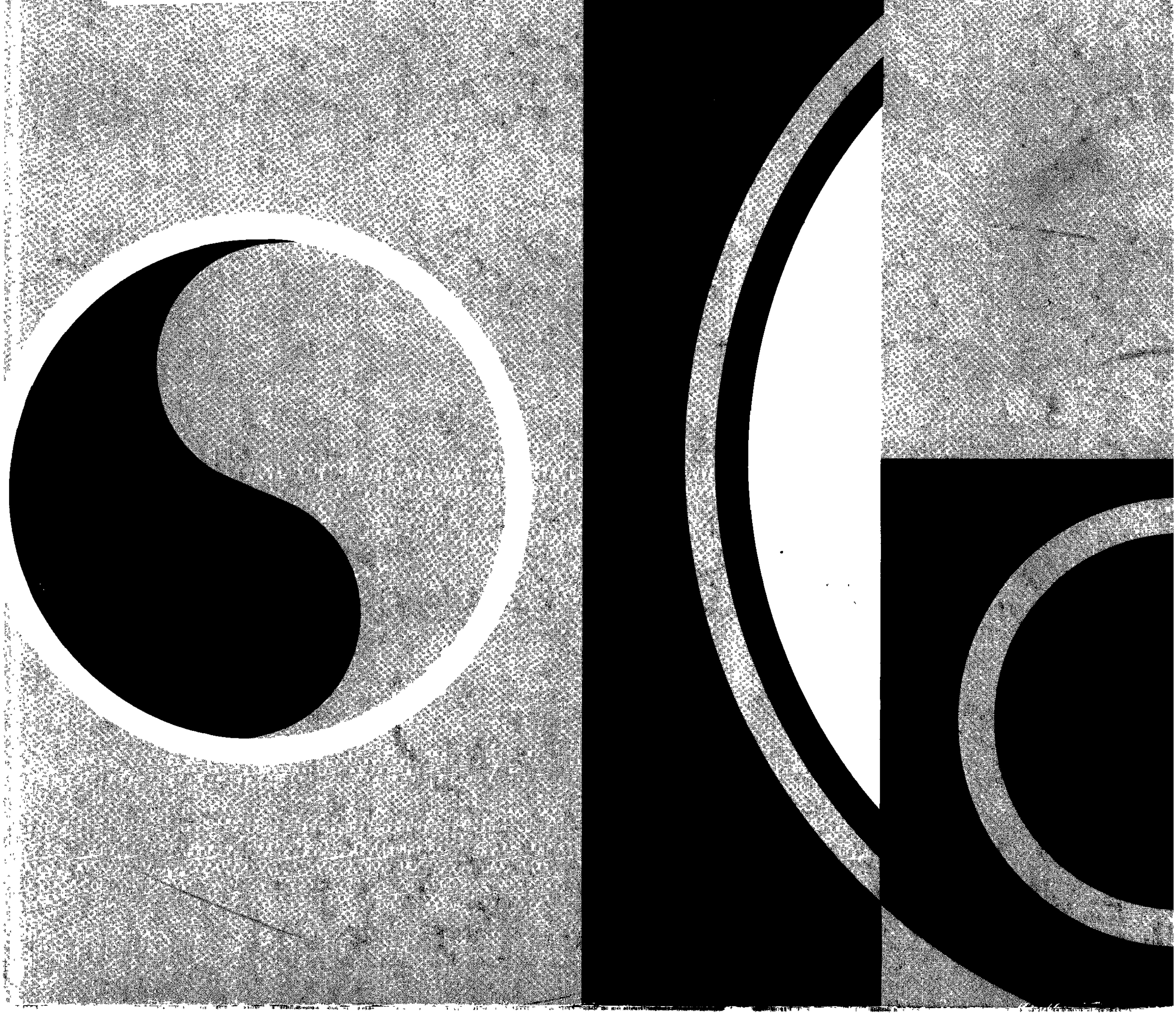
625477011B

Alternatives for Small Wastewater Treatment Systems

Pressure Sewers/Vacuum Sewers

EPA Technology Transfer Seminar Publication

This document has not been submitted to NTIS, therefore it should be retained.



EPA-625/4-77-011

ALTERNATIVES FOR SMALL WASTEWATER TREATMENT SYSTEMS

Pressure Sewers/Vacuum Sewers

ENVIRONMENTAL PROTECTION AGENCY • Technology Transfer

October 1977

ACKNOWLEDGMENTS

This seminar publication contains materials prepared for the Environmental Protection Agency Technology Transfer Program and presented at Design Seminars for Small Wastewater Treatment Systems.

Part I was prepared by James F. Kriessl, Sanitary Engineer, Municipal Environmental Research Laboratory, Environmental Protection Agency, Cincinnati, Ohio.

Part II was prepared by I. A. Cooper, Senior Project Manager, and J. W. Rezek, President, Rezek, Henry, Meisenheimer & Gende, Inc., Consulting Engineers, Libertyville, Ill.

NOTICE

The mention of trade names or commercial products in this publication is for illustration purposes, and does not constitute endorsement or recommendation for use by the U.S. Environmental Protection Agency.

CONTENTS

Part I

PRESSURE SEWERS

BACKGROUND

The disposal of human wastes in an environmentally acceptable manner has been a problem throughout history. Historical evidence of early man's struggle to overcome this problem has been recorded and related elsewhere.[1-3] Little progress has been recorded between the pre-Christian solutions described in the literature and the mid-19th century rebirth of municipal sewage in Europe and the United States, a development that has continued to the present. Between 1857, when the first large American system was installed in Brooklyn, N.Y., and 1905, about 28 million Americans were "sewered."[4] By 1940 more than half of the total U.S. population was served by sewers, and the figure now stands at about 71 percent.

A recent review of nearly 300 facilities' plans for rural communities in the United States produced the relationship shown in figure I-1.[5] Monthly charges much above $20 are considered excessive in rural areas where median incomes are generally significantly lower than in urban areas. Because most on-site wastewater disposal systems would cost significantly less than $20 monthly, the on-site approach has been generally used in these areas. Difficulties have arisen in areas where conventional on-site systems have failed because of unfavorable soil conditions. Typically, the result from this condition has almost invariably been a recommendation to install sewers in the community. Implementation of this recommendation depended on the financial status of the community, availability of Federal grants, and public attitude. Without getting into a lengthy discussion on the merits and demerits of the grant programs and centralized collection and treatment systems, it suffices to note that the cost of conventional sewers is extremely high for most small communities. In fact, it is not uncommon to see engineering estimates in excess of $10,000 per home. Also, the cost of the conventional collection system generally represents more than 80 percent of the total system capital cost in rural areas. Figure I-1 clearly illustrates the relationship between cost and population density, which is primarily explained by the greater length of sewer per contributor, greater problems with grade resulting in more lift stations or excessively deep sewers (see fig. I-2), and regulations that limit the smallest sewer pipe diameter.

Essentially because of the foregoing economics, the primary form of wastewater treatment and disposal in rural areas has been the septic tank-soil absorption system (ST-SAS). Before the passage of the Norris-Rayburn Act during the depression of the 1930's, few rural areas had the electricity necessary to provide for water carriage of human wastes. However, as the rural electrification program took effect the following decade, two major events occurred. First, the rural population obtained electricity that upgraded the standard of living, including pressurized water supplies and water carriage of wastes. Second, urban dwellers emigrated to previously rural areas, where they could enjoy the best of both societies. The disposal of wastewater generated in these unsewered areas was best accomplished by ST-SAS's, as shown in figure I-3.[6] Developers of these areas also found advantages in these systems because costs were directly related to the dwelling and offered a minimum of postconstruction responsibility. Unfortunately, many of these systems have failed because of faulty design and construction, unsuitable soil conditions, and owner negligence. Present estimates indicate, however, that 15 to 20 million ST-SAS's still exist in the United States, serving more than one-fourth the population.

Unfortunately, many situations have come about in recent years that cannot be solved by either of the traditional alternatives, and the results of attempting to apply either technology in

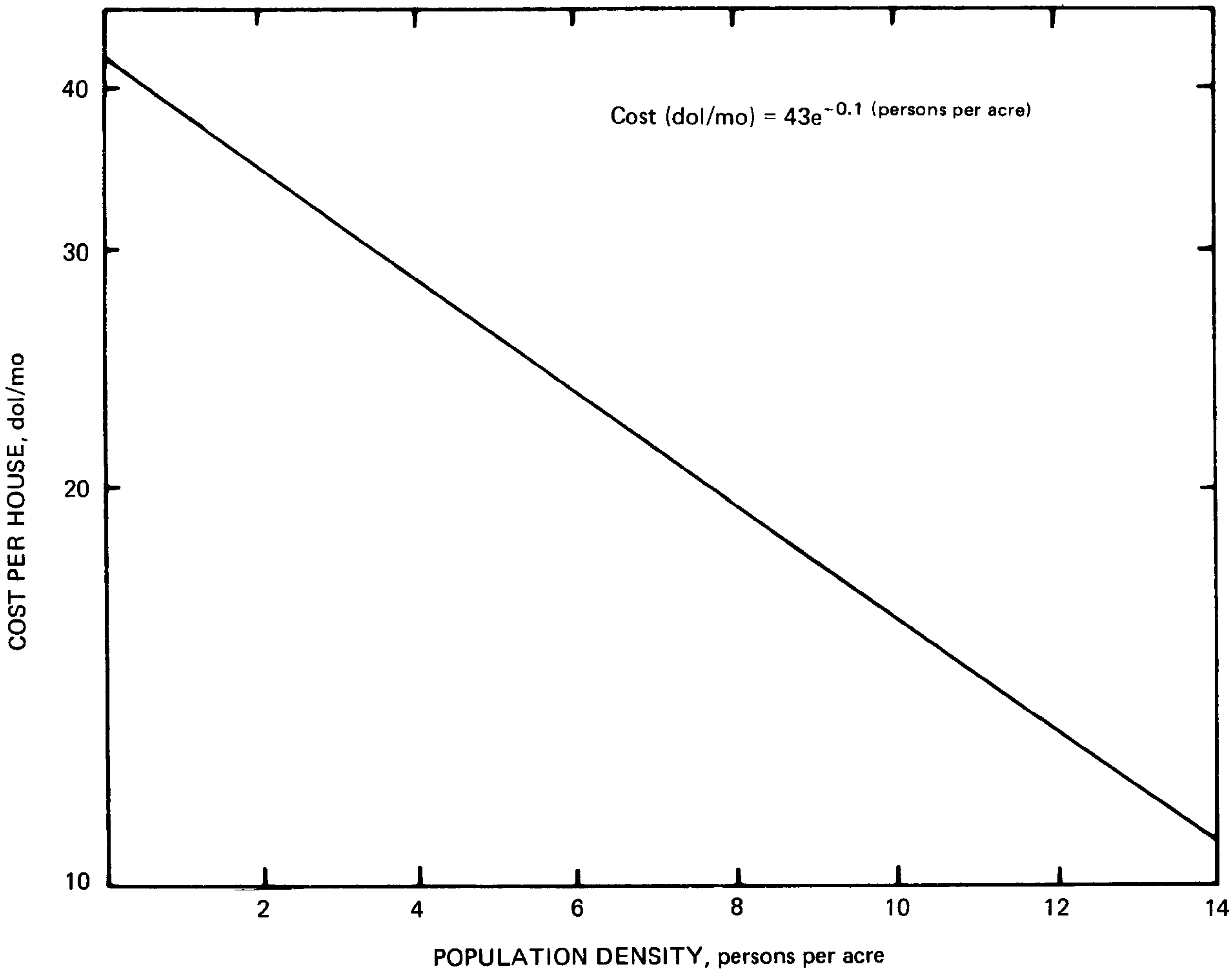


Figure I-1. Monthly cost of gravity sewers.

these situations have been unsatisfactory to all involved. For example, building moratoriums have been imposed that prevent the development of highly desirable land parcels; conventional sewers have been installed at tremendous cost to the homeowners serviced; and ST-SAS's have been constructed that cannot function properly, therefore contaminating the very environment that made the site so attractive originally. The problem has become so acute that the 92d Congress specifically directed the Environmental Protection Agency (EPA) in Section 104(q)(1) of Public Law 92-500 to

> conduct a comprehensive program of research and investigation. . . eliminating pollution from sewage in rural and other areas where collection of sewage in conventional, community-wide sewage collection systems is impractical, uneconomical or otherwise infeasible, or where soil conditions or other factors preclude the use of septic tank and drainage field systems.

INTRODUCTION

Although sewage pumping has been practiced for many years in municipal systems in the form of lift stations and force mains to avoid excessive depths of cut, and in many individual homes in the form of ejector or sump pumps, the wholesale use of small-diameter pressure collection systems

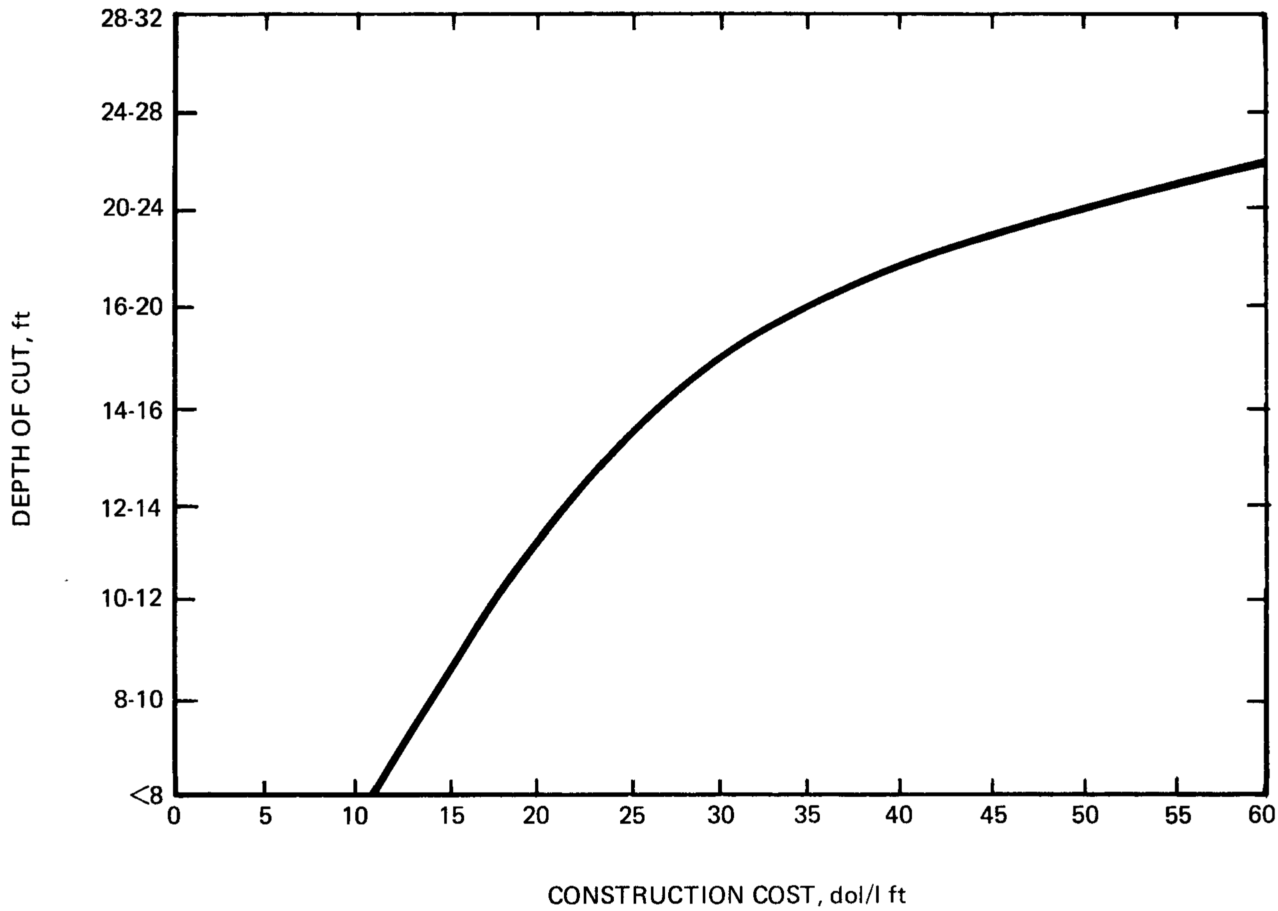


Figure I-2. Cost of sewer construction.

did not emerge until the latter part of the 1960's. The chief impetus at that time was provided by the late Gordon Maskew Fair who proposed that small-diameter pressurized sewers be placed inside larger combined sewers to carry sanitary sewage. Although the "sewer-within-a-sewer" concept was not found to be entirely feasible in a resulting study, the use of pressurized sewers carrying ground sewage was feasible.[7] This study was performed by the American Society of Civil Engineers (ASCE) for the Federal Water Pollution Control Administration and included numerous research studies on such topics as household wastewater generation patterns, critical velocities of flow, alternative system layouts, and prototype grinder pump (GP) performance.

Although experience with pressure sewer systems is limited in both number of installations and duration of service, some information is available on their economics. The EPA has sponsored full-scale evaluations of GP-pressure sewer systems at Albany, N.Y.; Phoenixville, Pa.; and Grandview Lake, Ind. Other communities have also used this technology. Significant data are available from these sources, and the purpose of this report is to present as much of this information as possible to assist the engineering profession in determining the applicability of and design criteria for pressure sewer systems.

A number of advantages of pressure sewers have been presented in the literature.[1-3, 7-12] These benefits are primarily related to installation costs and inherent system characteristics. Because these systems all use small-diameter plastic pipes buried just below the frost penetration depth, their installation costs can be quite low compared to conventional gravity systems in low-density areas. Other site conditions that enhance this cost differential include hilly terrain, rock outcropping, and

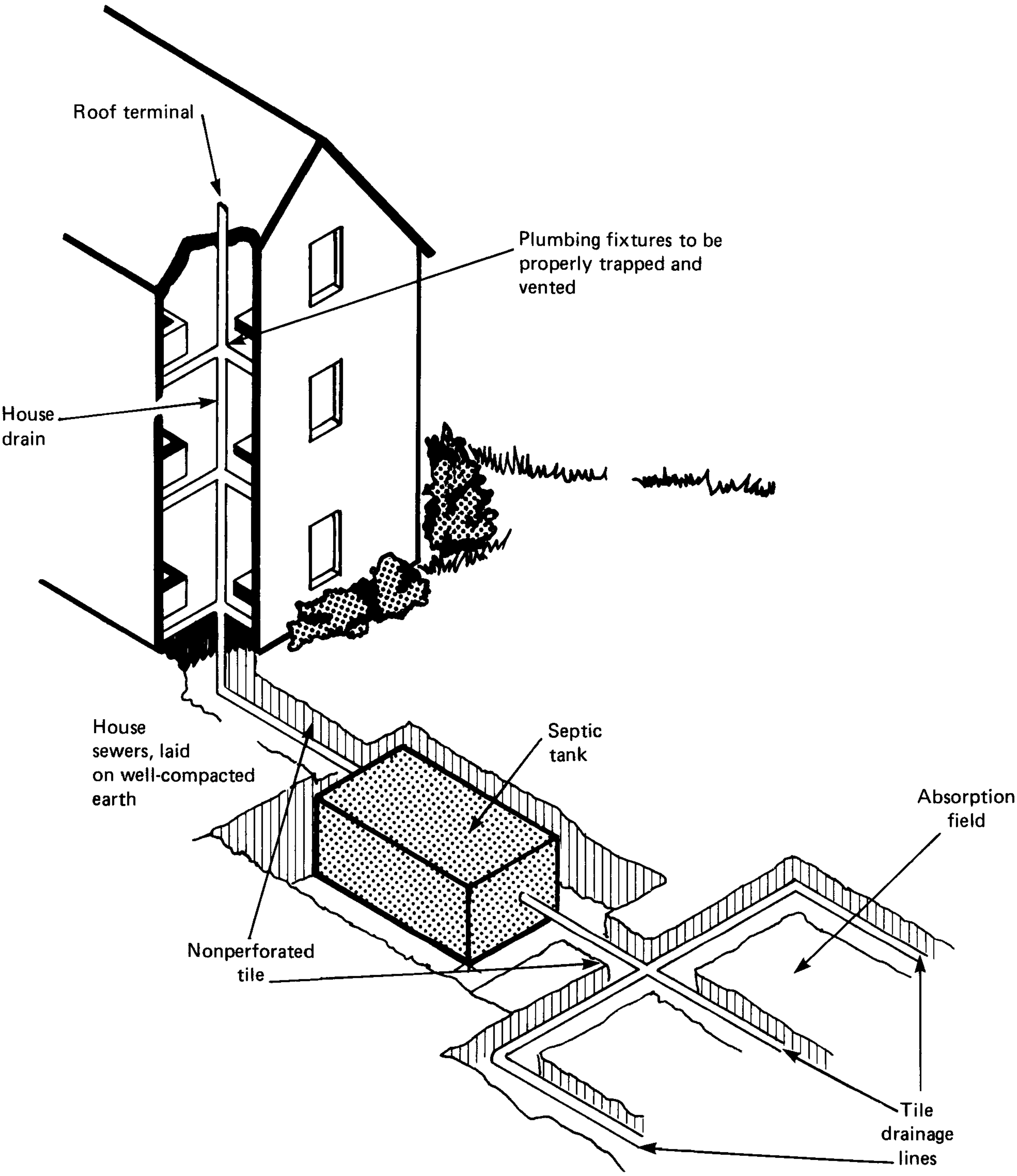


Figure I-3. Typical on-site system.

high water tables. Because pressure sewers are sealed conduits, there should be no opportunity for infiltration, and treatment plants can be designed to handle only the domestic sewage generated in the homes serviced, excluding the infiltration that occurs in most gravity systems.

As with any technology, certain disadvantages also exist. The disadvantages of pressure sewers include high operation and maintenance costs related to the use of mechanical equipment at each point of entry to the system. Also, depending on the type of system used, the wastewater conveyed to the treatment facility may be more concentrated than normal wastewater. It may require, therefore, a higher level of treatment to satisfy effluent requirements. A wastewater will also be devoid of oxygen.

DESCRIPTION

Essentially, a pressure sewer system is the reverse of a water distribution system. The latter employs a single inlet pressurization point and a number of user outlets, while the pressure sewer embodies a number of pressurizing inlet points and a single outlet, as shown in figure I-4. The user input to the pressure main follows a generally direct route to a treatment facility or to a gravity sewer, depending on the application. The primary purpose of this type of design is to minimize sewage retention time in the sewer.

The two major types of pressure sewer systems are the GP system and the septic tank effluent pump (STEP) system. These are depicted in figures I-5 and I-6. From these figures it is obvious that the major differences between the alternative systems are in the on-site equipment and layout. But some subtle differences also exist in the pressure main design methods and in the treatment systems required to reduce the pollutants in the collected wastewater to an environmentally acceptable level. Neither pressure sewer system alternative requires any modification of household plumbing, although neither precludes it if such modifications are deemed desirable.

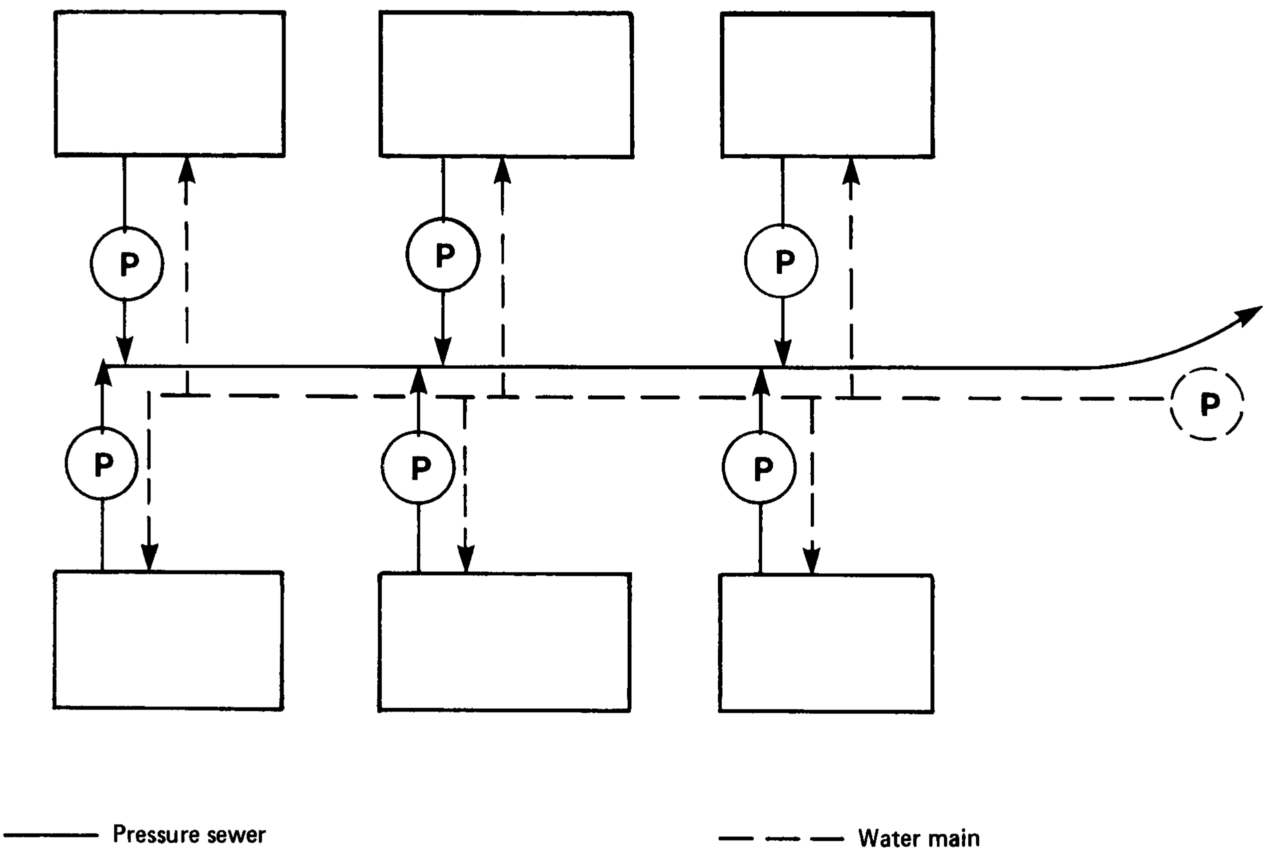


Figure I-4. Pressure sewer vs. water main.

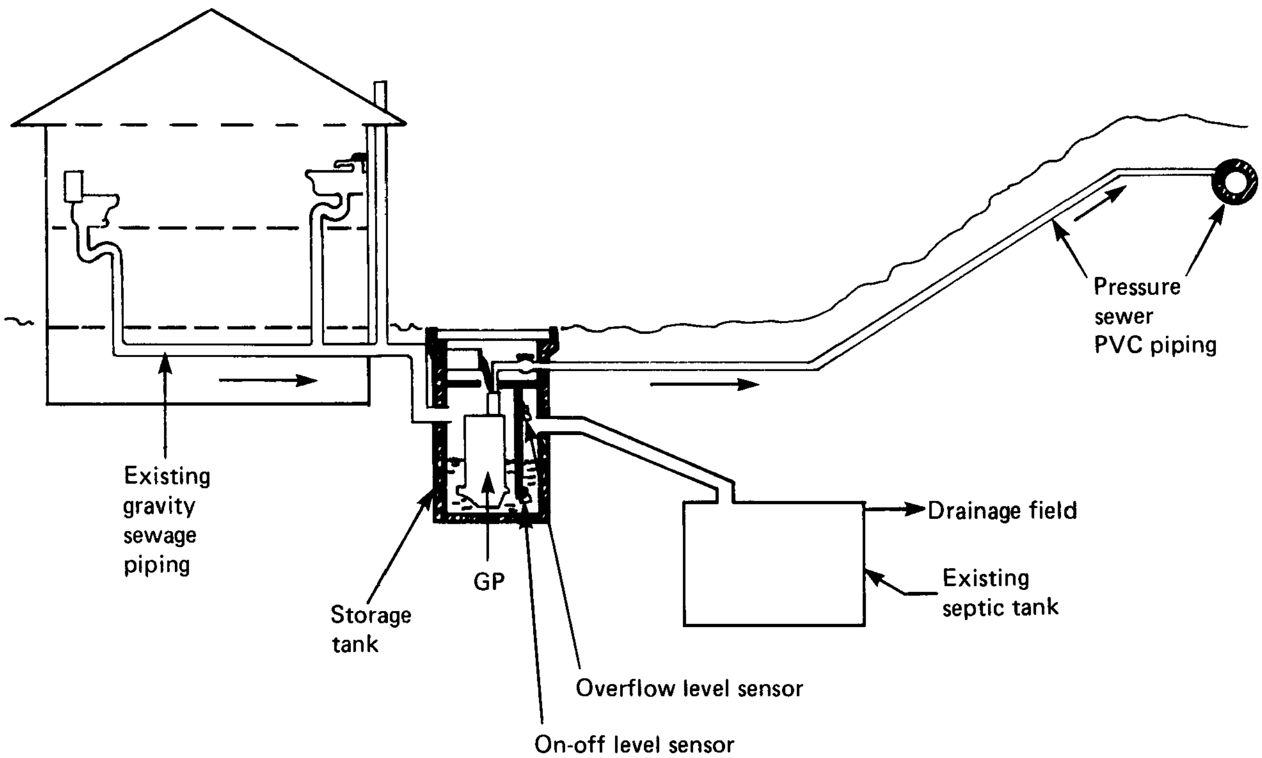


Figure I-5. Typical grinder pump installation.

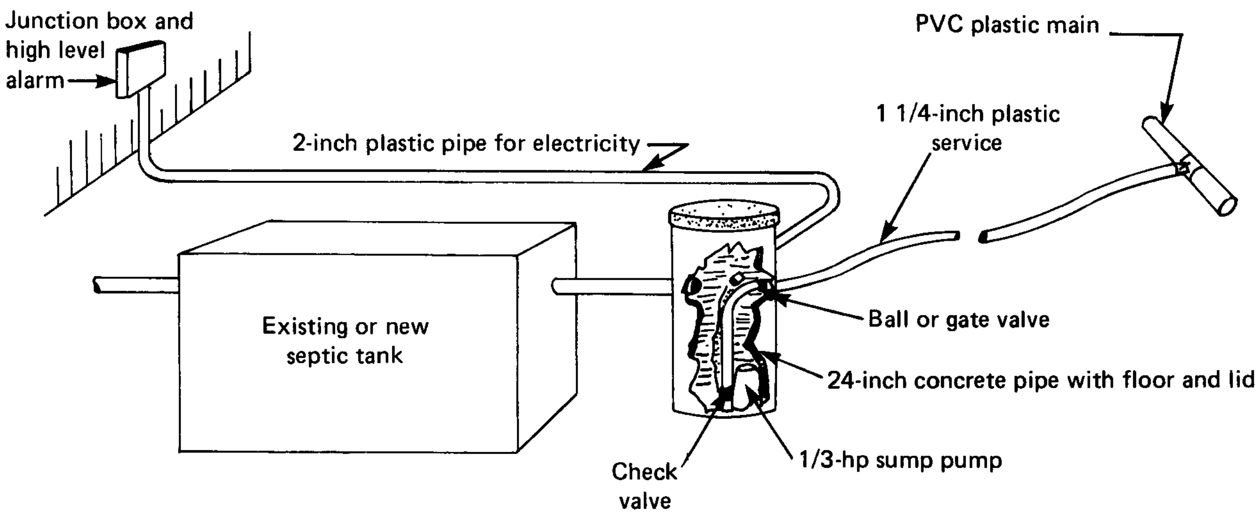


Figure I-6. Typical STEP system.

PREVIOUS EXPERIENCE

As noted earlier, the isolated pumping of sewage and septic tank effluent has been practiced for many years. The first attempt to use a pressure sewer system was reported by Clift[9] in 1968, but the system described did not employ the techniques and materials that are now considered standard design practice. To serve 42 customers in low-lying areas of Radcliff, Ky., it would have been necessary to finance $3,170 per connection, while the prototype pressure sewer system cost only $1,346 per connection. This prototype design used pneumatic ejector units at each connection, which discharged into a 3-inch (7.6-cm) cast iron lateral and a 4-inch (10.2-cm) cast iron main that emptied into a gravity sewer. Even though mechanical and electrical problems were encountered that eventually caused abandonment of the system, Clift reported that during the first 3 years of operation, no odors or blockage of pressure lines occurred.[9] Severe corrosion was encountered, however, and found to be the primary cause of the electrical and mechanical component shortcomings.

Clift also performed preliminary estimates on similar prototype pressure sewers for two other locations. In one case, 120 out of 280 lots around a lake were considered well suited to using a system with conventional gravity sewers, while the more inaccessible lots were thought to be better served by pressure sewers. In the second case a similar hybrid design approach was estimated to save 5.5 percent in capital costs.[9]

The most highly instrumented study of pressure sewers was performed on a group of 12 homes in Albany.[2,8,10] Each dwelling was equipped with a commercial GP and connected by laterals to a pressure main that emptied into a gravity sewer, as shown in figure I-7. The system operated well after the original prototype GP units were replaced with improved models. The pressure main had been oversized to allow all units to operate simultaneously. Subsequent accumulations of grease and fibrous materials reduced some pipe cross-sectional areas by as much as 40 percent. Valuable information was reported on design and construction methods and on the operational characteristics and maintenance requirements of the GP units. Monthly power costs of 10 to 27 cents per home were incurred, based on a rate of 2.3 cents per kilowatt hour. Wastewater from the pressure sewer was characterized and found to be more concentrated than normal municipal wastes, ostensibly because of the lack of sewer infiltration.

Another relatively short-term (6-month) study of a pressure sewer system with GP's was made at Phoenixville.[11] This system, as shown in figure I-8, was 2,800 feet (854 metres) long and discharged into a gravity sewer more than 60 feet (18.3 metres) above the farthest GP location. Another unique feature of this system was the inclusion of multiple-family dwellings serviced by a single GP. Data reported on construction costs were excellent. Some indirect evidence of pipe cross-sectional area reductions, similar to the Albany study, was also noted. The GP units used were similar to the Albany units, and their operation resulted in a monthly power cost of 11 to 25 cents per capita.

