can build up and be quite considerable.

The friction losses along the length of pipes are due to friction and consequently energy loss as the flow slides along the pipe's wall and the energy that is used up in the water as it slides around itself in turbulence.

Friction Losses along Pipelines

For now, consider only the major losses along pipelines. The features of a pipe line which influence the loss of energy along a pipe line include:

- Pipe's Length, L
- Pipe's Diameter, D
- Friction factors, f, or discharge factors, C.
- Velocity of water, V
- Viscosity of water that influences the friction or discharge factors.

The pipe's material results in a friction factor or conveyance factor which relates the surface roughness of the pipe material to the loss of energy due to friction as the water passes along the pipe boundary. All of these combine to result in the friction loss as water flows through a system. Various theoretical and empirical equations have been developed to help us compute these losses.

One of the two commonly used equations for head loss is the Darcy Weisbach equation:

$$H_l = f \text{ x } L/D \text{ x } V^2/2g$$

In the Darcy Weisbach equation the friction factor, f, is related to the pipes material and the condition of the fluids flow. Laminar and turbulent flows have different friction factors for a given pipe material but in most cases found in onsite wastewater it can be assumed that the friction factor is constant.

A second commonly used equation for determining the head loss due to friction along a pipe is the Hazen Williams formula. This formula can be solved for the head loss in a pipe and results in a working formula, which takes the following form:

$$\text{Head loss/100 ft of pipe} = 100 \times [Q/(0.285 \times C \times D^{2.63})]^{1.85}$$

Q = flow in gallons per minute
D = pipe diameter in inches
C = smoothness coefficient

Fortunately, for simple problems we don't have to solve this by hand. The head loss/100 foot of pipe is often tabulated for our convenience for common pipe sizes and materials. The following Table 3 contains such a chart of Schedule 40 PVC a material commonly used in on-site systems. Looking at the chart it becomes apparent that as the flow goes up for a given pipe size the friction loss also goes up.

Table 3 Head Loss Table (ft/100 ft)

Based on Chezy Discharge Equation

Schedule 40 PVC

	Nominal Pipe Size Inside Diameter in Inches						
	1 inch	1-1/4 inch	1-1/2 inch	2 inch	2.5 inch	3 inch	4 inch
Flow Rate	1.049	1.380	1.590	2.047	2.445	3.042	4.204
(gpm)							
1	0.09	0.02	0.01	0.00	0.00	0.00	0.00
2	0.31	0.08	0.04	0.01	0.01	0.00	0.00
3	0.66	0.17	0.09	0.03	0.01	0.00	0.00
4	1.12	0.29	0.15	0.04	0.02	0.01	0.00
5	1.69	0.45	0.22	0.07	0.03	0.01	0.00
6	2.37	0.62	0.31	0.09	0.04	0.01	0.00
7	3.16	0.83	0.42	0.12	0.05	0.02	0.00
8	4.04	1.06	0.53	0.16	0.07	0.02	0.00
9	5.03	1.32	0.66	0.19	0.08	0.03	0.01
10	6.11	1.61	0.81	0.24	0.10	0.03	0.01
12	8.56	2.25	1.13	0.33	0.14	0.05	0.01
14	11.39	3.00	1.50	0.44	0.18	0.06	0.01
16	14.58	3.84	1.92	0.56	0.24	0.08	0.02
18	18.14	4.77	2.39	0.70	0.29	0.10	0.02
20	22.05	5.80	2.91	0.85	0.36	0.12	0.03
22	26.30	6.92	3.47	1.01	0.43	0.15	0.03
24	30.90	8.13	4.08	1.19	0.50	0.17	0.04
26	35.84	9.43	4.73	1.38	0.58	0.20	0.04
28	41.11	10.81	5.42	1.58	0.67	0.23	0.05
30	46.72	12.29	6.16	1.80	0.76	0.26	0.05
35	62.15	16.34	8.20	2.40	1.01	0.35	0.07
40	79.58	20.93	10.50	3.07	1.29	0.45	0.09
45	98.98	26.03	13.06	3.81	1.61	0.55	0.11
50	120.31	31.64	15.87	4.64	1.95	0.67	0.14
55	143.53	37.75	18.93	5.53	2.33	0.80	0.17
60	168.63	44.35	22.24	6.50	2.74	0.94	0.20
70	224.34	59.00	29.59	8.65	3.64	1.26	0.26
80	287.27	75.55	37.90	11.07	4.66	1.61	0.33
90	357.29	93.96	47.13	13.77	5.80	2.00	0.41
100	434.26	114.20	57.29	16.74	7.04	2.43	0.50
110	518.09	136.25	68.35	19.97	8.40	2.90	0.60
120	608.68	160.07	80.30	23.46	9.87	3.41	0.70
130	705.93	185.64	93.13	27.21	11.45	3.95	0.82
140	809.77	212.95	106.82	31.21	13.14	4.53	0.94
150	920.13	241.98	121.38	35.46	14.93	5.15	1.07

NUMBERS IN CHART ARE HEAD LOSS/100 FT IN UNITS OF FT

If it is desired to use the Hazen Williams Formula from the text, use a “C” value of 140. If it is desired to use the Darcy Wesibach $H_l = f *(L/D)*V^2/(2g)$ be sure to use the actual inside pipe diameter and a friction factor “f” equal to 0.021

The friction head loss information allows the convenient resolution of complex pump design problems when used in conjunction with the simplified energy equation and pump head data prepared by pump manufacturers. In the next few sections a progression of example problems will apply these concepts in an increasing order of complexity and realism.

Minor Losses

In addition to frictional losses due to pipe materials and turbulence, losses also result from changes in direction, changes in flow area and changes in friction due to fittings. These losses are known as minor losses since they are usually much smaller in magnitude than the pipe wall friction losses. Equivalent lengths or loss coefficients are used to calculate minor losses.

Equivalent Lengths

Equivalent lengths (L_e) assume each fitting or flow variation produces a head loss that is equal to the losses caused by an equivalent length of the pipe. For example, a 2-inch gate valve may produce the same amount of friction as 1.5-feet {0.46 m} of 2-inch {5.08 cm} pipe. Therefore the equivalent length of the gate valve is 1.5-feet. The equivalent lengths for all of the minor losses are added to the pipe length term in the Darcy Weisbach or Hazen Williams equation.

The equivalent length method should be limited to turbulent flow. Equivalent lengths are easy to use, but you must have recommended equivalent length values. Table 4 is a sampling of equivalent lengths for common fittings. (Lindburg, 1992) The equivalent length of a fitting will vary between manufactures, materials and the method of attachment. Because of these variables, it may be necessary to use a generic table of equivalent lengths.

Table 4 Pipe Fitting Length Equivalents

	Equivalent Length in Feet		
	Pipe Sizes		
Fitting Type	1”	2”	4”
Regular 90° Elbow	5.2	8.5	13.0
Long Radius 90° Elbow	2.7	3.6	4.6
Regular 45° Elbow	1.3	2.7	5.5
Tee	3.2	7.7	17.0
180° Return Bend	5.2	8.5	13.0
Globe Valve	29	54.0	110.0
Gate Valve	0.84	1.5	2.5
Angle Valve	17	18.0	18.0
Swing Check Valve	11	19.0	38.0
Coupling or Union	0.29	0.45	0.65

Loss Coefficients

Each fitting has a loss coefficient, K, associated with it. This coefficient is multiplied by the kinetic energy to get the associated loss.

$$h_m = Kh_v$$

Where:
K = loss coefficient
h_m = head loss
h_v = velocity head

Loss coefficients for specific fittings and valves are generally determined empirically, in most cases they cannot be derived theoretically. There are two methods to determine the loss coefficient. The loss coefficient for any minor loss can be calculated if the equivalent length is known.

$$K = \frac{fL_e}{D}$$

Where:
K = loss coefficient
L_e = equivalent length
f = friction factor
D = pipe diameter

Otherwise, a field determination of the losses due to the fitting as a function of flow rage can be generated and a rating curve or equation can be developed and used.

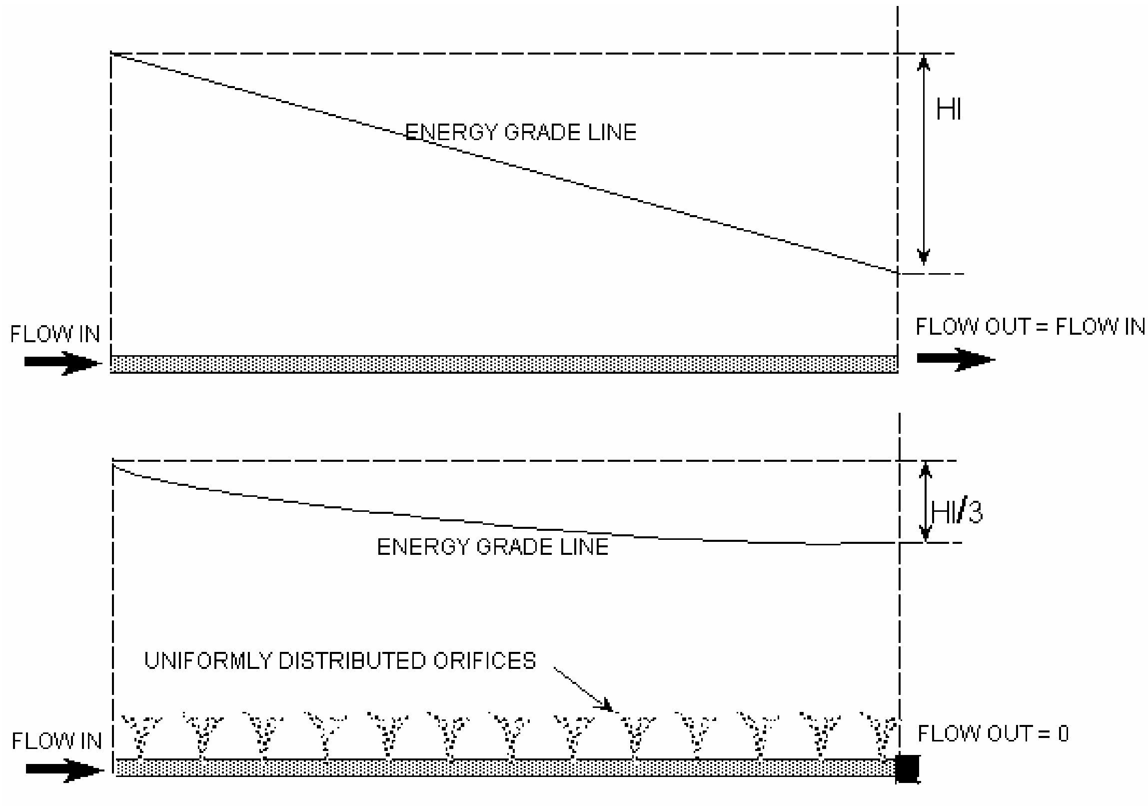
Head loss along a pipe with multiple orifices

Of particular importance in onsite hydraulics is the use of multiple orifices along a pipe. Multiple orifices along a pipe are used to uniformly dose treatment media as well as uniformly deliver treated effluent to the ground. In particular when multiple orifices are used it is important to maintain uniform flow along the length of the pipe. To guarantee that the discharge from all of the orifices falls within a tolerable range it is necessary to be able to estimate the head at the extremes of the pipe. This can be done by carefully calculating the head loss between each two orifices in turn, calculating the remaining head, calculating the discharge from each orifice in turn, diminishing the remaining flow by that discharge and continuing on down the pipe. This repetitive process will enable the determination of the discharge from each orifice, from the first to the last, which will then enable the designer to ascertain if significant discharge differences exist.

Since the total discharge down the pipe is diminishing linearly (to the extent that each orifice has essentially the same flow) the velocity in the line will also be diminishing linearly. Recalling that head loss relates to the velocity squared it can be deduced that the head loss along the line will decrease non-linearly with the greatest head losses occurring at the beginning and the smallest at the end.

Figure 13 illustrates this situation by comparing the head loss (Hl) for a length of pipe carrying all the flow for its entire length to the same pipe with uniformly spaced orifices discharging all the flow out the orifices along its entire length.

Figure 13 Head Loss Comparison: Non-Perforated versus Perforated Pipe



This problem has attracted the attention of designers looking for efficient ways to solve this problem easily. Calculus allows an elegant solution that will be left to the reader to pursue. If equal sized orifices are closely spaced, it has been shown that in the limit the total head loss across the entire line is 1/3 of the head loss that would be developed for the entire line carrying the entire flow. This useful conclusion can considerably facilitate the analysis of pipes with multiple orifices.

$$Hl_{\text{multiple orifices along a pipe}} = 1/3 \times Hl_{\text{total pipe carrying the total flow}}$$

Lifting and Discharging Effluent with a Constant Head Pump and no Friction

To take a first look at how the hydraulic energy provided by a pump can lift and discharge water at some distant location we will now look at one of the most common situations in on-site and decentralized systems, pumping effluent from a tank to a disposal field or into a sewer line. There are generally 4 elements to be considered in such analysis.

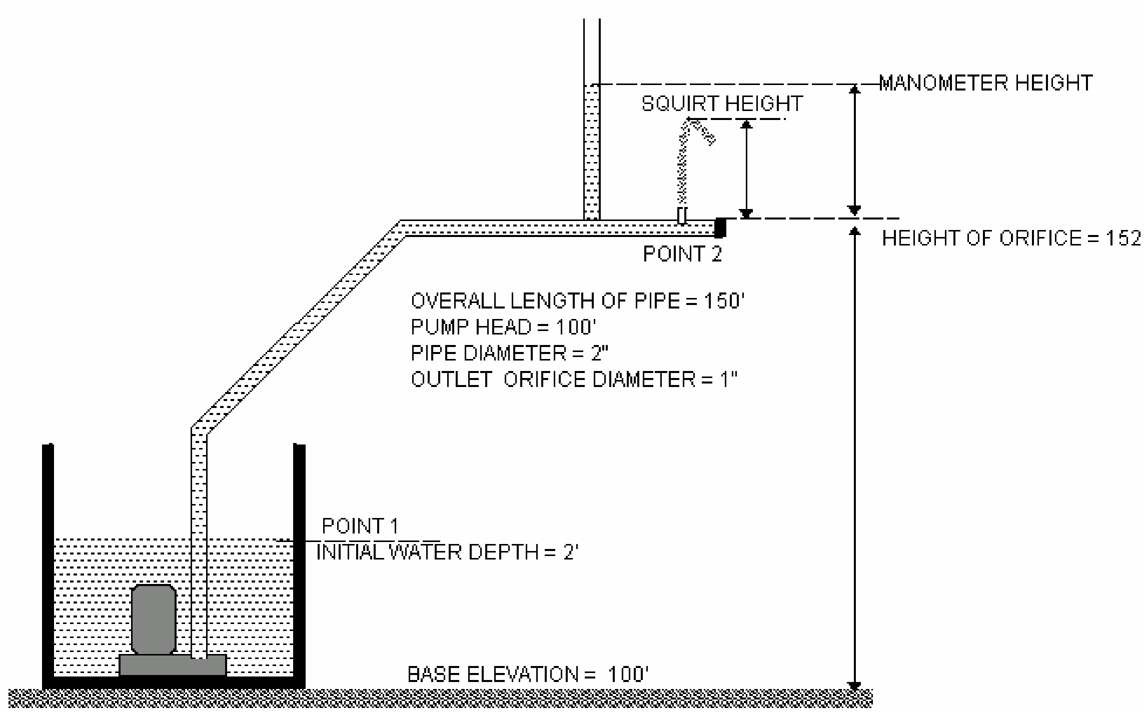
- The hydraulic characteristics of the pump.
- The head losses associated with the connecting pipes (and fittings)

- The change in elevation between the water surface in the lift station and the discharge feature or orifice.