The Grandview Lake pressure sewer system[12-14, a, b] was much larger in size (it served 93 homes). The need for this system was related to an engineering estimate for conventional sewerage

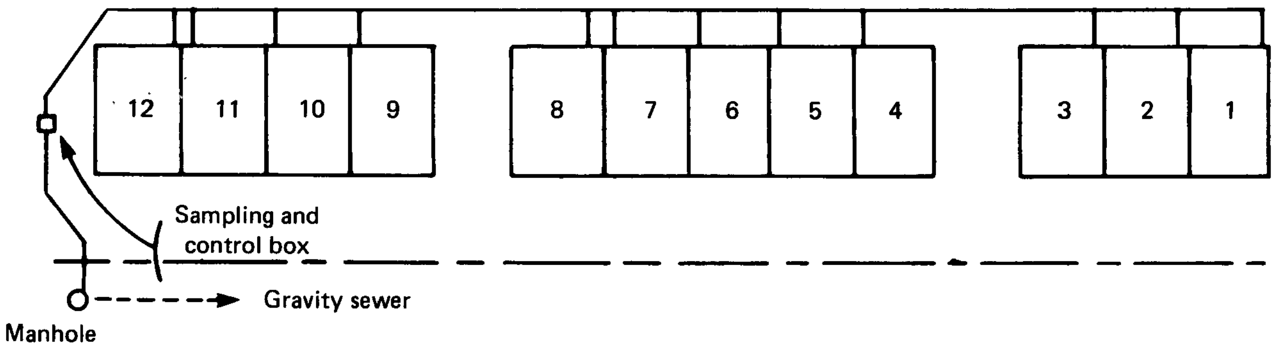


Figure I-7. Twelve homes in Albany system.

[a] S. M. Rees, SIECO, Inc., personal communication.
[b] R. P. Farrell, Environment/One Corporation, personal communication.

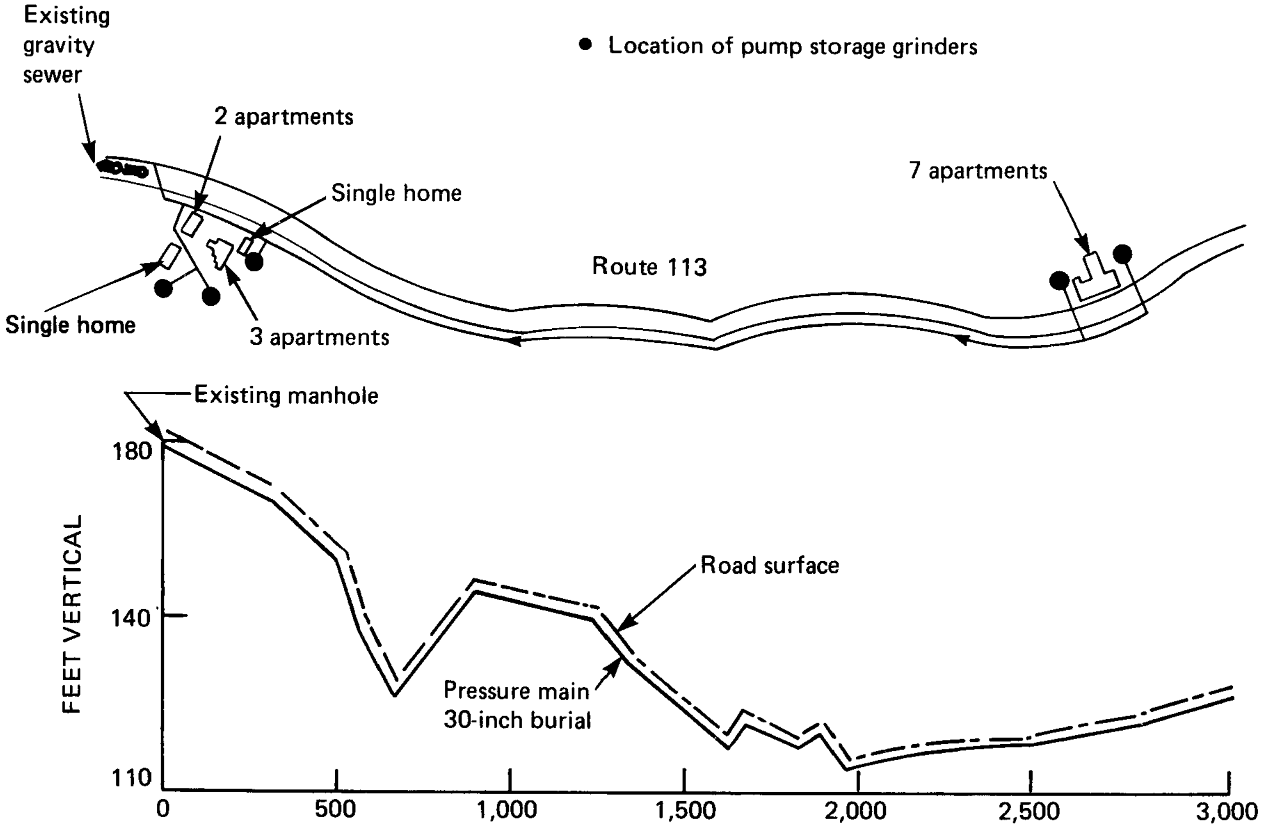


Figure I-8. Phoenixville pressure sewer system.

of $3,000 per lot or $10,000 per existing home because of unfavorable terrain and the resulting need for nine lift stations. This pressure sewer system included different types of GP units and a few STEP units. Approximately 29,000 feet (8,840 metres) of pressure main, six automatic and seven manual air release valves, and treatment by stabilization pond with effluent land spreading were used in the Grandview system, as illustrated in figure I-9. Grease problems plagued the system by causing faulty operation of automatic air-release valves and by promoting deposits on flow measuring devices at the plant. The installed cost of pressurization equipment and ancillary on-site components varied from $1,000 to $1,500 per home. A contingency provision for potential on-lot overflows during equipment or electrical outages was included in the system design. Existing SAS's were used whenever possible. Where these were not available, a small (2-day capacity) gravel-filled absorption bed was provided. Generally, 1-inch (2.5-cm) service connections were used to feed 3- and 3.5-inch (7.6- and 8.9-cm) pressure mains.

Other installations of GP-pressure sewer systems being designed, installed, or operated have been noted.[10, 15-17, b, c, d] Bowles[1] and Cochran[15] describe an installation at Horseshoe Bay on Lake Lyndon Baines Johnson, Tex. As many as 4,000 connections are planned for this development, with about 106,000 feet (32,300 metres) of a 2- to 12-inch (5.1- to 30.5-cm) pressure main. About 200 GP units were in operation.[15] Equipment problems relating to installation and design have been experienced, but corrections have been made and the system is now functioning satisfactorily. The sewage is treated by an activated sludge system with tertiary chemical clarification and filtration. Another Texas system, which has been in partial operation since 1972, is located at Point

[b] R. P. Farrell, Environment/One Corporation, personal communication.
[c] R. E. Lawford, Peabody Barnes Company, personal communication.
[d] J. Schultz, Becher-Hoppe Engineers, Inc., personal communication.

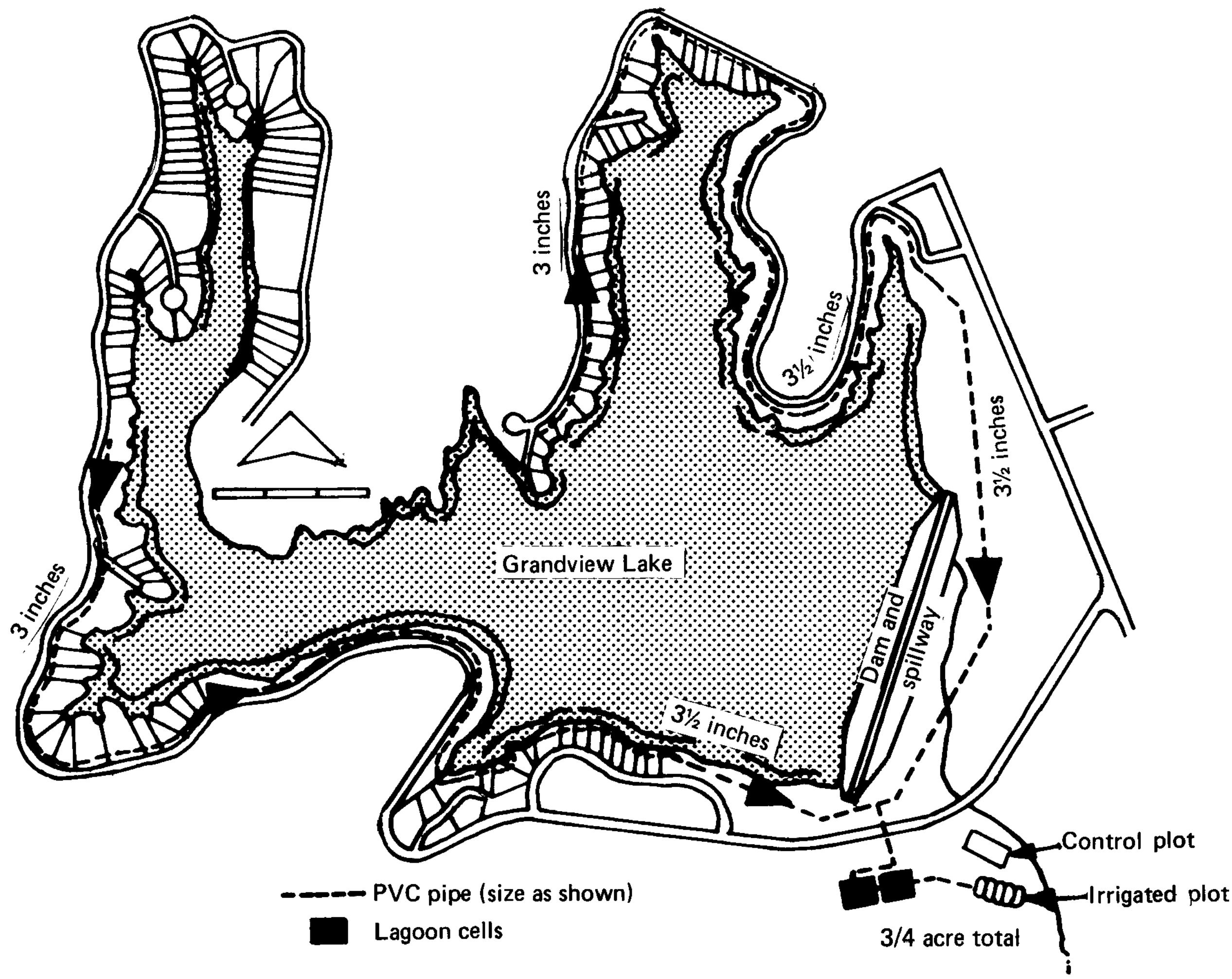


Figure I-9. Grandview Lake sewage research and demonstration project.

Venture on Lake Travis. This GP installation also suffered from initial problems resulting from construction activity but was functioning in an acceptable manner in 1974. Two more large GP systems and two smaller STEP systems have been approved in Texas.[15]

Gray[16] has reported on the circumstances that led to the design and construction of a GP-pressure sewer system at Weatherby Lake, Mo. The system now serves 330 homes and is expected ultimately to serve 900. The total bid cost of the pressure system was $1,030,108, compared to a conventional system (including eight pumping stations) estimate of $2,250,000. This system consists of 309 GP units, 35,000 feet (10.7 km) of pressure main, 37,100 feet (11.3 km) of service lines (polyvinyl chloride (PVC), SDR-26 with gasketed joints), 42 air-release valves, and 24 flushing and cleanout connections. Included in the above bid cost is 5,300 feet (1.6 km) of gravity interceptor to deliver the pressure sewer effluent to the Kansas City municipal system for treatment and disposal.

Other GP projects have been proposed, designed, or constructed in Saratoga, N.Y.; Clifton Park, N.Y.; and Kinnelon, N.J.[10, 18] A GP pressure sewer project on Madeline Island, Wis., is being built to serve a recreational development.[d]

---

[d] J. Schultz, Becher-Hoppe Engineers, Inc., personal communication.

The most noteworthy STEP-pressure sewer installations are located in Florida and Idaho. General Development Utilities, Inc., of Miami has installed two large systems—one in Port Charlotte, Fla., and the other in Port St. Lucie, Fla. The Port St. Lucie pressure sewer (125 homes), buried at a depth of 2 feet (0.61 metre), discharges into a gravity sewer; while the smaller (26 homes) system at Port Charlotte discharges into an extended aeration treatment plant. The Port Charlotte system is the oldest, having been in operation since August 1970. The pumping units are small centrifugal sump pumps, and the pump pits are vented via the building sewers in the same manner as the 900-gallon (3.4-m^3) septic tanks that pretreat the wastewater.[e]

Two separate pressure sewer installations located at Coolin and Kalispell Bay at Priest Lake, Idaho, serve 348 and 200 homes, respectively. One-third- and one-half-hp (0.25- and 0.37-kW) sump pumps, equipped with bronze impellers, are used to pump the septic tank effluent through 1.5-inch (3.8-cm) PVC, SDR-26 service lines and 3- to 6-inch (7.6- to 15.2-cm) PVC mains to lagoons for treatment.[f] Although some initial problems resulted from improper impellers that were supplied with the pumps, the operation of these systems and treatment facilities has been capably handled by one individual.

Sanson has described design methods used in planning STEP systems for two Indiana communities.[19] Additional pressure systems using STEP concepts have been planned or approved in Florida, Oregon, Idaho, Washington, Ohio, Wisconsin, and Arkansas.

Two privately owned and operated pressure sewer systems located in Oregon have been in operation for a significant period of time using centrifugal pumps to pressurize raw sewage directly from the source. One system services houseboats (approximately 500), while the other services a private housing development (approximately 150 homes). These systems are operating without excessive operation and maintenance requirements, despite the higher potential operation and maintenance costs with this design.[20, g]

Two major manufacturers of pressurization equipment have supplied information on present and future installations of pressure sewer systems.[b, c] The States listed below have approved at least one project that is either being designed, constructed, or operated:

Arkansas	Mississippi	South Carolina
California	Missouri	South Dakota
Delaware	Nebraska	Texas
Florida	New Jersey	Vermont
Idaho	New York	Virginia
Illinois	North Carolina	Washington
Indiana	Ohio	West Virginia
Kentucky	Oregon	Wisconsin
Michigan	Pennsylvania	

SYSTEM DESCRIPTION

A pressure sewer system consists of two major elements: the on-site or pressurization facility and the primary conduit or pressurized sewer main. Probably the widest divergence of opinion exists on the proper design and equipment selection for the pressurization facility. Opinion varies be-

[b] R. P. Farrell, Environment/One Corporation, personal communication.
[c] R. E. Lawford, Peabody Barnes Company, personal communication.
[e] H. Schmidt, General Development Utilities Company, personal communication.
[f] C. W. Rose, Farmer's Home Administration, personal communication.
[g] J. Ward, personal communication.

cause of the competition between proprietary mechanical devices of different designs and because of some basic attitudes on the relative merits of the available alternatives.

In all designs household wastes are collected in the building drain and conveyed therein to the pretreatment or pressurization facility. In most cases the piping arrangement includes at least one check valve and one gate valve to permit isolation of each pressurization system from the main sewer. The two major alternative systems, which are illustrated in figures I-5 and I-6, use a pressurization device that is located below ground in a manhole or access hole to collect the household wastes by gravity discharge. GP's also can be installed in the basement of a home to provide easier access for maintenance and greater protection from vandalism.[21]

The pressure main can take many forms, but it generally consists of a single, small-diameter conduit that has numerous feeder lines from each pressurization inlet. This type of arrangement has been deemed desirable to minimize sewer retention time.[7] A typical example of a pressure sewer flow diagram is illustrated in figure I-10.[22]

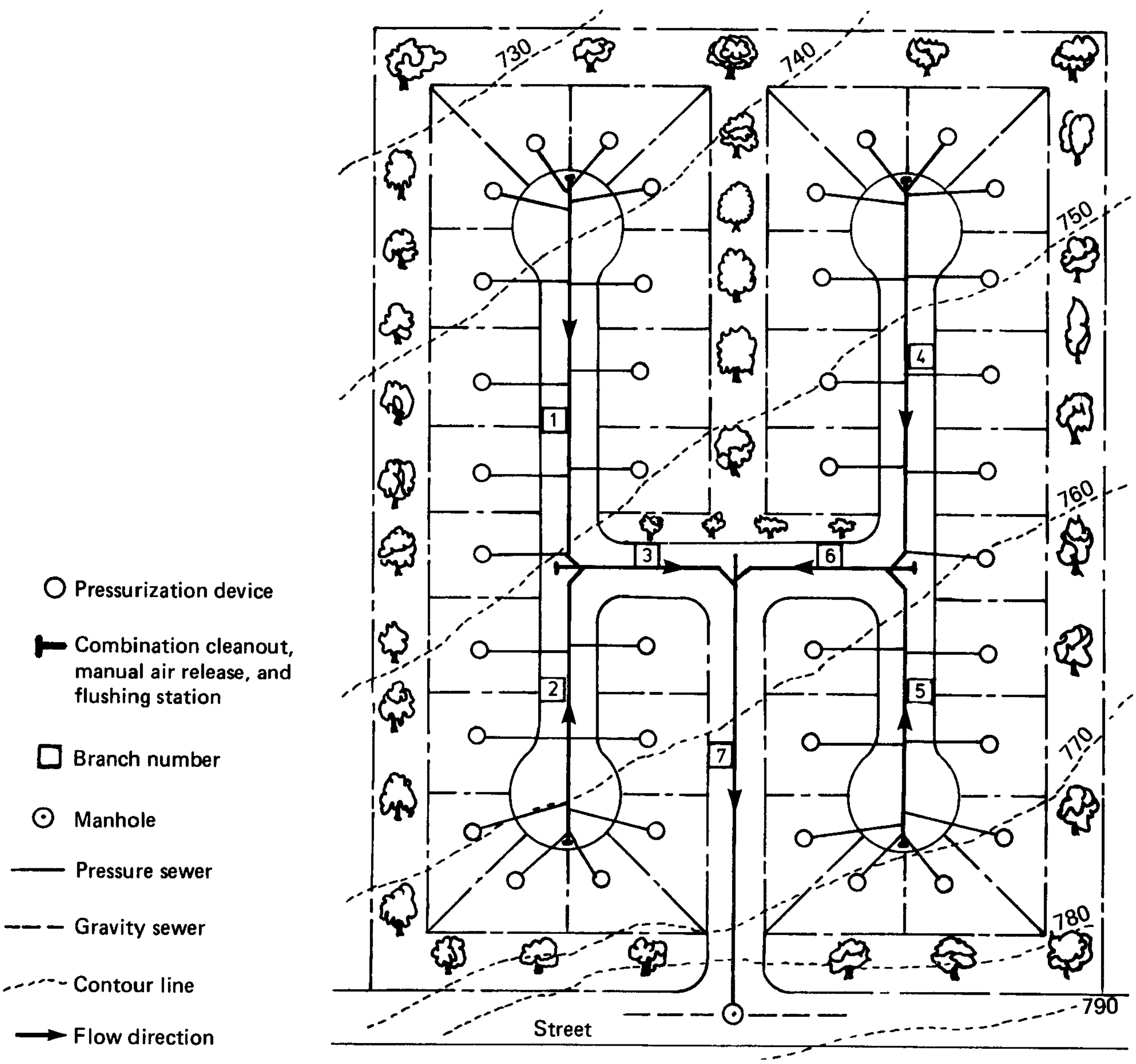


Figure I-10. Typical pressure sewer layout.

Pressurization Facilities

STEP. As noted earlier, household wastewater is collected by the building drain and transported from the home through the building sewer to a septic tank. A few investigators have characterized household wastewaters and per capita flows.[23-25] Mean flows have been found to vary from 43 to 50 gallons per capita per day (0.16 to 0.19 m^3 per capita per day). Table I-1 represents the results of the most extensive of these studies[24] for homes with and without garbage grinding against typical municipal wastewater analyses for the same parameters.[26] Generally, the wastewater generated at the home is more concentrated in most pollutant categories than a normal municipal waste (primarily of domestic origin), which has been diluted by infiltration and other extraneous water sources in municipal gravity sewer systems.

Significant treatment occurs in a septic tank. Primarily, the septic tank serves as a device for removing settleable solids and grease. Heavy solids settle during the multiday nominal detention period, while grease and other floatables collect in the scum layer. A cutaway view of a septic tank is shown in figure I-11. Anaerobic biological activity occurs sporadically, which causes some liquefaction of accumulated solids. This digestive action produces gas that rises as bubbles in the system, and the inlet flow patterns are quite variable. Both of these occurrences reduce the effectiveness of the septic tank in retaining captured solids. A well-designed septic tank generally removes from 80 to 90 percent of the hexane extractables (grease), 70 to 90 percent of the suspended solids (SS) (including all the grit), and 50 to 80 percent of the biochemical oxygen demand (BOD).[27-30] In the case of grease removal, the septic tank is an excellent grease trap, not only because of its inlet and outlet configurations but also because its size allows the grease to cool and congeal for easier separation. SS removal may be temporarily reduced or negated during extended hot summer periods because of increased anaerobic digestion and resulting gas production and mixing from rising bubbles. BOD removal is higher than that normally credited to primary sedimentation. Typical septic tank effluent may have the following analysis:

- BOD_5, 100-180 mg/l
- SS, 50-75 mg/l
- Grease, 10-20 mg/l

Table I-1.—*Household wastewater characterization*

[mg/l]

Parameter	Household wastes[24]		Typical munici-pal[26] medium strength
	Without grinder	With grinder	
BOD_5	415	465	200
TSS	296	394	200
VSS	222	309	150
TKN	51	52	40
NH_3-N	11	10	25
TP	33	32	10
Grease	123	129	100

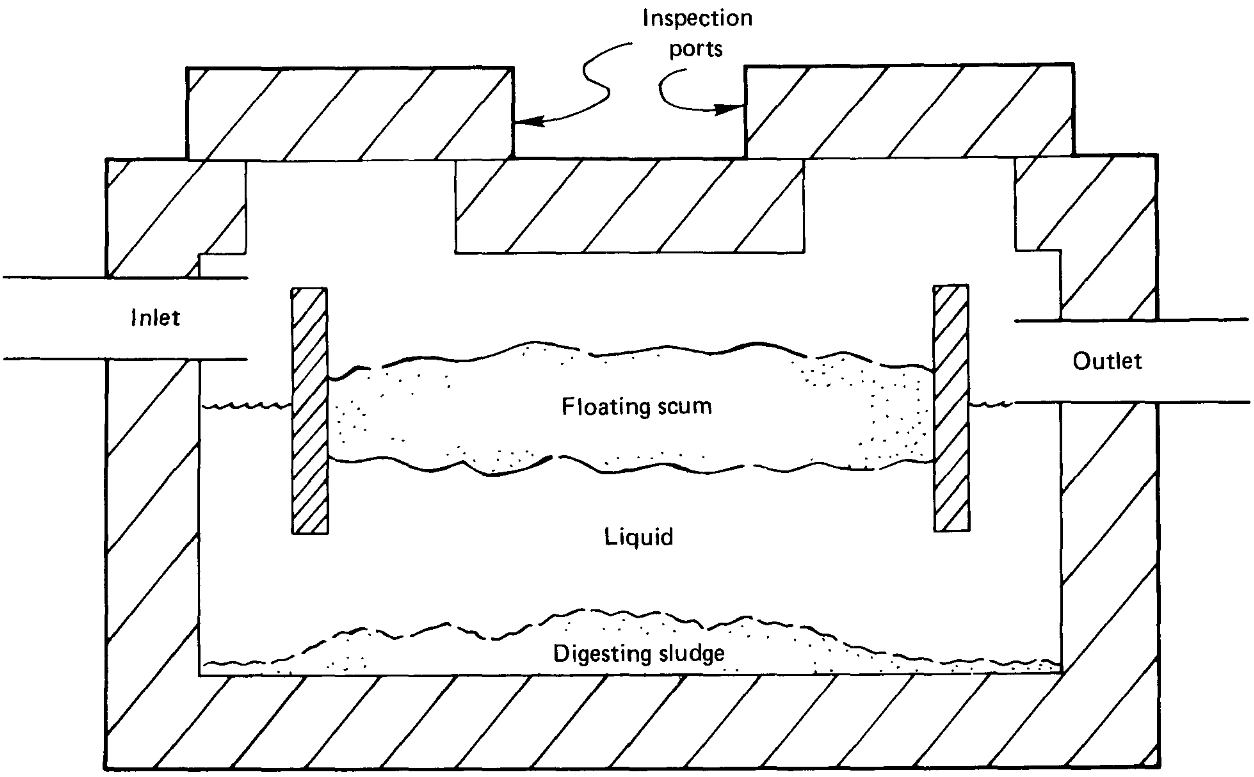


Figure I-11. Cutaway of septic tank.

The septic tank effluent then flows to a receiving tank, as depicted in figure I-6, which houses the pressurization device, control sensors, and valves required for a STEP system. The heart of the STEP system is, of course, the pressurization device. Normally, small centrifugal pumps have been designed or employed for the STEP systems. The oldest STEP systems are in the Miami area. The pumps used at Port Charlotte and Port St. Lucie are all manufactured by the Hydromatic Pump Company. Almost all of these units are Models SP 33A, with the head-discharge curve shown in figure I-12. The same pumps also are used in two STEP systems at Priest Lake, although some 0.5- and 1-hp (0.37- and 0.75-kW) pumps are also included in these systems for locations where higher heads were required. The EPA demonstration project at Bend, Ore., uses similar pumps manufactured by Peabody-Barnes, which are driven by 0.5-hp (0.37-kW) motors.

All of the above sump pumps are submersible and generally retail for around $200. Several units are equipped with bronze impellers said to reduce potential corrosion problems. Pumps with impellers made of plastic materials are now available from several manufacturers. This development should significantly extend the present 10-year life estimate for the pumps now in use. The cost of the new pumps is not significantly higher than the standard units.

The design of these systems requires a proper septic tank installation and an effluent holding tank containing the pump, level controls, valves, and piping. The discharge piping is essentially the same for any design alternative and will be discussed along with the pressure main design alternatives. Design decisions for effluent holding tank installations include material of construction, diameter, and working levels for the tank, pump choice, and ancillary needs.

The effluent holding tank can be made of any material suited for septic tank use, including properly cured precast or cast-in-place reinforced concrete and steel tanks meeting Commercial Standard 177-62 of the U.S. Department of Commerce, with proper anticorrosion coatings.[6] The Albany project report[8] indicated the epoxy-coated steel tanks underwent severe corrosion during

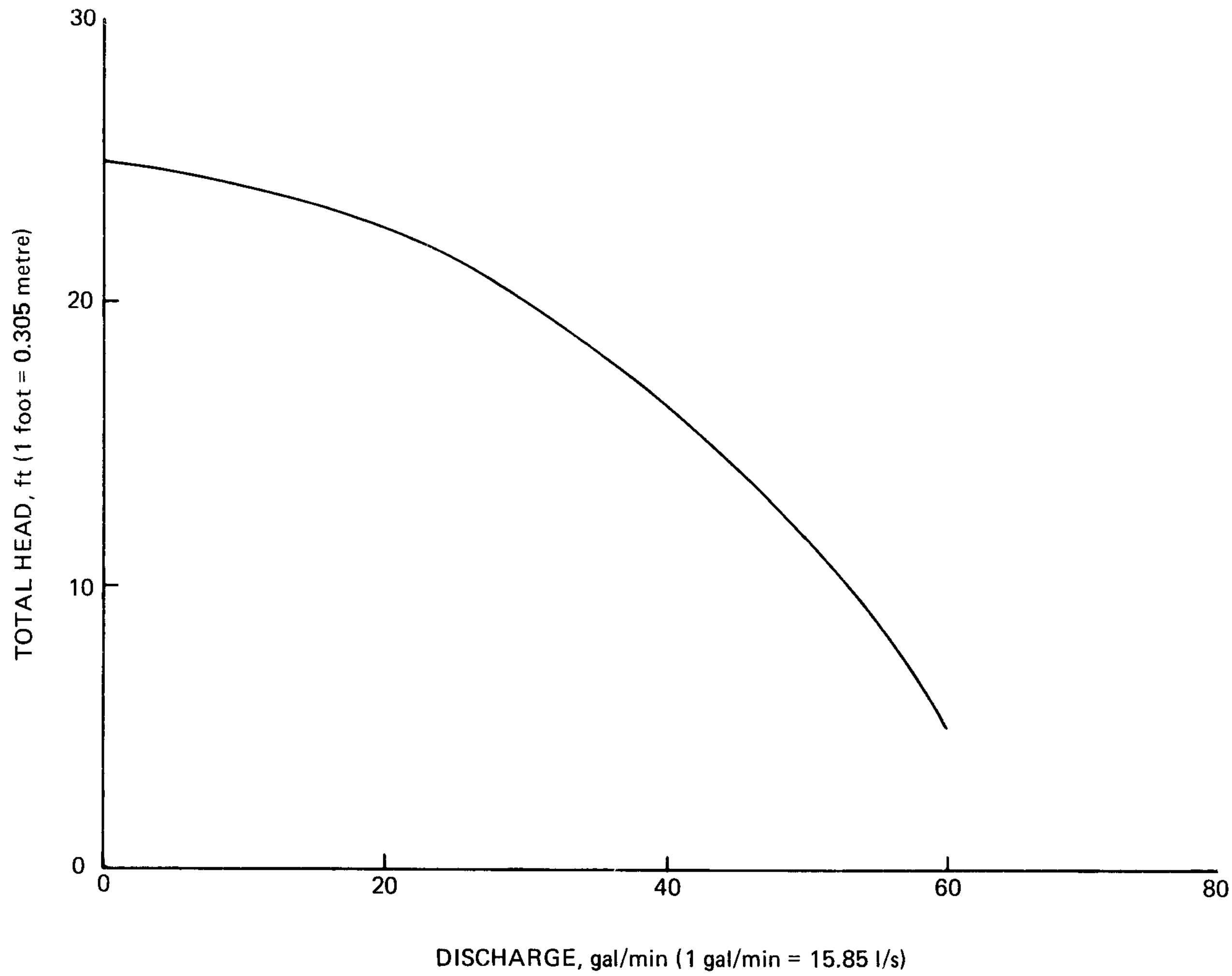


Figure I-12. Head-discharge curve for SP33A pump.

the study. Molded fiberglass, reinforced polyester (FRP) resin tanks were found to be quite acceptable on that project. The Phoenixville and Bend studies used concrete tanks with no apparent difficulty[11,h] while the Grandview Lake project[12] used FRP, precast concrete and steel tanks, with no mention of corrosion problems. The Miami systems use fiberglass tanks and the Priest Lake systems use steel tanks with a litumastic coating.[e] It should be noted, however, that these experiences have been of a relatively short duration and do not provide long-term information.

The size of the effluent holding tank is a function of a number of variables. A typical unit design is shown in figure I-13.[h] For single-family dwellings, the ASCE report[7] determined that a minimum of 30 gallons (114 litres) of net storage capacity was required for a 12 gal/min (0.76 litre/s) discharge rate. The concern for storage capacity relates to the submersible pump's ability to handle the maximum short-term flow from a home. Simultaneous discharge of a bathtub and an automatic washing machine was cited as the most likely maximum condition, producing a flow of 46 gallons (174 litres) in 2 minutes. The use of 30 gallons of net storage capacity (volume between cycle initiation level and overflow pipe) has proved adequate at Albany and Grandview Lake. A 70-gallon (276-litre) storage capacity was used at Phoenixville. Other considerations for required tank diameter include providing repair personnel with access to the defective pump or other mal-

[e]H. Schmidt, General Development Utilities Company, personal communication.
[h]L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.

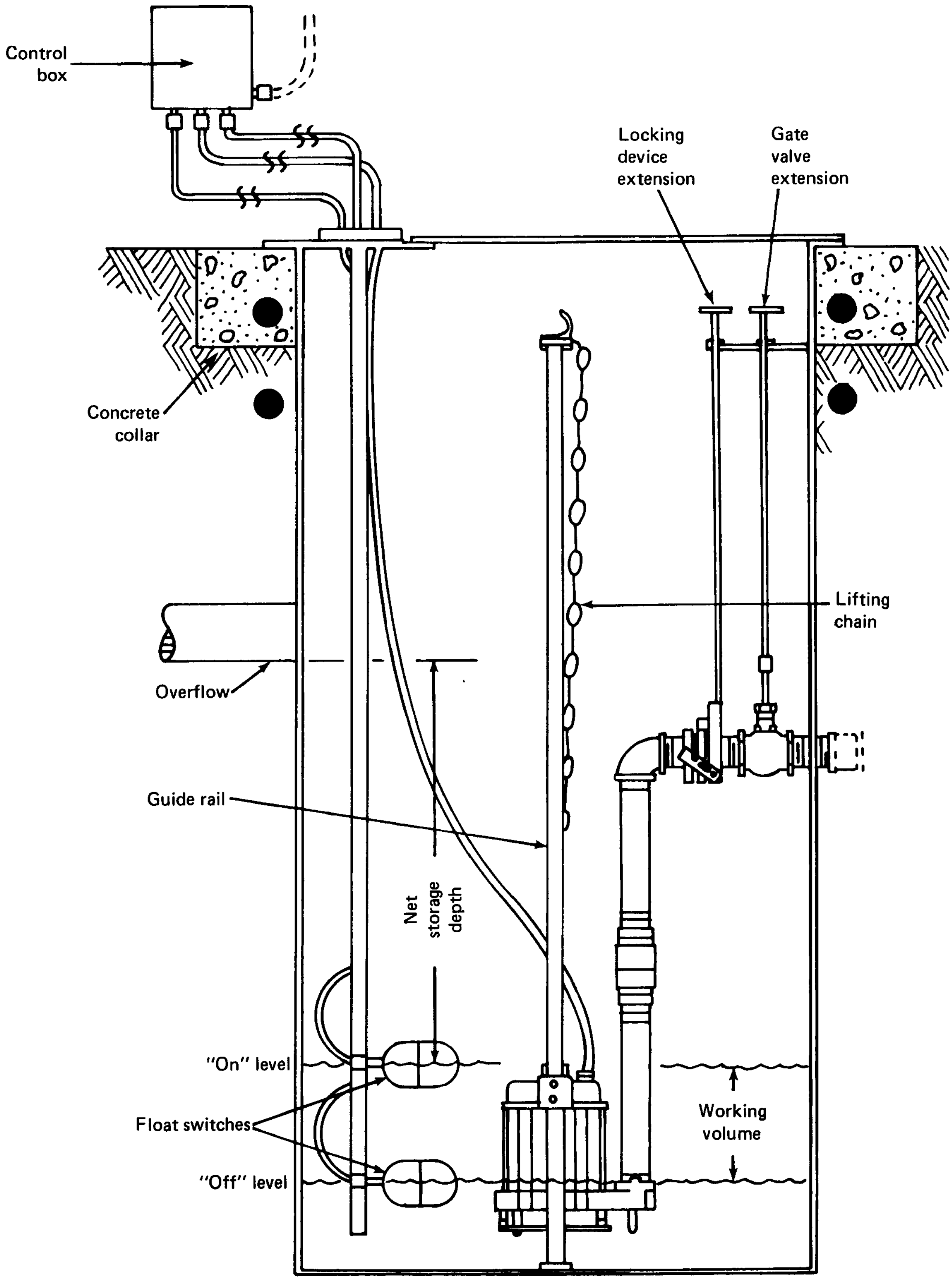


Figure I-13. Typical effluent pump chamber.

functioning item for repairs. If the unit is located within 3 feet of the surface, required diameter may be reduced to 24 to 30 inches (0.61 to 0.76 metre), but small diameter units have been troublesome at one GP location where grease accumulations interfered with float switches.[14] In locations where soil conditions do not provide adequate bearing strength or high ground water levels occur, a concrete pad or collar may be required, as shown in figure I-13.

The working levels in a tank are the levels at which the pump originates and terminates operation (fig. I-13). The volume of the tank between these levels is considered the working volume, which must be discharged during each cycle of operation. In actuality, the volume discharged during a cycle is always greater because the influent flow continues during the pumping cycle. This increase above the working volume is minimized by higher discharge (pumping) rates. At Albany the average operating cycle varied from 39 to 112 seconds for GP units having working volumes between 10 and 30 gallons (38 and 114 litres), while at Phoenixville monthly working volume averages varied from 33 to 137 seconds per 20 gallons (76 litres).[8,11] These times are a function of the pump characteristics, the total dynamic system head at the time of each cycle, the placement of on-off control sensors, and the wastewater flow and duration at the time of cycle inception. For a single home installation the Environment-One (E/One) design (Model GP 210) indicates a 10- to 14-inch (25- to 35-cm) differential that corresponds to about 20 to 28 gallons (77 to 106 litres) per cycle for their standard tank.[22] The Hydromatic GP design (Model CSPG-150A) indicates a 6-inch (15-cm) differential that would correspond to approximately 12 gallons (45 litres) in a similar tank.[31] The Miami STEP systems use 11- and 24-inch (28- and 61-cm) differentials, which correspond to 22 and 47 gallons (83 and 178 litres), respectively.[e] The differential determines the number of actuations or cycles per day and their duration if all other factors are equal. The relative value of fewer, longer cycles versus more, shorter cycles is not now quantified, but implications as to the relative merits of each can be derived in other sections of this seminar publication. Now it appears that manufacturers' standard designs control the issue.

A similar statement can be made about the type of pump control switches used in STEP systems. Many pump manufacturers offer "packages" that may include level control switches, control panels, wiring, and simplified maintenance systems. Although several types of control switches exist, only two types have been used in the manner required by pressure sewer designs. The first type is the pressure sensing tube, which is standard equipment on one GP unit and has been used in a privately owned system near Bend. In the Albany project 1-inch (2.5-cm) tube openings were rejected for GP units because of grease buildup that caused them to malfunction. After replacement with pressure-sensing tubes having 3-inch (7.6-cm) openings, no further problems occurred.[8] No pressure-sensing tube malfunctions have been noted in either the Phoenixville or Grandview Lake projects.[11,14] The Miami STEP systems use diaphragm-type pressure switches, and it is reported that these devices become the major source of maintenance problems after 2 years of service because the diaphragms lose their elasticity.[e]

The other major type of level control is the mercury float switch. This type of control device consists of a mercury switch sealed within a float made of a noncorrosive material. As the water level rises, it causes the float to either rotate or keel over to a position where the mercury switch actuates the pump or, conversely, terminates its operation. Several forms of mercury switches have been used in pressure systems, but usually the switch is either attached directly to the pump housing or is suspended from a stationary point above the liquid. This type of control is also standard equipment for several pump and GP malfunctions. Some difficulties were experienced with this type of control at Grandview Lake, but most problems related to faulty manufacture and shipping and installation problems rather than conceptual shortcomings. The Priest Lake and Bend systems use mercury float switches, with few reported problems.

Some of the ancillary factors that must be considered are the tank location, depth, covering, electrical connections, warning signals, and contingency items. Because the effluent holding tank

[e]H. Schmidt, General Development Utilities Company, personal communication.

follows a septic tank, gravity flow would dictate that the tank inlet be below the liquid level in the septic tank. In most cases it is good practice to locate the pumping chamber as close as possible to or as an integral part of the septic tank. Certain circumstances, however, may require greater physical separation of these tanks. Because septic tanks are usually located in the rear of a house and sewers are generally in the front, it may be more economical to locate the effluent holding tank in the front in cases where a natural slope exists toward the pressure main. Tanks or manways above tanks may be covered with any load-bearing material, such as prestressed concrete, protective-coated steel, or plastic. The covers should be attached in a watertight manner by gaskets or grooves and should be sufficiently above the ground to prevent entrance of normal surface runoff. They should be made as decorative as possible without impairing their accessibility. Trade-offs must be made between ease of access and protection against vandalism. The final design will have to take into account both factors as they relate to the locality under consideration. One suggestion is the use of covers that incorporate locks requiring the use of a special tool to open them.

Electrical connections to the main panel in the house must be made according to local codes. Approved underground wiring is recommended for both the pump and control circuits, which should be wired separately—that is, have separate fuses or circuit breakers—from other household lines and incorporate a fused electrical disconnect adjacent to the controls for use by the serviceman. The controls should be located in the garage or the basement, if an outside entrance is available. Outdoor locations must be designed to thwart would-be vandals. The choice of pump must be compatible with the available electrical service, for example, 110 or 220 volts, single or three phase. A high-level alarm light or audible device (bell or buzzer) should be located in the house where any malfunction can be quickly noted by the occupants. If an audible alarm is used, a reset button should be conveniently located so that relief can be easily and quickly obtained. The pump should be wired for automatic level control with a manual override located at the control panel. Electrical connections to the pump should be easy to disconnect if the pump must be removed for servicing, but they must also be completely watertight.

The primary contingency concerns of the designer are the possibility of a power failure and the ability of the system to handle a pump malfunction. It has been noted[22] that, during the period from 1968 to 1972, the 187 power outages recorded in the United States lasted for the times shown in table I-2. Outages of more than 9 hours' duration were caused by major natural disasters, such as floods, hurricanes, and earthquakes. Under such conditions, it is unlikely that some septic tank effluent overflow will significantly add to the total effect of the tragedy. Because 9 hours appear to be a reasonable maximum outage, the system should be able to absorb the flow from the house for that period. One can very conservatively assume that no more than 50 percent of a daily household wastewater flow of about 200 gallons (0.76 m^3) would occur in that period considering the probability of reduced water use during the power outage. Because septic tanks usually have 6 to 12 inches (0.16 to 0.33 metre) of freeboard, a rectangular 1,000-gallon (3.8-m^3) tank could hold anywhere from 100 to 200 gallons (0.38 to 0.76 m^3), excluding the capacities of the effluent holding tank and house sewer. Also, the loss of power in rural areas that are served by individual wells and cisterns essentially eliminates any possibility for wastewater generation because water supplies be-

Table I-2.—*Power outages recorded in the United States, 1968-72*

Percent of total outages	Duration, hours
53	<1
81	<2
89	<3
95	<5
97	<9

come inaccessible. Because the system seems to handle power outages, the primary potential difficulty appears to be malfunctioning mechanical components.

The time involved between determination of a malfunction by the alarm light and the arrival of a repair crew is a function of the institutional approach of the sewer district. The approach, in turn, would be influenced by such factors as the prior existence of soil absorption fields, the size of existing septic tanks, and the number of system contributors. For example, if solid absorption beds were previously in use, an overflow from the effluent holding tank to the bed could be sufficient to permit a normal 5-day workweek for repair personnel. Also, a larger septic tank, with its increased storage capacity above its normal water level, would allow a somewhat more generous response time than would a smaller one. A larger number of contributors would justify having a larger repair staff employed by the authority; if the number is smaller, a contract servicing arrangement with a private firm might be more advantageous.

Rose[3] has posed the question of who should purchase, install, and maintain pressurization facilities. The unanimous opinion of several authors[1,15,32-34] has been that the maintenance of the pressurization unit, house service connection, and pressure mains, and the installation of the mains should be the responsibility of the authority (district, county, and so forth). Bowles[1] and Cochrane[15] have recommended that a sewer district own the pump, install it and the service line, and tap the pressure main for a fee. The homeowner would then be responsible for installing all lines, tanks, and electrical connections to the pump and paying for the electricity to operate the pump. The district would perform maintenance and make repairs, on request, for a service charge. Voell indicates that homeowners prefer to have a utility arrangement whereby regular maintenance and repair would be performed by utility employees.[33] Leckman has discussed the option of public versus private ownership of the pressurization facility, with the authority providing all maintenance and repair services in preference to a private contractor.[34] All sources recommend a well-stocked repair shop with sufficient replacement units for quick and easy exchange with malfunctioning units to allow for repairs to be made at the shop.

The number of replacement pumps that must be available is primarily a function of their reliability, the number of units in the system, and the rate of repair. For example, if a system of 100 pumps with a reliability factor of 0.99 (work 99 percent of the time) were involved, the necessary number of standby units could be computed by use of the binomial distribution. Accordingly, the following probabilities exist for the respective situations:

- One pump fails, $p = 0.370$
- Two pumps fail, $p = 0.185$
- Three pumps fail, $p = 0.061$
- Four pumps fail, $p = 0.015$

On the basis of this type of analysis, the authority would then have to make a decision on the number of spare pumps. Other factors involved are the average time required to repair a pump in the shop, the size of the maintenance staff, and delivery times for spare pumps and parts. Under present conditions, the last item may be a major concern.

Assuming that a reasonable response time by the authority's maintenance and repair crews would be less than the 9-hour maximum power outage figure, the STEP system appears to require no auxiliary holding capacity. If, however, an existing SAS is available, the minor cost of installing an emergency overflow drain from the effluent pumping tank to that system could be a prudent investment. The major concern in using an existing soil field is infiltration during wet periods that could result in a reverse flow from the field to the effluent pumping tank through the overflow drain.[14,a]

---

[a]S. M. Rees, SIECO, Inc., personal communication.

The pump chosen for pumping septic tank effluent should be selected on the basis of reasonable cost, reliability, proper head versus discharge characteristics, and compatibility with the application. To be compatible the pump must be able to handle organic and some light inorganic solids (negligible grit) and be reasonably resistant to sulfide and other septic effluent corrosiveness. The reliability criterion should be satisfied by rugged construction and resistance to moisture penetration, for example, submersible or watertight properties. If the above criteria are satisfied, the pump choice becomes one of economics and proper performance characteristics. In this type of application a centrifugal pump is probably the most economic selection.

Although proper pump selection is well discussed in the literature,[7,35,c] a few other items should be taken into consideration. To avoid major problems arising from dynamic dissimilarity, it would be prudent to install only one kind of pumping unit for all installations. In this way the units would be both geometrically and dynamically identical. Because little design information is available on this specific application of multiple pumping units, the methods of analysis discussed by Metcalf and Eddy[c] and Flanigan and Cudnik[35] should be helpful to the designer. The original ASCE report[7] indicated that the maximum economical curb pressure head should be equivalent to 69 feet (21 metres) of water and that the minimum pressurization unit discharge pressure head be equivalent to 0 to 11.5 feet (0 to 3.5 metres) of water. Therefore, a maximum discharge pressure head of about 81 feet of water (24.7 metres) was required. These numbers are only a function of the assumptions made in this study, but they represent reasonable target pressures. Higher working pressures may require stronger and more expensive piping materials, while lower pressures may restrict the capabilities of the system. There is no reason, however, not to use less restrictive pressurization criteria when the conditions of a particular site do not demand them.

As noted earlier, the most popular pump for STEP systems has been the low-head, submersible sump pump. The primary reasons for this pump's popularity have been generally flat terrain applications, low cost, and availability of parts. Unfortunately, until very recently no commercial pumps were available that were specifically designed for this type of application. Alternative units are needed to meet the requirements of these applications. One alternative approach for the STEP-type pressure sewers is to use a pneumatic ejector. The unit shown in figure I-14 is one of two types of pneumatic ejectors that have been developed expressly for pumping septic tank effluents. These units are manufactured by Clow Corporation and Franklin Research Company. Both units require an air compressor to impart discharge pressures.

Design methods for selection of STEP's are not easily found in the literature. A number of these types of installations have been designed and built, but very little information on them has been published. The hydraulic conditions that must be satisfied are primarily related to system size, pipe sizes and lengths, probability of simultaneous pumping, and growth characteristics of development. Most of these conditions will be discussed with the pressure main and service line design alternatives. The methods used to determine the applicability of a given pump have been outlined by Bowne,[32] Metcalf and Eddy,[c] and Flanigan and Cudnik.[35] Starting from the head-discharge curve shown in figure I-15, the operating point of each pump can be found by the methods that follow (using English units).

The static head on the pump is determined by

$$H_s = h_t - h_p$$

where

H_s = static head, feet

h_t = elevation of discharge point of pressure main, feet

h_p = elevation of pump, feet

---

[c]R. E. Lawford, Peabody Barnes Company, personal communication.

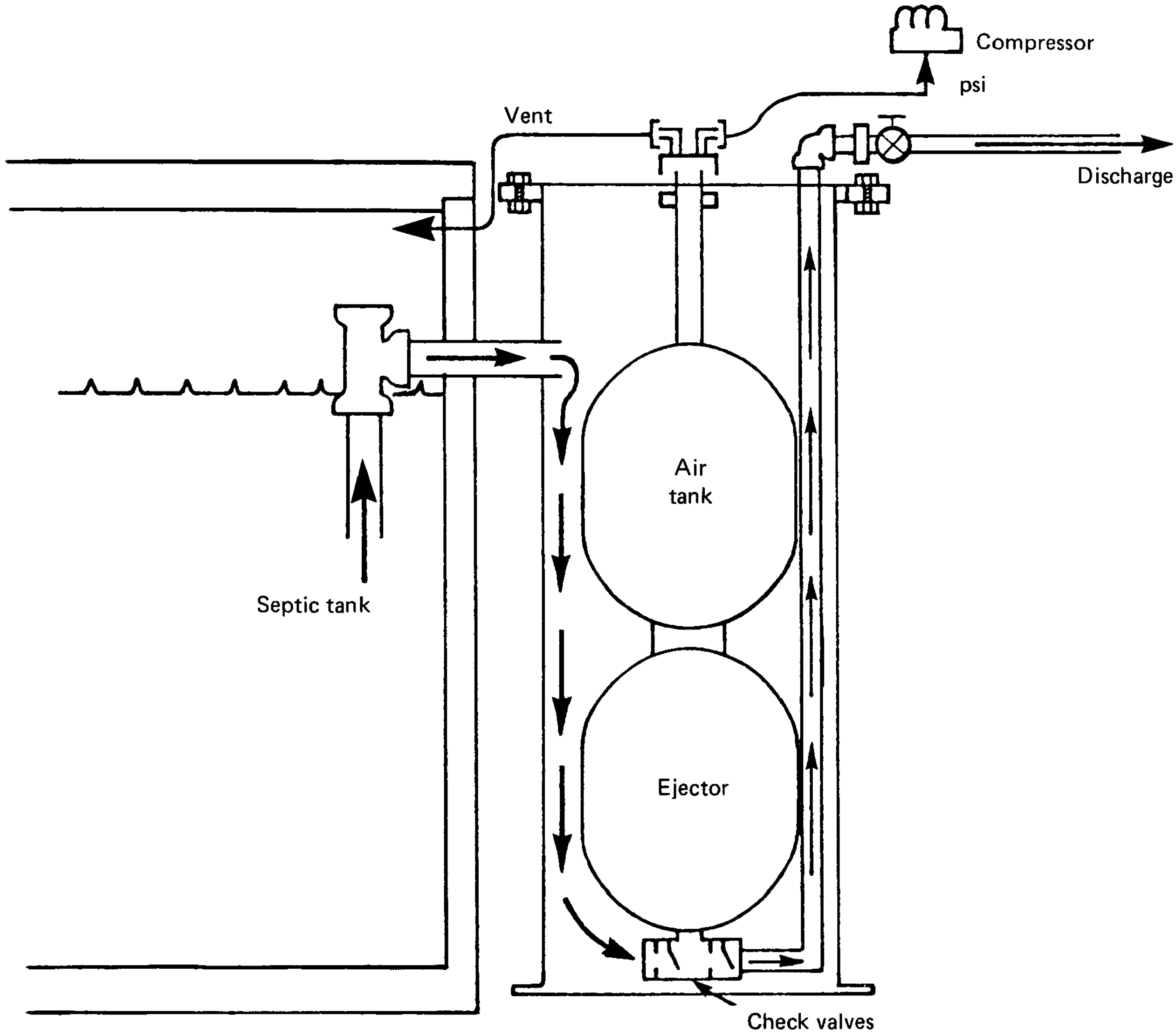


Figure I-14. Pneumatic ejector (courtesy of Clow Corporation).

The approximate dynamic head caused by pipe friction and other pipe constrictions, such as valves, bends, elbows, and other fittings, is determined. The pipe friction losses are usually computed by use of the Hazen-Williams formula,

$$H_{F1} = \frac{3.023}{d^{1.167}}\left(\frac{V}{C}\right)^{1.852}$$

where

H_{F1} = pipe friction head, feet

d = pipe diameter, feet

V = velocity of flow, ft/s

C = Hazen-Williams coefficient

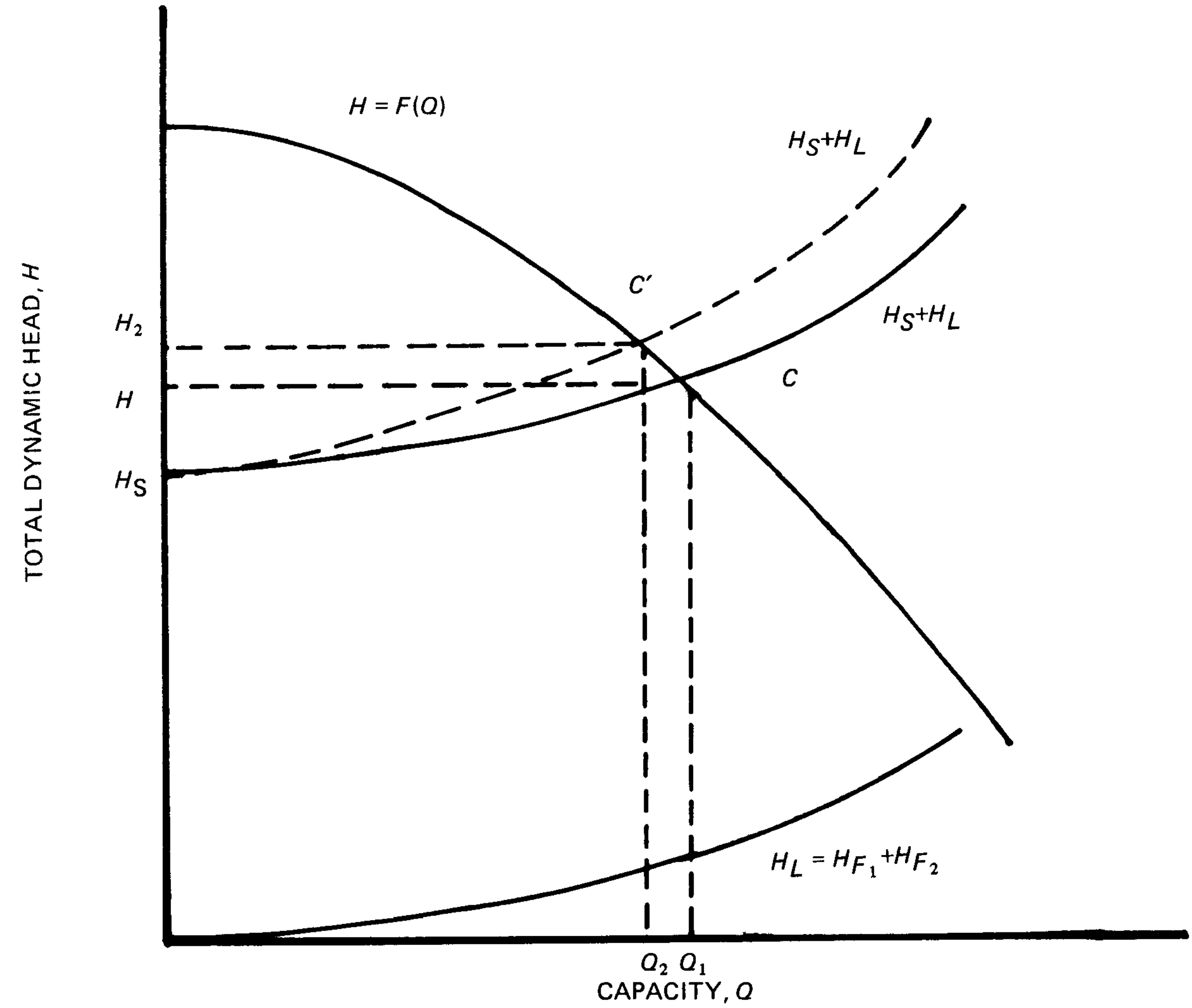


Figure I-15. Operating point of pump with characteristic $H = F(Q)$ and a pipeline with characteristics H_S+H_L.

while losses in valves and fittings are computed by use of the formula,

$$H_{F2} = K\frac{V^2}{2g}$$

where

H_{F2} = fitting friction head, feet

g = gravitational constant (32 ft/s^2)

K = fitting coefficient

Values of K for each type of fitting can be found in various hydraulics textbooks. The value of the Hazen-Williams coefficient C has been subject to interpretation. The Plastics Pipe Institute indicates that tests in several laboratories of new and used thermoplastic pipe resulted in C values ranging from 155 to 165, and recommends the use of a conservative value of 150.[36] The Albany and Phoenixville designs also used this value. Flanigan and Cudnik indicate that a C value of 150 is proper for clean water applications, but because of grease and other interfering matter present in wastewater, they recommend a C value of 140. They note that this conservative value should permit easier operation of the system during periods of stress and, if found through experience to be overly conservative, it can be revised upward. The Grandview Lake system was designed using a C value of 130.[a]

Applying the foregoing information at various values of Q (discharge, gal/min) will yield the total dynamic head (TDH) by:

$$\text{TDH} = H_s + H_{F1} = H_{F2}$$

Because H_{F1} and H_{F2} are functions of discharge, the TDH is represented by a nonlinear increasing curve in figure I-15, where $H_1 = H_{F1} + H_{F2}$. The intersection of these curves is called the "operating point." In normal pump selection design practice, the pump that has its optimum efficiency at this point would be chosen. However, because these pumps operate under varying conditions of TDH in a pressure sewer system and the cost of inefficient operation is negligible in this type of operation, this requirement is not very important.

This analysis is used to "plug in" the information of a tentative system design to the extreme cases, for example, the pumps requiring the most and least heads for operation. In the former case, a test of the highest TDH versus Q can be related to the adequacy of a particular pump design. In the latter case, a test can be made to determine any possible difficulties related to pump overloads and cavitation. Cavitation and overloading are unlikely with low-specific-speed pumps of the type used for this application, especially with discharge line losses. The maximum and minimum TDH analyses determine the variation in flows that can be expected from single-pump operation.

The problem of multiple-pump operation is far more complex. The pump head-capacity curve, shown in figure I-16, is assumed for all pumps in the system. Line and fitting losses, related to the service lateral that feeds the pressure main, are combined with the original head-capacity (*H-Q*) curve to produce a modified *H-Q* curve, as shown in figure I-16(c). Each pump then must be referenced to a single location on the main line, usually the point where the pump closest to the discharge end of the main enters the main line. This point or station is shown in figure I-17.

This referencing can be accomplished by a repeated series of combinations, that is, pumps 1 and 2 referenced to station 2 to get a combined *H-Q* curve at that point. This combination is then referenced to station 3 and combined with pump 3 and so on, until all the operating pumps are combined at the final point (in the case of fig. I-17, it is station 4 on the main line). The referencing process involves the conversion of the modified *H-Q* curve of the farthest pump (1 in this case) to the conditions at the main line connection of the closest pump (station 2 in this case) by converting the static and main line friction heads between the two stations. The major steps in the referencing process are shown graphically in figure I-16. This simplified example assumes that the pump and main line station are at the same elevation. In figure 16(a), the service discharge line losses are subtracted from the original *H-Q* curve to get the modified *H-Q* curve, and these losses are assumed to be the same for both identical pumps. In figure 16(b), the elevation difference and piping losses between stations 1 and 2 are shown and are applied to the modified curve from figure 16(a), to get the new *H-Q* curve for pump 1 at station 2. In figure 16(c), the pump curves at station 2 are shown separately and combined, along with the system curve, from station 2 to station 3. Theoretically, this type of analysis must be repeated until all the pumps in the system are related to station 4 in this example, or whatever the final station might be. Actually, a limited number of combinations will suffice, as discussed later in the section on pressure main design.

Bowne[32] has proposed using a much-simplified pump selection method, which takes advantage of a centrifugal pump's flexibility. Essentially, he establishes the hydraulic gradeline for the system

---

[a]S. M. Rees, SIECO, Inc., personal communication.

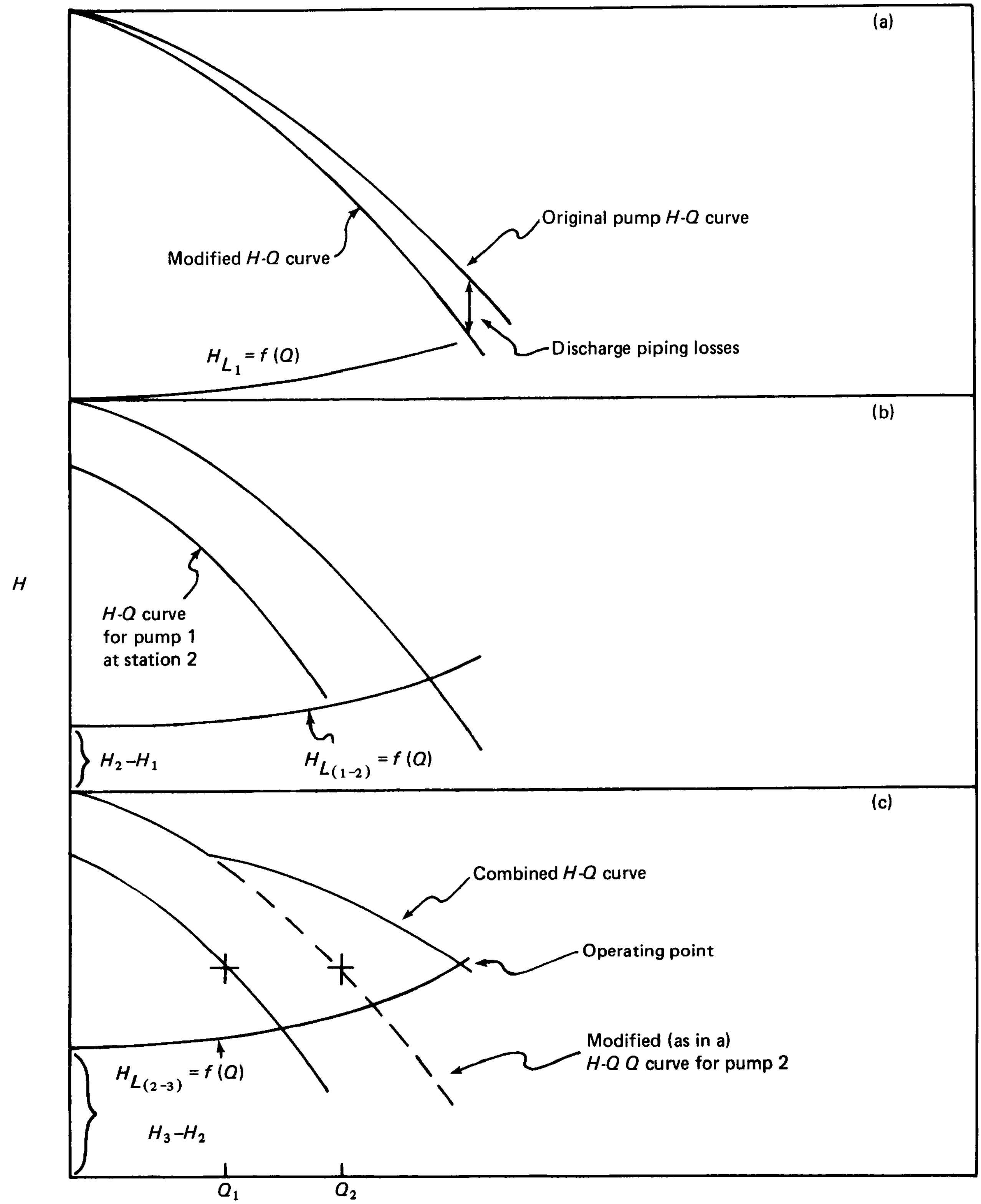


Figure I-16. Multiple pump operating analysis.

and determines the difference in elevation between the pump level and the hydraulic gradeline. Then, knowing the length and size of the service line, the modified pump H-Q curve will yield the adequacy of the pump at that location. By allowing a variance in the size of the pump discharge line, further design flexibility can be obtained. The number of simultaneous operations that may

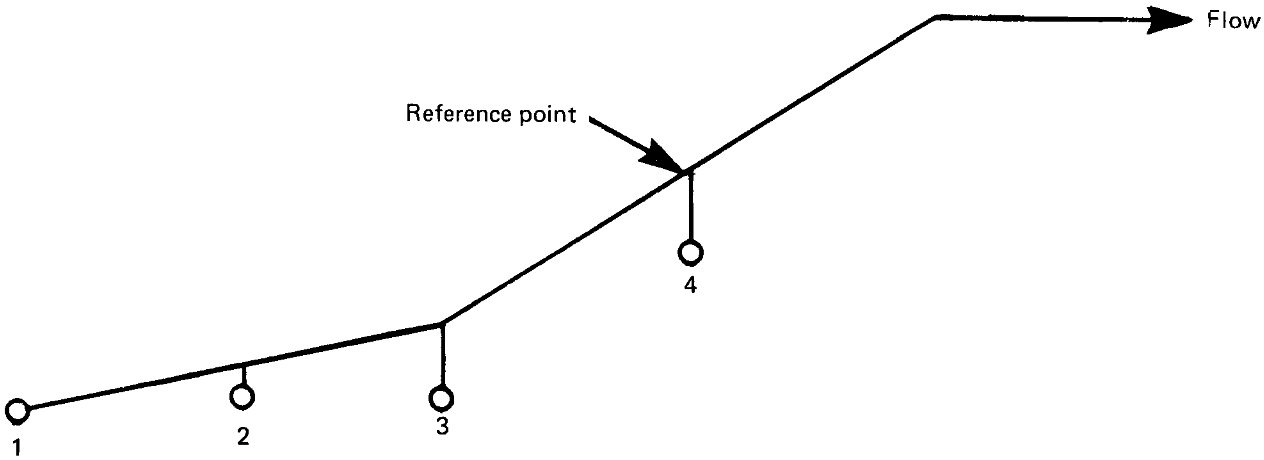


Figure I-17. Reference point for multiple pump operating analysis.

reasonably be expected will be discussed later, as will pipe sizes and other main line design factors that may impact pump selection. One of the advantages of centrifugal pumps is that they can operate for reasonable periods in a no discharge condition at heads greater than the maximum or shut-off head without significant damage to the unit. Flanigan and Cudnik relate an experiment wherein a 1.5-hp (1.2-kW) pump immersed in 70 gallons (265 litres) of water for 4 hours raised the temperature of the water to 122° F (50° C). This characteristic provides a cushion if a temporary high-head condition occurs in the pressure sewer system because of a malfunction or extraordinary flow. But pump discharge connections must be well designed and properly installed to resist the high pressure/ high temperature conditions of significant duration without serious damage.

GP's. Much of the preceding analysis provided for STEP stations is applicable to GP installations because the pressurization operation is essentially the same. Obviously, the septic tank is eliminated from the system along with the necessary inspections and cleanings that are associated with it. This exclusion may well be a determining economic factor when choosing the appropriate system for a home where no septic tank exists or for a multifamily dwelling that may require an exceedingly large septic tank to conform to local codes. It is also possible that a significant reduction in on-lot piping can be accomplished by not having to be tied to the existing septic tank location. Another degree of freedom is available with GP designs: the ability to install the unit in the basement of a home, as shown in figure I-10, for ease of maintenance and less-severe operating conditions. One other significant difference is the fact that more data are now available for GP systems than for STEP systems.

Design techniques will vary somewhat for GP installations compared to STEP installations, with respect to such things as emergency storage provisions and commercial package availability. But the basic design considerations are quite similar. The commercial aspect of the GP designs is worthy of discussion. As noted earlier, the concept of a GP for sanitary sewage transmission was integral to the original ASCE study.[7] As part of that study, General Electric Company developed a commercial GP unit in concert with waste generation, hydraulics, and other engineering factors. Certain members of the General Electric staff who were involved in this study later left and formed the E/One Corporation, which became a pioneer in GP-pressure sewer development. Since that time, E/One and the Hydromatic Pump Division of Weil-McLain Company have become the leading suppliers of GP units. There are other firms in competition with them, such as Robbins & Meyers, Toran, Peabody-Barnes, and Empo-Cornell (no attempt has been made to compile a complete list because any such list would be accurate only at the time it was compiled). E/One and Hydromatic units represent the two major GP design choices, that is, a progressing-cavity (semipositive-displacement) pumping element and a centrifugal pumping element, respectively.