- The hydraulic characteristics of the discharge feature.

These four elements are all linked together by the use of the energy concepts we have been developing in the previous sections or modules. To begin our analysis of real world pump effluent pump stations we will consider a hypothetical situation which limits the complexity by assuming constant characteristics for the pump and no frictional losses in the pipes. Figure 14, below, illustrates the features of the system being considered.

Figure 14 Pump & Pipe System



In this case we will assume we know that the pump is producing a constant 100 ft {30.5 m} of head but we do not know the resultant flow rate that will depend in part upon the discharge from the orifice at the top of the system. The orifice at the end of the pipe constricts the flow down to a 1-inch diameter opening. (Typical orifices in onsite or decentralized systems are less than 1 inch but 1 inch is used in this example problem to keep the computations simple.) The discharge from the orifice will depend in turn upon the head available at the orifice.

We will also assume that the pipe is frictionless. This is not realistic but let us consider one thing at a time. This problem simplifies that situation considerably by allowing all the water to come out at one point. In realistic effluent distribution systems the piping network will likely branch and effluent will emerge at multiple points along a lateral.

We will first determine the theoretical Manometer height. (The squirt height in the field will be less due to air friction and the break up of the flow into discrete particles or droplets.) The manometer height reveals the portion of the total energy available for driving water through the orifice constriction.

As was discussed earlier in this module, the total energy available at the manometer location includes the velocity head term but this energy is generally not available for enhancing the discharge or rise in water in the manometer tube. The pitot tube is a device which has its insertion point in the fluid turned up stream to “capture” the energy of the water’s velocity by creating a stagnation point immediately in front of the pitot tube. Therefore, the velocity head results in an increase in water level equal to the velocity head and can be used for computing the flow’s velocity and discharge.

As also discussed previously energy must be conserved and accounted for between each sequential point in the system. In problems such as this it is always easier to go from a point at the free water surface in the tank (point 1) to a point where the water again is only experiencing atmospheric pressure (point 2) because we know some of the energy terms at these points. The pressure is zero at both locations (pt 1 & pt 2) and the velocity is zero at the surface (pt 1) of the tank. In the middle points, energy is exchanging between elevation head, pressure head and velocity head and more work would be required to get some of the middle values. Therefore let us consider the conservation of energy between point 1 and point 2.

At point 1:	At point 2
Elevation Head = 102 feet {31.1 m}	Elevation Head = 152 feet {46.3 m}
Pressure Head = 0 psi (gage)	Pressure Head = O psi
Velocity Head = 0 ft/sec	Velocity Head = ?

Pump Head = 100 ft {30.5 m}
Head Loss Due to Friction = 0 ft (assumed)

Simple arithmetic shows us that the Velocity Head at point 2 must be 50 ft {15.2 m}. Reconsidering the development of the orifice equation we recall that:

Velocity head at the orifice = $V^2/2g = 50$ ft
Therefore, $V^2 = 50 * 2g$ or $V = (50 * 2g)^{0.5}$
Solving for V results in 56.7 ft/sec {17.3 m/s}.

If the orifice has a diameter of 1 inch, it will have an area of $x\ 1^2/4$
Or 0.785 {5.06 cm^2} square inches
Or 0.785/144 square ft.

If we remember that Q = A x V we can now figure the flow rate.
Q = 56.7 ft/sec x 0.785/144 sq. ft. = 0.31 cfs or 2.31 gallons/second {8.75 L/s}
or 138.7 gallons/min

The total energy at the manometer location will be equal to the energy at the outlet (50 ft {15.2 m}) minus the velocity head in the line at that location. Energy at the manometer will be the sum of the pressure head and the velocity head (in the pipe). The pipe diameter is twice as large as the orifice diameter. Therefore by continuity it will have ¼ the velocity.

The velocity in the pipe will be 56.7 ft/sec/4 = 14.2 ft/sec {4.33 m/s}

The velocity energy will be $V^2/2g$ or $14.2^2 / 2 * 32.2 = 3.13$ ft {0.95 m}

Therefore the pressure head at the end of the pipe will be 50 ft – 3.13 ft = 46.87 ft {14.3 m}

It should be obvious that with slow velocities at the end of laterals and small orifices we should see even closer agreement between the velocity head at the orifice and the pressure head measured close by with a manometer.

Lifting and Discharging Effluent with a Constant Head Pump and Friction

Consider the pump problem solved in the last section. The final flow calculated was to be 138.7 gallons per minute {525 L/min}. From the Hazen Williams Table we can see that at that flow the frictional head loss along the pipe would be about 46 feet {14 m}. This reduces the final head from 50 down to 4 but only if the pump continues to provide 100 feet {30.5 m} of head. With less velocity head at the exit there would be a considerably lower flow rate computed.

Lifting and Discharging Effluent with a Variable Head Pump and Friction

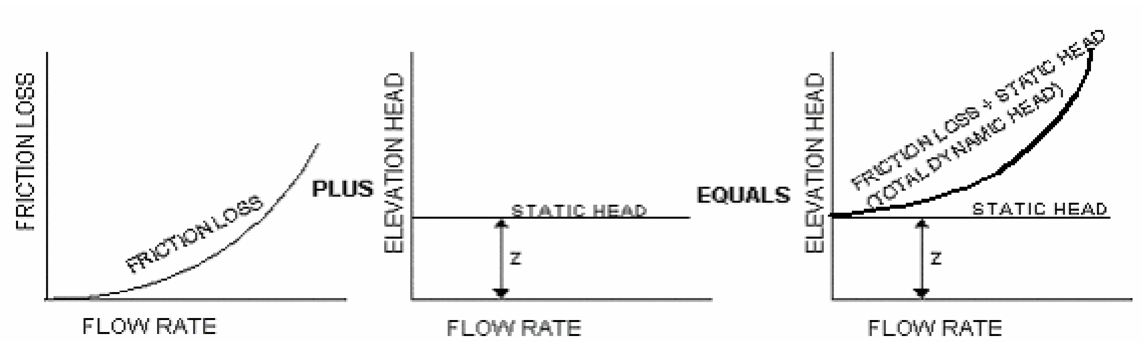
As discussed above the head a real pump provides will increase as the flow rate decreases and will decrease as the flow rate increases. A marriage between a system's energy requirements and a pump's energy availability will have to be developed to determine the actual final pressure and head for a system.

In the previous pump problem we solved for the pressure and flow at the orifice once we were given information about the pump's head, which we assumed, was constant. As it turns out any given pump provides a different amount of head for each different flow quantity. And for each different flow quantity the system needs a different amount of energy to overcome the friction. Therefore you can't solve for the flow until you know the pump head and you don't know the pump head until you know the flow. (This is like chasing your tail.) Fortunately the energy concept helps again because the amount of energy the pump provides and the amount of energy the piping system needs will come into balance once the system is turned on. The system can't run at a flow condition which demands more energy than the pump can provide at that flow condition. A balance must be struck.

The following, Figure 15, illustrates these concepts. The system curve is defined as the total of the static lift (the change in elevation) plus the friction loss in the piping system.

The static lift is generally constant (unless the system's outlet point is moving up or down) but the friction loss has a different value for different flow rates. As was discussed earlier, the friction loss is generally proportional to the velocity squared that relates to the discharge by the constant cross section area of the pipe. Therefore the friction loss increases with the flow squared. The sum of the static lift and the friction loss is referred to as the System Curve.

Figure 15 System Energy Considerations



Pump Energy Considerations

Pumps and turbines are devices for exchanging hydraulic energy for mechanical energy. Pumps driven by motors convey energy to fluids and fluids driving turbines can turn generators and make electricity. Both of these devices can be described in part by the same equation that relates the power of the pump or turbine to the flow rate of fluid and the head either generated (pump) or used (turbine).

The fundamental formula takes the form:

$$\text{Power} = C \times Q \times H$$

Where Power is the rate of doing work often expressed as Watts, or Horsepower, or BTU/time. Most of the pumps used in the onsite environment are rated by their horsepower so the formula can be rewritten with units commonly used with the hydraulic design of onsite systems:

$$HP = C \times Q \times H / 550$$

Where: HP is horsepower (550 ft-lbs/second {0.746 kW})

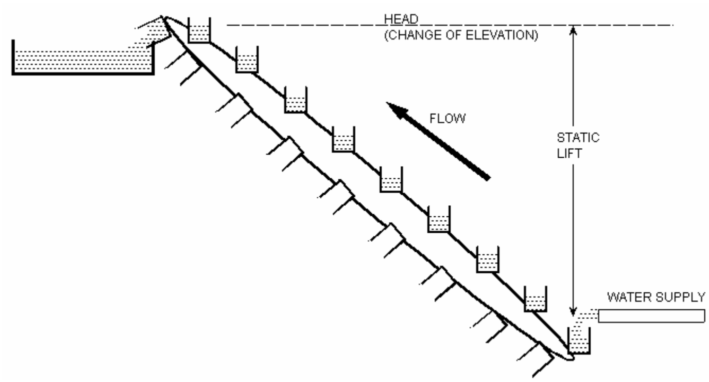
Q is the flow rate in cubic feet per second (cfs)

H is the head in ft

C is the weight of the water

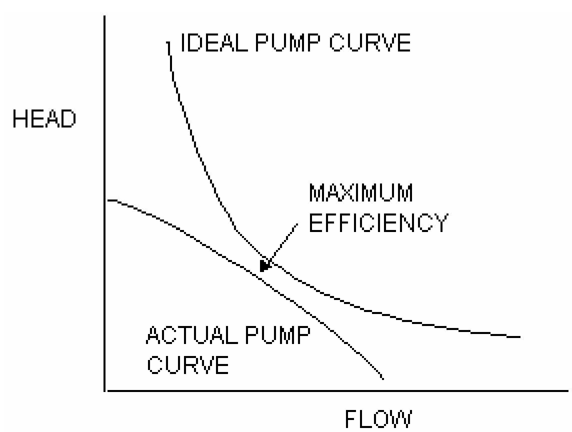
The relation can be visualized by thinking of an escalator carrying water up an incline in buckets. For each pound of water lifted one foot, one foot-pound of work is done. If 550 foot-pounds of work are done per second we call that 1 HP {0.746 kW}. This concept is illustrated in figure (Figure 4)

Figure 4 Hydraulic Machine



Ideally the hydraulic machinery equation describes a hyperbola where every combination of Q and H that results in the same HP is plotted as the possible combinations of Q and H for that machine. This is the ideal pump curve for a given horsepower. However, all machines have efficiencies somewhat less than 100% and pumps exhibit different efficiencies at different flow rates. This results in the typical pump curve departing more and more from the ideal curve as the flow departs more and more from its optimum design point.

Figure 5 Ideal versus Actual Pump Curves



Pump manufacturers make related series of pumps that have similar characteristics but different powers. These families of pump curves are often illustrated on the same graph. The following figure illustrates a typical family of pump curves.

The combination of system curve and pump curve or curves allows the rational design of a pump system. Figure 6 illustrates a system curve superimposed upon a series of pump curves.

Figure 6 A "Family" of Pump Curves

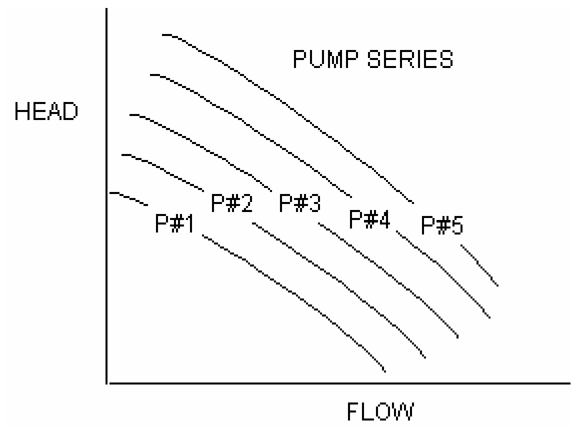
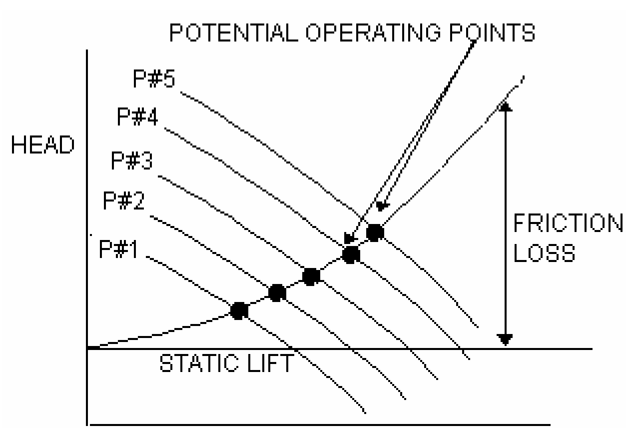


Figure 7 Pump Energy Considerations



Now, let us finish the hypothetical problem started before with some real pump data and find out what flow rate and final pressure will result if we put a specific pump in the system. Following the friction loss page is another page of data that contains typical pump curve information.

Two make our solution even more accurate it would be worthwhile to add in some of the minor losses that affect the system. These minor losses arise at the pipe fittings, joints, bends, and changes in diameter. There are two ways which are generally used to solve for the minor losses:

a. Friction loss factors which must be multiplied by the velocity term, $V^2/2g$

b. Equivalent length factors add a length of straight pipe to the total length of the pipe and results in the same additional head loss as would the pipe and bend individually.

Looking at Figure 14 we can see that there are two bends at 45 degrees. The chart following the pump data shows that for a 2 inch {5.08 cm} line and 45 degree elbow, there an additional 2.7 feet {0.81 m} of equivalent head loss per fitting. With two such fittings there is a net increase of 5.4 feet {1.62 m} of equivalent pipe. This length should be added to the original pipe length when determining the frictional head loss.

Hydraulics of STEP Type Systems

The term STEP system has been used loosely for systems with discharge either into an onsite disposal system or a community collection system. We will consider a community STEP system first. As discussed previously STEP systems are often a viable alternative when a continuous downhill path to the treatment facility is not available and/or when excavation costs are prohibitive.