The *H-Q* curves for the basic single-home models of each manufacturer presented in figure I-18 differ markedly, as would be expected. The E/One pump has been shown to be capable of operation

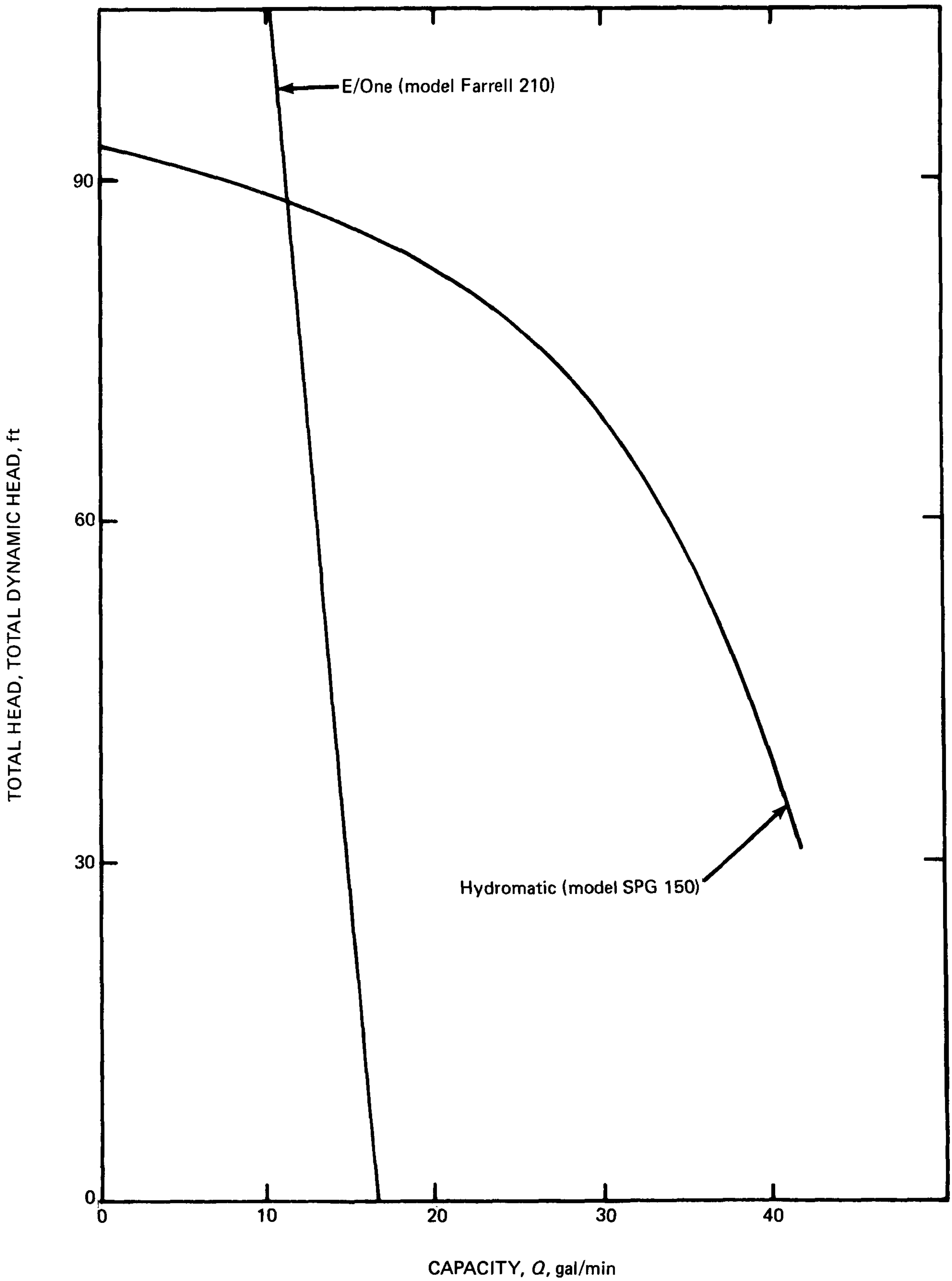


Figure I-18. Grinder pump characteristics.

above the 81-foot (24.7-metre) design limit for a considerable number of cycles by the National Sanitation Foundation (NSF).[37] The extreme condition of operation would occur immediately following restoration of power after a prolonged outage. Although it is unlikely that normal wastewater generation patterns would exist during such an outage, the assumption is that all or a significantly large number of the total units could be at or above their discharge or actuation levels. Therefore, at the instant power is restored, it could be assumed that all units would commence to discharge. However, because the resultant TDH would be greater than both maximum heads, a sequential pumpout would likely occur. The sequence would initially permit discharge from those units that pump against the least TDH, for example, in the case of a "flat" system, the units closest to the discharge point. The other units in the system that cannot discharge because of excessive TDH must wait their turn (thus, the sequential pumpout). During the period of excess TDH, centrifugal units will rotate without discharging, with the input energy being dissipated as heat. The previous section indicated that 4 hours of such operation, with a 1.5-hp (1.1-kW) GP, raised the temperature of 70 gallons (263 litres) of water to 122° F (50° C).[35] Because all the units in a small community system would likely be emptied in less time than this, no difficulties should be experienced. The progressing-cavity design, when pumping against excessive TDH, uses a thermal overload protector with automatic reset capability. Because this type of unit can pump at destructive pressures when unchecked, the thermal overload feature is intended to protect the motor and discharge piping from potentially damaging excessive pressure development. The automatic reset then allows the unit to cycle as many times as necessary until the tank contents can be discharged.

The minimum TDH operating condition is the single unit discharge. From figure I-18, it is apparent that the centrifugal unit will pump a given quantity of sewage in a significantly shorter time at any TDH below 88 feet (26.8 metres) of water. This provides a higher velocity in the discharge system and reduces the probability that simultaneous pump operations will occur.

Each manufacturer offers a package that includes the pump and its mounting, holding tank, controls, and other accessories. The E/One unit includes two flapper check valves, a pressure-sensing switch, and an antisiphon valve. The Hydromatic unit includes a ball check valve, a mercury level switch, a gate valve, and a control box. Each includes various ancillary items in their commercial packages.

The problem of emergency overflow storage was addressed in the previous section where septic tanks were found to offer sufficient storage capacity for extended power outages or system malfunctions. GP holding tanks are not normally large enough to provide proper storage of raw sewage. A solution proposed to handle this contingency includes the one shown in figure I-5. These absorption pits and beds were installed when an existing ST-SAS was not available. The specifications required that the bed have a minimum volume of 500 ft^3 (14.2 m^3) and be filled with pea gravel. The operating principle of this type of system is that the overflow will percolate into the ground at some finite rate. As pointed out previously, one caution with this type of system is that infiltration may occur from the bed or pit back to the GP tank during wet periods.[a]

Another design proposed by Schultz[d] is shown in figure I-19. This 200-gallon (0.76-m^3) holding tank lies between the house and the GP unit. Under normal conditions, the raw sewage flows along the bottom of the tank and does not accumulate. During an emergency overflow condition, the sewage backs up into the tank and not into the house because the top of the tank is at least 2 inches (5.1 cm) below the level of the lowest house drain. When power or system operation is restored, the emergency tank drains by gravity into the GP tank. The Phoenixville and Bend systems also use existing septic tank systems for emergency overflows.[11,h]

Leckman has discussed various other alternatives, such as standby power, water service termination, other holding tank designs, and interconnection with an adjacent GP unit.[34] He estimated

[a]S. M. Rees, SIECO, Inc., personal communication.

[d]J. Schultz, Becher-Hoppe Engineers, Inc., personal communication.

[h]L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.

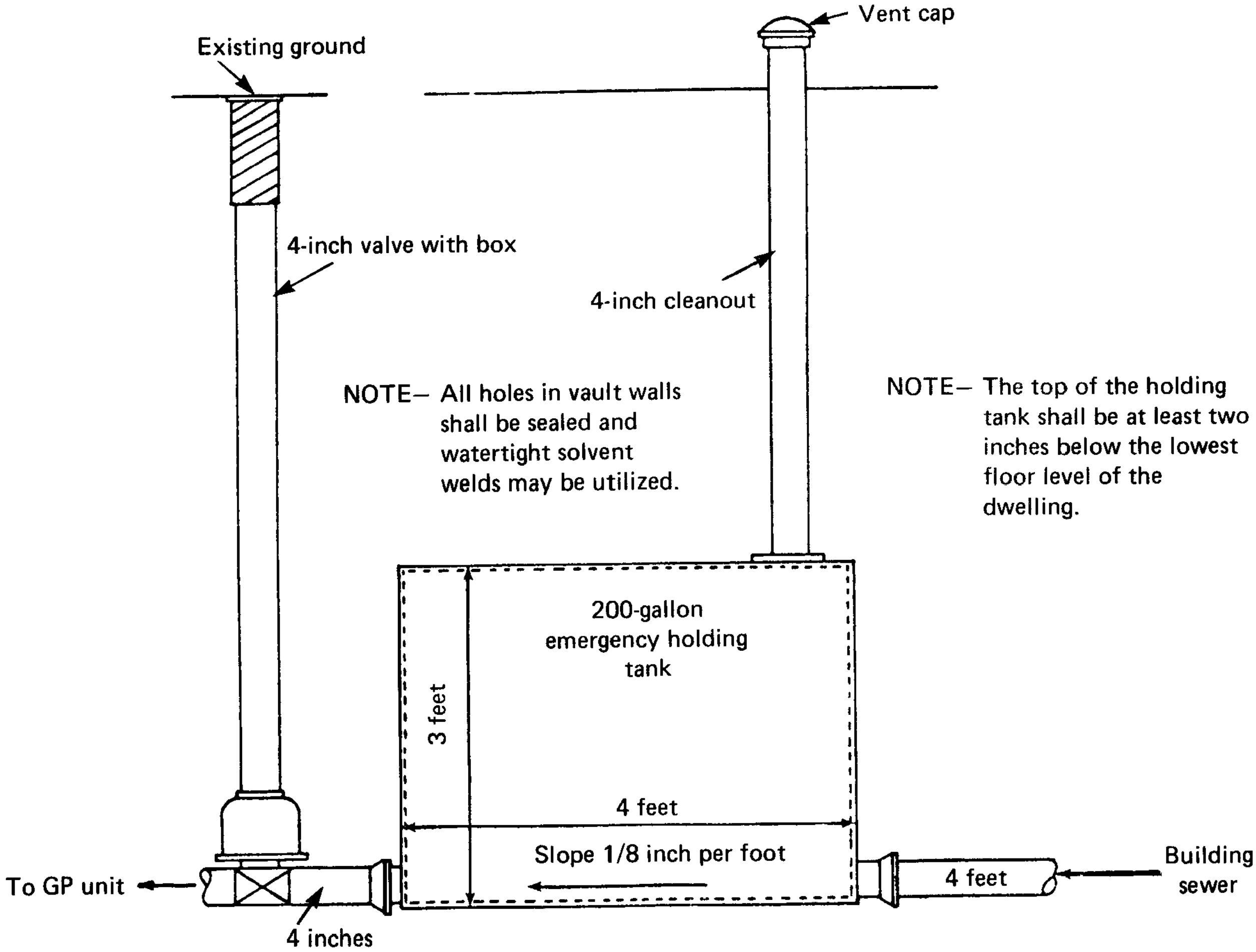


Figure I-19. Emergency tank design.

the cost of a standby generator at $750 (1972 prices) with substantial maintenance costs. Water main service could be terminated at the curb during power failures or planned maintenance activities to prevent overflow conditions. Holding tanks could be conveniently located and pumped out by septic tank pumpers, that is, interim holding tank systems. If GP units were close enough, interconnecting overflows would be possible during periods when malfunctioning of one unit occurs.

It is essential that GP systems have some type of contingency arrangement, based on the assumption that public health considerations preclude the possibility of raw sewage contamination of lawns, basements, and so forth. Confronted with this problem, the designer must choose one of the foregoing alternatives or develop a new means of storage for such emergencies. The choice should be influenced by the servicing arrangement, fluctuation of groundwater levels in the area, and available existing facilities. Also, the contingency approach should minimize the cost to homeowners.

System

Service Connections. Service connections between each pressurization facility and the pressure main would be similar in almost any design. Key elements in this design are pipe material, pipe size, valves, and the connection to the pressure main. The use of 1.25-inch (3.18-cm) inside diameter plastic pipe has been documented as the best compromise between minimum required scouring velocities for GP systems and minimum headloss considerations.[6] This analysis was based on the *H-Q* curve of the E/One GP unit, however, and is not necessarily valid for other pressurization facilities. Under these conditions the minimum discharge pipe velocity was greater than 2 ft/s (0.61 m/s) and headloss was less than 4 ft/100 ft (4 cm/m) of pipe length. The Albany project used 1.25-inch (3.18-cm) PVC, Type I, schedule 40 service lines with PVC, drain, waste and vent (DWV) fittings.[8] The DWV fittings were used because of their smoother transition properties compared to schedule 40 or 80 fittings. At the end of this project grease accumulations were found that resulted in reductions of as high as 40 percent of the lateral (service line) cross-sectional area. The Phoenixville system used 1.5-inch (3.81-cm) PVC, SDR-26 service lines with schedule 40 fittings.[11] The Grandview Lake project used two types of PVC (SDR-21 and SDR-13.5), polypropylene, and polybutylene service connections. The latter two types, with brass fittings, were found to be more costly and more difficult to install but performed in an excellent manner when properly installed. The PVC, SDR-13.5 was considered the best of all service lines, and the PVC, SDR-21 was considered the worst on the basis of installation and operation performance.[14] Most of the Grandview Lake service lines used 1.0-inch (2.54-cm) piping. Bowles[1] noted that the Lake Lyndon Baines Johnson Municipal Utility District project uses PVC service lines from 1.25 to 2.0 inches (3.18 to 5.08 cm) in nominal diameter. The STEP system at Priest Lake incorporates 1.5-inch (3.81-cm) PVC, SDR-26 service laterals.[f] The Miami systems use either 1.25- or 1.5-inch (2.54- or 3.81-cm) PVC, SDR-26 service lines, while the Bend system uses only 1.25-inch (2.54-cm) PVC, SDR-26 laterals.[e,h]

The original ASCE study report[7] indicated that polyethylene pipe had the advantage of being plowed in for quick sewer installation. Essentially, "plowing in" refers to a system in which trenching, feeding of coiled tubing or pipe, and backfilling are accomplished in a single operation. Despite this advantage and the lower cost, polyethylene has not been used in any pressure sewer systems. The reason appears to be that pressure-resistant fittings are not available for polyethylene. Therefore, PVC has been almost exclusively used for pressure sewer mains and service connections, despite its greater cost and its unadaptability to plowing methods. It is not inconceivable, however, that future development will allow the use of polyethylene.

The need for check and gate valves in service connections is obvious. Because the main is under pressure at all times, especially when one or more pressurization facilities are operating, check valves or backflow preventers are required on all pressurization units. In addition, some type of gate valve or equivalent is necessary to allow the isolation of each unit for repairs. As a result of the Albany study, Carcich et al.[2] indicated that shut-off valves are necessary both on the discharge side of the pressurization facility (GP or effluent pump) and on the service connection just before the pressure main (curb stop) to allow isolation of the pressurization facility or the pressure main. They also noted that the flapper-type pump discharge check valves required a horizontal run of pipe on the pump's downstream side to prevent the accumulation of solids that impeded normal operation of the flapper. Their project report[8] recommended that a 1-foot (0.30-metre) run of horizontal pipe be used, while Leckman[14] recommended 2 feet (0.61 metre). The Phoenixville system used a bronze swing-type check valve in a horizontal section of pipe.[11] The Grandview Lake installation used two types of bronze check valves, one with a vertical hanging gate and one with a 45° seating from the vertical. This latter design was reported to be superior.[14] The need for a horizontal run following GP discharge check valves was also documented, along with a preference for swing check valves over ball check valves.[14]

The Miami systems use a single check valve and gate valve on the discharge line of the pressurization unit. The check valves successfully used include plastic ball check valves, brass flapper valves, and plastic flapper valves. The gate valves used for these systems are made of brass or plastic. All of the above types of valves have been trouble free for as long as 5 years. Serious problems have been noted in these systems with valves made of steel or iron because of corrosion (iron sulfide), which caused failure within 2 years.[e]

---

[e] H. Schmidt, General Development Utilities Company, personal communication.
[f] C. W. Rose, Farmer's Home Administration, personal communication.
[h] L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.

At least two manufacturers now offer flexible rubberlike check valves for pressure sewer systems. Although conceptually attractive, these devices will require field experience to determine if they can retain their elasticity for an acceptable number of years under these severe conditions of service.

On-off valve selection and location should also be described. A study of the relative merits of gate and ball valves indicates that ball valves are more compact and lighter than gate valves, and they offer slightly less fluid friction (pressure drop) when in the fully open position. These differences are not normally so significant, however, that relative economics will not be the primary determinant of final choice.

Although some designers have indicated a preference for locating the on-off valve outside of the pressurization holding tank in order to permit valve operation without removing the tank cover, such a design decision would necessarily be a function of the ease with which the cover can be removed, redundancy of on-off valves on the service line, and economic trade-offs for the initial cost of the on-lot facilities.[e,32] It is likely that the valve would be located within a holding tank of sufficient size where a curb valve is also used for the service line.

Some designers prefer backup check and on-off (gate or ball) valves; others feel that such backup is superfluous.[e,32] The use of two on-off valves has been more common than dual check valves. Usually, a corporation stop or curb valve is located near the service line-pressure main connection to permit isolation of the entire service line and pressurization facility from the main, in addition to the on-off valve, which is generally located at the pump holding tank. This arrangement permits isolation of the pressurization unit from the service line in addition to providing a redundancy in isolation from the main. The use of two check valves is less clear. If such a design is chosen, however, it could be accomplished by two separate check valves or a dual check valve in a single housing.

In order to connect each service lateral to the pressure main at Albany and Phoenixville, sanitary tees with 45° elbows for 1.25-inch (3.18-cm) connections were used.[8,11] The Grandview Lake system used curb, cock-tapping saddle connections to the main, as shown in figure I-20.[14] The choice of service line connection may be based on relative economics or headloss considerations. One alternative would be to use standard 20-foot (6.1-metre) lengths of PVC pipe in laying the pressure main with later return to tap the main for each service line connection at its most advantageous location based on the user's lot geometry. Saddles and tapping tools for PVC are commonly available in most locations. Another alternative would be to provide a tee or valve connection between standard lengths of PVC main during construction so that each potential user would have a connection for his lateral. Variations on these two alternatives are manifold, and the economic trade-offs must be weighed against the objectives of the sewer authority. A list of suppliers and manufacturers of the necessary service line pipe and fittings, as well as pressure main items, can be obtained from the Plastics Pipe Institute, 250 Park Ave., New York, N.Y., 10017.

As noted previously, the size of service connection lines is normally between 1 and 2 inches (2.5 and 5.1 cm) in diameter. However, the designer should determine the proper size based primarily on the characteristics of the pressurization pumps used in his particular system; that is, a pipe size that is proper for a 15-gal/min (0.95-l/s) pump may not be proper for a 30-gal/min (1.89-l/s) pump. The choice, which is based on the trade-off between headloss (which includes service line length) and minimum scouring velocity, becomes more difficult with increasing system size. For smaller installations, where the static head pumped against is minimal, the use of smaller service lines may be prudent. In larger systems, having significant static and dynamic head, the use of larger laterals may be necessary. Greater flexibility in pipe sizing is available with STEP systems because of less-stringent minimum velocity concerns. When the complete system hydraulics is analyzed, the

[e] H. Schmidt, General Development Utilities Company, personal communication.

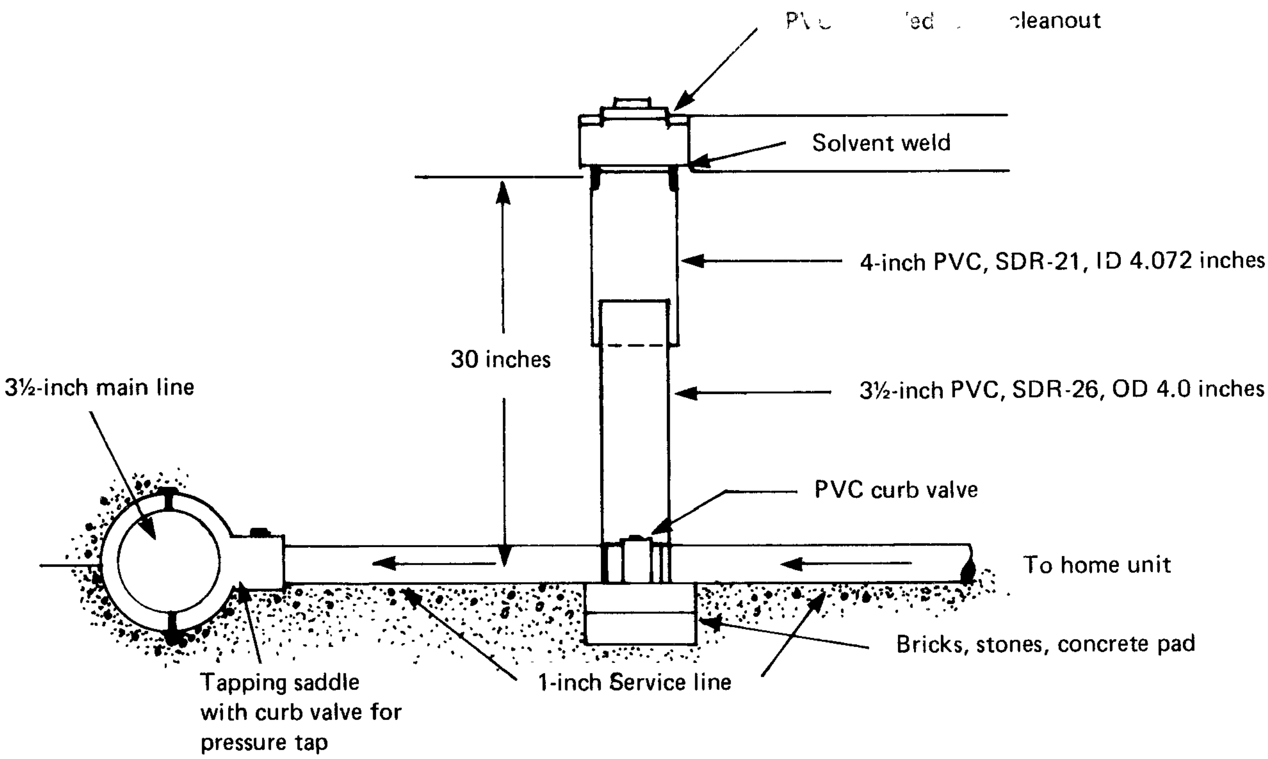


Figure I-20. Curb valve and riser installation.

decision can be made in context with all other hydraulic considerations, including friction losses caused by valves and fittings. In the Albany study approximately 32 percent of the total friction loss was computed to be caused by these items.[8]

Pressure Mains. Design considerations for pressure mains include the general layout, hydraulic load factors, appurtenances, maintenance provisions, and the type and size of piping. The general layout of a pressure sewer system was given careful scrutiny during the ASCE study.[7] One of the original objectives was to minimize the residence time of sewage in the system. Eliminating loops in the system results in reduced storage time, less opportunity for solids settling and odor production, fewer abrupt direction changes, and less headloss because of fittings. One potential difficulty with eliminating loops is problems with pressure main repair, which will be discussed later. By employing a single main with unidirectional flow, the designer can route that main via the most direct path to its effluent point, taking into account grade and existing natural and manmade barriers. The E/One design handbook also recommends the use of separate sections or branches, where possible, to minimize difficulties that result from occasional main repairs.[22] An example of this approach is shown in figure I-11, where one alternative could have been to extend the main (branch 7) vertically to the top of the figure and feed it directly from each pressurization unit.

Because the pressure sewer systems at Albany and Phoenixville pumped against a static head, that is, the terminal level of the pressure main was at a higher elevation than the remainder of the system, maintaining a positive pressure in the main was no problem. Difficulties can occur if a positive pressure cannot be maintained because of high points in the system. Several investigators[7,32,35] have recommended positive pressure maintenance at all points within a system, but such a conclusion must be weighed against a possible combined pressure gravity system where the latter is feasible. If a system were located on flat terrain, the use of a standpipe could accomplish the need for a positive pressure gradient. The investigators[7,32,35] all describe the use of pressure-sustaining and pressure-control valves to provide sufficient back pressure or artificial head to prevent draining of portions of the line and the associated problems of siphoning and flow impairment. If undulating terrain or long downhill runs are involved, these devices may be necessary if a combined system is not feasible. There is no reported experience with these devices, but the ASCE report[7] and Bowne[26] describe their function in greater detail.

The Grandview Lake, Phoenixville, Radcliff, and Priest Lake systems used air-release valves; the E/One design handbook,[22] the ASCE report,[7] and Flanigan and Cudnik[35] recommend them. However, exact criteria for their need and placement are lacking. Air or any gas accumulation in pressure mains will increase flow resistance and the headloss against which the sewage must be pumped. The potential sources of air in the sewer include insufficient purging after main filling and testing, malfunctioning pumps, or the release of air that was in solution at the time of pumping. Other gases could also be released from the wastewater in the pressure main by pressure reduction, biological activity, or chemical reactions. Gas bubbles accumulate along the crown of the sewer and ultimately move toward the high points in the pressure main. Once located there, the multibubble or "foamlike" structure tends to disappear in favor of a single large bubble configuration. This large bubble continues to grow until the drag force of the flowing liquid exceeds the pipe centerline component of the buoyant forces long enough to carry the bubble in the direction of flow (with minimum slippage) beyond the next low point in the system. If the duration of necessary flow is insufficient, the bubble will return to its original location. Kent has determined the relationship between slope, velocity, and bubble size.[38] According to his studies, at a 10° downslope, a bubble 3 inches (7.6 cm) long requires a velocity of almost 1.2 ft/s (0.36 m/s) to move through a 2-inch (5.1-cm) pipe. The additional headloss caused by the bubble is a function of relative volume of air to water. For the example above, the additional headloss because of air could be as high as 50 ft/1,000 ft (1.5 to 15 mm/m) of pipe length. Flanigan and Cudnik[35] and Farrell[39] have also examined this required bubble transport velocity but have not defined the requirement for sufficient duration of velocities in excess of this minimum. The hydraulic calculations are best described by Kent, but the requirement for sufficient duration is quite difficult to achieve because pressurization units rarely operate for much more than 1 minute or 2. If sufficient velocity were present in the main to move the bubble at a velocity of 1 ft/s (0.30 m/s), a 1-minute operation would only move the bubble 60 feet (18.3 metres). Because each pumping generally contains about 15 gallons (56.9 litres) of sewage, the internal volumes of three common PVC pipes would correlate to the displacement distances (1 foot = 0.305 metre) shown in table I-3. Minimum slippage is likely to occur between the liquid and the bubble so that the bubble will be displaced at a rate essentially equal to the average velocity of the fluid times the duration of flow.[38] By using some engineering judgment, combined with calculated velocities, displacement volumes, and system geometry, the designer can make reasonable decisions about the need and placement of air-release valves. (Bowne presents an interesting discussion on this subject.[32])

When it is determined that an air-release valve is required, the type (automatic or manual) must be chosen. Automatic air-release valves are available for sewer application that permit accumulated gas to escape at the valve without loss of liquid. Such valves require regular maintenance in the form of inspection and flushing to minimize clogging by sewage solids and grease. Manual valves are essentially vertical risers attached by a corporation cock to the crown of the pressure main at a high point. Although maintenance is minimal, a regular schedule of manual operation must be used to release trapped air. Both types require an access way for required operation and maintenance.

Table I-3.—*Internal volumes of PVC pipes and displacement distances*

Nominal size, in	Schedule 40, ft	SDR-21, ft	SDR-26, ft
1.25	193	163	157
1.5	142	124	120
2.0	86	77	76
3.0	39	37	35

The pressure sewer main is subject to malfunctions that directly or indirectly cause shutdowns. When these occur, all or some of the homes will be deprived of service for a period of time. Because the branched or dendriform design of a pressure sewer system has already been shown to be desirable, all homes upstream of the shutoff point will be without service. Two questions that then must be answered are:

- How quickly can repairs be made?
- Can the shutdown area be bypassed?

The design aspects of these questions are:

- What main line ancillary facilities are required?
- What would be an optimum spacing?

The question of main line valve requirements and spacing has been dealt with in numerous cases, even though they were not present in the Albany system and were not adequately used either at Phoenixville or Grandview Lake. In the Grandview Lake project, however, the desirability of using main line gate valves to isolate sections for repair was noted by the engineer.[14] The NSF sewer layout described in the ASCE report[7] suggested 600-foot (183-metre) intervals between valve and cleanout facilities, while Leckman[34] suggested a maximum distance of 400 feet (122 metres). The typical in-line cleanout facility is shown in figure I-21. Two facts should be noted regarding this figure. First, this design does not provide for a temporary bypass to minimize the number of units out of operation; second, the space and depth requirements of the valve box are obviously reduced compared to a conventional manhole. The second factor constitutes additional capital cost savings. The distance between cleanouts finally chosen by the designer will be a function of the size of the system, topography, and layout. On long runs in larger systems the 400- to 600-foot (122- to 183-

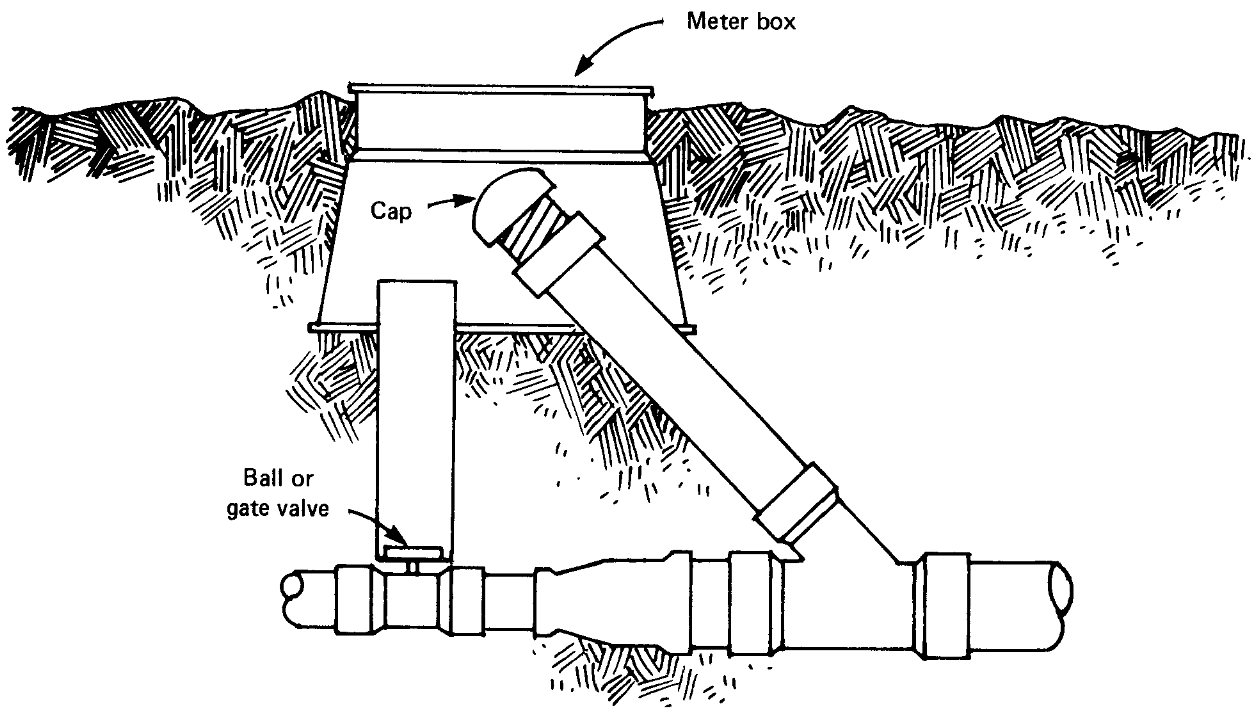


Figure I-21. In-line cleanout.

metre) maximum separation would relate to the density of contributors and the capabilities of mechanical cleaning equipment. A minimum requirement would include one of these facilities located on the main line wherever a branch main is to connect, as shown in figure I-10, or wherever pipe sizes change. State and local regulatory officials will necessarily be the final arbiters in all cases.

The problem of bypassing between valve locations relates to the system's physical design. Because some main line repairs are likely to require significant time to complete, the question of emergency overflow capacity again arises. The primary design question involves the relationship between the isolated (down) portion of the system and the upstream portion. Why do the upstream users need service when the isolated users do not? Two things that affect the validity of this question (but do not eliminate it) are: Isolated users can more easily be contacted to request reduced use of water during the outage and major repairs may take longer than the normal excess holding capacity can accommodate. In answering the question of bypassing, the designer should give strong consideration to the maximum repair operation, which would probably involve locating, excavating, and repairing a main break, as it compares to the emergency overflow capacity of the contributary units. If, after the comparison is made, the repair time does appear to be excessive, some form of bypassing should be considered. If the expected frequency of such occurrences is low, the bypassing arrangement considered should require minimal investment; if high, a more sophisticated approach may be required.

Two bypassing arrangements that have been proposed involve temporary hose connections and parallel mains. Voell discussed the use of temporary firehose connections. The only modifications required to implement this approach would involve the use of tees immediately upstream and downstream of a main line ball or gate valve, with a ball or gate valve and threaded fitting attached to each tee stem in the main line valve box. The provision of parallel lines at the time of construction is a relatively expensive solution. However, when the hydraulic load factor is low at the time of construction, one alternative technique that will be discussed later is the use of parallel lines of different sizes. If this approach were adopted, the existence of these parallel lines might offer a ready solution to the bypass problem.

Another item relating to the maintenance of the system is the terminal cleanout provision. A cleanout facility should be provided at the ends of each branch of a pressure sewer system. Figure I-22 shows a typical design of one of these facilities. It should be noted that watertightness is not necessarily required for these valve boxes, but local conditions such as high ground water levels and poor soil drainage should be weighed in making this determination.

Another design consideration is that of thrust blocking. As in water transmission practice, the designer must consider the need for thrust blocking or anchoring any bends, plugs, or caps by using standard techniques of calculation. The Grandview Lake system used these anchoring techniques on plugs, caps, and bends exceeding 22.5°.[14] Thrust blocks must not obstruct access to joints or other fittings of the system.

New housing developments represent a typical example of the load factor problem. When the population served at the time of construction is significantly less than the projected maximum, the hydraulic design may be extremely difficult, especially for GP systems where critical velocities are required. Obviously, if the ultimate number of contributors is twice the construction period number, velocities in the pressure main will also be related by a similar ratio. The entire aspect of hydraulic design is dealt with later, but some suggestions on the problem of varying load factors will be discussed here. One of the reasons for recommending the branched layout shown in figure I-11 is that smaller branch mains can be used efficiently by a community developer. Although the practice can be difficult, it is not uncommon for sections to be fully or at least greatly developed before new sections are opened for house construction. This approach offers many logistical advantages to a builder during the construction period.

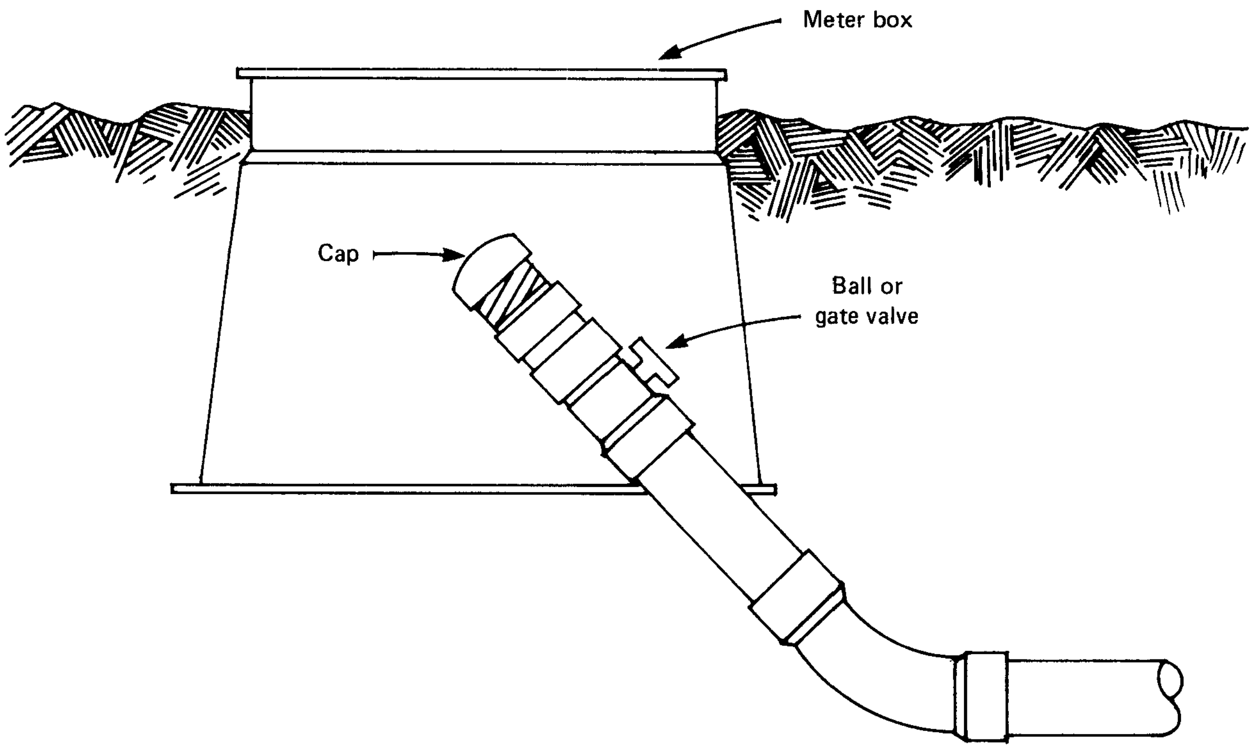


Figure I-22. Terminal cleanout.