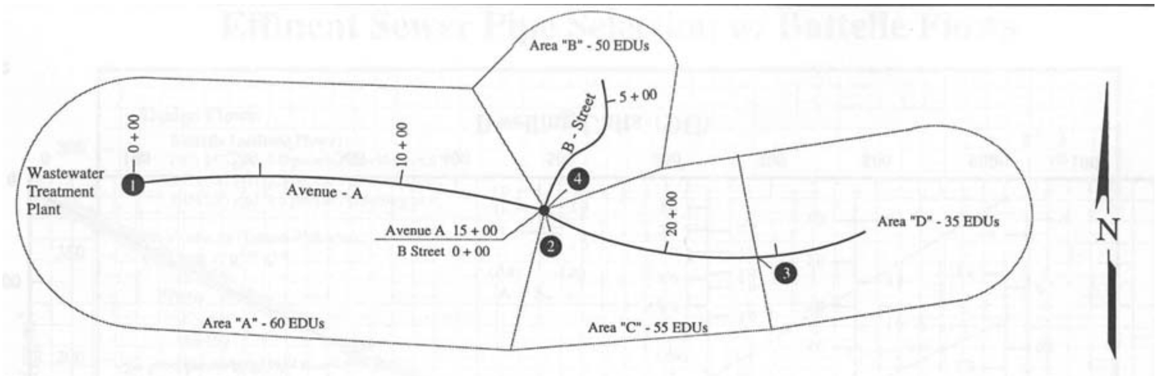
Design of Common Transport Line

The following discussion highlights the major design steps and considerations for a community STEP system. The reader is referred to more detailed design guidelines for design specifics.

Determining Flows along the Line

The first step in the design of a community STEP system is the application of pump hydraulic considerations to the common line serving the entire community. Figure 16 below illustrates the plan view for a small STEP system. Generally the design flows for community sewer systems do not consider the hydraulics of the collection line at each connection. Instead areas of the community being served are grouped together to result in a larger combined flow entering the system at a node or junction. The hypothetical community shown in the figure has 4 areas identified (A, B, C&D) and 4 associated nodes (1, 2, 3&4) where the aggregate discharge from the area is added to the flow in the main line. The flows entering the system at each node in the common system will be determined as discussed above in the section dealing with design flows. The total flow reaches the treatment facility at station 0+00, node 1.

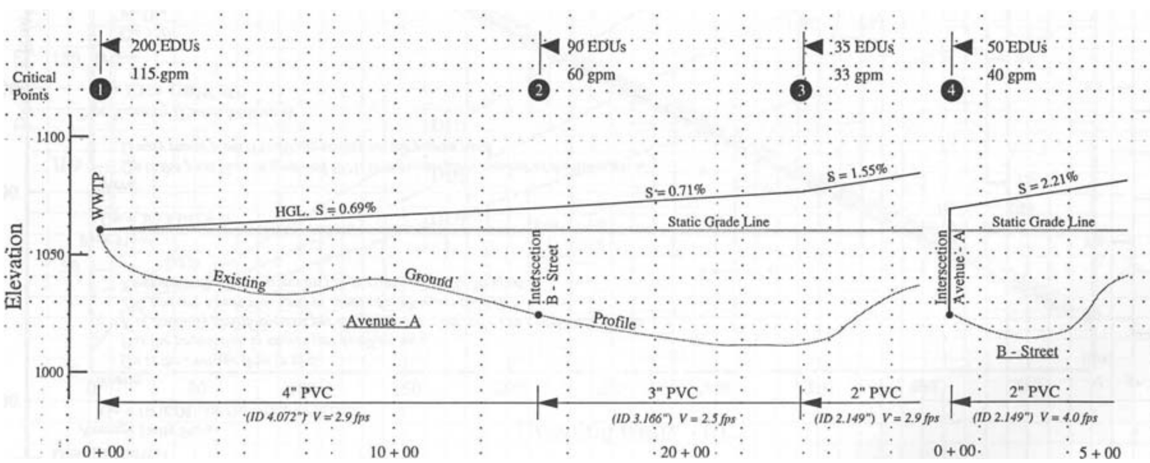
Figure 16 Hypothetical Decentralized STEP system (Orenco, 1996)



Establishing the Hydraulic Grade Line for the System

The design proceeds upstream from the final discharge point. The design starts with the required elevation and exit pressure or head and adds the head losses in each section of pipe based upon the computed design flow for that section of pipe. Figure 17 shows the hypothetical profile for the situation illustrated above.

Figure 17 Hypothetical Profile (Orenco, 1996)



To understand the profile drawing shown consider the following:

- The system's horizontal alignment is not represented.
- Area B's profile is shown as a separate part of the drawing at the right. It connects at the point labeled "Intersection B-Street" in the main profile.
- The proposed pipe itself is not shown. It will be parallel to the Existing Ground Profile, generally 30 to 36 inches below existing grade.

The shallow depth of the proposed pipe is one of the advantages of these types of systems. Conventional Sewer system profiles will always show the gravity flow sewer pipe due to its greater depth and slopes which differ from the natural ground slope.

The hydraulic grade line (HGL) depicts the sum of the elevation head and the pressure head at any point in the system. The measured change in elevation from the pipe itself (about 30 inches below grade) to the plotted elevation of the HGL indicates the pressure (in feet or meters) that will be expected in the pipe when it is carrying the design flow.

At each step in the design the elevation of the hydraulic grade line should be determined and plotted on the profile drawing. As the design proceeds toward the extremity of the system the design flow will decrease as fewer homes contribute to the design flow at the predetermined nodes. This reduced flow will likely decrease the slope of the hydraulic grade line unless the designer has decreased the diameter of the common line in which case the slope of the hydraulic grade line may not decrease as you approach the beginning of the system.

The elevation of the hydraulic grade line at the location of each house connection will influence the design of the pressure line from the septic tank to the common line.

Minimum pressures should be maintained and the HGL should not descend below the elevation of the pipe or negative pressures will result. The flow will proceed in the direction of the descending HGL but negative pressures along the way can cause unanticipated hydraulic problems if service connections are made in areas where the pipe pressure is negative.

From Septic Tank to Common Line

The second step in the design of a STEP system is the application of pump hydraulic considerations to the pressure line from the individual pump to the common transport system.

Figure 18 illustrates graphically the hydraulic analysis associated with the design of the individual STEP system.

The analysis of this system is similar to the analysis of the pump systems discussed previously. In brief, the design procedure considers:

- Achieving a pressure at the end of the service line equivalent to the pressure in the Pressure Sewer Main (h_p) when design flow conditions are being experienced in the Pressure Serer Line is the major design goal.

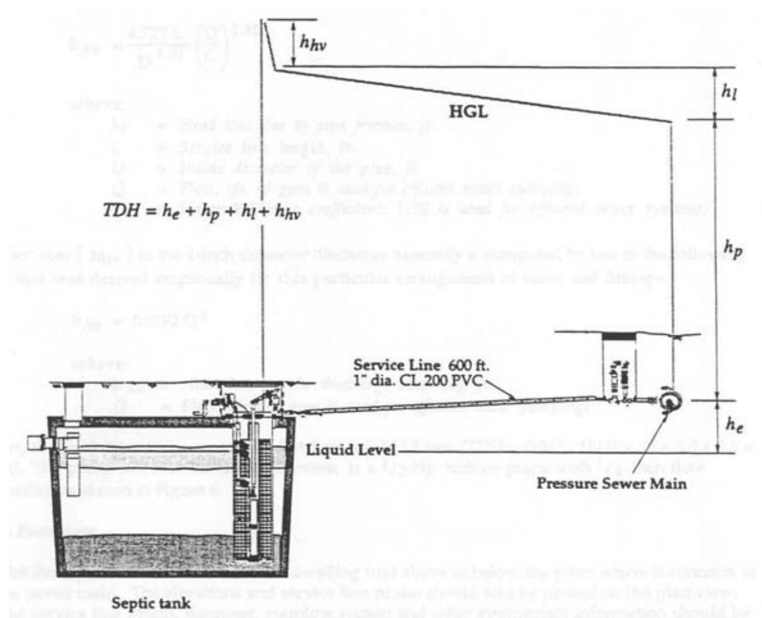
- The chosen pump must overcome the friction loss along the Service Line (h_l) when operating at the chosen discharge rate. As the friction acts continuously along the service line the HGL descends toward the elevation of the HGL at the street.

- The chosen pump must also overcome any additional head losses (h_{hv}) that occur in the pump vault (across the effluent screen) and at any of the "minor" hydraulic elements that connect the pump to the Pressure Sewer Main.

- And finally, the pump must produce enough head to overcome any change of elevation from the water level in the septic tank to the elevation of the Pressure Sewer Line in the Street (h_e). Therefore the Total Dynamic Head required from the pump when discharging at its design flow rate must equal:

$$TDH = h_e + h_p + h_l + h_{hv}$$

Figure 18 STEP from Septic Tank to Pressure Sewer Main (Orenco 1996)



From Septic Tank to Dispersal System (LPP/LPD)

One of the more common applications of pump hydraulics in the onsite/decentralized arena is the use of pumps to deliver effluent to a dispersal field. There are two main variants of this type of system:

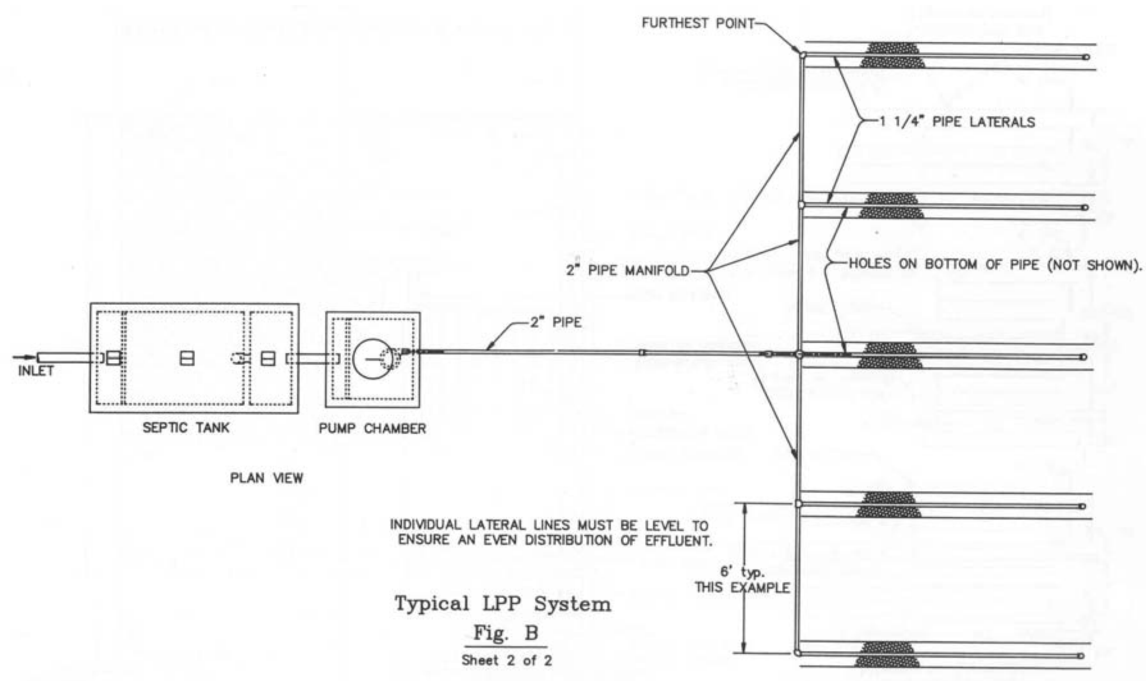
- Pressure dosing the dispersal laterals (or drip line)
- Pressure delivery to the beginning of a gravity flow dispersal (or conveyance) system

Figure 19 illustrates the plan view of an onsite system that discharges the septic tank effluent under pressure to a dispersal field. The pump system achieves enhanced control over the discharge to the dispersal field due to:

- The pump actuation control system standardizes the dose volume.
- Timers (if used) allow standardized "rest" intervals between doses that are believed to enhance the continued treatment of the effluent in the trench.

- The pressurized effluent laterals deliver effluent uniformly along the length of the trench through evenly spaced orifices.

Figure 19 An (enhanced flow) STEP (or LPP) system (SSPMA 1998)



Design Process for (Enhanced Flow) STEP (LPP/LPD)

The design process for these systems starts with the appropriate soil-loading rate determined by the soil scientist. Ideally the soil recommended soil-loading rate would include:

- Total discharge per unit area per day
- Maximum discharge rate per unit time
- Minimum desirable interval between doses.

Often this data is not available and the hydraulic designer will generally start with the maximum allowable discharge per unit area per day. With typical trench widths ranging from 1 to 3 feet the total required length of trench can easily be computed. Site constraints will dictate the overall configuration of pipes (i.e., how many laterals, lateral length and lateral spacing.) Together the soil limitations and the site limitations will influence the overall system configuration.

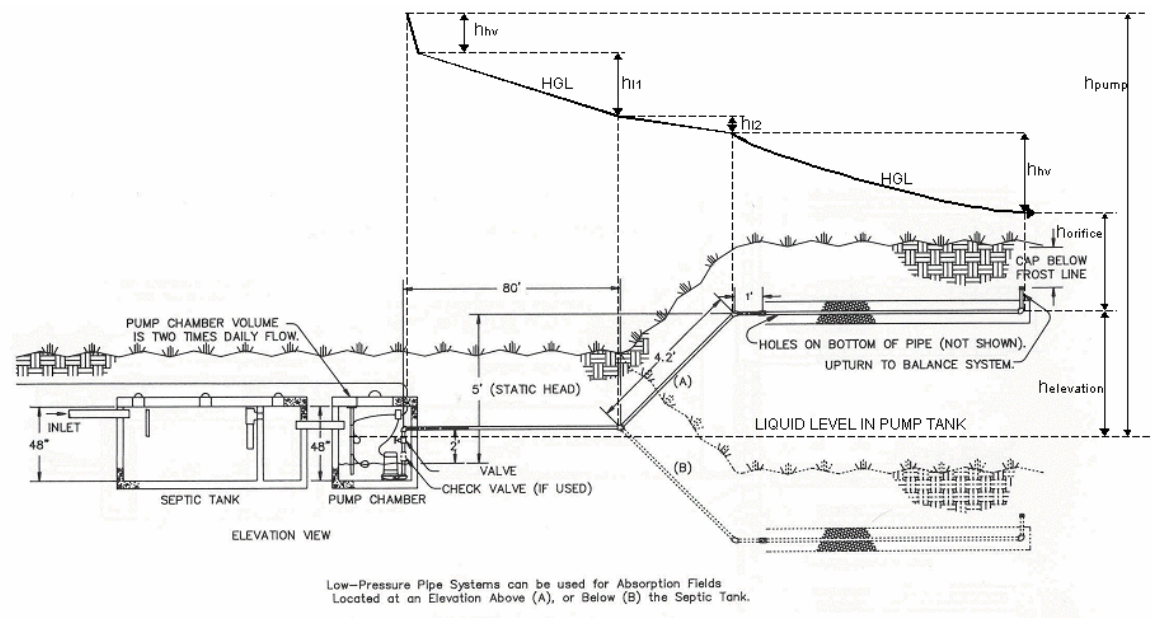
The design proceeds as follows:

- Using the orifice equation and related information determine the required pressure at the most distant location sufficient to achieve the desired discharge for a given orifice diameter.

- Using the lateral length, diameter, orifice spacing and head loss relationship for spaced orifices determine the total flow and total head loss for an individual lateral.
- Compute the head loss along the manifold as lateral flows are added.
- Compute the head loss along the transmission line for the total flow.
- Determine the change of elevation between the water surface in the pump tank and the discharge point of the orifice
- Continue the design of the pump, pump chamber and control configuration similar to the procedure for other STEP variants discussed.

In the final analysis the required pump head will be the sum of the contributing head terms. Figure 20 illustrates the profile and hydraulic grade line for this type of system.

Figure 20 Profile and Hydraulic Grade Line for Enhanced STEP-LLP/LLD system (SSPMA 1998)



Design Process for simple STEP Discharge to Gravity Dispersal

The design process for these systems follows the same procedure as the previous system with several simplifications. Again the design starts with the appropriate soil-loading rate determined by the soil scientist. Together the soil limitations and the site limitations will influence the overall system configuration.
The design proceeds as follows:

- Since the discharge will be at atmospheric pressure in the gravity flow manifold or distribution box the HGL will descend to the actual elevation of the bottom of the manifold or distribution box.
- Using the lateral length, diameter, orifice spacing and head loss relationship for spaced orifices determine the total flow and total head loss for an individual lateral.
- Compute the head loss along the manifold as lateral flows are added.
- Compute the head loss along the transmission line for the total flow.
- Determine the change of elevation between the water surface in the pump tank and the discharge point of the orifice
- Continue the design of the pump, pump chamber and control configuration similar to the procedure for other STEP variants discussed.

Figure 21 Plan View Simple STEP Discharge to Gravity Flow (SSPMA 1998)

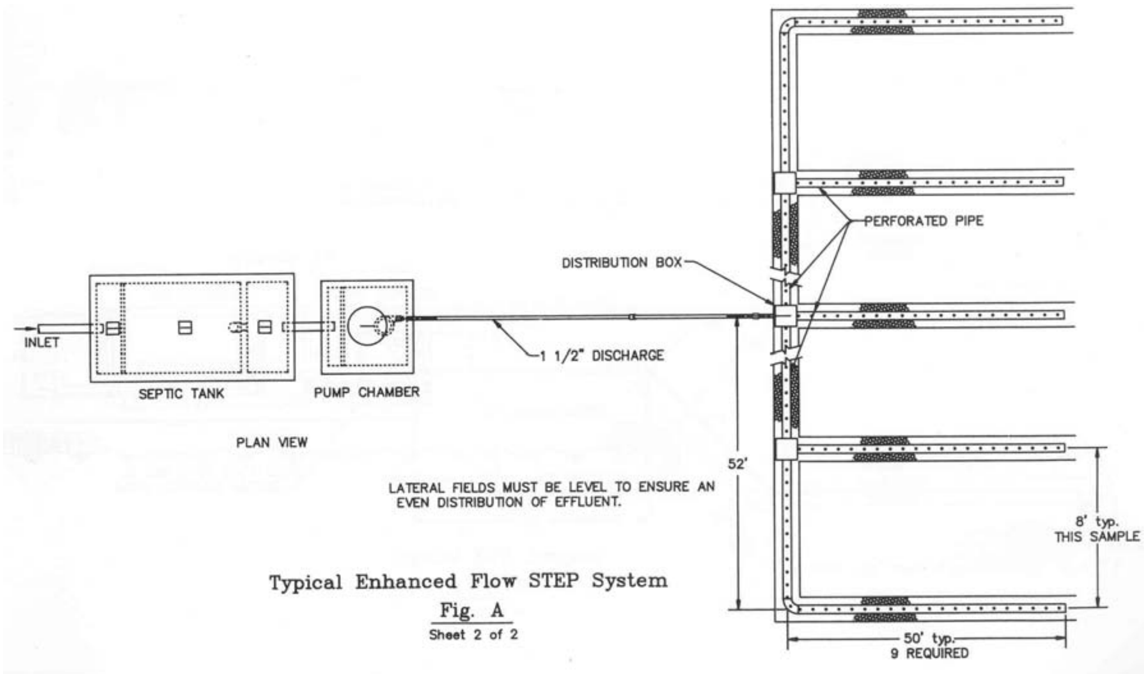
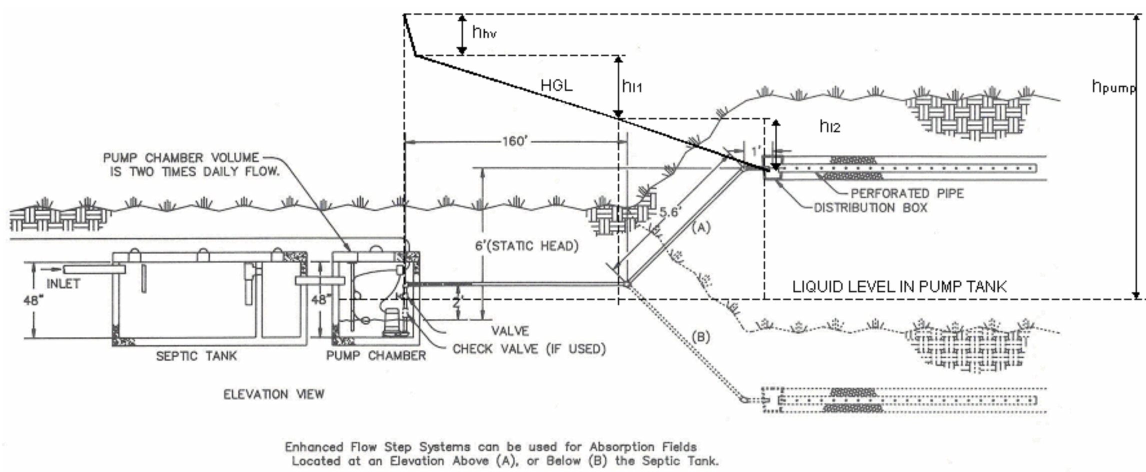


Figure 22 Profile View Simple STEP Discharge to Gravity Flow (SSPMA 1998)



Gravity Conveyance in Onsite & Decentralized

Overview of Gravity Flow in Individual Onsite Systems

The use of gravity flow hydraulics for moving wastewater is very common throughout our communities. Whether our homes, schools or businesses are connected to a centralized facility, a decentralized facility or an individual onsite system, plumbing features that depend upon gravity flow are very common.

Collection

Under every sink, shower, washing machine and toilet in every house is an extensive under drain system that conveys wastewater to a single household discharge point. Such household plumbing features rely upon gravity to convey the water from the point of origination (as wastewater) to its discharge point. The design of the under drain systems in our buildings is subject to the standards codified in our plumbing and building codes. These codes address not only the hydraulics of flow but issues of health and safety, reliability, avoidance of nuisance (smells), and structural integrity. In many cases they are proscriptive standards, meaning that specific design values are mandated. In most cases the design proceeds by computing the number of fixture units connected to the system at any point and providing the minimum pipe diameters and slopes recommended by the codes. The reader is referred to their local plumbing and building codes for learning more about indoor sanitary plumbing.

Conveyance

Once the effluent is collected to a major drain line that emerges from the building it must be conveyed to either the community collection system or to the onsite treatment and dispersal system. For most common installations the plumbing and building codes will suffice for establishing the minimum diameter and slope and other design feature required for a home or building of any particular size. For uncommon situations performance requirements are often established. Performance requirements do not stipulate the values of particular design features such as diameter or slope but instead stipulate the required performance such as required velocities and depths of flows. In these cases the designer must consult fundamental hydraulic data and references to design such systems. The material presented in this module will provide much of that needed information.

Treatment

Most common treatment (or pretreatment) devices used in onsite systems rely upon gravity flow to move the effluent through the system. Internal weirs, ports, orifices, siphons, and screens are used to control gravity flow through the treatment device and in the process separate clarified and treated effluent from sludge and scum.

Distribution

Once the effluent is treated and conveyed to where it will be dispersed, there are often needs to divide the flow into uniform fractions to supply one of several dispersal components. Devices which accomplish this task may be manufactured for this purpose or may be built individually at the project site. These distribution devices are discussed later in this section.

Dispersal

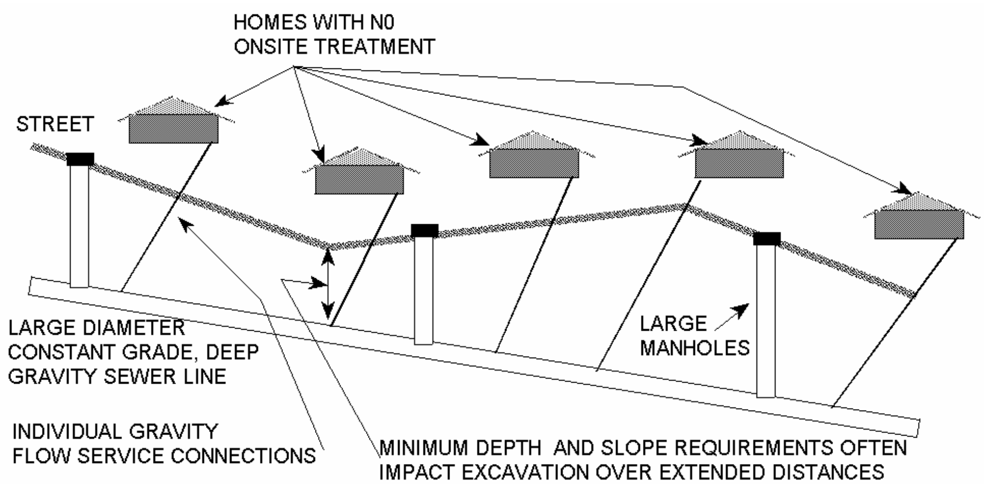
Most individual onsite systems rely upon gravity flow to deliver treated effluent to the dispersal system. In many cases the dispersal system consists of underground trenches into which effluent is released to flow by gravity in, hopefully, a predetermined way. Dispersal trenches will also be discussed later in this section.

Overview of Gravity Collection and Conveyance in Decentralized Systems

Decentralized systems tend to serve smaller communities or groups of homes and/or businesses. The upper size limits on decentralized systems has not been firmly established. Conversely, conventional sewer systems (which serve larger communities, towns and cities) similarly have no established lower size limit. For purposes of this curriculum the distinction between decentralized collection and conveyance systems and conventional collection and conveyance systems will be based upon the difference in design philosophy and resulting overall configuration of the system being considered. The following figures illustrate some of the main differences between conventional and decentralized gravity conveyance systems.

Figure 23 illustrates a conventional sewer collection system. Until recently systems of this sort were the only alternative considered to passive individual onsite systems (i.e., septic systems or cesspools).

Figure 23 Conventional Gravity Sewer System



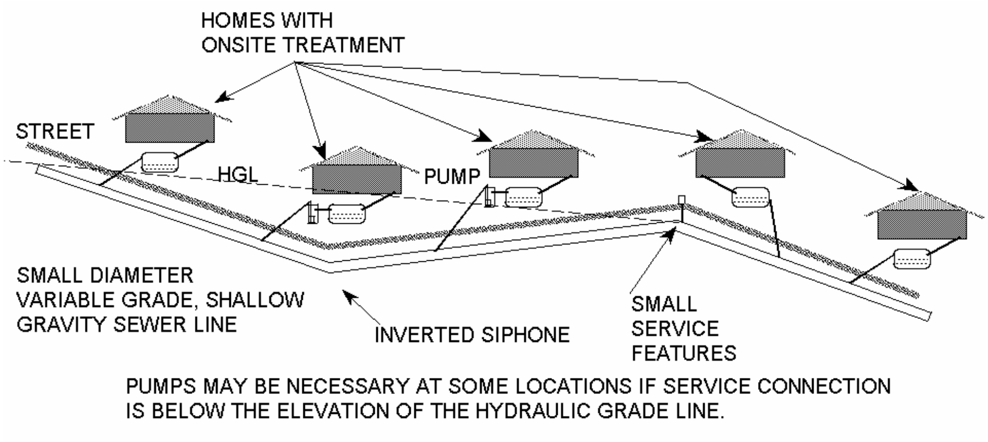
All gravity flow lines must be placed at a sufficient depth to prevent freezing and to receive wastewater from the lowest location served. The cost of installation will depend in large part on the depth of the sanitary sewer. In areas with shallow soils over bedrock the cost of installing conventional gravity sewers could be unfeasible if the sewer must be installed deep. Furthermore, the maximum depth of the sanitary sewer requires knowledge of the overburden loads that will be placed on the pipe. The pipe must have the structural strength to avoid breaking or collapsing due to the weight of the cover materials and any surface loads (i.e., vehicles). Large diameter pipes even if placed at greater depths are often still more susceptible to crushing than small diameter shallow pipes.

Conventional sewer systems carry raw wastewater. Due to the solids loads carried by raw wastewater the chances of blockages are high. Cleanouts (manholes) are used to allow access to the gravity sewer for maintenance such as unclogging a blockage. Cleanouts need to be located at convenient intervals, changes in slope, changes in diameter or at sharp bends to allow access to the pipe for cleaning. Sewers laid on flat grades require periodic flushing and/or cleaning to remove deposited solids and to prevent plugging. The location of the cleanouts is generally dictated by the Unified Plumbing Code, however many local regulatory agencies may have more conservative requirements.

Sanitary sewers are meant to carry wastewater that comes from fixtures such as sinks, toilets, bathtubs, showers and washers. Infiltration is groundwater that enters sanitary sewers through leaks in pipes. Inflow is storm water that is directed to the sanitary sewers through connections such as roof downspouts, driveway drains and groundwater sump pumps. When infiltration and inflow enter the sanitary sewer, they take up pipe capacity that is required for the wastewater. The infiltration and inflow can cause sewer backups and overflow into the environment during wet weather. They can also cause overloading at the treatment facility itself. Large diameter, deep sewer systems are more susceptible to infiltration and inflow due to greater soil moisture pressure at greater depth, larger diameter pipes and therefore larger pipe joints where leaks likely occur, and manhole structures which can conduct street drainage into the sewer system.

Conventional sewer systems may be the best alternative in many situations but should not be considered the only alternative to individual onsite systems.
Figure 24 illustrates a decentralized gravity system. In this figure each house has its own septic tank for pretreatment. The effluent from the septic tank flows by gravity (or through a pump) to a sewer line in the street that conveys the sewage (also by gravity) to the community sewage dispersal and/or treatment system. Systems of this type are often referred to as Septic Tank Effluent Gravity (STEG) or Variable Grade Sewer (VGS) systems. If a pump is used at all, its purpose is to deliver the effluent to a flooded portion of the gravity flow line. Where the gravity flow line is flooded it may be slightly pressurized depending upon the extent of the flooding condition. The pump is not intended, however, to further pressurize the entire system as is does in a STEP system where the pumps in the aggregate collectively provide the energy to move the effluent to the community system.