Voell has discussed the use of dual lines, whereby one pressure sewer is sized for present needs and a second line is constructed for later development.[33] The pitfalls of this approach are manifold in redundant hookups and ancillary needs, which can translate into excessive costs as pointed out by Rees and Hendricks.[13] However, the earlier discussion on the need for main line contingency bypassing represents a possible advantage for this approach.

Probably the most widely accepted approach is the use of flushing to provide scouring velocity for minimization of grease and solids accumulations. These flushing units should discharge a volume of liquid that is at least equal to the total internal volume of the pressurized branch being flushed. Although Bowles[1] and Voell[33] suggest using flushing devices that employ potable water supplied with approved air breaks, this approach negates one of the advantages of pressure sewers, that is, reduced sewage volumes. The Grandview Lake design used 1,000-gallon (3.8-m^3) holding tanks equipped with end suction and 0.75-hp (0.56-kW) centrifugal pumps to flush the pressure mains.[14] These pumps were actuated by timers and pumping was stopped by low-level float switches. The flushing liquid was septic tank effluent. This sytem, therefore, did not increase the quantity of wastewater to be treated from the pressurized system. Also, the timers can be set to provide flushing during minimum flow periods (between 12 a.m. and 7 a.m.).

If initial flows are significantly less than the design flows, the use of flushing units appears to be the most advantageous design procedure, especially for GP systems. Considerations in their design are: the volume of main to be flushed, required scouring velocities, the number of household sources required to provide the necessary volume, and proper pump selection to obtain the necessary flows at the projected TDH.

The hydraulic design of the pressure main has been discussed by a number of authors.[2,8,11,19,22,32,34,35,40] The original work for the ASCE project was done by Hobbs.[40] His work determined the relationship between sewage characteristics and carrying velocity for pressure sewers. He found that the minimum scouring and maximum redepositing velocities for 2- to 8-inch (5.1- to 20.3-cm) plastic pipe are nearly identical and can be estimated by the equation,

$$V_s = \sqrt{d}/2$$

where

V_s = minimum scouring velocity, ft/s

d = inside diameter of pipe, inches

The critical material in the sewage tested was found to be sand. Although some evidence of a sand size versus V_s relationship was evident, insufficient data were taken to define it. The sewage tested had grease concentrations that ranged from 15 to 365 mg/l. Tests were conducted in their entirety *without* lengthy no-flow periods.

Carcich et al.[2] noted that the above criterion refers only to prevention of solids-settling. In the Albany study, the accumulations (which amounted to as much as 40 percent of the pipe cross-sectional area) were all located in the crown of the pipe and consisted primarily of grease. In explaining the problem, it was noted that during periods of inoperation ($V = 0$), grease accumulation at the crown is inevitable. These periods allow the release of gases that float the solids that combine with the grease at the crown to create a solidified mass of substantial strength and durability, which could be highly resistant to dislodgement. The crown-oriented mass creates greater flow resistance in the pipe.

The problem of grease accumulations affects the hydraulic design problem in two ways: The friction coefficient for the pipe will be different from its nominal "clean water" value, and scouring velocities are thereby required to minimize the effects of these accumulations. The magnitude of the grease problem is obviously greater in systems using GP units than in those using STEP systems because grit, grease, and SS are removed in the septic tank.

The Albany project was designed on the following assumptions:

- The maximum number of GP units operating simultaneously would be all 12 in the system.
- The minimum number of units routinely operating would be four.
- Four units would then provide a flow greater than the minimum scouring velocity.[8]

The study determined that the system was hydraulically overdesigned, because the following frequencies were actually obtained:

- Two simultaneous GP operations 20 times per day
- Three simultaneous operations at least once per day
- Four simultaneous operations about once every 14 days[2,8]

Two major GP system design decisions were modified on the basis of the Albany data. First, a minimum velocity of 2 ft/s (0.61 m/s) is required in all pipe sizes normally used in GP pressure systems.[2,8] Second, on the basis of these data and other information, the E/One Corporation produced the design table shown in table I-4.[22] There also appeared to be an inverse relationship between the number of users of a particular section of pipe and the amount of grease that accumu-

Table I-4.—*Simultaneous operat ı of grinder pump units*

Grinder pump units	Maximum operating simulta-neously
1	1
2-3	2
4-9	3
10-18	4
19-30	5
31-50	6
51-80	7
81-113	8
114-146	9
147-179	10
180-212	11
213-245	12
246-278	13
279-311	14
312-344	15

lated in each pipe size,[2,8] reinforcing the theory that no-flow periods allow grease accumulations to develop. However, grease accumulation in the pressure laterals did not follow this pattern.[8]

The Phoenixville GP system design was tested with a computer program that used the Hazen-Williams formula along with a mathematical expression for the *H-Q* relationship of the GP units.[11] This program checked all possible operating conditions to determine the headlosses and velocities that could occur at various locations in the system. An increase of the discharge head of one GP, from 81 feet (24.7 metres) at the start of the study to 123 feet (37.5 metres) at the end indicated that some constriction had developed in the system.

The Grandview Lake design was based on the design engineer's peak water demand curve. A value of 80 percent of the peak water demand was used to determine the maximum simultaneous use of home units. A Hazen-Williams *C* factor of 130 was used to estimate pipe friction losses.[a] As a result of the Grandview Lake experience, the engineer is designing new STEP systems with a peak flow of 70 percent of their peak water demand rate and a minimum velocity of 2 ft/s (0.61 m/s) instead of the 1 ft/s (0.30 m/s) minimum used at Grandview Lake.[19]

A hydraulic design procedure for GP systems has been recommended by the E/One Corporation.[22] After a preliminary layout of the branches and pressurization facilities, a tabular analysis of the system is made. Table I-5 represents this type of analysis for the GP system design shown in figure I-10. The maximum number of units operating simultaneously is estimated by using table I-4, and the maximum flows are assumed to be 11 gal/min (0.69 l/s) per GP unit. From these data, the maximum velocity is obtained by using the pipe cross-sectional area. The velocity relates to specific headlosses per unit length of pipe (available from the company based on a Hazen-Williams *C* factor of 150). Flanigan and Cudnik[35] present a strong case for the use of $C = 140$ to allow for grease and other deposits on the inside of the pipe. The accumulated frictional headlosses at this maximum flow condition are determined, starting at the discharge point. The E/One procedure then superimposes the static head difference between the highest elevation in the system between the pumps

[a]S. M. Rees, SIECO, Inc., personal communication.

Table I-5.—*Pressure sewer system PVC, SDR-21 pipe schedule and branch analysis sheet*

Branch no.	No. of pumps	Cumulative total	Maximum on	Maximum flow, gal/min	Size, in	Maximum velocity, ft/s	Length, ft	Friction loss, ft/100 ft	Friction loss, total ft	Headloss, total ft	Max. elevation main, ft	Pump site elevation, ft	Elevation difference, ft	Maximum total head, ft
1	3	3	2	22	2	2.03	60	0.79	0.48	34.18	785	735	50	84.18
	6	9	3	33	2	3.05	500	1.68	8.40	33.70	785	738	47	80.70
	2	11	4	44	2	4.07	180	2.85	5.12	25.30	785	748	37	62.30
2	3	3	2	22	2	2.03	80	.79	.79	28.39	785	765	20	48.39
	5	8	3	33	2	3.05	450	1.68	7.58	27.76	785	760	25	52.76
1+2		19												
3	0	19	5	55	2	5.08	300	4.31	12.90	20.18	785	751	34	54.18
4	3	3	2	22	2	2.03	60	.79	.48	34.18	785	744	41	75.18
	6	9	3	33	2	3.05	500	1.68	8.40	33.70	785	746	39	72.70
	2	11	4	44	2	4.07	180	2.85	5.12	25.30	785	755	30	55.30
5	3	3	2	22	2	2.03	80	.79	.63	28.39	785	770	15	43.39
	5	8	3	33	2	3.05	450	1.68	7.58	27.76	785	768	17	44.76
4+5		19												
6	0	19	5	55	2	5.08	300	4.31	12.90	20.18	785	758	27	47.18
3+6		38												
7	0	38	6	66	3	2.81	800	.91	7.28	7.28	785	755	30	37.28

in question and the discharge point. The sum of the dynamic and static heads should then be made approximately equal to 81 feet (24.7 metres) by the E/One method. One possible difficulty is the use of a Hazen-Williams *C* factor of 150. Although nonspecific on the subject, the E/One tables suggest that pipe sized to accommodate a minimum velocity of about 1.8 ft/s (0.55 m/s) be used.

Flanigan and Cudnik recommend that velocities ranging from 2 to 5 ft/s (0.61 to 1.52 m/s) be used and that flushing be provided. They also offer a series of tables of suggested design flows based on number of units, occupants per dwelling unit, and level of affluence to occupants. As in the E/One approach, the design flows are expected to occur once or twice per day. As noted earlier, a *C* factor of 140 is recommended at this time, with a possible revision upward with more experience.[35]

Sanson[19] has indicated the latest approach of SIECO, Inc., which relates to the company's experience with peak water supply demands. Pressure mains are sized on the basis of a sewage flow of 70 percent of the peak water demand. This assumption is based on a pump that pumps 10 gal/min (0.63 l/s) at a design head of 80 feet (24.4 metres). A minimum velocity of 2 ft/s (0.61 m/s) is also required. The use of a *C* factor of 130 or less has been indicated.[a]

Figure I-23 is a plot of the foregoing design flows for various sizes of pressure sewer systems. The figure includes the recommendations of Sanson[19] and E/One[22] and the maximum and minimum recommendations of Flanigan and Cudnik.[34] This last report recommended eight levels of design, but only the two extremes are shown in the figure. These extremes represent daily flows of 400 and 175 gallons (0.61 and 1.53 m^3) per household. Sanson's curve is based on 200 gal/d (9.76 m^3/d) per connection, while the E/One curve is said to be based on peak flows obtained in the Albany project and other existing pressure sewer systems.[22] In terms of peak to average flow, the Flanigan and Cudnik curves for smaller systems depend on the capacity versus TDH curves of the pumping units chosen. For example, the peak flow predicted by wastewater diurnal flow equations is usually less than the author's suggested design peak flow. The reasons relate to the possibility of simultaneous operation of pumping units with much greater flow capacity. Because this assumption is also the basis of the E/One curve, it may explain the similarity of the E/One and Flanigan and Cudnik curves for systems of 20 or fewer homes. Sanson's curve,[19] however, is based on the fact that centrifugal STEP's will seek their own equilibrium condition at all times and will not affect sewer design beyond the peak flow assumption as a percent of peak water demand. This peak flow assumption is based on the *H-Q* curve of the pump at the assumed maximum pressure of the system. For the curve plotted in figure I-23, a pump capable of pumping 10 gal/min (0.63 l/s) at 81 feet (24.7 metres) of TDH was assumed along with a maximum peak household water demand of 15 gal/min (9.95 l/s) to obtain a (10/15) (100) ≅ 70 percent factor to be applied to the peak water demand curve. Bowne[32] has chosen the Flanigan and Cudnik curve, which is based on 215 gal/d (0.81 m^3/d), for his design. The generally lower peak flows reflected by Sanson's curve take into consideration the flow-smoothing capabilities of centrifugal pumps, that is, the ability of these pumps to adjust to prevailing hydraulic conditions.

Once design flows have been chosen, the pipe sizes can be determined by using the Hazen-Williams formula. At this time, with minimum experience in the use of these systems, a maximum *C* factor of 140 seems prudent to provide some safety factor for flows in excess of the design flow and to allow for grease or other accumulations on the inner walls of the pipe. Other design checks should include the load factor and possible need for flushing. Another factor to consider is the headloss because of valves and fittings in the system. The Albany report estimated that the losses caused by valves and fittings were 32 percent of the total friction loss.[8] This reinforces the need to use a more conservative *C* factor and also implies that designers should incorporate additional safety factors when considering critical conditions. Hydraulic friction loss data for PVC valves and fittings are becoming generally available for use as part of the design calculations.

Bowne offers a simplified design procedure for use in STEP systems with centrifugal pumping.[32] After determining the number of homes to be served by the system, a peak flow is estab-

[a]S. M. Rees, SIECO, Inc., personal communication.

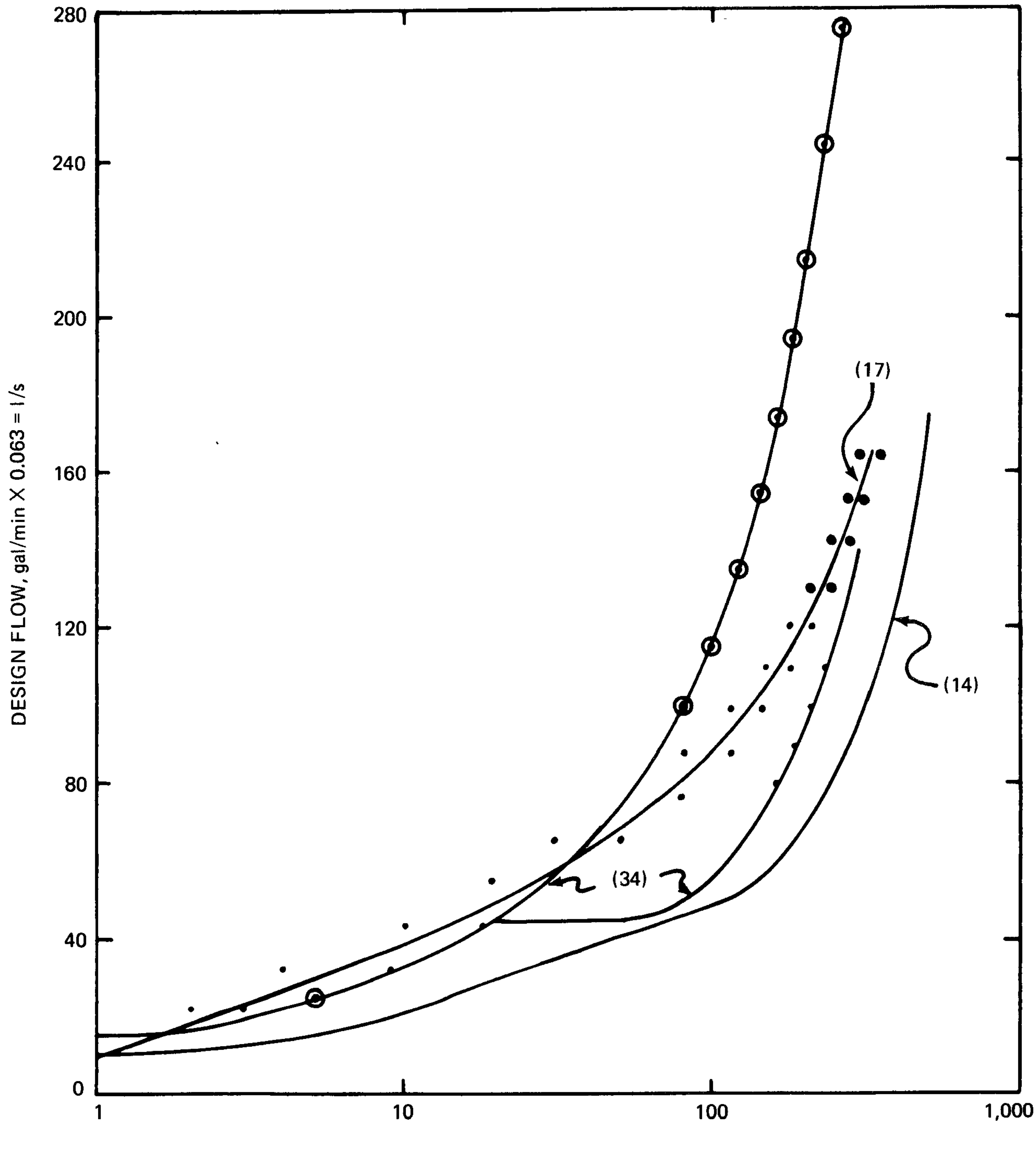


Figure I-23. Suggested design flow.

lished based on one of the above sources or a recognized equivalent. A profile of the proposed system is then prepared, and hydraulic gradelines corresponding to various pipe sizes are plotted, as shown in figure I-24.

Because a reasonable approximation of pump characteristics is already known based on economics, pressure limitations, and other factors, any pipe size that indicates an excessive TDH requirement (difference in elevation between the hydraulic gradeline and ground or sewer profile) is

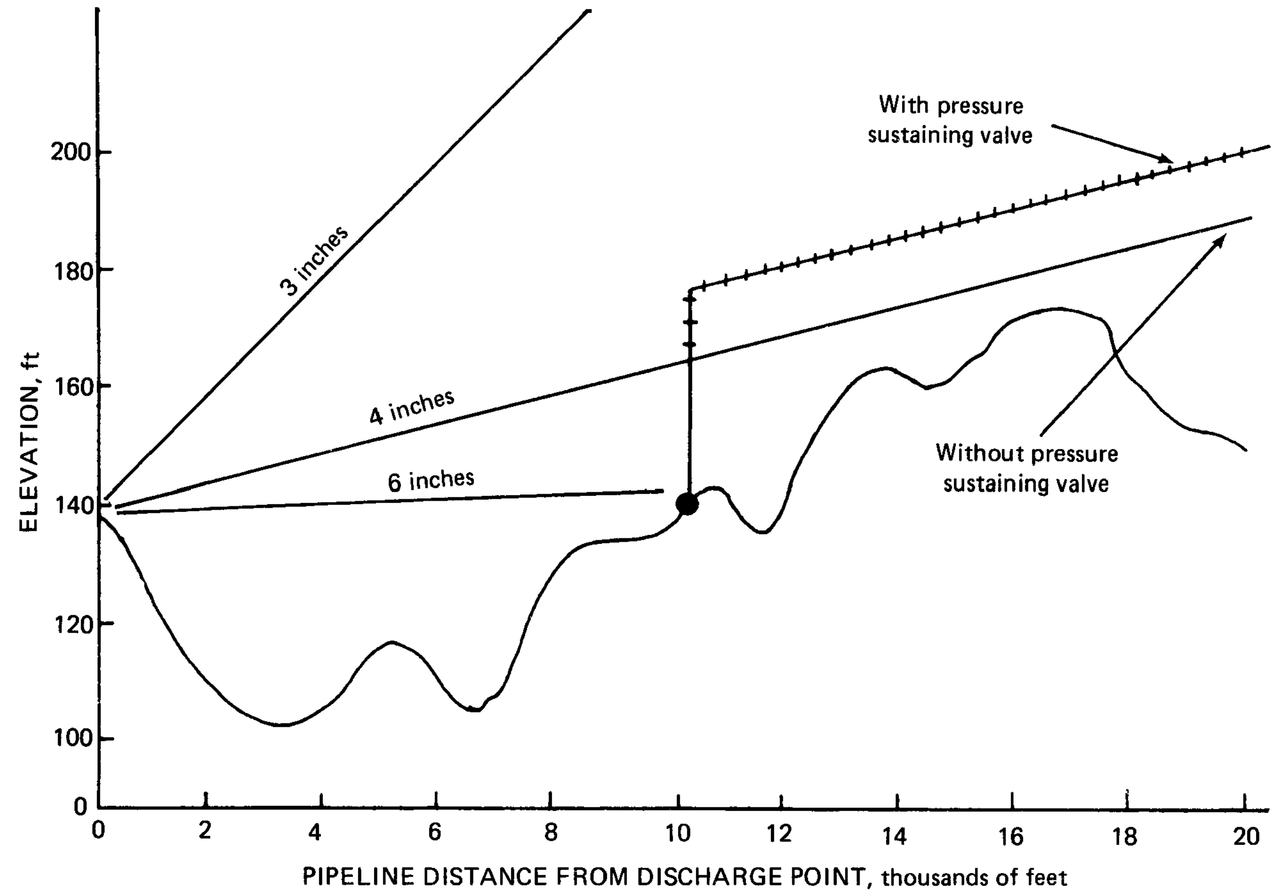


Figure I-24. Pipe sizing procedure.

sequentially discarded until a satisfactory pipe size is found. In figure I-24 the 3-inch (7.62-cm) pipe requires greater pumping capacity than is feasible and the 6-inch (15.24-cm) size is excessive from the standpoint of cost and low velocities. Close examination of the system profile for this example indicates the need for a pressure-sustaining valve and possibly one or more air-release valves to prevent problems caused by air pockets. With the introduction of the pressure-sustaining valve, a new dynamic hydraulic gradeline results and is plotted, as shown in figure I-24. Individual pump characteristics can then be tested for sufficiency by the elevation difference between the proposed pump elevation and the elevation of the dynamic hydraulic gradeline at the mainline station where the pump lateral intersects. To make this test, the modified *H-Q* curve of the proposed pump, which includes service line losses for various sizes and lengths of discharge pipe, is plotted, as shown in figure I-25. The previously noted head requirement can then be located on the *H-Q* diagram at the design flow to determine the adequacy of the pump and the most suitable pipe size.

The type of plastic pipe chosen can significantly affect the design and economics of the pressure sewer system. PVC pipe has been used almost exclusively. The pressure sewer mains at Albany were PVC Type I, Schedule 40, with PVC, DWV fittings, and the joints were solvent welded.[8] At Phoenixville, PVC Type I, SDR-26 piping was used with PVC Schedule 40 fittings.[11] At Grandview Lake PVC, SDR-26 pipe (solvent welded) and PVC fittings were used. The Miami systems use PVC, SDR-26 pipes with slip-ring joints, as do the Priest Lake installations.[e,f] PVC, SDR-26 pipe with solvent welded joints is used in the Weatherby Lake pressure sewer system. Flanigan and Cudnik recommend the use of PVC, SDR-26 piping in all systems whose pumping heads do not exceed 90

[e]H. Schmidt, General Development Utilities Company, personal communication.
[f]C. W. Rose, Farmer's Home Administration, personal communication.

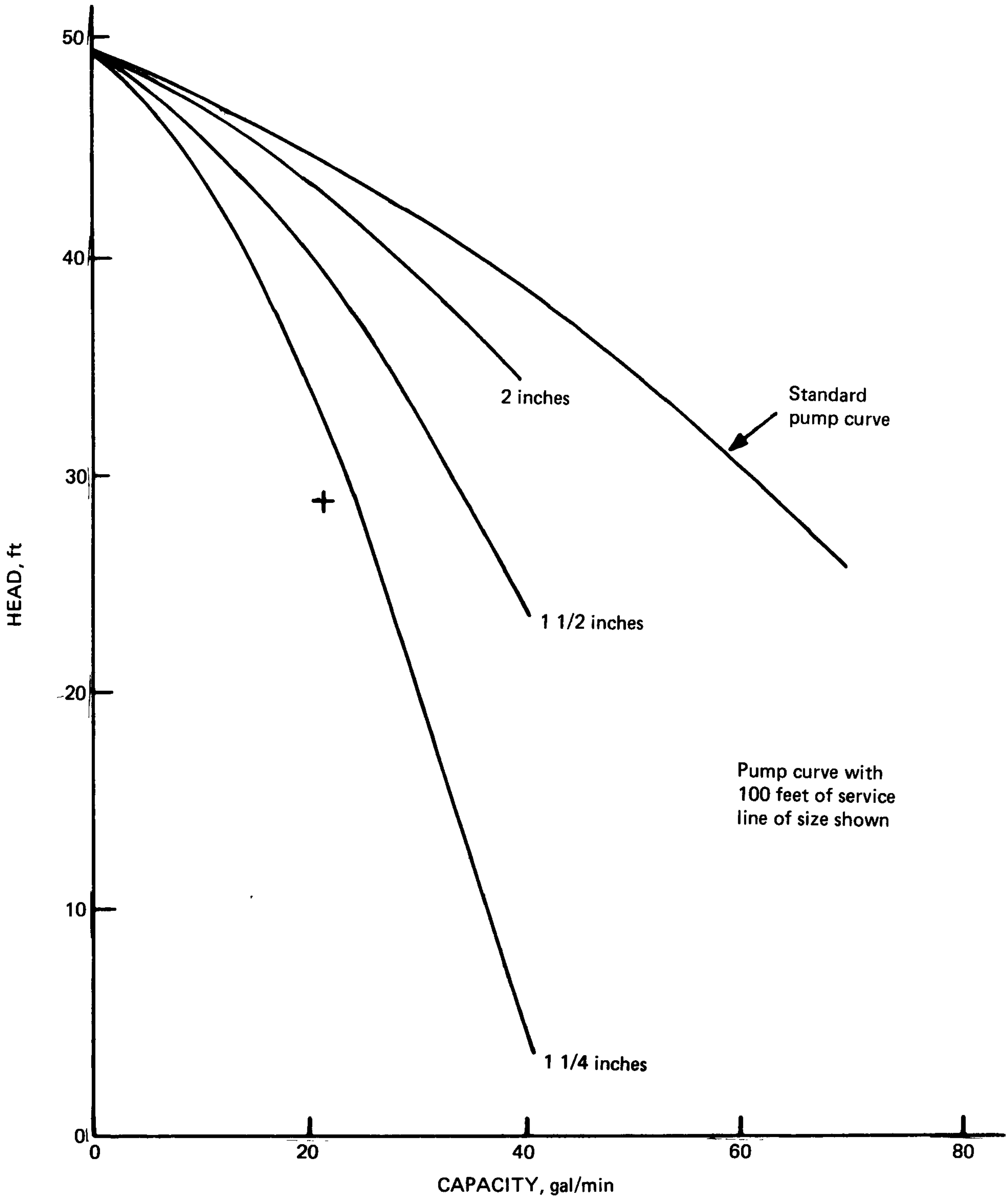


Figure I-25. Pump and service line testing procedure.

feet (30.5 metres).[35] E/One recommends, in order, SDR-21, Schedule 40 pipes and SDR-26.[22] The pressure rating of SDR-26 pipe is 160 psi (1,110 kPa); SDR-21 is rated at 200 psi (1,380 kPa), while Schedule 40 pipe may vary. Schedule 40 pipes of 2 and 3 inches (5.1 and 7.6 cm) are rated at 277 and 263 psi (1,920 and 1,820 kPa), respectively.[34] All pressure ratings are at a temperature of 73°F (22.8°C) and are generally reduced at higher temperatures to the extent that PVC is not recommended above 150° F (65.6° C). The higher pressure-rated pipe recommended by E/One may be related to its GP's ability to operate at very high pressures. The safety factor between pressure ratings of SDR-26 pipe and system design pressures in almost all cases exceeds 4. Because the other recommended PVC pipes all have greater pressure ratings, their safety factors are larger. Because a safety factor of 4 is common for water supply systems where water hammer conditions are more likely, the safety factor for all PVC pipes discussed appears adequate.

As a matter of interest, one system has been designed with polyethylene pipe in northern Michigan. This system is a combination pressure-gravity design that uses GP's. The details of how the polyethylene pipe was adapted to the pressure system are not available at this time, but 40-foot (12.2-metre) lengths of polyethylene pipe will be fused together through the use of heat and pressure before being laid in the trench.[17]

Some concern has been expressed about the shallow depth of pressure mains and their increased susceptibility to damage by excavating equipment. Bowne has suggested that markers be set along the pipe route warning of its presence.[32] He further suggests burial of a copper wire with the pipe for easy location by a cable finder and the institution of a permit system for excavation to minimize such accidents. Accurate "as built" drawings are a necessity in all cases. Leckman[34] has raised the related question of differentiation between water and sewer lines made of PVC. He also suggests markings where applicable and other methods such as standardized relative locations of sewer and water lines. The Grandview Lake system uses brown-colored PVC pipe to simplify differentiation, while the Miami systems use green PVC pipe in one location and red striped pipe in another.[e, 14] This type of color coding is required by the Pennsylvania Department of Environmental Resources.[41]

CONSTRUCTION CONSIDERATIONS

The small pressure sewer system at Albany was installed by a plumbing contractor. A temporary variance was obtained from the Plumbing Code of the City of Albany to use PVC, DWV pipe. The system was pressure-tested at 80 psig (553 kPa) before backfilling took place. A number of leaks were found and repaired at that time. These leaks were attributed to the plumbers' lack of familiarity with the use of PVC. After the leaks were repaired, the pipe was covered with 18 inches (45.7 cm) of sand.[8]

The Phoenixville system was installed by a general contractor. The trenching machine used allowed construction in a trench of less than 4 inches (10.2 cm) in width at an average pipe depth of 2.5 feet (0.76 metre). In areas where rock removal was required, a backhoe was used. Where the pipe crossed under public or private roads, it was encased in a 4-inch (10.2-cm) asbestos cement pipe to protect it from traffic or vehicle loads. Upon completion the pressure system was leak checked at 100 psig (692 kPa) with potable water before backfilling.

The system at Grandview Lake also was installed by a general contractor. Where rock was encountered, 5 inches (12.7 cm) of sand bedding were required.[a] Several 8-inch (20.3-cm) layers of granular fill with tamping were required where pipes passed under roads. The normal pipe depth was 3 feet (0.92 metre) below the ground surface. Joints were solvent welded and the solvent was allowed to set up before "snaking" the pipe into the trench. After the pipe was laid in the trench it was pressure tested at 100 psig (692 kPa) for at least 30 minutes after all excess air had been expelled. After being in place for about 2 weeks, the pipe was leak tested at 150 psig (1,040 kPa). The pipe was backfilled only when the temperature was below 65° F (18.3° C).[14]

[a]S. M. Rees, SIECO, Inc., personal communication

[e]H. Schmidt, General Development Utilities Company, personal communication.

Bowne has emphasized the need for care during construction to avoid scoring PVC pipe, which could result in strength reduction.[32] He also noted that bedding and backfill requirements are less stringent when compared to conventional pipe because of the reduced brittleness of PVC at moderate temperatures. His preference for pipe protection and ease of installation under roadways is to encase the plastic pipe in a steel pipe. His specifications are illustrated in figure I-26.[i]

The British Standards Institution (BSI) provides an excellent set of guidelines for plastic pipe application.[42,43] In relation to construction or pipe laying procedures, the BSI specifies:

- Trench width should be equal to or greater than the sum of the outside diameter of the pipe plus 12 inches (30 cm).
- Depth of bedding below the pipe barrel should be no less than 4 inches (10 cm).
- Bedding material should not exceed 0.5 inch (1.0 cm) in size.
- Sidefilling around the pipe should be done in 3-inch (7.5-cm) layers, compacting each layer by hand.
- Pipes should be partially backfilled (leaving joints exposed for inspection) before pressure testing.
- Pressure testing at 1.5 times the maximum working pressure at the point of maximum stress should be done for 1 hour.

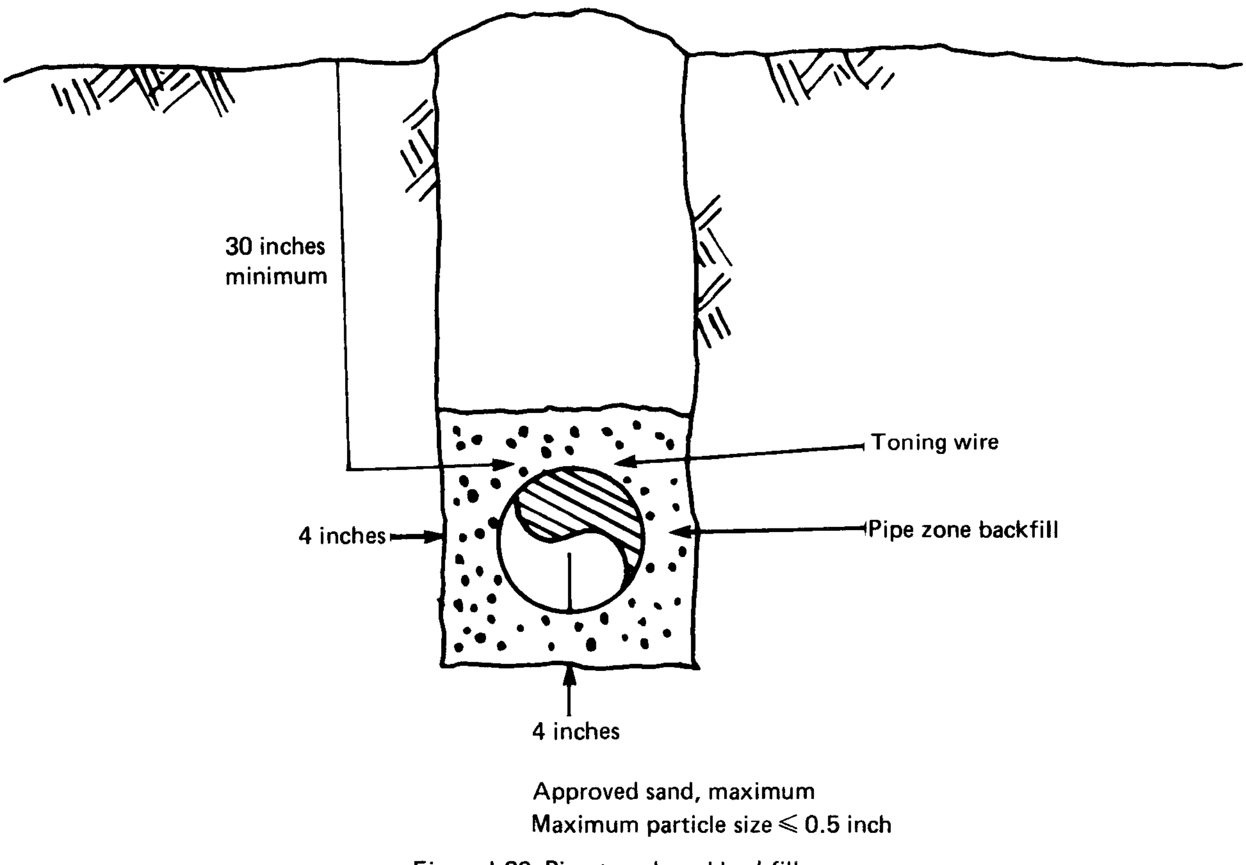


Figure I-26. Pipe trench and backfill.