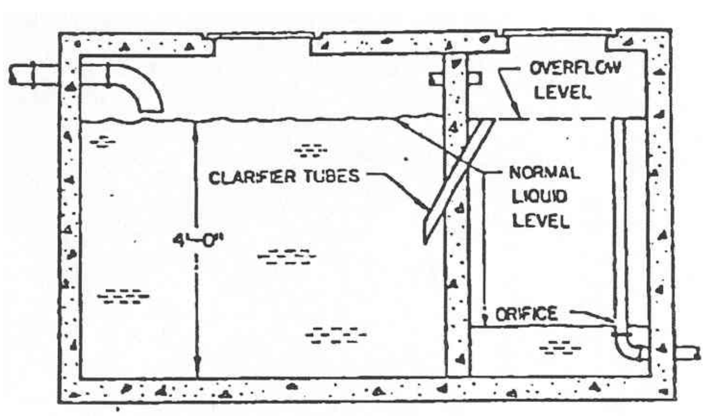
Figure 24 Decentralized Gravity System (STEG/VGS)



Although standard septic tanks can be used with STEG/VGS systems it is recommended that as a minimum the septic tanks have two compartments with effluent filters or upflow clarifier tubes mounted between the first and second compartment. Additional modifications to standard septic tanks have been recommended to provide some flow equalization. Figure 25 illustrates the use of several of these modifications:

- Upflow Clarifier Tubes reduce solids carryover into second compartment.
- Low Level Outlet allows a much of the second compartment to be used for flow equalization and surge suppression.
- Low Level Orifice into discharge line meters flows into community collection system reducing peak flows.
- Overflow Discharge line extended up to maximum design working level of tank. Surges and sustained high flows beyond the hydraulic capacity of the orifice will cause the effluent in second compartment to rise possibly to the point of overflow into the open vertical end of the discharge line.

Figure 25 Modified Septic Tank for STEG/VGS (Simmons & Newman 1982)



The configuration suggested above could reduce the peak flows allowing the use of smaller community gravity lines. Such a configuration would only be economical for new construction where septic tanks modified to this specification could be built and installed in large quantities. It is also worth noting that this configuration will require that the tanks be set at higher elevations considering the requirement that the tank drain down to the level of the orifice.

Differences in Design Philosophy Relative to Conventional Sewage Collection Systems

STEG/VGS sewer systems, while still relying upon gravity to move the effluent along are different in several significant ways from the more conventional municipal sewer system. The shallower depths used in STEG/VGS systems result in highly reduced excavation requirements. The smaller diameters used in STEG/VGS result in a reduced need for mechanical cleaning (pigging) line. Pneumatic cleaning of lines is facilitated in which air or water is used to blow debris out of the lines.

Figure 24 above shows a section of a STEG/VGS system with includes the use of an inverted siphon. Inverted siphons are relatively short segments of pipeline with dips or low sections with invert elevations below the invert elevations of the upstream and downstream sections to which it is connected. This results in pipe segments with positive or neutral slopes. The design of the pipe line can follow natural contours up and down to some extent. Any connection to the STEG/VGS line along an inverted siphon section must originate from a discharge point at an elevation high enough to be above the hydraulic grade line or have an individual pump to inject the effluent into the line.

The conveyance of partially clarified effluent has fewer or less stringent design constraints than does the design of municipal sewage intended to carry raw sewage. Among the significant differences in the design philosophy of STEG/VGS systems include less concern about minimum velocities. With the absence of most of the settleable solids typically found in wastewater a STEG/VGS system can be designed with less concern for maintaining velocities high enough to keep solids entrained in the flow. This is particularly true for those sections of the STEG/VGS system with a constant negative (downhill) slope. For those sections of the STEG/VGS system that have low areas (see inverted siphons above) there is still the possibility of solids collecting in the low point of the pipe so care must be taken to insure that velocities of at least 2 ft/sec are achieved during peak flow periods. Additionally, due to the absence of dense settleable solids such as sand and gravel which are often present in conventional sewer lines less expensive materials with less scour resistance may be used.

Differences in Design Philosophy Relative to STEP Systems

STEG/VGS systems may be a more cost effective solution to effluent transfer in areas where a predominant downhill path can be developed. STEG/VGS systems differ from STEP systems in the following ways:

Pumps

The pump, wet well and control system which are required for every connection to a STEP systems are not needed. Some connections may need a pump if they are located near an inverted siphon and the elevation of the home is below the hydraulic grade line of the line at the point of connection.

Grades

STEG/VGS systems must have a predominantly downhill layout. Although short areas may have positive slopes, gravity is still the prime mover of the effluent and a negative pipe slope is necessary for the continuous downhill flow.

Diameter

The STEG/VGS system will generally have larger diameter pipes compared to STEP systems but still have smaller diameter pipes when compared to conventional municipal sewer systems.

Flow Equalization

STEG/VGS or VGS systems can use simple stand pipes and orifice holes to even out peak flows to some extent. This may enable the use of lower peak flows for overall system design depending upon the base assumptions used in the community. Some experts recommend using a flow of 0.6 gpm per residence plus a constant of 10 gpm in situations where no flow equalization results from either the pre-treatment (septic) tank or a stand pipe and orifice. When flow equalization is available they recommend 0.4 gpm per residence. These recommendations are based upon an assumed overall average flow rate of 0.1 gpm that is closely equivalent to 150 gallons/day. (Simons and Newman 1982) STEP systems, on the other hand, can actually generate spike flows that must be balanced out statistically and/or hydraulically by other connections along the system.

Overview of Gravity Conveyance Hydraulics in Onsite & Decentralized

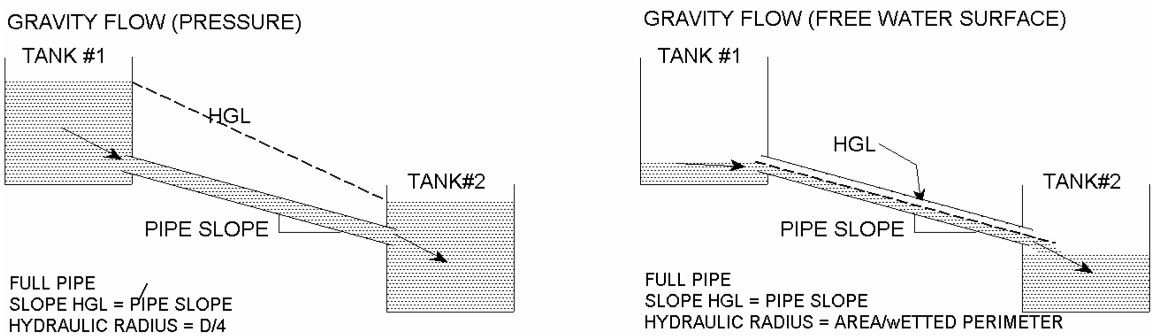
In gravity flow onsite systems, gravity is the only driving force for moving the sewage and treated effluent through the system. As will be seen later, this puts several limitations on the design but results in a simpler system to operate requiring no external sources of power and minimizing maintenance requirements. Gravity conveyance most often refers to open channel flow in which earth's gravitational field provides the force and energy necessary to overcome friction in the pipes. In such cases there is generally a free water surface exposed to the atmosphere or at least at atmospheric pressure.

In this module we will expand the concept of gravity conveyance to include systems in which earth's gravitational field provides the force and energy necessary to overcome friction in the pipe but where there can be pressure flow in closed conduits. Gravity systems therefore include all systems which do not have pumps. The concept of the hydraulic grade line is useful for illustrating the differences between these two gravity flow conditions.

Figure 26 illustrates two basic conditions for gravity flow. On the right can be seen a system flowing by gravity in which the pipe is flowing partially full and the free water surface coincides with the hydraulic grade line.

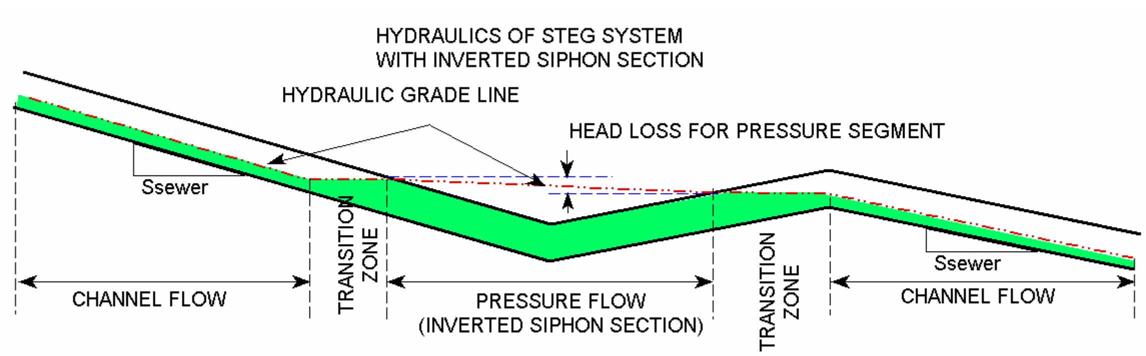
On the left can be seen a system flowing by gravity in which the pipe is flowing completely full and the free water surface does not coincide with the hydraulic grade line.

Figure 26 Gravity Flow Conditions



STEG/VGS systems can flow in either of the conditions shown above. As the sewer pipe's grade and elevation follow the natural contours there are sections of the pipe which flow full under slight hydrostatic pressure and there are areas which flow with a free water surface as open channels. We have referred to this condition as an inverted siphon Figure 27 illustrates this situation.

Figure 27 Flow Types In STEG/VGS Systems.



Important hydraulic considerations for gravity flow

Slope

If the pipe can be assumed to flow as steady uniform open channel flow with a free water surface it is the slope of the sewer which dictates the amount of energy available to overcome the friction in the pipe. Steady flow indicates that the flow quantity is not changing with time and uniform flow indicates that the flow velocity is not changing along the pipe. If the pipe is flooded, flowing full, and under slight pressure from the upstream head which has built up it is the slope of the hydraulic grade line which dictates the amount of energy available to overcome the friction in the pipe. In conventional gravity flow systems as well as STEG/VGS systems the designer attempts to develop a system layout which follows the natural topography in a generally down hill direction. If the slope of the local topography is nil or runs the opposite direction of the desired flow direction either excavation and lift stations will be necessary to result in a consistent down (hydraulic) gradient or the designer may take advantage of relatively short sections with positive gradient and create inverted siphons.

Diameter

The diameter affects the ratio of the wetted surface of the pipe which is in contact with the liquid and the interior area of the pipe where less friction is encountered. In general as the pipes diameter increases less resistance to large flows is encountered. However, when large diameter pipes are carrying very low flows the situations reversed and more friction is encountered as the flow is carried in only a small fraction of the pipe's bottom where there is large contact with the bottom of the circular pipe.

Roughness

Different pipe materials offer different resistances to fluid flow. Sewer pipes range from concrete, clay, and ceramic to plastic materials and in some cases metal. Each material has a different roughness factor which must be taken into consideration.

Velocity (min & max)

Large municipal sewer systems, small STEG/VGS systems, as well as individual onsite systems can flow with large variations of velocity. It is desirable to maintain sufficient flow velocity during low flow periods or during the initial phase of a projects life when only a few homes are connected so that there is no buildup of settled solids collecting on the bottom of the pipe. There are also concerns about high velocities resultant from intermittent high flows or due to the systems design. High velocities can result in physical abrasion of the pipes due to solids carried in the flow and it is claimed by some plumbers that high fluid velocities in small household lines can result in clogging due to solids being left behind and building up in the line. Although this idea runs counter to the accepted concepts of sedimentation and fluid drag the claim is still made and forms part of the basis for limiting velocities in gravity flow lines.

Fundamentals of Gravity Conveyance Hydraulics

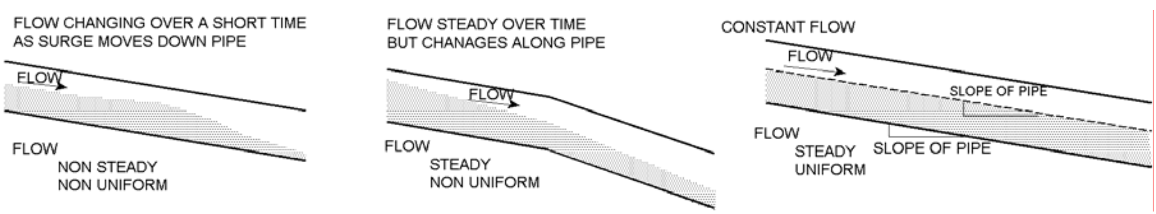
A variety of hydraulic analysis tools are available to help the designer size and analyze pressure flow and gravity flow conveyance systems. In the pressure flow portion of this module we discussed the use of the Hazen Williams formula, the Darcy Weisbach formula and the Chezy formula. For gravity flow situations the Manning's equation is generally used although the Chezy formula is occasionally used to estimate the maximum gravity flow discharge for a pipe flowing full but not under pressure.

Manning's Equation for Analyzing and Designing Gravity Flow Systems

Both community and onsite gravity flow wastewater lines are generally designed to flow as open channels without pressure. The water (whether raw sewage from the source or treated effluent) flows downstream in the pipe by gravity. The velocity of the flow generally depends on the slope and friction resistance of the pipe.

The Manning equation is most often used for determining flow or velocity in open channels. The use of the Manning equation assumes steady and uniform open channel flow. For small-scale systems where the flows are intermittent and come in pulses, the assumptions of steady uniform flow are not always valid. Figure 28 illustrates some of the flow conditions that can be present in a gravity flow sewer line. The left segment of this figure illustrates a short surge of effluent moving down the pipe. This flow is neither steady nor uniform. The analysis of such a flow is complicated and beyond the scope of this module and is generally not considered in the design of gravity flow sewers. The middle segment shows the drawdown of the water surface as a steady flow approaches a change in slope. While possibly steady (if the flow through the pipe is constant) it is not uniform and continuity dictates that the velocity is changing as the depth changes. The right segment shows steady uniform flow in which the flow neither changes with time nor location along the pipe. In this third case the slope of the pipe equals the slope of the free water surface that coincides with the hydraulic grade line. The use of Manning's equation assumes that this condition is most representative for design.

Figure 28 Common Open Channel Flow Conditions



Manning's equation enables the designer to estimate flow depths and velocities for the average and extreme range of discharges expected. Often times large flows may result in velocities which are high enough to cause scour in the pipes when dense materials (ex. sand) carried in the wastewater abrade the sides of the pipes. At the other extreme, small flows may result in velocities that are so low that solids settle out of the flow and are deposited on the bottom of the pipe where they can accumulate, congeal and form blockages. Both short duration high flows as well as low flows are used as constant design flows.

$$Q = \frac{1.486 \times R^{2/3} \times S^{1/2} \times A}{n}$$

Q = Flow or discharge in cubic feet per second
n = Coefficient of roughness or friction factor

A = cross-sectional area of flow in square feet
R = Hydraulic radius in feet
S = Slope of the hydraulic gradient in feet per foot (must equal the slope of the pipe because of the assumption of steady flow).

Knowing that the velocity in the pipe is equivalent to the flow (Q) divided by the flow area (A) enables Manning's formula to be used directly for computing velocity as well.

$$V = \frac{1.486 \times R^{2/3} \times S^{1/2}}{n}$$

The friction is based on the pipe material, the pipe condition and the jointing method. Many texts display tables of Manning roughness coefficients based on materials and condition. Table 1 provides some commonly used Manning's coefficients.

Use of Manning's Equation in Pressure Flow Situations

Previous portions of this module suggested the use of the energy equation and Darcy Weisbach's formula for friction losses to analyze pipes flowing full under pressure. This is one of the most common approaches. Manning's equation however can also be used for pipes flowing full under pressure if two significant considerations are addressed:

- The Hydraulic Radius for a pipe flowing full = D/4. The definition of the hydraulic radius easily enables the simple substitution of D/4 for the hydraulic radius of circular pipes flowing full.
- The slope used in Manning's equation is, formally, the slope of the energy grade line that is equivalent to the slope of the hydraulic grade line if the velocity remains constant over the sewer section being considered.

Therefore, Manning's equation can be conveniently used for computing flows through an inverted siphon section if D/4 is used for the hydraulic radius and the anticipated hydraulic grade line (HGL) is used for the slope in the formula.

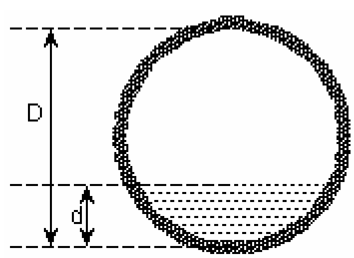
Table 1 Roughness Coefficient, Manning's N

channel material	n
clean, uncoated cast iron	0.013–0.015
clean, coated cast iron	0.012–0.014
dirty, tuberculated cast iron	0.015–0.035
riveted steel	0.015–0.017
lock-bar and welded	0.012–0.013
galvanized iron	0.015–0.017
brass and glass	0.009–0.013
wood stave	
small diameter	0.011–0.012
large diameter	0.012–0.013
concrete	
with rough joints	0.016–0.017
dry mix, rough forms	0.015–0.016
wet mix, steel forms	0.012–0.014
very smooth, finished	0.011–0.012
vitrified sewer	0.013–0.015
common-clay drainage tile	0.012–0.014
asbestos	0.011
planed timber	0.011
canvas	0.012
unplaned timber	0.014
brick	0.016
rubble masonry	0.017
smooth earth	0.018
firm gravel	0.023
corrugated metal pipe	0.022
natural channels, good condition	0.025
natural channels with stones and weeds	0.035
very poor natural channels	0.060

Hydraulic Analysis of Partially Full Pipes

Manning's equation is often used for evaluating gravity flow pipes flowing partially full. Manning's equation requires the determination of the hydraulic radius, which is the cross-sectional area, divided by the wetted perimeter. Determining the hydraulic radius for circular pipes following partially full can be difficult do the geometry relating to the hydraulic radius and wetted perimeter for circular pipes flowing partially full.

Figure 29 Partially Full Pipe

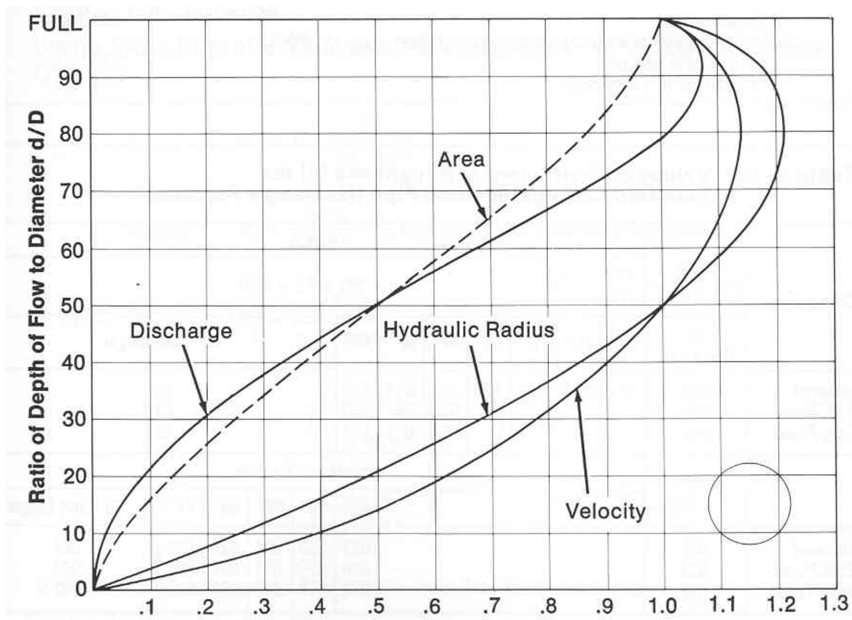


Fortunately, Figure 30 can be used to solve a variety of unknowns based on the ratio of the flow depth to the pipe diameter. The chart is based upon precompiled ratios between the hydraulic properties of a pipe flowing full to the same pipe flowing less than full. The vertical access represents the single ration of d/D, the actual flow depth, d, under the given conditions to the full diameter, D. The 4 plotted curves relate to corresponding ratios of:

- Actual flow to Full Flow, q/Q,
- Actual conveyance area to the full pipe cross-section area, a/A,
- Actual hydraulic radius to the flowing full hydraulic radius, h_r/H_r, and
- Actual velocity to the full flow velocity, v/V

The chart can be used in a variety of ways depending upon the purpose of the analysis.

Figure 30 Hydraulic Properties of Circular Pipe (AISI 1980)



It is interesting to note that greatest flow occurs when the pipe is flowing at 93percent of its maximum depth and the greatest velocity is when the pipe is flowing at 80% its maximum depth.

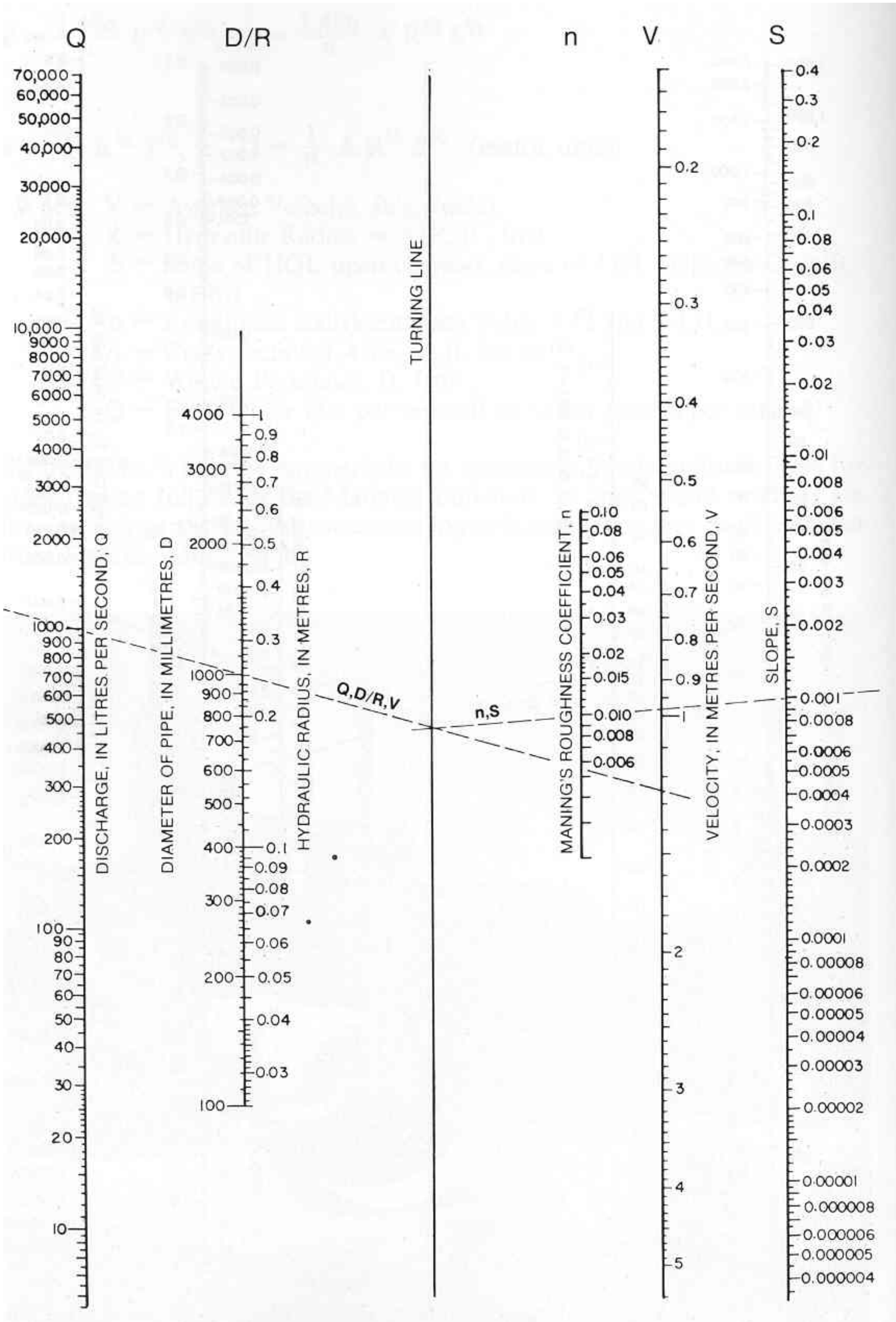
Overview of Design Process for Conventional Sewage Collection

Centralized sewer systems most often depend upon extensive gravity flow sewer systems converging through a dendritic pattern to the centralized sewage treatment plant. Therefore, the analysis and design of gravity flow sewer systems is a highly developed field and is covered extensively in several civil engineering texts.

Manning's Equation Solution Nomographs

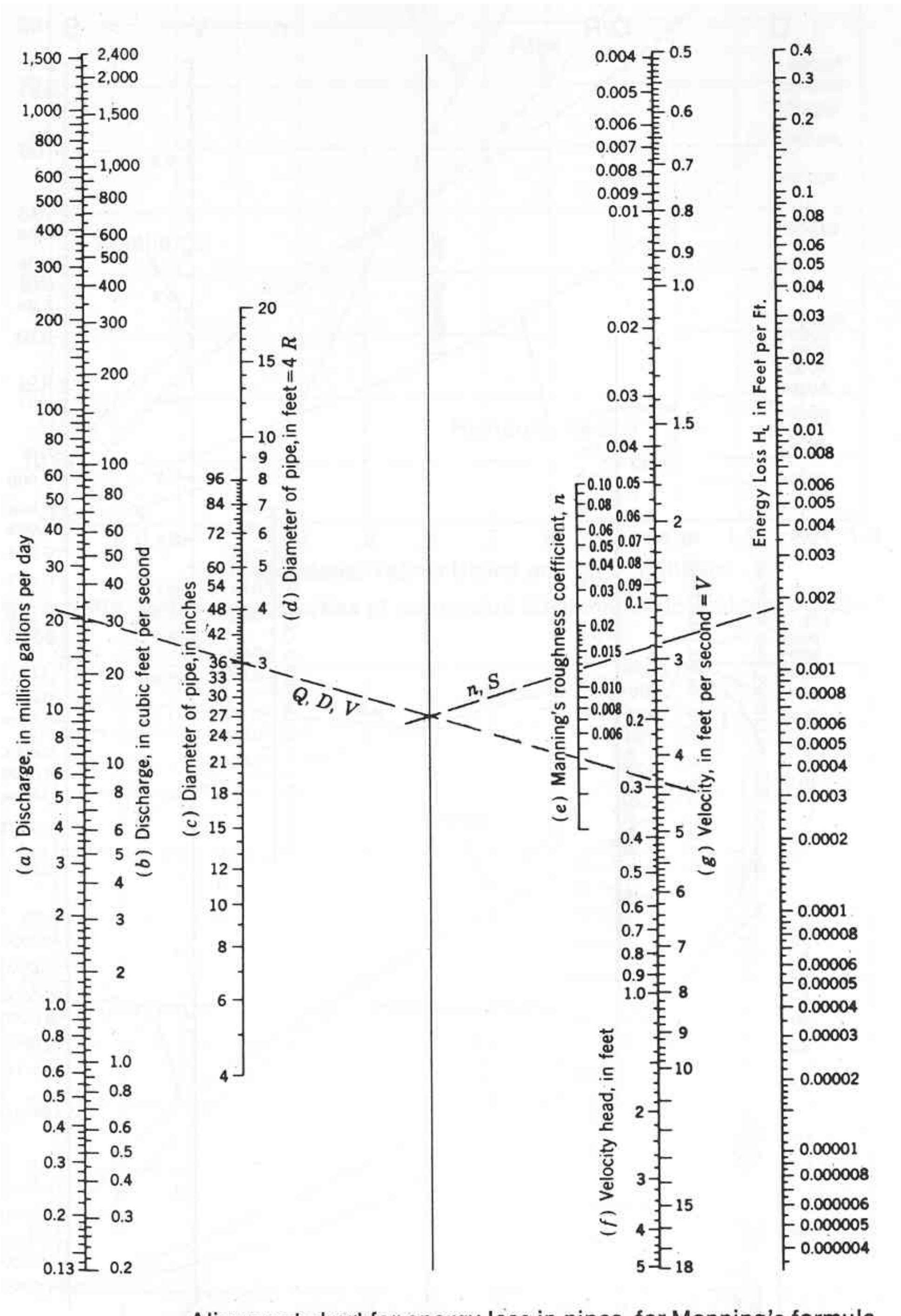
Manning's Equation Nomograph – Metric

Figure 31 Nomograph for Solution of Manning's Formula SI Units (AISI, 1985)



Manning's Equation Nomograph – English

Figure 32 Nomograph for Solution of Manning's Formula English Units (AISI, 1985)



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Applications of Gravity Conveyance in Onsite & Decentralized

Gravity Flow in Individual and Small Onsite Systems

Gravity dispersal systems are used to disperse treated wastewater back into the environment through downward infiltration followed by percolation or upward to the atmosphere through evapotranspiration. Many different designs and configurations are used and it is not the intent of this chapter to discuss the variations in design alternatives, but to give an overview of the gravity processes of getting the effluent to the dispersal area. For systems using infiltration the subsurface environment the effluent is discharged on or over the natural soil bottom of a dispersal trench. For systems using evapotranspiration to distribute the wastewater, the water is discharged into clay lined or synthetic lined bed or trench specifically designed to enhance capillary rise and evapotranspiration.