---

[i]W. C. Bowne, Douglas County (Oregon) Road Department, personal communication.

- A successful pressure test must not lose more water than 0.24 gal/1,000 ft of length per in of nominal diameter per day per 43.4 psi of test pressure (3 l/km per 2.5 cm/d per 300 kPa).

- No construction should be undertaken at temperatures below 14° F (-10° C), because of reduced impact strength of PVC pipe in cold weather.

- Scoring of pipes by dragging them over rough ground or other careless handling must be avoided.

For any pressure sewer installation, one of the desired advantages, which may be maximized, is the ease of construction. Where rock is absent and progress is unimpaired by natural or manmade barriers, trenches can be dug in a very short time by mobile trenching devices. In difficult areas, backhoes may be required, although the Bend system used a special "rock-saw" trenching device for main construction.[h] In any case, the rate and ease of construction, compared to conventional gravity systems, are obvious. For example, at Weatherby Lake the contractor laid 900 feet of 2- and 3-inch (5.1- and 7.6-cm) main on the first day of work.[16]

In any established area where pressure sewers are installed, care must be taken to avoid damaging existing water, electrical, and gas service lines. At Grandview Lake, where the homeowners' recollections were the only guide to the location of service lines, an average of 1.3 existing service lines was cut during pressure sewer service line installation at each home. The maximum for one home was seven.[14] Ideally, "as built" drawings should be obtained, but these often do not exist in rural areas.

OPERATION AND MAINTENANCE

Obviously, any system that uses numerous pressurization facilities and other more sophisticated mechanical equipment will require a significant amount of operation and maintenance. The institutional concepts alluded to earlier for the pressurization facilities must also take into account the needs of the entire pressure sewer. These have generally been described in the design considerations for the pressure main. For the most part, operation and maintenance can be divided between on-lot needs (pressurization units and service lines) and main line needs.

The Albany project started with 12 E/One prototype units, most of which were replaced during the study by modified units.[8] The number of service calls then fell off sharply. Most of the 44 malfunctions reported were caused by faulty pressure sensors. Because the modified units had improved pressure-sensing tubes, only 5 of the 44 malfunctions noted involved modified units. The malfunctions took the form of excessive noise (because of their in-house location), continuous motor operation and nonfunctioning units resulting in overflows. An operation ratio based on any greater-than-15-minute malfunction was calculated. The operation ratio consisted of the number of days when no malfunction occurred over the total service days. Because it is a measurement of reliability, the ratio provides a meaningful account of expected service requirements. The operation ratio over the entire project period varied from 0.90 to 0.99 at the 12 homes. A separate operation ratio was calculated to be 0.995 for the modified units. A number somewhat less than this would probably be more accurate because all the startup difficulties that would normally be expected in the system were not included in this latter calculation. The corresponding downtime, defined as the hours out of service over the total hours of possible service, was 2.69 percent for the prototype units and 0.27 percent for the modified GP units. The difference was primarily because of the improved pressure sensors of the modified units.

---

[h] L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.

Power consumption at Albany was measured for 2 of the 12 units.[8] Monthly power consumption averaged 10.2 and 5.3 kWh for the units, which also averaged 28.5 and 16.0 min/d of operation. Because the wastewater flows were not measured at each home, the operational time could be converted to an approximate flow per home by multiplying the operating time by an average GP pumping rate. If 15 gal/min (0.94 l/s) is assumed as an average, the operating times represent about 427 and 240 gallons (1.62 and 0.91 m^3) per day, respectively, and the power required per unit volume is 0.795 and 0.736 Wh/gal (0.210 and 0.195 kWh/m^3), respectively. A conservative average would then appear to be about 0.8 Wh/gal (0.212 kWh/m^3) for the GP units and conditions of this study.

The Phoenixville system experienced startup difficulties, primarily caused by oversights during installation.[11] During the course of the study, problems with faulty GP discharge line materials, causing electrical short circuiting, a faulty circuit breaker, and faulty grinder assemblies were recorded. If the operating ratio analysis used in the Albany report[8] had been applied to the Phoenixville data, the operating ratios for units 1 through 5 would have been 1.0, 1.0, 0.975, 0.97, and 0.97, respectively. The average operating time per day for GP 3 was 33 minutes at an average discharge pressure of 11 psig (76 kPa), while drawing an average of 12.5 A. For unit 4, the valves were 59 minutes at 44 psig (304 kPa) and 14.0 A. Therefore, by converting pressure to flow by the *H-Q* curve and calculating kilowatt-hours from the voltage, amperes and time of operation, the daily flows and power requirements can be calculated to be 461 and 590 gallons (1.74 and 2.23 m^3) and 0.37 and 1.51 kWh for units 3 and 4, respectively. These convert to 0.803 and 2.56 Wh/gal (0.214 and 0.677 kWh/m^3), respectively. These unit costs are in close agreement with the Albany data.

The Grandview Lake operation and maintenance information is extensive.[14] Of the three major commercial GP units used, (E/One, Hydromatic, and Tulsa), the E/One unit required the lowest number of service calls per number of units installed, while the Hydromatic had the next lowest, and the Tulsa had the highest. The data are, however, only approximate on the E/One and Hydromatic units because service calls were often made directly to the private maintenance services for each of these units and the number of units was continually changing as the number of connections grew. The calls reported to the engineer were classified according to the nature of the malfunction, as shown in table I-6.[14] The E/One unit was found to fail because foreign particles scored the metal rotor and excessive delay in thermal overload activation.[14] The overload was caused by excessive air in the pressure sewer line. Maintenance difficulties were compounded by the weight of the unit, which generally required more than one person to install or remove it. The Hydromatic unit was often found to leak at its quick-disconnect fittings because of excessive pressures in the system. The ball check valves also were found to require substantial maintenance, as did the float switch that controlled operation.[14] Some float switch problems occurred because of grease accumulation brought on by a lack of sufficient swirling action in the tank. The lack of an antisiphon device also caused some problems.[14] Although lighter than the E/One, this GP was also difficult to install and remove. A number of design problems were cited that resulted in the increased rate of service calls for these units.[14] The report recommended redesign and upgrading of the units used in the project. Both the E/One and Hydromatic GP units have been modified since the study.

One of the difficulties encountered in analyzing the Grandview Lake report[b] is that the dynamic nature of the system precluded determination of precise operation ratios, downtimes, and so forth. It is fair to say that the experimental nature of this large installation produced better practical engineering data than the Phoenixville or Albany projects, but less mechanical-electrical performance information. The number of commercial unit service calls for the size of installation involved (shown in table I-6), excluding an unsuccessful noncommercial GP unit, appears to be quite low, despite any data shortcomings. The E/One Corporation indicates that 19 service calls were made by its representative during 11,800 unit days of possible operation in the Grandview system.[b] This would yield an operation ratio of 0.9984, assuming the numbers are accurate.

[b] R. P. Farrell, Environment/One Corporation, personal communication.

Table . Summary of maintenance frequency

[2-year period]

Cause	Locally manu-factured unit	E/One unit[a]	Hydro-matic unit[a]	Tulsa unit
Pump failure	52	4	8	2
Grinder failure	25	0	1	2
Piping failure (within tank)	41	4	7	1
Electrical failure (excluding controls)	66	3	2	0
Control failure	23	0	10	2
Piping failure (outside tank)	11	2	8	0
Infiltration/inflow of water	56	1	0	0
Collection system malfunction	7	1	2	4
Improper installation	9	2	2	0
Miscellaneous	81	0	12	5
Total	371	17	52	16
Maximum number of units	27	15	28	2

[a]Maintenance of the E/One units was done primarily by the manufacturer's representative. The Hydromatic units were serviced by the manufacturer's field personnel as malfunctions were reported to the factory. Therefore, the figures listed above were based on the field notes taken by the engineer's maintenance crew and may not include all of the service calls by others.

The Horseshoe Bay (Lake Lyndon Baines Johnson) GP installation has also experienced some operation and maintenance problems.[15] These have been identified as excessive wear and failure of the GP stators because of grit particles in the sewage and improper ventilation of pump motors. Similar grit problems were noted during the early stages of a similar GP system at Point Venture (Lake Travis).[15]

The Miami systems represent the longest history of operation and maintenance of all the pressure sewer systems. Thus far, the experience with these pressure sewer systems indicates that the operation and maintenance costs are the same as for gravity systems. Currently, an annual preventive maintenance inspection is employed. This inspection consists of the following:

- The pump is removed from the holding tank, inspected for corrosion and suction plate condition, and cleaned (if necessary).
- The check valve and gate valve are inspected for proper functioning.
- The pump is returned to its operating position and tested.

A two-man crew normally requires 30 minutes to complete such a preventive maintenance procedure. This preventive maintenance program, therefore, amounts to 1 man-hour per year. In addition, replacement of diaphragm switches and refurbishing of brass disconnect fittings are included in this program every other year. On one of these systems, which contains 26 STEP units, one emergency repair has been reported in 3 years of operation.[e]

[e]H. Schmidt, General Development Utilities Company, personal communication.

Routine maintenance of the Priest Lake systems is reported to have resulted in service calls to about 8 percent of the STEP units (approximately 500 total units in systems) in the first year and only 2 percent in the second year.[32] During the third year the service calls averaged about five per week, with an average service time of 30 minutes per call. One man services both Priest Lake systems, routinely inspects and pumps out the system septic tanks, and operates the treatment facilities. Experience has indicated that this individual can remove 30 pumps per day, replacing the impellers and returning them to service.[32]

The combined experiences of several STEP systems and some raw sewage pumping systems indicate that a 10-year life can be expected for the submersible sump pumps that have been used thus far before rebuilding is required.[e,32] One of the major causes for servicing of STEP systems is the buildup of iron sulfide on the pump impeller after lengthy periods of inactivity, such as in a vacation home.[i]

Operation and maintenance of pressure mains includes periodic cleaning, repairing of leaks, and major replacement of sections. Because of the short length of the Albany project's main, no maintenance was reported.[8] At Phoenixville, a 2-day shutdown was incurred when an air-release valve was damaged by heavy equipment during routine snow removal.[11] At Grandview Lake, a number of service calls were required because the tapping tool, used to connect individual home services to the pressure main, was improperly used, resulting in leaks.[14] Additional leaks and breaks were caused by heavy equipment, earthslides, and improper installation. Line cleanouts were necessary during the early stages of operation because of low flows and concomitant solids buildups. On four occasions, supplementary flushing of the lines was accomplished with lake water. Maintenance of the air-release valves was necessary and was somewhat more difficult for the automatic valves because no pressurized water source was available at the valve locations. Some odors were reported when these valves were actuated. Earth shifts at one location in the Grandview Lake system caused repeated breaks in the pressure main. The eventual solution to this maintenance problem was effected by replacing the PVC pipe with a looped section of flexible pipe.[14] No maintenance of the Miami or Priest Lake systems' mains has been necessary.[e,32]

In STEP systems, periodic septic tank pumping is considered an operation and maintenance requirement. The cost of pumping varies with geographic location, but regular surveillance of solids buildup in these tanks is required to prevent significant reductions in their grease and solids removal capabilities. Pumping is not normally required at intervals of less than 3 years[6] and may greatly exceed this length of service. For instance, the Miami STEP systems have not required any pumping of septic tanks in 5.5 years of service.[e] Because the scum and sludge accumulation is highly variable, annual inspection of septic tanks is initially recommended until sufficient data are generated to determine a proper interval for pumping individual tanks.

VARIATIONS

It is obvious that any technology as new as the one described in this seminar is not limited to the design and construction methods discussed herein. One example that is conspicuous by its absence is the use of a single collection tank and pressurization device for more than one dwelling unit. Rose[3,f] has long been an advocate of this approach and the Bend system employs one three-home and three two-home installations. The potential savings in cost are obvious because the number of pumps or GP units needed is reduced. Rees has expressed concern about the possible problems resulting from allocation of operation and maintenance costs among contributors that could make such an approach difficult to implement.[a] Another variation that could occur relates to the

[a]S. M. Rees, SIECO, Inc., personal communication.
[e]H. Schmidt, General Development Utilities Company, personal communication.
[f]C. W. Rose, Farmer's Home Administration, personal communication.
[i]W. C. Bowne, Douglas County (Oregon) Road Department, personal communication.

inclusion of multifamily or recreational/commercial contributors. Although the Phoenixville system appeared to handle a small apartment building without difficulty,[11] some changes in design, such as holding tank size and pipe sizing changes based on specific flow patterns, will probably be necessary.

CODES

Information on State and local codes that restrict or govern the use of pressure sewer technology must be obtained from the responsible agencies. A partial list of States that undoubtedly have permanent or temporary regulations for pressure sewer installations appeared in the section on previous experience. States not listed should be contacted before planning begins.

Leckman[34] contacted 11 Illinois local government units to determine whether 1.25-inch (3.18-cm) polyethylene pipe would be allowed for transmitting sewage. All 11 local authorities indicated that it would be in violation of their plumbing codes. When asked about the possibility of granting conditional or special permission to use a GP system, 9 out of 11 authorities said they would refuse permission. Leckman also analyzed the 1971 Ten State Standards[44] as they applied to pressure sewer systems. Serious conflicts were found in chapters 20 and 30 concerning design considerations, such as per capita flow, minimum sewer size and slope, sewer alignment, pump openings, wet well requirements, emergency operation, and minimum sewer velocity.

WASTEWATER CHARACTERIZATION AND TREATMENT

As noted earlier, wastewater from an individual home is more concentrated than normal municipal wastewater. Some data from pressure sewer systems have been obtained. At Albany, a thorough sampling and analysis program produced the results summarized in table I-7. These data are reasonably consistent with those in table I-1.

The Phoenixville and Grandview Lake systems were sampled (generally grab samples) only a few times; in contrast, more than 50 composite samples were analyzed at Albany.[11,14] The concentrations of pollutants analyzed were generally diluted by table I-1 standards. The Phoenixville data confirmed the absence of dissolved oxygen, as would be expected in a pressure sewer.[11] Because the Grandview Lake system used both GP and STEP units, the combined wastewater does not necessarily follow the pattern indicated in Table I-1. Twenty-four-hour composites yielded SS, 5-day BOD (BOD_5) and chemical oxygen demand (COD) concentrations ranging from 80 to 265 mg/l, 100 to 310 mg/l, and 230 to 462 mg/l, respectively.[14]

Table I-7.—*Albany wastewater characterization*

[mg/l]

Parameter	Concentration	
	Mean	Range
BOD_5	330	216-504
COD	855	570-1,450
TSS	310	138-468
TKN	80	41-144
Total phosphorus	15.9	7.2-49.3
Grease	81	31-140
pH (units)	–	7.1-8.7

The GP-pressure sewer effluent at the Horseshoe Bay project is treated by advanced treatment methods. The treatment sequence involves an activated sludge system followed by chemical precipitation and filtration. The final effluent is then used to irrigate a nearby golf course. Occasional odor problems in the treatment plant lift station have been controlled by the use of odor-masking compounds.[15]

The amenability of pressure sewer wastewater to treatment is of primary concern. Hobbs investigated the effects on sewage solids, both from grinding and comminution.[40] He found no apparent difference in required solids transport velocities, but noted that the grinder did appear to yield finer solids than the comminutor. If this condition were significant, the possibility of reduced sedimentation efficiency in the primary clarifier could be significant in the design of treatment facilities.

At Albany, hydrogen sulfide odors were detected at the discharge of the pressure main, and daily grab samples analyzed for sulfide showed concentrations of up to 2.5 mg/l.[8] Some method of freshening the sewage when it reached the treatment plant was suggested. Also, settleability tests were made on the sewage from the pressure sewer and compared to some local residential sewage collected in gravity sewers. Although the details of the comparison were not presented, the data show that SS removal at equivalent overflow rates is somewhat less for the pressure sewer wastewater, when expressed as a percent. Because of its high initial strength, the primary effluent resulting from the GP pressure sewer wastewater was significantly stronger in terms of SS and organic matter, representing a higher loading to subsequent biological treatment processes. The consequences are increased sludge production in the biological reactor and a need to maintain a higher mixed liquor SS concentration for equivalent treatment. These considerations are not likely to offset the advantage of treating lower flow volumes. Because the collection system represents from 75 to 90 percent of the total cost of conventional wastewater management in smaller communities, a 50-percent savings in the collection system cost could be offset only by doubling the treatment cost.

The special characteristics of wastewater from pressure sewer systems require that the designers of treatment facilities use their best engineering judgment in matching the type of treatment and specific modifications with the application, at least until further data are obtained. If, however, the pressure sewer terminates by discharge to a gravity interceptor, the effects of these special characteristics on the ultimate treatment facility will be a function of the relative contributions of the pressure and gravity systems. If the pressure sewer contribution is locally or totally significant, sulfide control methods may be needed at the pressure main connection to the gravity interceptor. If this is the case, the use of control methods described by Pomeroy[45] are applicable.

In the case of a fully pressurized system, two effects of the pressure sewer wastewater on normal treatment systems must be considered—the tremendous variation in flow and the anaerobic condition. Because a completely pressurized sewer will normally be located in a relatively small community, the treatment system employed should be consistent with normal small-flow methods. Likely choices, therefore, would include trickling filters, oxidation ditches, activated sludge package plants, and lagoons, with possible land application of effluents during all or part of the year. Treatment systems can be divided into lagoons and conventional biological processes.

The Grandview Lake pressure sewer system used lagoons with land disposal of the effluent.[14] The Grandview Lake system had some difficulties from a physical standpoint (levee failure), but runoff from the effluent-irrigated and nonirrigated hay fields was not significantly different in quality. The Priest Lake design incorporates two-stage lagoons with supplemental aeration followed by land spreading. Primarily because of a high evaporation rate, however, no effluent has yet been produced for land spreading. Bowne has also suggested facultative lagoons followed by slow sand filtration as a means of obtaining a high quality effluent.[i]

---

[i]W. C. Bowne, Douglas County (Oregon) Road Department, personal communication.

Facultative lagoons, with or without mechanical surface aeration, represent a convenient method of handling both the noxious gases potentially emitted from pressure sewer wastewater and the wide flow variations. Inlets should be located near the bottom of the lagoon to allow proper dispersion of the influent into the anaerobic bottom area. Designers must consider the relative strengths of GP wastewater and septic tank effluent compared to normal domestic loading rates. In the case of GP-pressure systems, any sludge allowances normally made in depth dimensioning might be increased to allow for higher influent solids and sludge accumulation rates. In the case of STEP systems, these allowances may be reduced.

When conventional biological systems are to be used, the problems of extreme flow variation and hydrogen sulfide odor potential must be accommodated in the design. Introducing anaerobic wastewater directly into the aeration compartment of a system of the activated sludge type appears attractive, but high sulfide concentrations in wastewater can encourage the growth of filamentous organisms in such a system.[49] However, the STEP system near Port Charlotte is using this approach with no ill effects. No odors have been reported around the extended aeration unit, and effluent BOD_5 and SS concentrations have averaged 6.9 and 14.3 mg/l over 3 years of operation.[e]

Other methods of sulfide control include U-tube aeration, chlorination, and ozonation.[45] The latter two methods are expensive and somewhat inconsistent with smaller treatment systems, while the first results in substantial headlosses. Any oxygen addition method would be most suitable at the discharge end of a pressure sewer because of the air-binding hydraulic problems discussed earlier.

Of all the aspects of pressure sewer technology, treatment problems are possibly the least studied, primarily because very few systems have been totally pressurized. Most have constituted a small portion of an overall sewer system and have not demonstrated major effects on treatment. However, the few totally pressurized systems from which information is available have reported no major difficulties in treatment.[e,14,i]

One additional point should be stressed. There may be a major difference in treatment requirements between STEP systems and GP systems. In the former case, relatively weak wastewater in terms of BOD_5 and SS must be treated, while additional maintenance in the form of septic tank pumping is required. In the latter case, very concentrated wastewater in terms of BOD_5 and SS must be treated. The trade-offs must be weighed by the designer. Further experience with both systems will expedite design selections.

COSTS

Capital Costs

No new technology is valuable unless its total costs are competitive with those of existing technology in a significant number of situations in engineering practice. Situations where these systems have been found economically superior to conventional sewerage include hilly terrain, outcropping rocky areas, and low areas with unfavorable on-site disposal capability. Other favorable conditions also exist, such as high ground water regions and low population density communities. For practical purposes, these areas generally fall into the following categories:

- Low density areas unsuited for on-site disposal

---

[e] H. Schmidt, General Development Utilities Company, personal communication.
[i] W. C. Bowne, Douglas County (Oregon) Road Department, personal communication.

- Geological conditions unsuited for normal excavation
- Undulating or hilly terrain

The practicality of pressure sewers is not limited to any of these categories nor are pressure sewers the exclusive answer in such categories.

Some information is available on the relative economics of pressure sewers as compared to existing technology. Often these comparisons are "broad brush" in nature and do not represent itemized cost comparisons. All information on costs is instructive, however, when the high cost of sewerage is considered.

Numerous citations in the literature document the excessive cost of installing conventional sewers in low density areas. Voell has estimated that it would cost $2.6 million for conventional sewers at the Central Chautauqua Lake Sewer District of New York, as compared with an estimate for a GP-pressure sewer system of $1.2 million. Grandview Lake sewerage estimates of $3,000 per lot and $10,000 per existing home were reduced to less than $2,000 per home for the pressure sewer system.[32] An estimate of $100,000 for a 10-house sewer system in Saratoga was rejected in favor of a cost of $20,000 for a pressure sewer-GP system. Conventional sewerage estimates for two communities at Priest Lake were about $12 million with treatment.[f] A STEP-pressure sewer system was subsequently built for less than $1 million. Bowne[20] has presented a present worth comparison of a conventional gravity system with a STEP-pressure sewer system using average costs in his region of Oregon. On a total cost basis for this rural-suburban area, Bowne has estimated present worth costs of $4.7 million for conventional sewers and $2.4 million for a STEP-pressure system. A list of seven unidentified municipalities receiving GP-pressure sewer and conventional sewer estimates has been provided by E/One and is shown in table I-8. The two locations where the highest percent of savings are noted (3 and 5) involve low-lying areas below existing sewer grades.

To determine the limits of pressure sewer application, it may be assumed that low density housing areas, where soil conditions are favorable, will be better served by on-site wastewater treatment and disposal systems. This assumption leads to the question, At what level of housing density

Table I-8.—*Cost comparisons*

Location	Connections	*Engineering News Record* index	Grinder pump-pressure sewer, dol per connection	Conventional sewer, dol per connection	Savings for grinder pump-pressure sewer, percent
1	285	1700	1,930	4,570	48
2	30	1895	2,800	4,667	40
3	9	2014	2,222	10,000	78
4	309	1753	3,240	6,176	49
5	10	1753	1,653	10,000	83
6	320	1895	2,709	4,088	34
7	100	2098	1,360	2,350	42

NOTE: All estimates by consulting engineering firm except 2 and 5, which are bid prices for grinder-pump/pressure sewer systems. Conversion of estimates to current costs requires use of present *Engineering News Record* index.

[f] C. W. Rose, Farmer's Home Administration, personal communication.

would conventional sewer technology, rather than pressure sewers, better serve the population? Although computation of this density would be a most valuable piece of information from a theoretical point of view, it would be highly subjective and would vary with each assumption made or existing physical condition for any potential application. It is probable that the experience gained in each new application in the coming years will permit reasonably reliable definitions of the limits of applicability of pressure sewers.

The type of data required includes capital costs in various regions of the country, various climate conditions, and various soil and other geological conditions. For example, the cost of pumps, pipe, valves, labor, and so forth, varies by region. The requirement that pipes be buried below the frostline makes pressure sewers in Florida less expensive than in Minnesota, if all other conditions are the same. In rocky areas, a cost comparison between pressure sewers and conventional sewers will be more dramatic than in areas where more favorable soil conditions exist. Although some information has been generated on capital costs, a great deal more is necessary to allow reasonable estimation of the limits of pressure sewer applicability.

Data are also needed on operation and maintenance requirements in terms of manpower, skills, and costs. This aspect of the total picture will remain the least-definable of the entire equation until information on these requirements becomes available. Only well-documented experience from a number of installations over a significant period of time will permit solid estimates of these factors.

To initiate the task outlined above, a series of data points can be found in the literature, but a considerable variation in physical constraints exists for each. Also, some bid prices may reflect adjustable profit margins; for example, if a large profit was figured for one category of work, a different category bid may reflect smaller profits to complement the other. Also, it should be noted that all cost estimates, bid prices, and actual costs can vary significantly on any job. All costs are related to the month and year estimated or incurred and to geographic location.

In determining capital costs, a number of subelements must be considered, such as engineering, valve boxes, fittings, cleanouts, flushing arrangements, testing. Some of these items may be difficult to estimate; therefore, a few basic facts will be presented with expanded information based on traditional practice. One primary cost category is the pressure main cost per lineal foot. At Phoenixville[11] the cost of PVC pipe, excavation, installation, lateral tie-ins, and restoration for 2,800 feet (854 metres) of main was $2.00 per foot ($6.56 per metre). Additional costs of rock removal, restoration of streets and driveways, load protection, an automatic air-release valve ($350), and a terminal cleanout ($350) resulted in a total pressure sewer cost of $2.82 per foot ($9.25 per metre). Pressurization facilities and service connections amounted to $2,050, including $900 per GP. In this total, service line costs of $2.50 per foot ($8.20 per metre), circuit breaker costs of $60 each, and power cable connection costs of $3.00 per foot ($9.84 per metre) were incurred. The overall (January 1971) capital cost for the pressure system was $19,020. Dividing this total among the users is difficult because of the presence of apartment units. The costs, however, can be allocated as $595 per person, $1,270 per dwelling unit, $2,720 per structure, or $3,810 per GP. The engineering fee for design and supervision of construction was $2,100 and the legal charges were $2,500. This additional $4,600 increases the above unit costs to $864 per person, $1,842 per dwelling unit, $3,946 per structure, and $5,530 per GP, respectively.

Early bids on the Grandview Lake project were rejected because they were almost twice the engineering estimates. Subsequent bids were acceptable. The final cost of the 28,352-foot (8,640-metre) pressure main was $35,491, or about $1.25 per foot ($4.11 per metre). This total includes blacktop road repair, manual ($125 each) and automatic ($200 each) air-release valves, gate valves and boxes ($100 each), and a small vacuum collection station. The total on-lot costs are difficult to establish from the information available, but some of the data are useful. The bid price for 6,600 feet (2,014 metres) of 1-inch (2.54-cm) service line was 60 cents per foot ($2.07 per metre); for 360 feet (110 metres) of 1.5-inch (3.81-cm) line, it was 95 cents per foot ($3.12 per metre). The cost of the curb cock is shown to be $20. In addition, a 1971 report on the project showed that on-lot installed costs for the various GP and STEP units ranged from $1,000 to $1,500, assuming 150 feet (45.8 metres) of service line and no electrical hookup.[12] The original costs at Grandview reflected the use of homemade GP units, which were later replaced with commercial ones.[14] Engineering and legal fees amounted to $23,384 at 1969 rates.

Several costs have been accumulated on the various equipment components of a pressure sewer system. Installed prices for PVC pipe, for example, have ranged from $1.00 to $3.00 per lineal foot ($3.28 to $9.84 per metre) in sizes up to 6-inches (15.2-metres) nominal diameter.[11,a,f] In difficult locations, where more expensive methods of installation than simple trenching are required, these costs have risen as high as $6.00 to $9.00 per lineal foot ($19.68 to $29.52 per metre).[18,d,h] Carcich et al.[2] and Bowne[20] indicate that costs could be as high as $15.00 per lineal foot ($49.20 per metre) for some areas.

Main line accessories, such as air-release valves, have been installed at a cost of $120 to $350 each.[11,a,f] Cleanouts have been estimated to cost from $150 to $400 each.[2,11,18] Valve boxes cost between $100 and $900 each, depending on their design and construction conditions.[a,d]

GP's are estimated to cost from $1,000 to $2,000 each, with an additional $300 to $700 installation cost per unit.[2] STEP's cost about $200 to $400. Depending on the amount of ancillary equipment (alarms, valves, sensors, switches, tankage, removal mechanism), STEP units may be installed for $1,000 to $2,000 each.[20,h] Bowne breaks down his estimate for a septic tank that must be replaced in an existing home to $450 for replacement, $150 for the pump vault, $250 for the pump, $150 for electrical work, and $400 for the balance. This results in a grand total of $1,325 for the on-lot facilities.[20]

Operation and Maintenance Costs

Operation and maintenance costs are generally unknown for pressure sewer systems. For all systems, the cost of main repairs and cleaning must be added to the operation and maintenance needs of the pressurization facility. Leckman[34] has estimated the maintenance cost on GP units to be $4 to $8 per month, and Dounoucos[46] has estimated a GP maintenance cost of 1.4 to 2.0 percent of the on-lot capital cost per year. Bowne relates that GP service contracts have been instituted at rates of $48 and $60 per year.[20] The Priest Lake system operator indicates that effluent pumps can be rebuilt for $50 to $100 each.[i] Replacement motors cost less than $100, while seals, bearings, and capacitors cost about $7, $5, and $9, respectively.[20] Bowne has estimated that on-lot systems will require an operation and maintenance cost of $50 per year, and that pressure main operation and maintenance will cost $100 per year per mile ($62 per year per kilometre).[20]

Septic tank cleaning generally costs $30 to $50 and is required at 3- to 5-year intervals to protect the grease and solids removal capability of the tank. This assumption is based on traditional septic tank practice, where pumping is performed to protect the subsurface disposal field from potential clogging caused by wholesale unloading of grease and solids. In the case of STEP systems, the results of septic tank failure may not be as severe when occurring at an isolated location, but simultaneous failures of several tanks might result in serious problems because of fouling of pumps, controls, or pressure mains. Schmidt surveyed 12 septic tanks in the Miami STEP systems and discovered that accumulations of sludge and scum were significantly less than earlier U.S. Public Health Service studies.[27-29,e] Bowne, like Schmidt, suggests a 10-year interval between pumpings, tempered initially by yearly inspections to determine individual site accumulation rates for developing rational

---

[a] S. M. Rees, SIECO, Inc., personal communication.
[d] J. Schultz, Becher-Hoppe Engineers, Inc., personal communication.
[e] H. Schmidt, General Development Utilities Company, personal communication.
[f] C. W. Rose, Farmer's Home Administration, personal communication.
[h] L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.
[i] W. C. Bowne, Douglas County (Oregon) Road Department, personal communication.

pumping schedules.[20,e] The cost of inspection and pumping must be included in the operation and maintenance cost estimate for STEP systems.

Operation and maintenance costs on a pressure sewer main are difficult to assess but are likely to be less than a conventional gravity system. Based on the data of Smith and Eilers,[5] an updated average cost for gravity sewer operation and maintenance is probably between 7 and 8 cents per year per foot (23 and 26 cents per year per metre) of pipe, which converts to about $400 per year per mile ($248 per year per kilometre). Bowne used actual operation and maintenance costs for rural water supply systems to obtain a pressure main operation and maintenance estimate of $100 per hour per mile ($62 per year per kilometre).[20] Because burial depths of water mains and pressure sewer mains, as well as many of their other physical features, are quite similar, this analogy is probably the best estimate available at this time.

A few bits of data may be of use in checking some of the above assumptions. These data include a 0.0033 service ratio for the E/One GP units at Albany.[8] In terms of days per year requiring service calls, this corresponds to 1.2. At Grandview Lake, E/One GP units required 19 service calls in 11,800 unit days of operation,[b] reflecting a service ratio of 0.0016 or about 0.6 service days per year. Hendricks and Rees present similar numbers for the same project and GP unit.[14] Their information indicates that the Hydromatic and Tulsa GP units indicate approximately 1 and 4 service days per year, respectively. From the above data and the factory improvements and modifications that have occurred in the interim, it would seem prudent to assume a conservative service requirement for the GP units of about 1 day per year.

The Priest Lake STEP systems had problems with 8 percent and 2 percent of the pumps (total approximately 500), respectively, during the first 2 years of operation.[20] Much of the first year's operation and maintenance was because the supplier provided improper impellers on the pumps. During the summer season of the third year when all units were in operation, approximately five service calls per week were required. The operation and maintenance charge per home is $5 per month for these systems.

The Miami STEP systems are serviced by the developer for the same monthly charge that is levied on gravity system contributors.[e] This includes about 1 man-hour per year of preventive maintenance on each STEP unit. One-half hour of operation and maintenance was required on the Big Bend system during the first month of operation.[h]

Some Oregon systems are using submersible sump pumps of the same type as those employed in STEP systems for pumping raw sewage.[20,h] The operation and maintenance experience for one installation is said to be five service calls per month for a system of about 150 pumping units.[g]

The consensus, from the foregoing experiences, is that an effective preventive maintenance and overall operation and maintenance program for STEP systems would involve a yearly inspection of each pressurization facility (septic tank, pump, sensors, valves, and so forth) and about 0.5 service call per year. Pressure main operation and maintenance should be less than that required for GP pressure mains because most of the problem-causing material (fibers, grease, and so forth) is removed by the septic tank.

The power cost of GP or STEP units can be estimated. The Albany and Phoenixville GP information conservatively indicates the need for about 1 Wh/gal (0.264 kWh/m^3). The power cost can be estimated by multiplying this figure by the number of occupants per home and the average

---

[b]R. P. Farrell, Environment/One Corporation, personal communication.
[e]H. Schmidt, General Development Utilities Company, personal communication.
[g]J. Ward, personal communication.
[h]L. R. Clark and J. E. Eblen, C & G Engineering, personal communication.

wastewater flow generated daily per capita. For example, if a GP were to be used by a four-person family with an average wastewater flow of 50 gallons per capita per day (0.19 m^3 per capita per day), the monthly power cost would be about 15 cents on the basis of 2.5 cents per kilowatt-hour. Bowne estimates that the power cost for 0.33 hp (0.25 kW) STEPs would be about 10 cents per month.[32] A conservative estimate for both STEP and GP units would be about 20 cents per month, or about the same as an electric coffeemaker.[b] Actual costs for each installation will depend on the number of people served, the cost of electricity, and the specific pressurization device chosen.

Estimating Procedures

The system depicted in figure I-10 can be used as a simplified example, assuming a scale of 1 inch = 300 feet (1 cm = 36 metres). Additional assumptions are: PVC mains of 2-inch (5.1-cm) nominal diameter, except for a 3-inch (7.6-cm) interceptor (branch 7); service lines of 1.25-inch (3.2-cm) nominal diameter PVC; and GP units.

Table I-9 shows assumed unit costs and a rough estimate of the capital cost of the system. The total of $104,850 represents a cost per house of $2,760.

Two things are shown in this example: the economical nature of the pressure sewer and the high cost of GP's. It is because of the latter factors that the use of STEP's is being investigated and tried by many.

In many rural areas sewers are required because poor soil conditions have precluded the continued use of the original ST-SAS. If this were the case in the example location, the data developed by Bowne indicate that STEP units can be substituted for the GP units at a cost of about $1,000 per installation as compared to $2,000 for the GP installation. This substitution in the foregoing example would reduce the cost per home to about $1,760.