This section will concentrate on percolating systems. For percolating systems the gravity distribution system is located in permeable, unsaturated natural soil or imported fill material so the pretreated wastewater can infiltrate and percolate through the underlying soil to the ground water. A trained soil scientist, engineer or sanitarian determines the appropriate soil application rate. The primary infiltrative surface may be the bottom of the excavation, the sidewalls below the distribution pipe or both the bottom and sidewalls of the excavation. The determination of the infiltrative surface varies from region to region and is predominantly controlled by state environmental quality offices, county health departments or possibly sanitary districts. There is ongoing discussion about which portion of a trench is most critical for design. Some would say that using the sidewall assumes that the trenches are flooded and they are, therefore, not operating as ideal treatment and dispersal devices.

Gravity flow can be used for the dispersal of effluent where there is sufficient elevation difference between the treatment outlet and the disposal plumbing. Gravity flow systems are simple, passive and inexpensive, but are the least efficient method of distribution. Although conveyance to the dispersal trenches by gravity is relatively predictable, distribution within the dispersal trenches is uneven over the infiltration media. This can results in localized overloading.

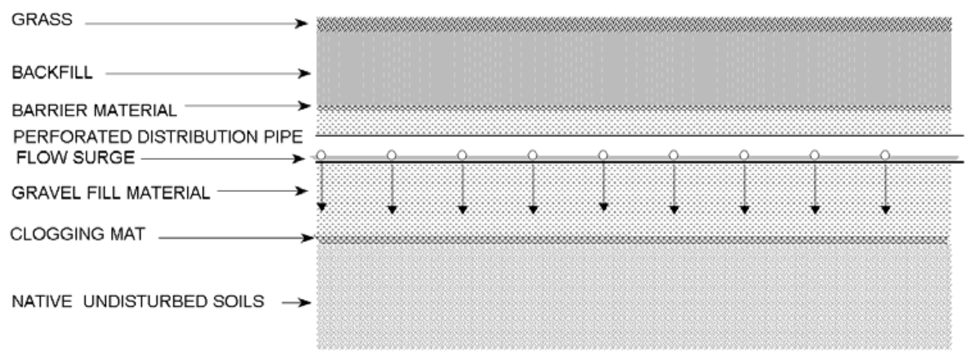
Trenches should be laid out along contour lines. The horizontal alignment of the trenches need not be straight and using PVC pipe or other flexible pipe the trenches and their laterals can be curved to fit contours and avoid trees. On sloping lots with multiple trench systems distribution boxes or drop boxes can be utilized to adequately disperse the wastewater between trenches. A discussion of these devices follows.

As a biomat forms on the infiltration surface it clogs the soils and slows down the draining of the soil, forcing the effluent to flow through the media of the trench until it reaches an unclogged surface. This occurs until the entire trench or bed is ponded and the sidewalls become the infiltration surface. Without extended periods of resting the biomat clogs more and more of the infiltration surface until hydraulic failure occurs. This may take decades for some soils and months for others.

Gravity Flow Perforated Pipe Used In Onsite Subsurface Soil Absorption Trenches.

Perforated pipe is still the most common piping material installed to distribute the wastewater into the distribution system. Gravity leach pipe is usually 3 or 4-inch {7.62 cm or 10.2 cm} perforated polyvinyl chloride (PVC), flexible corrugated polyethylene (PE) or acrylonitrile-butadiene-styrene (ABS) pipe. The perforations are usually ½ inch {1.27 cm} orifices placed 60 degrees up on each side of the flow line of the pipe and spaced 12 inches {30.5 cm} apart. The pipe is constructed in this manner to allow treated effluent to pond up within the pipe to the orifice opening and evenly discharge to the leach trench. Figure 33 illustrates a gravity flow lateral in an idealized typical trench.

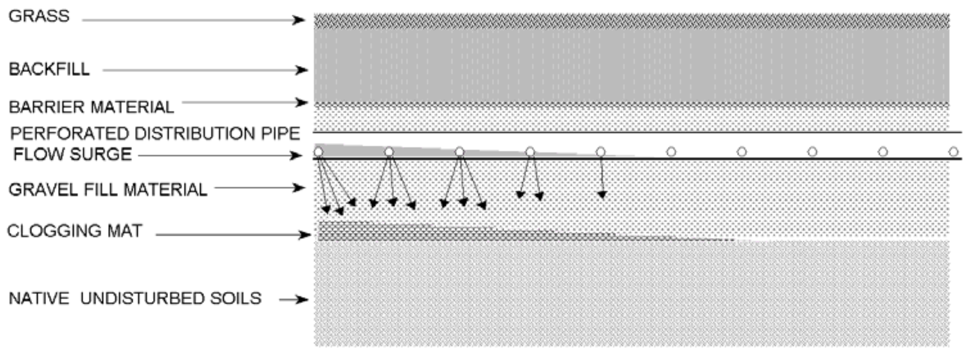
Figure 33 Idealized Trench with Uniform Flow Distribution



In this scenario, the effluent is discharged uniformly down the length of the pipe. The effluent drains through the gravel fill material where additional treatment similar to what takes place in a trickling filter may take place. When the effluent reaches the native undisturbed soils, which, ideally, are finer materials, the organic content of the partially treated effluent becomes food for the local microbes. As the microbes consume the residual organic material a clogging bio-mat is formed on and in the first layer of the native soils. This bio-mat adds to the filtration of the effluent but most often also decreases the local absorption into the soils. As one area becomes less penetrable to the effluent, the effluent will flow to less restrictive areas by sheet flow similar to water spreading out on a somewhat horizontal surface finding low spots and avoiding high areas.

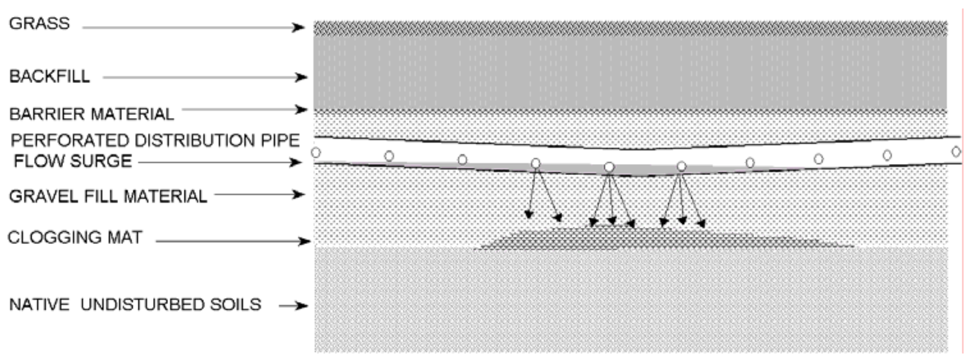
Even under the best of circumstances in which the pipe is installed absolutely level flow non-uniformities will result from the intermittent surges of water that are typical of small onsite systems. Figure 34 illustrates both the non uniform distribution of effluent to the gravel fill material but also the non uniform establishment of the clogging mat.

Figure 34 Expected Flow Distribution in Well Constructed Trench



Unfortunately, often do to less than perfect installation, unleveled gravel beds, crowning or sagging of the pipe resulting from local conditions at the time of installation, the actual discharges between orifices is unpredictable. Figure 35 illustrates the conditions actually expected in gravity flow distribution laterals.

Figure 35 Uneven Flow Distribution in a Sagging Pipe



Despite the less than total control of the distribution provided by the gravity flow distribution pipe, gravity flow distribution is still the most common method of effluent dispersal. The pipe is placed within a porous aggregate fill material. Fortunately the gravel fill aggregate material placed in the trench not only supports the pipe and backfilled material over the trench but also enables the spreading of the localized flow from the distribution pipes across the entire excavation. In some areas the gravel fill aggregate is referred to as “distribution rock” recognizing its role in promoting the lateral movement of the effluent. Effluent accumulations or ponding in any one area of the bottom of the trench will tend to move laterally as sheet flow to lower areas where ponding has not yet occurred.

Dispersal Trenches Considerations

Though both the sidewalls and the bottom of the trench may act as infiltrative surfaces (when the trenches are flooded), many design guidelines call for the area of the drain field to be based only on the area of the bottom of the trench. For example, the infiltrative area of a 2-foot wide trench that is 60 feet long would be 120 square feet if the sidewalls are not considered. When gravity systems are first installed and initially dosed, the bottom of the trench acts as the primary infiltrative surface. After continued application of wastewater, the bottom surface can become sufficiently clogged to pond liquid above it. At this point the sidewalls of the trench may become infiltrative surfaces as well.

Individual trenches are constructed as shallow, reasonably level excavations from 1-3 feet deep and 1-3 feet wide. The bottom of the trench is filled with approximately 6 inches of washed, crushed rock or gravel over which the distribution pipe is laid. More gravel is placed over the pipe, the gravel covered with a geotextile or other semi-permeable barrier or to prevent silts, clays from backfilling operations from penetrating the gravel layer, and potentially clogging pore spaces or pipe openings. The trench is then backfilled to grade with soil.

During construction of trench systems care should be taken to maintain and protect the infiltrative properties of the soil. In particular, one should avoid smearing and or sealing of the surfaces on the bottom and sides of the trench. This can normally be avoided by not digging when the soil is wet enough to smear or compact easily. Any unavoidable sealing of the soil can be amended by raking the soil to a depth of approximately 1 inch and removing the loose material. Local codes may specify minimum trench separation distances as well as trench width, slopes, depths of gravel and other fill materials, depths to groundwater or bedrock and allowable barrier materials. General guidelines recommend that the trenches should be uniformly sloped from 0-4 inches per 100 feet and spaced at least 8 feet apart from center to center. Always check local regulations for applicable design criteria.

As a biomat forms on the infiltration surface it clogs the soils and slows down the draining of the soil, forcing the effluent to flow through the media of the trench until it reaches an unclogged surface. This occurs until the entire trench or bed is ponded and the sidewalls become the infiltration surface. Without extended periods of resting the biomat clogs more and more of the infiltration surface until hydraulic failure occurs. This may take decades for some soils and months for others.

Distribution Boxes and Manifolds

Effluent can be distributed to the disposal trenches in a variety of ways. In general the gravity flow hydraulics of these systems can be either serial loading or parallel loading. Serial systems are constructed using tees and other fittings; drop boxes, and alternating valve systems. Parallel systems are constructed using distribution boxes or manifolds. A distribution system has three parts:

1. From pretreatment (septic tank) to the distribution device
2. From the distribution device to trenches (soil dispersal trenches)
3. Dispersal within the trenches

To allow even distribution of the treated effluent to each of several dispersal trenches various devices have been developed. Such devices include:

1. distribution boxes (d-boxes)
2. manifolds

Distribution Boxes for Parallel Trench Loading

Distribution boxes, or D-boxes, are used to divide the incoming flow among multiple distribution lines. Distribution boxes can also be used to take individual trenches out of service by blocking the outlet to the particular line. They are typically shallow, flat-bottomed, watertight structures with a single inlet and multiple outlets. D-boxes are typically constructed of plastic or concrete. Access is typically provided for cleaning and adjustment of weir elevation if applicable. Equal distribution of trickle flows from a septic tank to a distribution box is difficult to achieve do to:

1. Placement on improperly leveled base.
2. Uneven settlement of D-box and base material
3. Unequal flow hydraulics within the D-box
4. Uneven growth of biological material at the flow lines of the exit ports
5. Surface tension effects causing sporadic and unpredictable trickle flow paths.

All outlets are at the same elevation, typically 1 to 2 inches below the inlet. The d-box must be laid level on a stable footing to divide the flow evenly among all outlets. Uneven settlement or frost heave will result in unequal flow to lateral lines because the outlets are no longer level. Several manufacturers now make d-boxes with adjustable box leveling, adjustable outlets, or have adjustable weir controls on the outlets. Figure 36 shows a plastic “D” box used for distributing flow to several trenches Figure 37 shows the Dial a Flow system (Courtesy of American Manufacturing) that has an adjustable eccentric circular weir insert commonly used in new and retro-fit installations to allow adjustable outlet control for a “D” box. As the circular plate is rotated the bottom of the off center circular hole moves up or down. This allows the adjustment to correct for uneven flows between outlets.

Figure 36 Plastic “D” Box

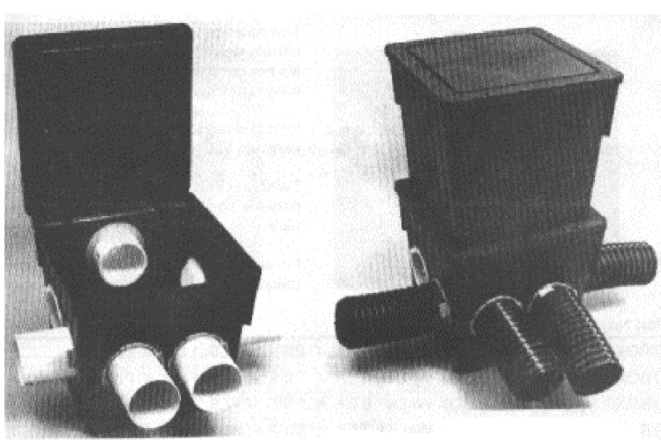
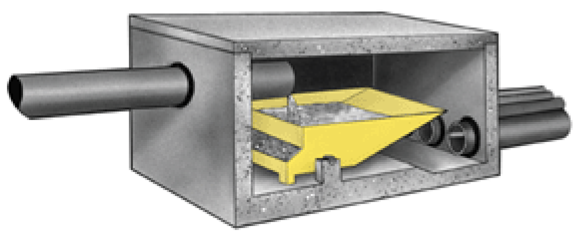


Figure 37 Adjustable Weir Control

It should be noted that distribution boxes do address equal distribution from each outlet of the box, but do not address equal distribution along the trenches. Another variation of a distribution box is the tipping bucket distribution system illustrated in Figure 38. The concept is to “pulse” or “charge” the outlets so that the same volume leaves each outlet with each pulse.

Figure 38 Tipping Bucket Distribution System



Manifolds are simply lengths of larger or equal diameter pipe with “T” fittings spaced to match the spacing of the trenches. Such manifolds should be designed as symmetrically as possible with ideally no more than two branches at each stage of the manifold system.

In areas where there is little slope, i.e. flat areas no d-boxes are necessarily used. Instead a pipe network consisting of a header pipe and parallel distribution lines are used. This configuration is similar to those used in bed type systems, but each lateral has an individual trench.

Serial Distribution

Serial distribution is an alternative to parallel distribution in soil dispersal systems. It is useful in situations where slopes are steeper than suitable for evenly distributed systems. Some characteristics of the operation of serially distributed systems are:

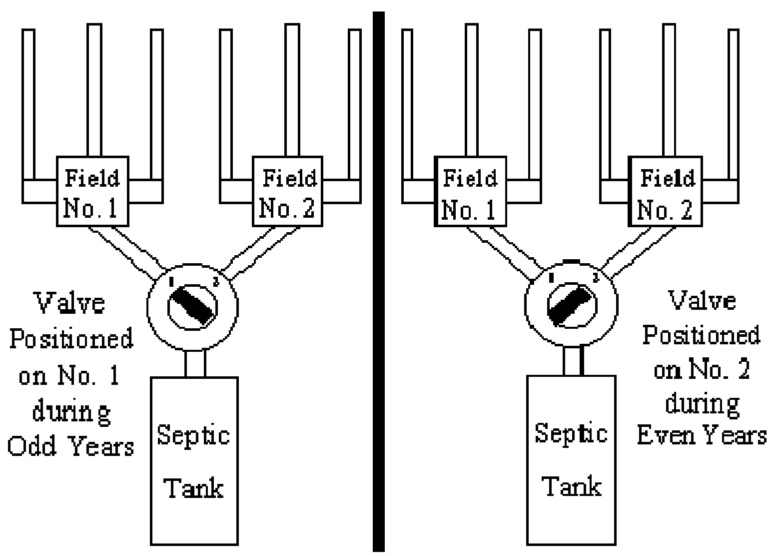
1. Serial systems form a biomat in first trench fairly rapidly

2. Lower trenches are used in wet weather and under relatively high loading conditions.
3. Upper trenches rejuvenate in dry weather
4. If you have true serial distribution, flow goes full length of the trench(es) in service

Items 2 and 3 are enhanced with the use of diversion valves. With diversion valves, individual lines or entire sections of drain fields can be "rested" as necessary. Figure 39 illustrates one such diversion valve knows as a "Bull Run Valve" (Courtesy of American Manufacturing) and Figure 40 illustrates the use of a diversion valve in a dispersal system.

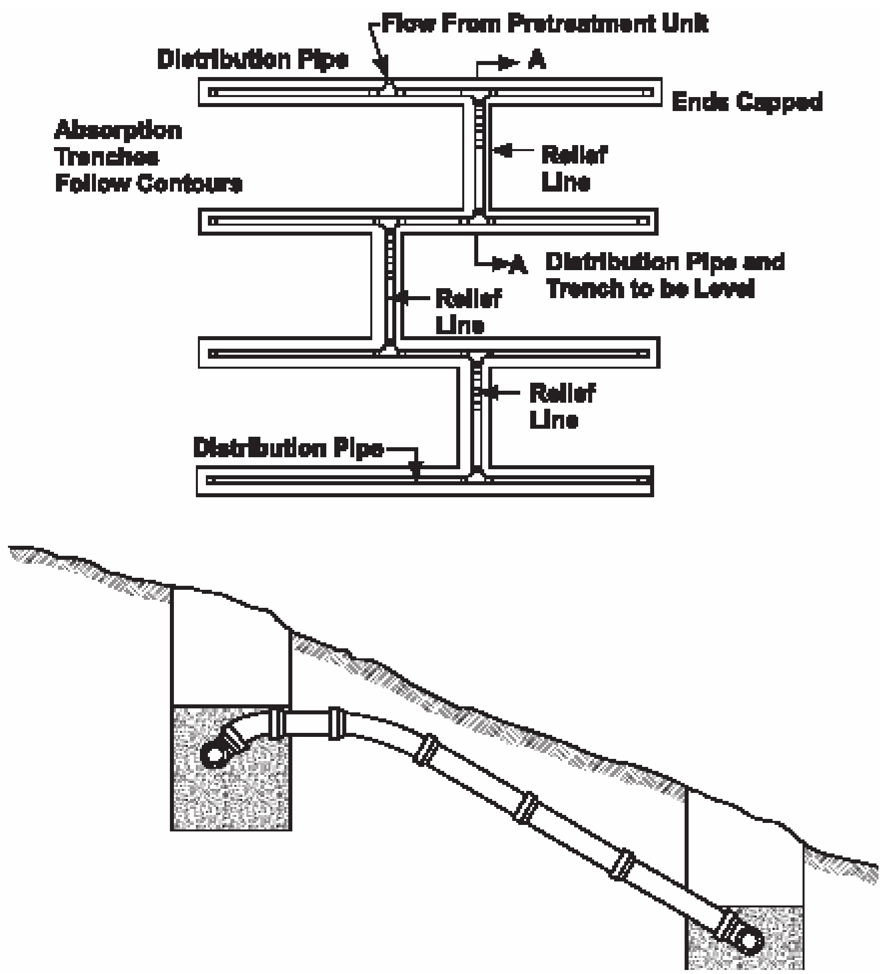
Figure 39 "Bull Run Valve" (American Manufacturing)

Figure 40 Use of a Diversion Valve with Parallel Trenches



Serial distribution distributes wastewater to a series of trenches on a slope. Instead of dividing the flow equally among the trenches as in a system using "D" boxes, the highest trench is loaded until completely flooded before the next (lower) trench receives effluent and so on down slope. This loading method is typically accomplished by using serial relief lines installed between successive trenches. This is illustrated in Figure 41. These lines act as overflow lines connecting one trench to the next lower trench. Successive relief lines should be separated 5 to 10 feet in order to avoid short-circuiting. A theoretical advantage to these type systems is that they make full use of all infiltration surfaces in the trenches and create maximum hydrostatic head over the bottom infiltration surfaces to force the effluent into the soil. It does not matter whether even distribution between successive trenches is obtained. As infiltrative capacity in the upper trenches is reduced, effluent progresses to the next trench in line. However, because “ponding” in the trenches is necessary for proper function, hydraulic failure occurs in some instances. This type of distribution can also be accomplished with serial tees. In other words, tee fittings and sections of pipe are used to emulate the effect of the serial relief lines.

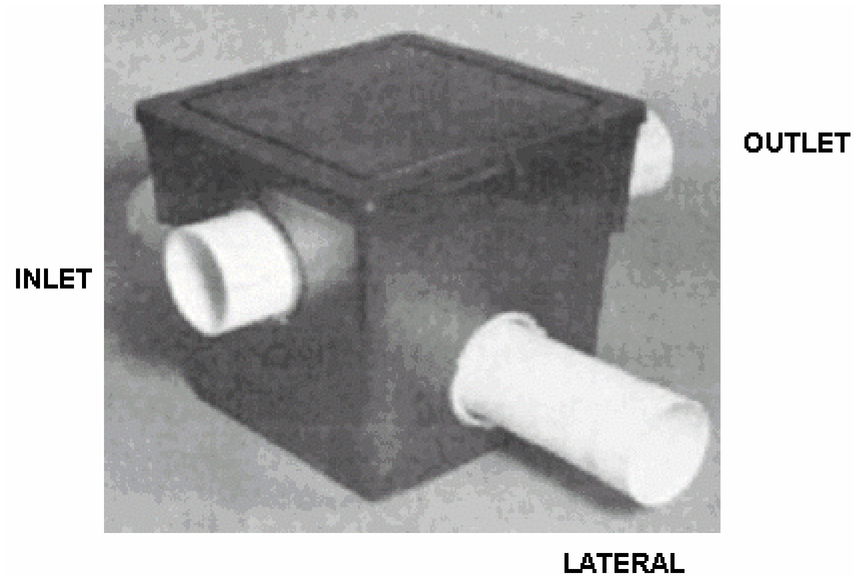
Figure 41 Serial Loading Configuration (USEPA, 1980)



Drop boxes are used in serial systems in place of relief lines. An example of a plastic drop box is shown in Figure 42 below. Drop boxes are installed for each trench. The boxes are typically connected to trenches above and below by solid piping. The outlet invert is placed near the top of each trench if serial distribution is desired in order to successively fill each trench.

Drop boxes are versatile in that individual trenches can easily be taken out of service and trenches can be added to the system more easily than with serial relief systems.

Figure 42 Drop Box for Serial Loading (American Manufacturing, 2002)



Some potential advantages of serial distribution include use of drop boxes to provide maintenance points, parts of the system rest part of the year, serial systems can be installed on sloping sites, they force early use of sidewall infiltration, can be installed in shallow soils, trench length can vary, serial systems can be expanded by just adding trenches or trench length.

Potential disadvantages include: high loading rates in top trenches until biomat forms, little or no biomat in lower trenches as they start up and a perception of progressive failure progressing from the upper trenches to lower. However, effective management techniques using drop boxes can overcome the progressive failures and other issues by resting upper trenches periodically

Gravity Flow in Decentralized Community Systems

STEG/VGS

Design Procedure for STEG/VGS System.

1. Determine basic systems requirements including the design peak flow rate, the pipeline material, proposed and its friction factor and the minimum nominal pipe size.

2. Prepare plans and profiles showing elevations. Establish stationing. Subdivide the system based upon convenient pipeline sections and logical system discontinuities including locations where branches join the system and or where changes in overall slope are anticipated.

3. For each pipeline segment determine the cumulative flow to be expected at the end of that segment. Do not design or analyze the system incrementally from one service connection to the next.

4. For each pipeline segment determine its length and change of elevation.

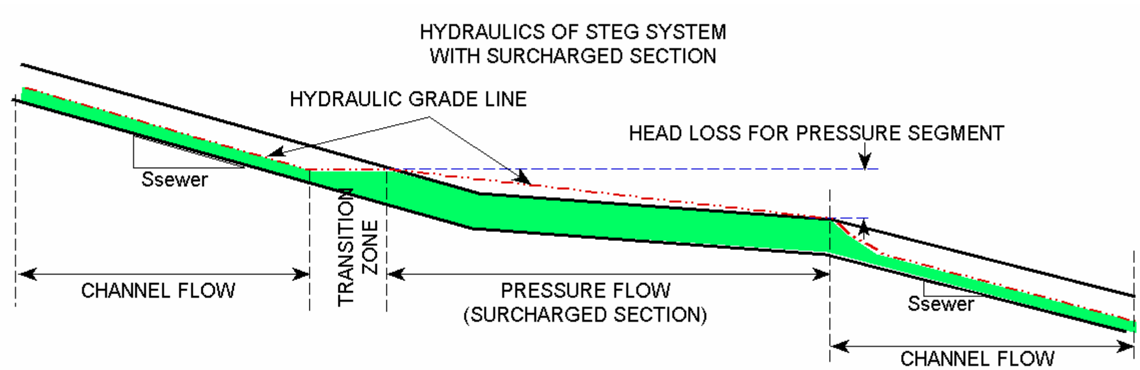
5. Compute the slope for each pipeline segment.

6. Using the preferred design equation iteratively design the pipe segment to achieve the desired balance between slope (ideally close to ground slope) and diameter.

7. If neither acceptable slopes nor diameters result in a flowing full capacity which is less than the design flow the section may flow under surcharge conditions during peak flow conditions. If surcharge conditions extend over significant lengths of the system, determine the hydraulic grade line through the surcharged section and compare to expected treatment system invert elevations at each service locations. Adjust pipe size and/or slope if necessary.

Surcharged conditions may exist in a STEG/VGS sewer system without an inverted siphon being present. A true inverted siphon implies that a middle section of pipe is physically lower than either of it ends. A surcharge condition may occur if the slope in a particular section of pipe is flanked by steeper sloping pipe segments. Figure 43 illustrates a section of STEG/VGS sewer line with a surcharged section operating under slight pressure.

Figure 43 STEG/VGS with Surcharged Section



A more detailed design procedure with worked out design examples may be found in Crites and Tchobanoglous, 1998. Table provides some typical design data for STEG/VGS.

Figure 44 Typical Design Data for STEG/VGS System

Typical design data for STEG sewer wastewater collection systems

Item	Unit	Range	Typical
Service lateral pipeline diameter	in	2.0–4.0	3.0
Collector main pipeline diameter	in	4.0–8.0	6.0
Trench depth*	in	24–36	30
Cleanout intervals†	ft	400–1000	500
Service connection discharge flow rate	gal/min	0.1–1.0	0.4

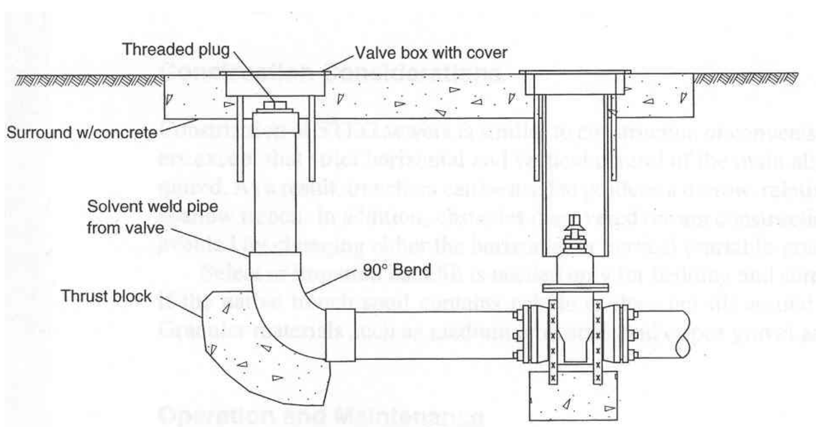
*Use frost depth in cold climate areas (when insulated or heat-traced piping not used).
†Pigging stations can be farther apart, depending on pipe size variation.

Appurtenances

In addition to the building conveyance lines and septic tank STEG/VGS may have the following additional components:

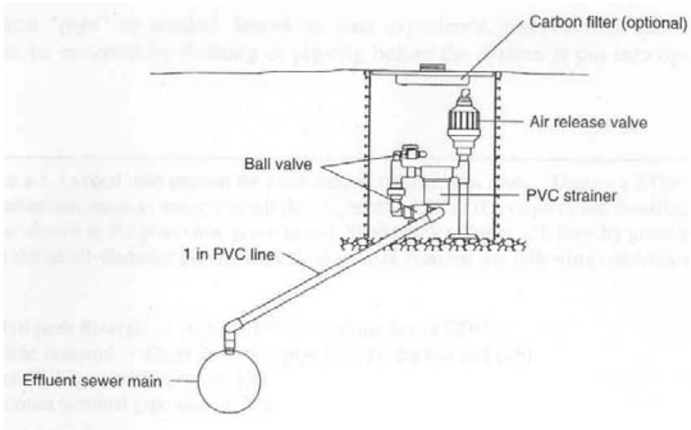
- Cleanouts access ports located at junctions on the main lines, at changes in pipe sizes, at high points, changes in direction and at standardized intervals of from 500 to 1000 feet (150 to 300 m).Figure 45 illustrates a cleanout for a STEG/VGS systems.

Figure 45 Cleanout for STEG/VGS (Crites and Tchobanoglous, 1998)



- Vents and air release valves sometimes in combination with vacuum relief valves are often located at cleanout locations.
- Odor control filters. Illustrates a valve box for a STEG/VGS system showing an air release valve and an odor control filter.

Figure 46 Air Release Valve and Odor Control Filter for STEG/VGS (Crites and Tchobanoglous, 1998)



Effluent Conveyance
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Metcalf & Eddy, Inc (Tchobanoglous, Burton and Stensel). Wastewater Engineering, Treatment and Reuse, Forth Edition. McGraw-Hill Higher Education, 2003 (page 188) Figure 1 Typical flow rates for individual residences: (a) single home, (b) average of about 5 homes, (c) average of 61 homes (b,c courtesy of Baker, 1990) (Metcalf and Eddy

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