Approximate operation and maintenance costs for the example installation appear in table I-10. The total of $2,161 per year amounts to $56.87 per home, or a monthly cost of about $4.74. The amortized capital cost must be added to this amount to get the total monthly cost.

Table I-9.—*Assumed unit costs and estimated capital cost of typical pressure sewer installation*[a]

Component	Unit cost	Quantity	Estimated system cost, dollars
3-inch PVC pipe	$4 per foot	800 feet	3,200
2-inch PVC pipe	$3 per foot	3,140 feet	9,420
1.25-inch PVC pipe	$2 per foot	5,700 feet	11,400
Service line connections	$35 each	38 each	1,330
GP units, including electrical hookup	$2,000 each	38 each	76,000
Cleanouts with manual air release valves	$500 each	7 each	3,500
Total			104,850

[a]Shown in figure I-10.

[b]R. P. Farrell, Environment/One Corporation, personal communication.

Table I-10.—*Operation and maintenance costs of typical pressure sewer installation*[a]

Component	Unit cost	Quantity	Approximate cost per year, dollars
Pipe	$100 per mile per year	9,640 feet	18
GP	$54 each per year	38 each	2,052
Power	$2.40 each year	38 each	91
Total			2,161

[a]Shown in figure I-10.

For this example, no engineering or legal fees or other additional costs will be considered. Therefore, amortization of the $104,850 capital cost over 20 years at 5-7/8 percent interest (municipal rate) yields an annual cost of $238 per home, or $19.83 per month. The total monthly cost per home is then $24.57. Because these systems are eligible for EPA Construction Grant funding, the cost per home could be reduced to a fraction of that amount.

From the discussion of the operation and maintenance costs for GP and STEP systems, there is not enough evidence available at this time to justify a difference in the operation and maintenance cost estimates for these two types of pressure systems. Therefore, the substitution of STEP units for GP units in the above example yields a monthly cost per home of $4.74 for operation and maintenance, in addition to the amortization cost of the STEP system computed on the same basis as the GP system. The amortization of the $66,850 capital cost yields an annual cost of $151 per home, or $12.65 per month. The total monthly cost for the STEP system is, therefore, $17.39. Grant eligibility is the same for both systems, except that new septic tanks required for the STEP approach are not eligible.

The example is admittedly crude, but it gives some idea of the cost-estimating procedures necessary to evaluate proposed pressure sewer systems. Additional factors will have to be evaluated to properly accomplish such an estimate in a real situation.

CONCLUSIONS

Pressure sewer systems are a viable alternative technology and should be considered in any cost/effective analysis of alternative wastewater management systems in rural communities.

Pressure sewers offer many advantages over conventional gravity sewers in areas where:

- Population density is low.
- Severe rocky conditions exist.
- High ground water or unstable soils prevail.
- Undulating terrain predominates.

The most serious impediment to wider adoption of pressure sewer technology is the present lack of comprehensive long-term operation and maintenance data and treatment information.

Lower capital costs and significantly shorter construction times are inherent in pressure sewer technology, as compared to conventional methods.

Pressure sewers should only be considered with properly conceived management arrangements. Failure to do so could seriously limit the effectiveness of this technology.

Two major types of pressure sewer system designs are available: GP systems and STEP systems. The relative merits of the systems should be weighed by the engineer in his cost/effective evaluation.

REFERENCES

[1] E. J. Bowles, "Pressurized Waste Water Collection," presented at the Water Pollution Control Federation Conference, Atlanta, Ga., 1972.

[2] I. G. Carcich, L. F. Hetling, and R. P. Farrell, "Pressure Sewer Demonstration," *J. Environ.*, Eng. Div., Am. Soc. Civ. Eng., *100*, 1, 25-40, 1974.

[3] C. W. Rose, "Rural Wastes: Ideas Needed," *Water Wastes Eng.*, *9*, 2, 46-47, 1972.

[4] P. L. Gainey and T. H. Lord, *Microbiology of Water and Sewage*, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1957.

[5] R. Smith and R. G. Eilers, *Cost to the Consumer for Collection and Treatment*, U.S. Environmental Protection Agency, Report No. 17090-07/70, 1970.

[6] *Manual of Septic-Tank Practice*, U.S. Public Health Service, Publication No. 526, 1969.

[7] American Society of Civil Engineers, *Combined Sewer Separation Using Pressure Sewers*, U.S. Department of the Interior, Federal Water Pollution Control Administration, FWPCA Report No. ORD-4, 1969.

[8] I. G. Carcich, L. J. Hetling, and R. P. Farrell, *A Pressure Sewer Demonstration*, U.S. Environmental Protection Agency, Report No. R2-72-091, 1972.

[9] M. A. Clift, "Experience with Pressure Sewerage," *J. Sanit.*, Eng. Div., Am. Soc. Civ. Eng., *94*, 5, 849-865, 1968.

[10] I. G. Carcich, L. J. Hetling, and R. P. Farrell, "The Pressure Sewer: A New Alternative to Gravity Sewers," *Civ. Eng.*, *44*, 5, 50-53, 1974.

[11] G. Mekosh and D. Ramos, *Pressure Sewer Demonstration at the Borough of Phoenixville, Pennsylvania*, U.S. Environmental Protection Agency, Report No. R2-73-270, 1973.

[12] G. F. Hendricks, "Pressure Sewage System and Treatment at Grandview Lake, Indiana," presented to the American Society of Agricultural Engineers Convention, Pullman, Wash., 1971.

[13] S. M. Rees and G. F. Hendricks, *Grandview Lake Sewage Research and Demonstration Project—Annual Report*, prepared for the U.S. Environmental Protection Agency, 1971.

[14] G. F. Hendricks and S. M. Rees, *Economical Residential Pressure Sewage System with No Effluent*, U.S. Environmental Protection Agency, Technology Series, Report No. EPA-600/2-75-072, December 1975.

[15]D. M. Cochrane, "Pressurized Sewer Systems: Regulatory Agency's Viewpoint," presented at the Water Pollution Control Federation Conference, Denver, Colo., October 1974.

[16]G. C. Gray, "Environmental Constraints Challenge Designers of Shoreline Community near Kansas City, Missouri," *Prof. Eng.*, *45*, 6, 1975.

[17]T. C. Williams, "Plastic Pipe and Pressure Sewers Mark Expansion," *Water Wastes Eng.*, *12*, 11, 85, 1975.

[18]Alfred Crew Consulting Engineers, Inc., *Preliminary Report on Water Pollution Control System-Siek Road Area (Addendum No. 2)*, prepared for Borough of Kinnelon, N.J., 1974.

[19]R. L. Sanson, "Design Procedure for a Rural Pressure Sewer System," *Public Works*, *10*, 86-87, 1973.

[20]W. C. Bowne, *Glide-Idleyld Park Sewerage Study*, Douglas County, Ore., Report, 1975.

[21]*Detail Drawings and Specifications SA-2021B-137 through SA-2021B-144*, Environment/One Corporation, 1973.

[22]*Design Handbook for Low Pressure Sewer Systems*, Environment/One Corporation, 1973.

[23]E. R. Bennett and K. D. Linstedt, *Individual Home Wastewater Characterization and Treatment*, Colorado State Environmental Resources Center, Completion Report No. 66, July 1975.

[24]M. Witt, R. Siegrist, and W. C. Boyle, "Rural Household Wastewater Characterization," *Home Sewage Disposal*, Proc. Am. Soc. Ag. Eng., *175*, 79-88, 1975.

[25]E. E. Jones, Jr., "Domestic Water Use in Individual Homes and Hydraulic Loading of and Discharge from Septic Tanks," *Home Sewage Disposal*, Proc. Am. Soc. Ag. Eng., *175*, 89-103, 1975.

[26]Metcalf and Eddy, Inc., *Wastewater Engineering*, McGraw Hill, New York, 1972.

[27]S. R. Weibel, C. P. Straub, and J. R. Thoman, *Studies on Household Sewage Disposal Systems—Part I*, U.S. Public Health Service Publication, 1949.

[28]T. W. Bendixen, M. Berk, J. P. Sheehy, and S. R. Weibel, *Studies on Household Sewage Disposal Systems—Part II*, U.S. Public Health Service, Publication, 1950.

[29]S. R. Weibel, T. W. Bendixen, and J. B. Coulter, *Studies on Household Sewage Disposal Systems—Part III*, U.S. Public Health Service, Publication No. 397, 1954.

[30]J. W. Patterson, R. A. Minear, and T. K. Nedoed, *Septic Tanks and the Environment*, prepared for Illinois Institute for Environmental Quality, 1971.

[31]*Specifications for Hydromatic Model CSPG-150A*, Hydromatic Pump Company, 1973.

[32]W. C. Bowne, *Pressure Sewer Systems*, prepared for Douglas County, Ore., 1974.

[33]A. T. Voell, *Investigation of Pressure Sewer System and Advanced Waste Treatment*, prepared for Chatauqua County, N.Y., Health Department, 1972.

[34]J. Leckman, *Pressurized Sewer Collection Systems*, prepared for Illinois Institute of Environmental Quality, 1972.

[35] L. J. Flanigan and R. A. Cudnik, *Review and Considerations for the Design of Pressure Sewer Systems*, Hydromatic Pump Division, Weil-McLain Company, 1974.

[36] *Water Flow Characteristics of Thermoplastic Pipe*, Plastics Pipe Institute, Technical Report No. PPI-TR14, 1971.

[37] *Report on the Performance of Grinder Pump Model Farrell 210*, National Sanitation Foundation Testing Laboratory, Report No. C-9-3, 1973.

[38] J. C. Kent, *The Entrainment of Air by Water Flowing in Circular Conduits with Downgrade Slopes*, master's thesis, University of California, 1952.

[39] R. P. Farrell, *Operation of Grinder Pumps and Pressure Sewer Systems under Abnormal Conditions*, information bulletin from Environment/One Corporation, 1974.

[40] M. F. Hobbs, *Relationship of Sewage Characteristics to Carrying Velocity*, Report No. R-2598 to Am. Soc. of Chem. Eng., 1967.

[41] *Sewerage Manual*, Pennsylvania Dept. of Environmental Resources, Bureau of Water Quality Management, Publication No. 1, 3d ed., Harrisburg, Pa., 1976.

[42] *Code of Practice for Plastics Pipework: Part 1—General Principles and Choice of Material*, British Standards Institution, Publication CP 312 (Gr B), London, England, 1973.

[43] *Code of Practice for Plastics Pipework: Part 2—Unplasticized PVC Pipework for the Conveyance of Liquids under Pressure*, British Standards Institution, Publication CP 312 (Gr B), London, England, 1973.

[44] Great Lakes—Upper Mississippi River Board of State Sanitary Engineers, *Recommended Standards for Sewage Works*, Health Education Service, Albany, N.Y., 1971.

[45] R. D. Pomeroy, *Process Design Manual for Sulfide Control in Sanitary Sewerage Systems*, U.S. Environmental Protection Agency, Technology Transfer Publication, 1974.

[46] A. Dounoucos, "Sanitary System Construction Costs Turn Engineering Attention to Alternate Solutions," *Prof. Eng.*, *44*, 8, 1974.

Part II

VACUUM SEWERS

BACKGROUND

Vacuum sewers are one of the alternatives in sewer systems for smaller communities, land developments, and rural areas. The advantages of these systems may include substantial reductions in water use, material costs, pipe size, excavation costs, and treatment expenses, and a potential for overall cost/effectiveness.

Vacuum systems depend on a central vacuum source constantly maintaining 15 to 25 inches of mercury on small-diameter collection mains (fig. II-1). A gravity vacuum interface valve separates atmospheric pressure from the vacuum in the mains. The valve can be either in the home sanitary sewer service line or in a vacuum toilet. When the interface valve opens, a volume of sewage enters the main, followed by a volume of atmospheric air. After a preset interval, the valve closes. The packet of liquid, called a slug, is propelled into the main by the differential pressure of vacuum in the main and the higher atmospheric pressure air behind the slug. After a distance, the slug breaks down by shear and gravitational forces, allowing the higher pressure air behind the slug to slip past the liquid. With no differential pressure across it, the liquid then flows to the lowest local elevation, and vacuum is restored to the interface valve for the subsequent operation. When the next upstream interface valve operates, identical actions occur, with that slug breaking down and air rushing across the second slug. That air then impacts the first slug and forces it further down the system. After a number of operations, the first slug arrives at the central vacuum source. When sufficient liquid volume accumulates in the collection tank at the central vacuum source, a transfer device, such as a sewage pump, delivers the accumulated sewage to a treatment plant (fig. II-2).

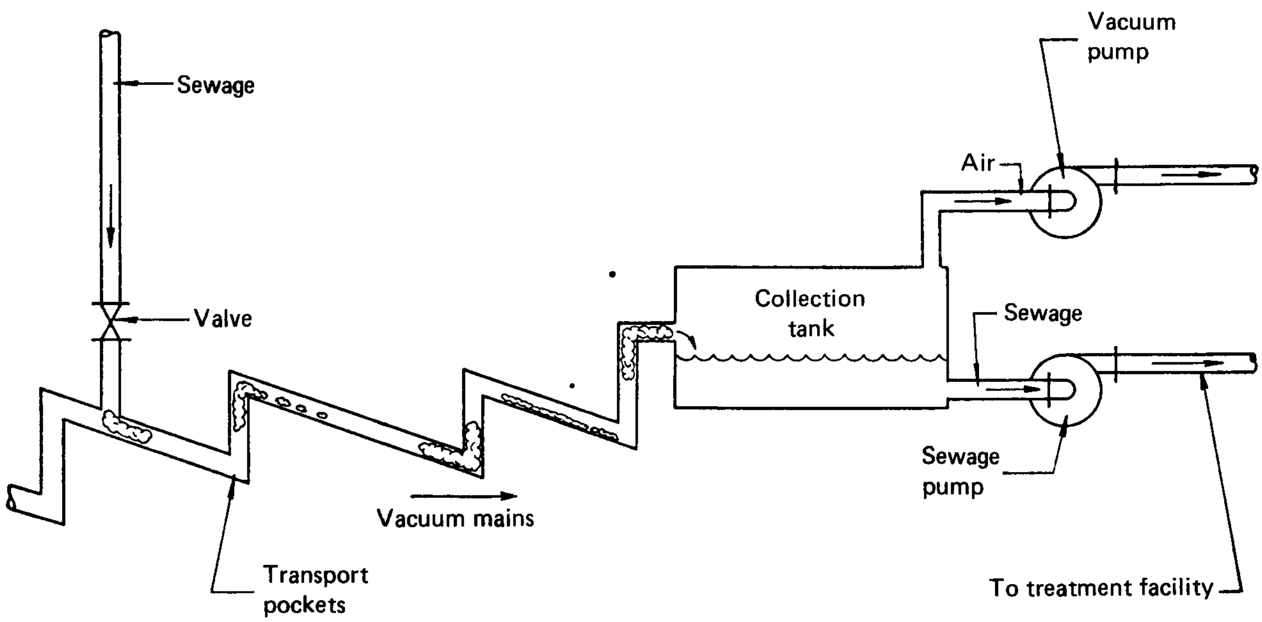


Figure II-1. Elements of a vacuum sewage system.

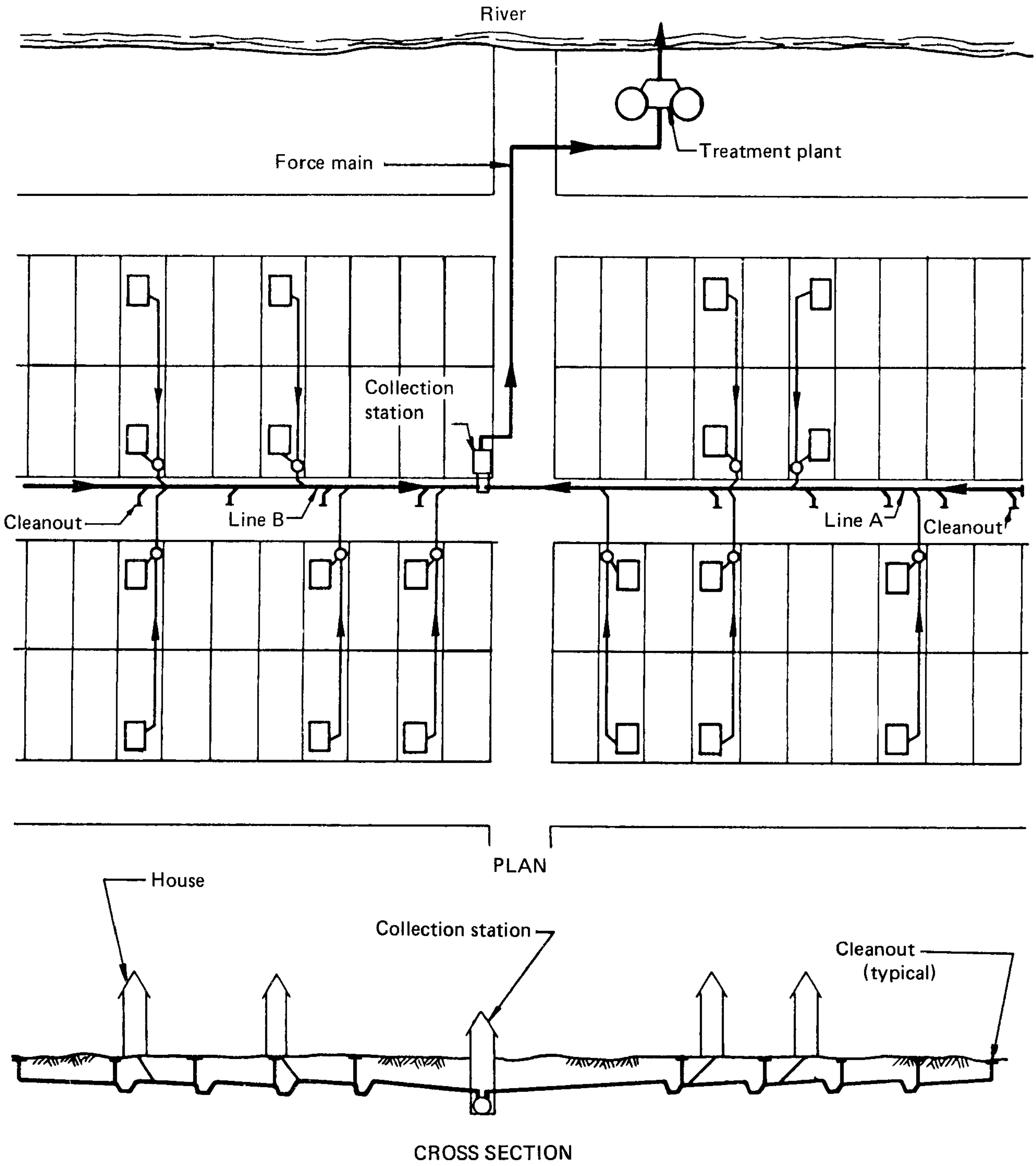


Figure II-2. Typical layout—vacuum sewer system.

Vacuum sewage collection systems have been patented in the United States since 1888, when Adrian LeMarquand invented a system of sewage collection by barometrical depression.[1] The first commercial applications of such systems were by the Liljendahl Corporation (now known as Electrolux) of Sweden in 1959.[2] Currently, several companies in the United States are actively marketing vacuum equipment for residential systems in this country. There are significant differences among the designs of the four types of currently installed systems covered in this paper: Liljendahl-Electrolux, Colt Envirovac, AIRVAC, and Vac-Q-Tec.

The Liljendahl-Electrolux system was introduced to this hemisphere in the Bahamas (fig. II-3) between 1965 and 1970. This concept uses separate black and gray water collection mains. Black water refers to toilet wastes and gray water includes all other domestic wastewater. Black water is generated at a vacuum toilet (fig. II-4), which discharges about 3 pints per flush. The gray water generated in the home is discharged into the system by a specially designed vacuum valve. The wastewater is transported through separate black and gray water vacuum mains to vacuum collection stations for disposal. About 1,600 vacuum toilets are components of 14 separate systems in the Bahamas. The 90-percent reduction in toilet wastewater volume was a definite consideration in the selection of these systems for critically water-short Nassau.

The first residential vacuum collection system in the United States was designed by Vac-Q-Tec and serves the Lake of the Woods development near Fredericksburg, Va. Vac-Q-Tec has designed several other residential vacuum sewage collection systems, all in use by private developers. This system uses concepts of the Liljendahl system but has many important differences.[3] The Vac-Q-Tec system requires no inside vacuum toilets or vacuum plumbing and has combined black and gray water vacuum collection mains. The system includes a 750-gallon storage tank at the homesite; pneumatically operated, electrically controlled vacuum valves; 4-inch polyvinyl chloride (PVC) collection lines; and 13 vacuum collection stations.

The Colt Envirovac system, depicted in figure II-5, is the direct descendant of the Liljendahl-Electrolux system.[4] Colt is currently marketing self-contained vacuum sewage comfort stations, as well as preengineered community vacuum collection systems. The Colt system at South Seas Plantation near Fort Myers, Fla., serves 33 residences, with separate building plumbing for gray and black wastewater. A gray water valve serves each residence. Black water piping from the vacuum toilet joins the gray water vacuum piping immediately downstream of the gray water valve. The system then functions as a single pipe network to the vacuum collection station.

The AIRVAC Company markets a pneumatically controlled and operated vacuum valve for combined gray and black water systems (fig. II-6). A design manual illustrating the use of the AIR-

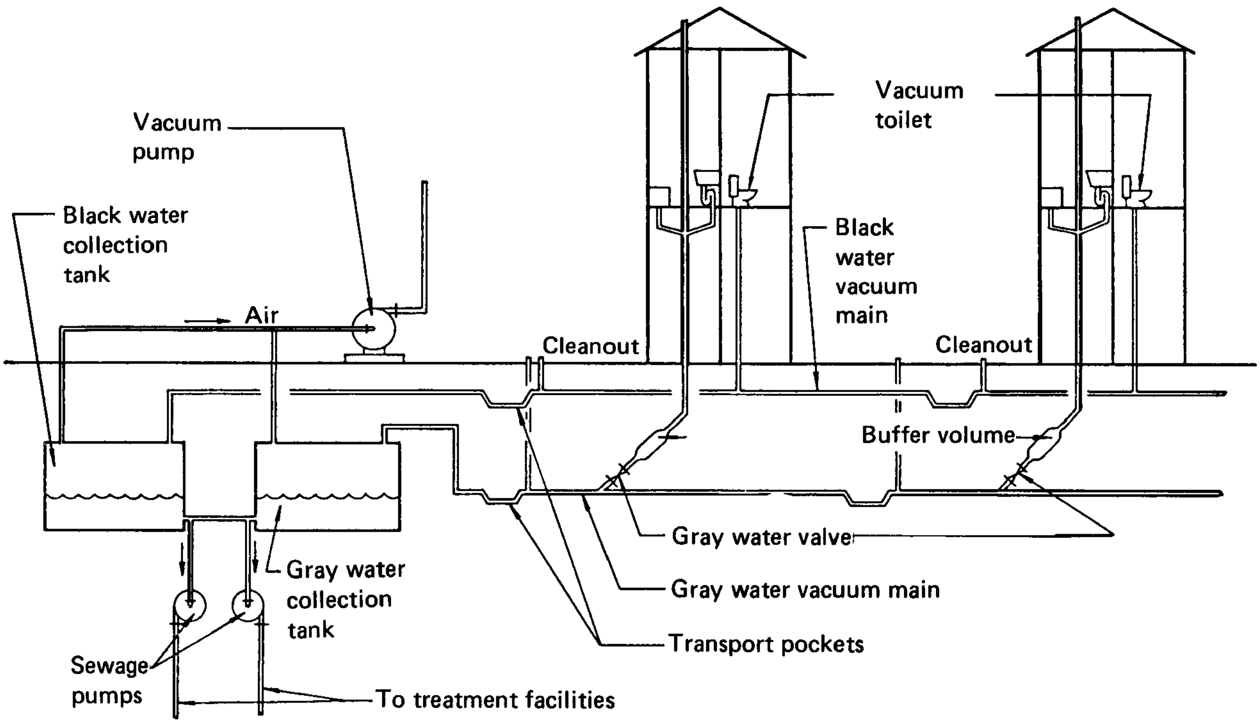


Figure II-3. Liljendahl-Electrolux vacuum sewer collection system—Nassau.

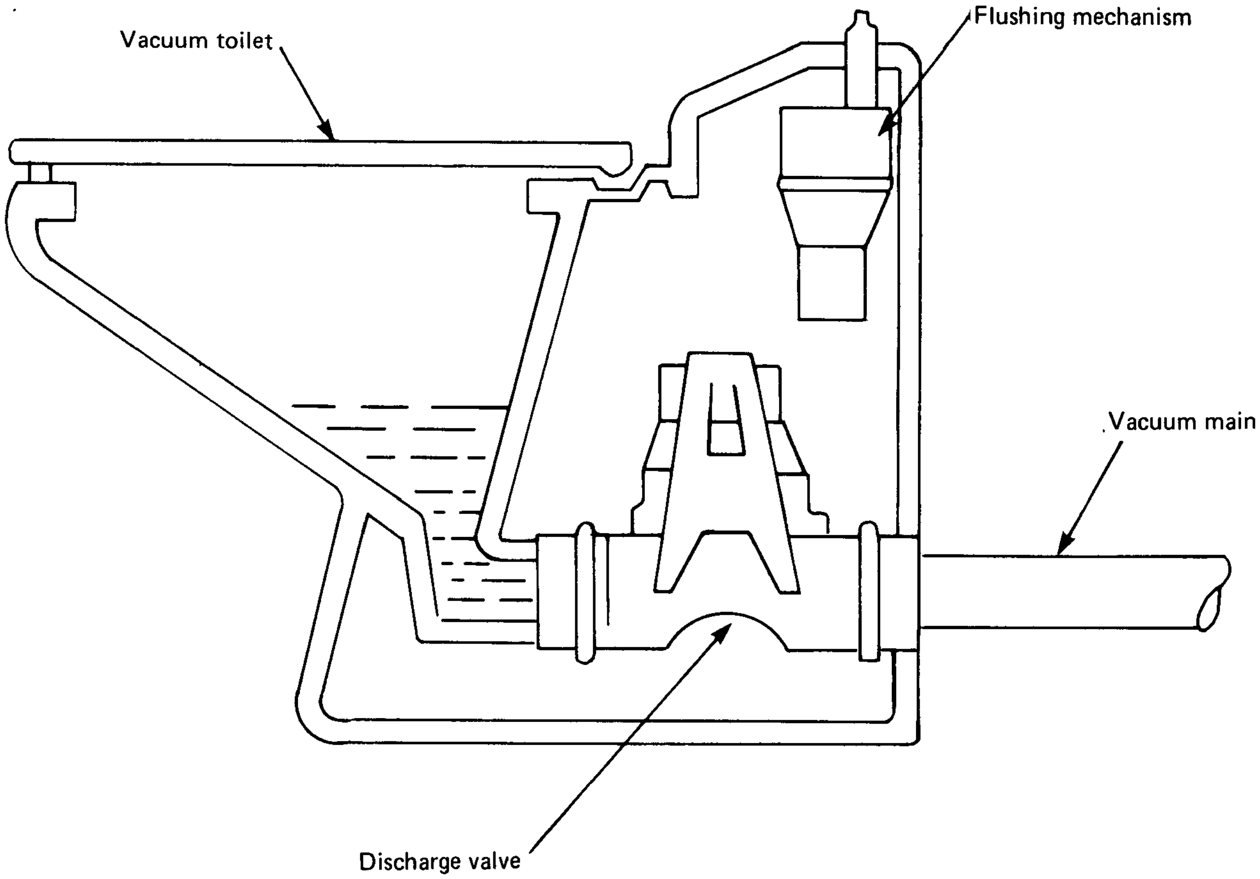


Figure II-4. Vacuum toilet.

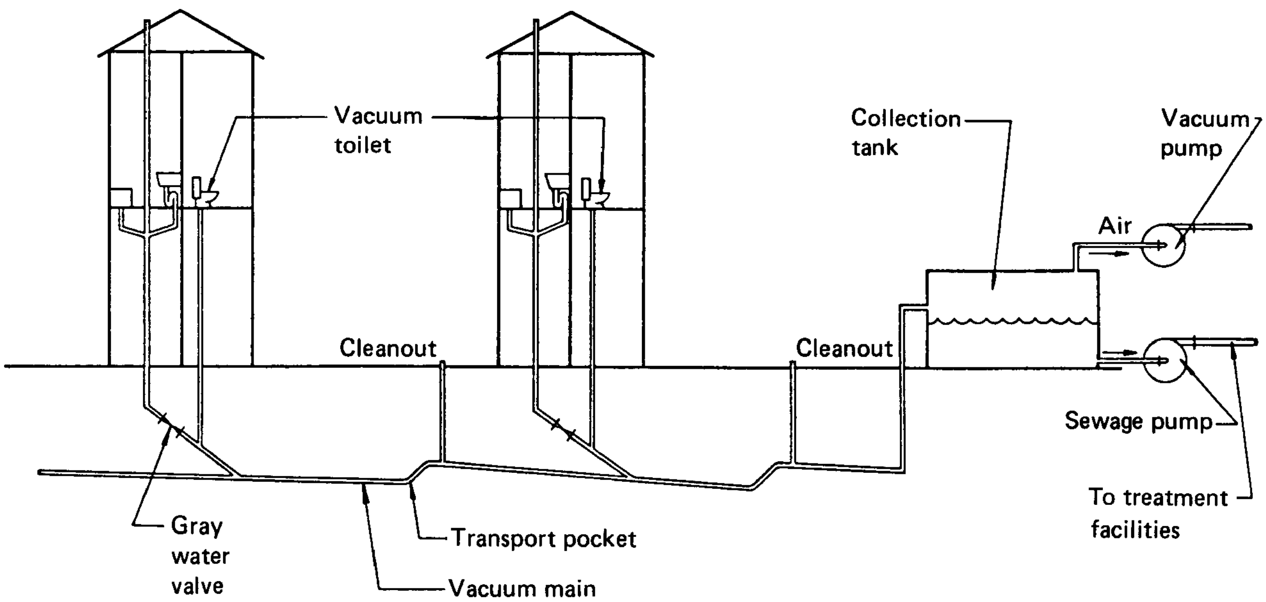


Figure II-5. Colt Envirovac vacuum sewage collection system.

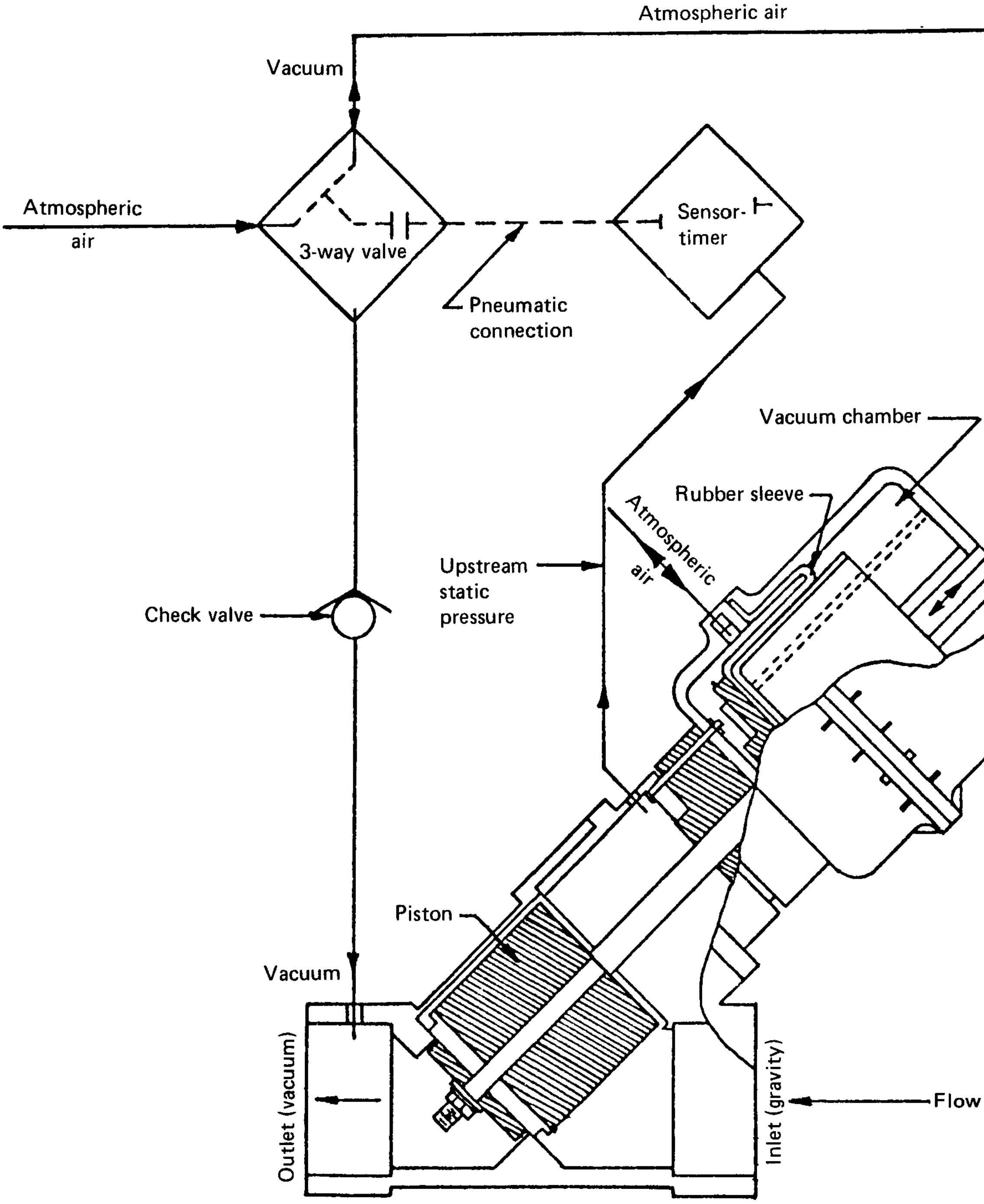


Figure II-6. AIRVAC 3-inch valve.

VAC product is currently available. AIRVAC has seven operating residential systems throughout the United States, with about 1,000 valves installed. The AIRVAC system (fig. II-7) uses conventional gravity household plumbing, with wastewater discharging into their 3-inch valve. The vacuum valve is located in a valve pit. This valve starts its operation cycle when it senses approximately 10 gallons of accumulated sewage, admitting that liquid and a quantity of air to the mains.[5] Sewage travels through 3-inch, 4-inch, or 6-inch mains to a vacuum collection station.

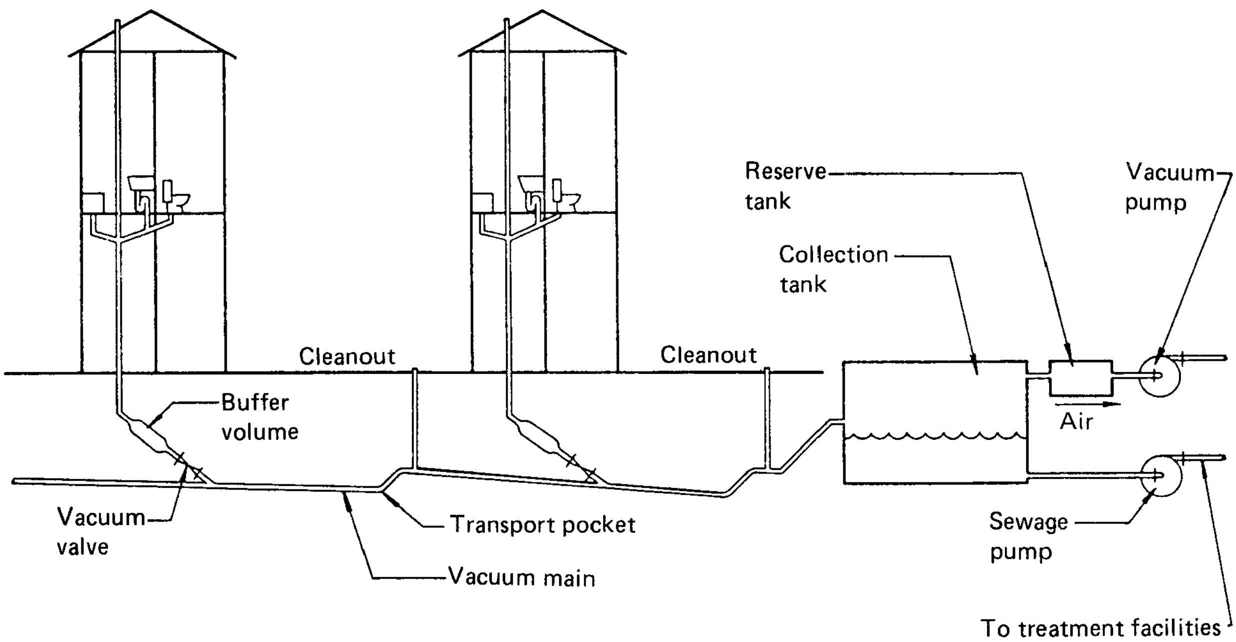


Figure II-7. AIRVAC vacuum sewage collection system.

SYSTEM PARAMETERS

General

A comparison was made of the basic system parameters of each of the four types of vacuum sewage collection installations. The comparison was based on manufacturers' promotional literature, design guidelines, and site visits to 18 systems throughout the Western Hemisphere. The major differences among these collection systems are shown in table II-I. The water-saving feature of the Electrolux and Colt systems is reported to be as much as 27 percent of the total in a domestic application with the use of vacuum toilets.[4,6,7] AIRVAC and Vac-Q-Tec systems can be altered to accommodate these water-saving devices.

Vacuum Valves

Vacuum toilets are flushed after each use, while vacuum valves operate automatically, based on the volume of gray or gray and black water behind the valve. When a predetermined volume has accumulated, the valve opens, provided there is adequate vacuum available. Atmospheric air forces the sewage into the mains, which is followed by a volume of air. The valve is actuated by a pneumatic controller in all systems except the Vac-Q-Tec. Vac-Q-Tec's valve operation requires a separate electrical power source at each valve site to control valve operation.

The Vac-Q-Tec's gravity-vacuum interface valve assembly is unique in that it requires an external electrical power source. The valve's position can be monitored and the valve operated from the collection station through an extra set of contacts in the controller. A separate cycling mode, called Auto-Scan, offers added flexibility. This mode locks out the accumulated volume-cycle command from each valve, and sequentially operates each valve during low-flow periods. Additional operating and skilled electronics technicians are required to maintain these more complex systems.

The capability for shaving peak flows is possible with the Vac-Q-Tec system when additional controls are added to the base system. During high-flow periods, wastewater is stored in the 750-

Table II-1.—*Vacuum collection system parameters*

System type	House piping	Valve type	Discharge volume	Piping profile	Cleanouts	Collection line
Electrolux	Black and gray, separate	Black, vacuum toilets; gray, pneumatic valves, 2 in	Black, 3 pt Gray, 10 gal	Set configuration with traps	200-250 ft	Black, 1½ and 2 in; gray, 2 and 3 in; PVC, solvent weld
Colt	Black and gray, separate	Black, vacuum toilets; gray, pneumatic valves, 2 and 3 in	Black, 3 pt Gray, 10 gal	Set configuration with traps	200-250 ft	Single main, 3, 4, and 6 in; PVC, special "O" ring
Vac-Q-Tec	Conventional plumbing	Electrically actuated pneumatic valve	75-100 gal	Parallels terrain with traps	No	Single main, 4 in; PVC
AIRVAC	Conventional plumbing	Pneumatic valve, 3 in	10-15 gal	Set configuration with traps	Yes	Single main, 3, 4, and 6 in; PVC, solvent weld

gallon residential tank and is released later during low-flow periods. All other systems must be designed to handle peak flows.

The amount of water entering the system with each valve operation varies with each manufacturer and the type of gravity vacuum interface valve. The vacuum toilet admits approximately 0.3 to 0.4 gallon per flush, and the pneumatically controlled valves admit 10 to 15 gallons per cycle. U.S. Navy research[8] has reported that good transport characteristics are found with sufficient inlet air and small enough slug loadings for the available pressure differential to overcome the liquid's inertia. This results in rapid slug breakdown, reestablishing vacuum quickly at upstream valves.

Piping Systems

Piping profiles vary in each system. The manufacturer recommends differing profiles, depending on uphill, downhill, or level terrain. Vacuum reformation traps are located where the designer wishes to reform a slug of water for transport purposes. Traps also are used to gain elevation by raising the mains closer to the ground surface or to conform to terrain variations.

PVC pipe is common to all systems. Solvent-weld and O-ring joints have been used; when O-ring is used, however, it must have a joint designed to seal against vacuum. Studies showed in some systems initial savings in capital expenditures from the use of low-cost, smaller diameter PVC pipes rather than gravity sewer mains.[7] Construction cost savings were also realized by not having to shore deep, sandy trenches or blast deep rock trenches to install gravity sewers.

Collection Stations

Collection station (often called vacuum central) design parameters, shown in table II-2, vary with each manufacturer. Electrolux and Colt vary their use of vacuum reserve tanks with each installation, while AIRVAC and Vac-Q-Tec use vacuum reserve tanks between the receiving tanks and their vacuum pumps. The use of vacuum reserve tanks extends vacuum pump life by reducing pump cycling.

Vacuum pump construction has been both sliding vane and liquid ring. Pros and cons are numerous on the use of each, and no standard has yet emerged. Liquid ring pumps, however, have been used more frequently in vacuum sewer applications. The contents in the vacuum collection tank must be transferred to a treatment facility after sufficient volume has been collected. Nonclog

Table II-2.—*Vacuum collection station parameters*

System type	Receiving tank	Receiving tank evacuation device	Valve monitoring and control capability
Electrolux . . .	Separate black and gray water vessels. Reserve tank use varies by installation.	Sewage pumps	No
Colt	Common receiving vessels. Reserve tank use varies.	Sewage pumps	No
Vac-Q-Tec . . .	One receiving vessel plus reserve tank.	Pneumatic ejectors	Yes
AIRVAC	One receiving vessel plus reserve tank.	Sewage pumps	No

sewage pumps with sufficient net positive suction head (NPSH) to overcome tank vacuum are generally used. NPSH refers to the total suction lift in feet measured at the suction nozzle less the vapor pressure of the liquid in feet. It is important to use pumps whose shaft seals close against vacuum; otherwise, system vacuum will be depleted during low-flow periods.

VACUUM SEWAGE CHARACTERISTICS

In a contractual study for EPA, a sampling and monitoring program was initiated to determine sewage flow and strength characteristics from AIRVAC's Mathews site. This level site, with persistent high ground water, offered an excellent baseline study as the newly completed vacuum system served residences and businesses, while an existing gravity system served the central portion of the town. Wastewater flow and strength characteristics were studied for a continuous 7-day period. Monitoring was accomplished by Manning flow meters, level recorders, and automatic samplers located at the influent splitter box of a 100,000-gal/d contact stabilization treatment plant.

Mathews

Gravity wastewater flows exhibited a characteristic diurnal flow pattern (fig. II-8). This composite of 7 days' flow data showed vacuum wastewater flows lagged the gravity flow pattern by up to 2 hours, depending on the time of day. Flows from the vacuum system showed greater hourly flow fluctuations than gravity flows because of the storage and intermittent discharge features of the vacuum station. The midnight to 6 a.m. time period points to possible reduced infiltration of the vacuum system over the gravity system.

Dissolved oxygen (DO) of the gravity sewage, collected from sources close to the monitoring station, exhibited an uncharacteristically high 6.0 to 7.0 mg/l, possibly because of closeness to the

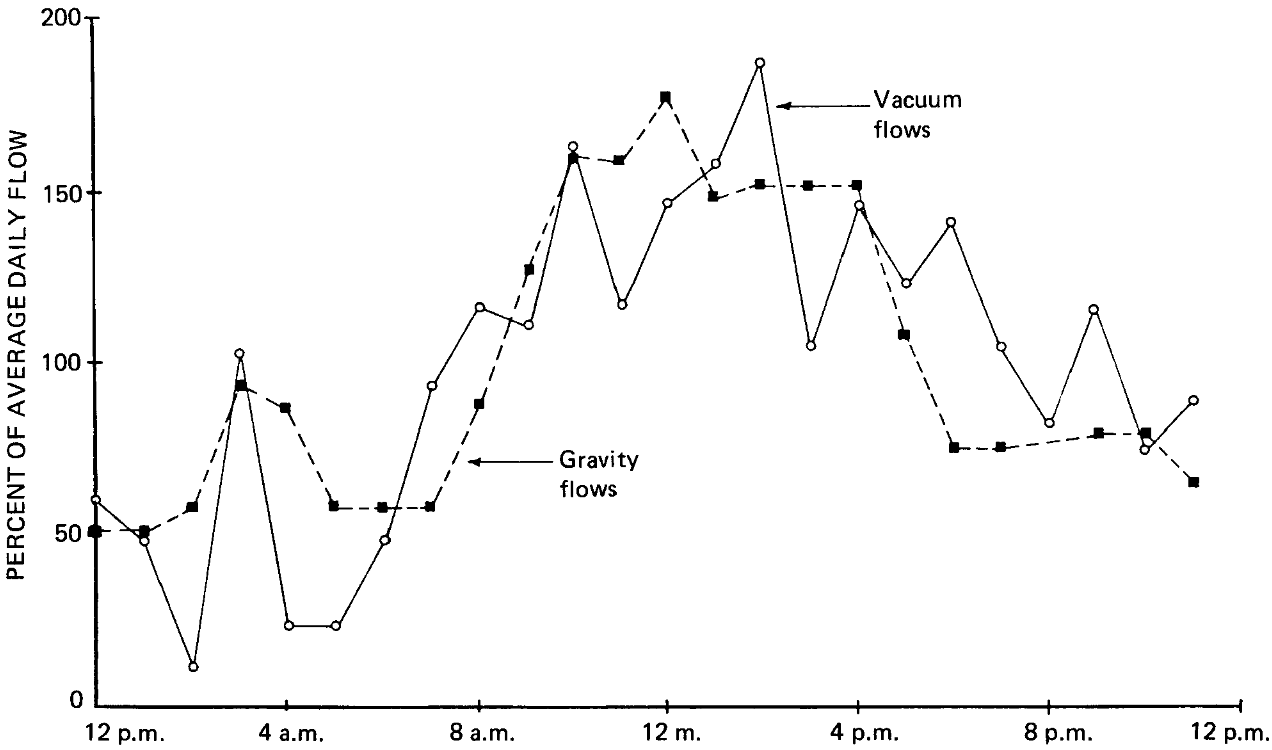


Figure II-8. 7-day composite wastewater flows at Mathews treatment plant.

source (± 300 feet) and turbulence from pumping immediately before the sampling point. Vacuum sewage consistently maintained a DO in the range of 0.6 to 1.0 mg/l. The low DO readings could have been caused by the collection system vacuum conditions removing DO. From dye tests performed during a morning high usage period, discharges from a vacuum valve 6,000 feet from the collection station reached the treatment plant in 10 hours. In 2 hours the flow introduced by a valve 1,500 feet from the collection station appeared at the plant. The sewage residence time in the receiving tank at the collection station is 1 hour or less. This time allows for a reduction in the receiving tank of the sewage's oxygen content.

Biochemical oxygen demand (BOD) tests on both gravity and vacuum sewage composite samples were run for the 7-day monitoring period. Vacuum sewage exhibited a 5-day BOD (BOD_5) of 136 mg/l, while gravity sewage contained 76 mg/l. The lower BOD in gravity sewage can be attributed to infiltration in the old gravity mains located in the high ground water area. Reaction constants for biological degradation of raw sewage, or K values, were determined for both gravity and vacuum sewage. Gravity-collected sewage exhibited a K value of 0.176, while vacuum sewage yielded a K value of 0.168. From these limited data, no substantial difference can be seen in the treatment of these two wastewaters.

Vacuum-collected sewage was observed to be more homogeneous, with solids much more finely divided than gravity sewage, a consequence of the turbulence during vacuum transport.

South Seas Plantation

Sewage samples from Colt's South Seas Plantation vacuum system were also collected over a 7-day period. With vacuum toilets, a correspondingly higher BOD_5 would be expected. The BOD_5 from the 33 residential connections was 371 mg/l. During the test period, a substantial savings in water was realized by the Colt system using vacuum toilets.

COST COMPARISONS

The life cycle cost of the vacuum sewage collection system was compared to the alternatively bid gravity system for Mathews, shown in table II-3. This comparison is not intended to be typical but represents one system where alternative bids were taken and data were available. This value engineering analysis[9] brings all costs to the present worth position and then amortizes these costs over an assumed period—20 years in this case.

Initial capital expenditures are bid prices or installed costs and are in the present worth form. These costs are amortized over the bond issue life at 6 percent, which converts these figures to annual capital recovery cost amounts. The costs for replacing equipment included rewinding motors in the seven pumping stations, which would be required in the gravity alternative at 10 years and 20 years. The identical time periods are selected for replacement of vacuum valve assemblies and manholes. No allowance has been made for price increases because of inflation. Operating experience in the United States is limited, and data on experienced replacement periods are not available. The replacement costs for each alternative are brought back to their present worth based on the year of their replacement and then amortized by the capital recovery annuity over the life of the project.

Annual operation and maintenance costs for each system are based on actual previous year expenditures for the vacuum system or from similar costs incurred in other gravity-serviced communities. An additional amount is added to the annual cost of the gravity system to account for treatment of infiltration at a rate of 100 gal/in diameter per mile per day. The annual difference

Table II-3.—*Life-cycle cost analysis, Mathews, Va., sewer system*

[Dollars]

Costs	System	
	Gravity	Vacuum
Initial:		
Instant contract:		
Base	555,325	270,773
Interface:		
Pump stations	88,267	
Vacuum collection station		105,028
Other (collateral)	0	0
Total	643,592	375,801
Replacement (life cycle):		
Year 10 at 6 percent	6,000	60,062
Present worth of future replacement cost (0.5584)	3,350	33,538
Year 20 at 6 percent	6,000	60,062
Present worth of future replacement cost (0.3118)	1,870	18,727
Life-cycle:		
Annual owning and operating:		
Capital recovery of total initial cost amortized at 6-percent, 20-year initial factor (0.08718)	56,108	32,762
Capital recovery of present worth of replacement cost at 6 percent:		
Year 10	292	2,923
Year 20	163	1,632
Annual cost:		
Maintenance	4,101	7,235
Operation	1,769	2,452
Infiltration at 100 gal/in diameter per mile per day	1,009	
Total	63,442	47,004
Annual difference		16,438
Present worth of annual difference		188,543

(fig. II-9) shows a savings of almost $16,438 per year for the vacuum system, which is equivalent to a life cycle present worth of $188,543. The cost differential between this vacuum system and its gravity alternative would be less if an inflation component were incorporated into the replacement cost expenditures.

Recreational and second-home developers may see a significant initial cost advantage. Valve assemblies need to be added to the basic piping and collection station only when an owner decides to build on his property. The cost of the valve is then paid by the homeowner. This cost is usually in excess of the connection fee cost assessed by municipalities.

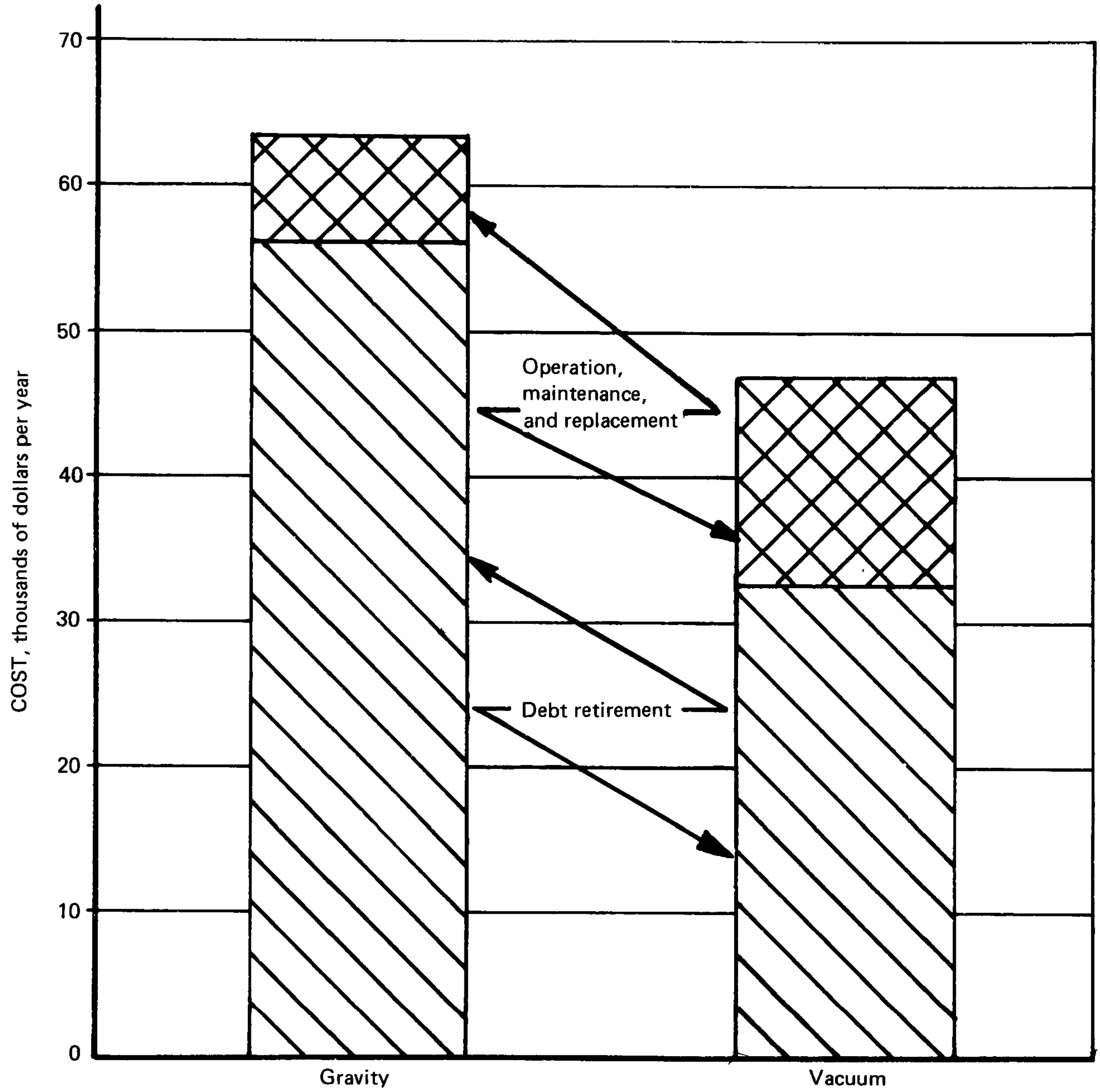


Figure II-9. Annual operation and maintenance costs, Mathews treatment plant.

CRITIQUE OF EXISTING SYSTEMS

Earlier vacuum sewage collection systems were often plagued with consistent operational problems. Although this situation has improved with successive generations of systems, some problems still exist.

Deficiencies in vacuum systems can be broadly defined into three areas: system design, component reliability, and lack of operation and maintenance guidance.

System Experiences

Although vacuum systems provide the means for installing economical sewer systems in problem areas, early systems were installed without sufficient prototype or field testing of equipment components. As a result, there were several operational problems. Significant field experience has provided an opportunity for upgrading the original systems, and most problems have now been eliminated.

Problem Areas

Early U.S. systems, particularly Vac-Q-Tec's, were installed without a thorough investigation of two-phase transport characteristics. As a result, these systems experienced significant problems in transporting sewage from as few as 10 percent of the design population[10] because of improperly planned vacuum main profiles, too large slug volumes, and insufficient air-admittance techniques.

Vac-Q-Tec's early systems, particularly at Lake of the Woods, located sensitive electronic valve control equipment in 55-gallon drums adjacent to the sewage holding tanks. These drums and electronics control boxes often corroded, causing numerous and continued malfunctions. Electrical control wires were buried alongside the sewage mains, which resulted in signal failures. The system's complex electronics, which includes monitoring and controlling valve operation from the collection station, proved a significant drawback. Highly skilled electronics technicians were required to keep the system in operating condition. Aware of these problems, current management is undertaking a replacement program aimed at simplifying the system and correcting existing problems.

Several AIRVAC and Colt systems were found to lack components now generally accepted as minimum design standards. These items include standby power and system malfunction reporting devices. The lack of standby power in the Colt system has caused valve boots to rupture when power outages occurred. An excessive amount of liquid built up behind the valves during outages. When power was restored and the valves cycled, there was not enough time to discharge the higher than normal column of water instead of air. The momentum of sewage resulted in the ruptured valve boots. With the addition of standby power, these problems should be alleviated.

The use of weak materials in early AIRVAC valve manholes caused problems. Manholes made of tar-impregnated paper deformed when placed in unstable soil or areas subject to vehicle traffic. Consequently, the transite cover bolt holes would not align with the manhole bolt clips, preventing adequate fastening and resulting in damage in some cases. Improved manhole materials are now recommended, such as the spun-wrapped fiberglass valve pit, counterweighted to prevent flotation with a cast-iron manhole cover, and should eliminate these problems. A breather tube extension above potential water levels and controller modifications have also minimized past reliability problems.

Additional problems have resulted from the use of manholes without bottoms in high ground water areas. During high ground water periods, standing water was able to enter the sensitive sensor-controller pneumatic circuit, causing valves to continually cycle and deplete system vacuum. These valve reliability problems were evident in Eastpoint, Fla., and Mathews systems. Reliability has been improved through recent modifications.

Designers must be aware of a public health hazard that may exist if a house on a vacuum system has a vent stack smaller than a 7.62-cm (3-inch) bore. When the valve operates, it may evacuate those water traps, allowing sewer gas from the local holding tank to enter the house. A 7.62- to 10.16-cm (3- to 4-inch) vent stack, installed on the gravity sewer lateral adjacent to the house wall, will eliminate this problem.

Valve failure also can cause failure of a system or a large portion of a system. If a valve fails in an open position or cycles continuously, available vacuum in the system may fall below acceptable levels.

OPERATION AND MAINTENANCE

Because of the complexity of vacuum equipment, operating personnel must be properly trained to maintain a vacuum sewer system. Some early installations suffered for lack of proper operation and maintenance manuals and other aids that would have assisted the operators in coping with this new technology. Manufacturers are now recognizing this need and are reacting accordingly with improved technical assistance and operation and maintenance manuals to assist system operating personnel.

Operation and Maintenance Tasks

Operation and maintenance tasks for vacuum systems can be divided into normal operation, preventive maintenance tasks, and breakdown or emergency operation and maintenance tasks. Each of these headings can be further divided into tasks related to vacuum valves and to the collection station.

Valves. Depending on a system's emergency breakdown history, some periodic valve inspection is required. As a starting point, semimonthly inspection and manual operation of each valve is suggested. An experienced operator learns the sounds a well-functioning valve and controller make and can use this tool as a preventive maintenance device. The breather lines in the AIRVAC valve should be inspected for liquid accumulation, which, if found, should be removed. Yearly maintenance includes an exterior cleaning of AIRVAC's valve breather cap and Colt's sensor controller mechanism.

In hard-water areas, AIRVAC suggests that valves be removed and overhauled every 3 to 4 years for scale removal. At 6-year intervals, new seals and valve seats are recommended.

A card file listing each valve location and any preventive or breakdown maintenance performed will identify problem locations. This procedure is consistent with good management practice.

Collection Stations. Specific components will vary from station to station, as will the required operation and maintenance details. Some maintenance procedures common to all vacuum systems include a daily record of pump running hours, ammeter readings, and oil levels. Weekly procedures include checking battery terminals and battery condition of the standby generator, exercising the standby generator, blowing down sight glass of the collection tank (if present), and checking mechanical seal pressurizers on sewage discharge pumps (which prevent loss of system vacuum). Yearly preventive maintenance might include inspection of check valves on sewage discharge and gas evacuation lines.

Breakdown Maintenance

Vacuum system malfunctions occur in one of three places: the valve, the piping system, or the collection station. Malfunctions in the collection station are usually the result of a pump, motor, or electrical control breakdown and will not be discussed here.

If a valve malfunctions in the closed position, identification of the broken device is simplified as the homeowner will experience a backup of sewage in or near his house. A complaint call invariably follows. Replacing the sensor is usually the quickest solution to this problem.

If the system experiences low vacuum, characterized by a low-vacuum relay energizing an alarm system or by vacuum pumps running excessively with vacuum below normal, a vacuum leak has occurred. A vacuum leak is possible from either a break in the vacuum transport piping or from a malfunctioning valve. Breaks in the transport piping usually have been the result of underground construction (e.g., by a telephone or gas company) in the area cutting the vacuum line rather than a passive piping system failure. Valve malfunctions that result in low-system vacuum occur either when a valve sticks in the open position (also very rare), when an AIRVAC valve continually cycles, or when a Colt valve's boot ruptures. The AIRVAC problem is caused by accumulated moisture in the sensor lines and is a more common occurrence. Successive generations of controller designs with preventive maintenance may provide a satisfactory solution to this problem. System vacuum loss from a Colt valve has been reported after rupture of the valve boot in a situation described earlier. Programed boot replacement with mandatory standby power should prove effective in eliminating this problem.

An outline procedure for locating the source of vacuum failure has been documented by AIRVAC as follows:[5]

- When a low-vacuum condition occurs in a system, isolate, in turn, each incoming line to the collection tank to identify the problem line.
- Close off the line with low vacuum. Open remaining lines to clear sewage from them.
- Allow vacuum in operational lines to reach maximum vacuum; then close valves on all incoming valves.
- Open line with problem. Sometimes high vacuum applied quickly may correct malfunction. Leave line open to the collection tank.
- Starting at the collection tank, go to the first system isolation valve on problem line. Connect vacuum gage to vacuum valve prior to isolation valve. Close isolation valve and observe if vacuum builds up. If it does not, problem is between collection tank and isolation valve. If vacuum rises, repeat process on next isolation valve. Before reopening each isolation valve, allow vacuum to build up in nonproblem sections of sewer to clear that section's sewage.
- After isolating problem section, check each valve pit to locate malfunction. Often this can be accomplished by driving to each pit and listening from the vehicle window.
- After locating the malfunctioning valve, pump accumulated water and remove any debris from valve manhole. The manufacturer's emergency maintenance procedures should then be followed.
- If no valves are malfunctioning, check for underground construction that could have caused a break in the transport piping.
- If construction activity did not cause the leak, isolate leak by plugging vacuum main with test balloons at selected cleanout point locations.
- After plugging a small segment, inspect segment by walking the line to audibly determine the location of the underground break.
- Repair pipe section following specific pipe manufacturer's repair procedures.

Based on the Mathews system, it is estimated that 4 hours per connection per year should be allocated to operation and preventive maintenance. Breakdown maintenance will require time inputs in addition to preventive maintenance tasks. Total system operation and maintenance time required was found to range from about 4 hours per connection per year in systems with few problems to over 30 hours per connection per year in systems with significant problems.

At AIRVAC's Plainville, Ind., location, an attempt was made to determine the time necessary to locate a failed valve. A valve was caused to fail at a location unknown to maintenance personnel, who located the failed valve and placed the system back in operation after only 21 minutes had elapsed. No sewage backups or service interruption occurred during this short time period. A key component in continual operation is an effective alarm system, coupled with constantly available maintenance personnel.

CONCLUSIONS

Vacuum sewer systems in new and existing communities offer potential cost/effectiveness through:

- Lower construction costs from smaller diameter mains installed closer to the ground surface
- Decreased infiltration/inflow
- Reduced water consumption with use of vacuum toilets
- Ease of installation

Vacuum sewer systems are relatively new in the United States. Each system has provided information and operating experience that have generally improved subsequent system designs. System reliability, costs, design, and applicability to a particular site should be evaluated by the design professional before selection of a sewage transport system.

Contractors have reported that small-diameter PVC vacuum sewers laid close to the ground surface have been installed more quickly and at less cost under the following conditions:

- High ground water areas in permeable soil
- Rocky areas

While vacuum sewers contain many of the same advantages as the concurrently developed pressure sewer systems, some apparent differences are worth mentioning. The advantages of vacuum sewers over pressure systems can be called the three C's—conservation, centralization, contamination.

- Conservation—with the water-saving feature of the vacuum toilet, water conservation is possible.
- Centralization—because the motive force of a vacuum system depends on vacuum pumps operating from a central source, power outages would not affect a vacuum collection station equipped with a standby power source. It would be impractical, however, to provide standby power to each pump unit in a pressure sewer system.
- Contamination—vacuum systems are subject to infiltration during pipe leak or break, which is undesirable and expensive. Pressure systems in the same situation will force contaminated sewage into the soil. This feature is especially important in systems serving marinas, ship facilities, or warm climate systems where piping may be exposed. Leaking sewer lines in these instances would be a health hazard. Water mains might be laid closer to vacuum lines as compared to pressure or gravity systems, with a substantially lower risk of contamination.

RECOMMENDATIONS

The consultant must consider, at a minimum, the following items when selecting and designing a sewage transport system:

- Application in specific terrain
- System reliability
- Operation and maintenance requirements
- System life
- Standby equipment
- Alarm systems
- Emergency operating procedures during partial or total system failure
- Standby power requirements
- Cost analyses
- Simplicity of operation
- Recommendations of manufacturers

All the factors in this section should be evaluated by consulting engineers and appropriate government agencies before the selection and design of a vacuum transport system.

REFERENCES

[1] A. LeMarquand, "Sewerage or Drainage of Houses, Towns, or Districts, and Apparatus Therefor," Patent No. 377681, Feb. 7, 1888.

[2] "Water, Our Most Precious Possession," Liljendahl Vacuum Company, Ltd., Stockholm, Sweden, undated.

[3] B. C. Burns et al., "Method and Apparatus for Conveying Sewage," Patent No. 3,730,884, May 1, 1973.

[4] *Envirovac Technical Information*, Colt Industries, Beloit, Wis., undated.

[5] *Design Criteria Manual*, AIRVAC, The Vacuum Sewer Systems, Rochester, Ind., May 1976.

[6] D. W. Averil and G. W. Heinke, "Vacuum Sewer Systems," report prepared for the Northern Science Group of the Canadian Department of Indian Affairs and Northern Development, Jan. 1973.

[7] I. A. Cooper and J. W. Rezek, "Vacuum Sewer System Overview," presented at the 49th Annual Water Pollution Control Federation Conference, Minneapolis, Minn., Oct. 3-8, 1976.

[8] E. P. Skillman, "Characteristics of Vacuum Wastewater Transfer Systems," presented at the American Society of Mechanical Engineers Conference on Environmental Systems, July 1976.

[9] A. J. Dell 'isola, *Value Engineering in the Construction Industry*, Construction Publishing Co., Inc., New York, 1974.

[10] J. W. Rezek and I. A. Cooper, "Preliminary Report on Vacuum Sewer System at Lake of the Woods, Va., Volume I: Hydraulic Field Test Program," report prepared for Lake of the Woods Service Company and Utilities, Inc., Rezek, Henry, Meisenheimer and Gende, Inc., Libertyville, Ill., Nov. 1974.

Appendix A
VACUUM SEWAGE FLOW CHARACTERISTICS

FLOW REGIMES

Vacuum sewers exhibit characteristic two-phase flow regimes. The air (gaseous phase) and sewage (liquid phase) occur in segregated, intermittent, or distributed flow, as shown in figure A-1.

Segregated flow includes stratified flow, wavy flow, and annular flow, and can be identified by a continuity of liquid and gas flows for a long length of pipe.

Intermittent flow is characterized by intermittent segments of liquid and gas, such as in plug or slug flow.

Distributive flow more nearly approaches a homogeneous fluid than other flow regimes. It exists only momentarily in vacuum sewer systems when a slug breaks down. Mist and bubble flow are forms of this regime.

Baker developed a correlation for predicting flow regimes from air-water data, as shown in Figure A-2.[1] Other investigators, however, have reported problems in the accuracy of these predictions.[2] Baker's abscissa is defined by a viscosity compensation parameter:

$$\Psi = (73/\sigma)[\mu_l(62.3/\rho_l)^2]^{1/3} \tag{1}$$

which is found with the other parameters in the section on nomenclature.

Because this flow map is valid only under steady state conditions, predictions using this technique may not be typical, as consistently variable flow rates are the norm rather than the exception in vacuum sewer systems.[3] One author has pointed out that with a nonvarying flow, it may take weeks to achieve steady state conditions.[2]

SLIP

In hydraulic tests at Lake of the Woods, Va., vacuum sewer system, Brockmeier[4] added various volumes of air through a 2.54-cm (1-inch) opening (fig. A-3), after adding 284 litres (75 gallons) of liquid to the system. When adequate vacuum was available (usually $\geqslant$ 25.4 cm, or $\geqslant$ 10 inches of mercury), a rate of 1,135 l/min (40 f^3/min) air entered the mains. Skillman[5] found an inlet orifice between 3.18 and 10.16 cm (1.25 and 4 inches) in diameter will allow 1.4 times the rate of air as a 2.54-cm (1-inch) orifice under an initial 50.8-cm (20-inch) vacuum. Using this correlation, 1,590 l/min (56 f^3/min) of air would enter through a larger sized orifice.

Applying this correlation to an AIRVAC system that allows about 38 litres (10 gallons) to enter in 3-5 seconds of the 10-second valve cycle, an air/liquid ratio of 5:1 enters the mains. Under an average vacuum of 38 cm (15 inches), this ratio becomes 10:1 inside the main. Thus, in order for 10 volumes of air to be transported through the same conduit as 1 volume of liquid, the air must be

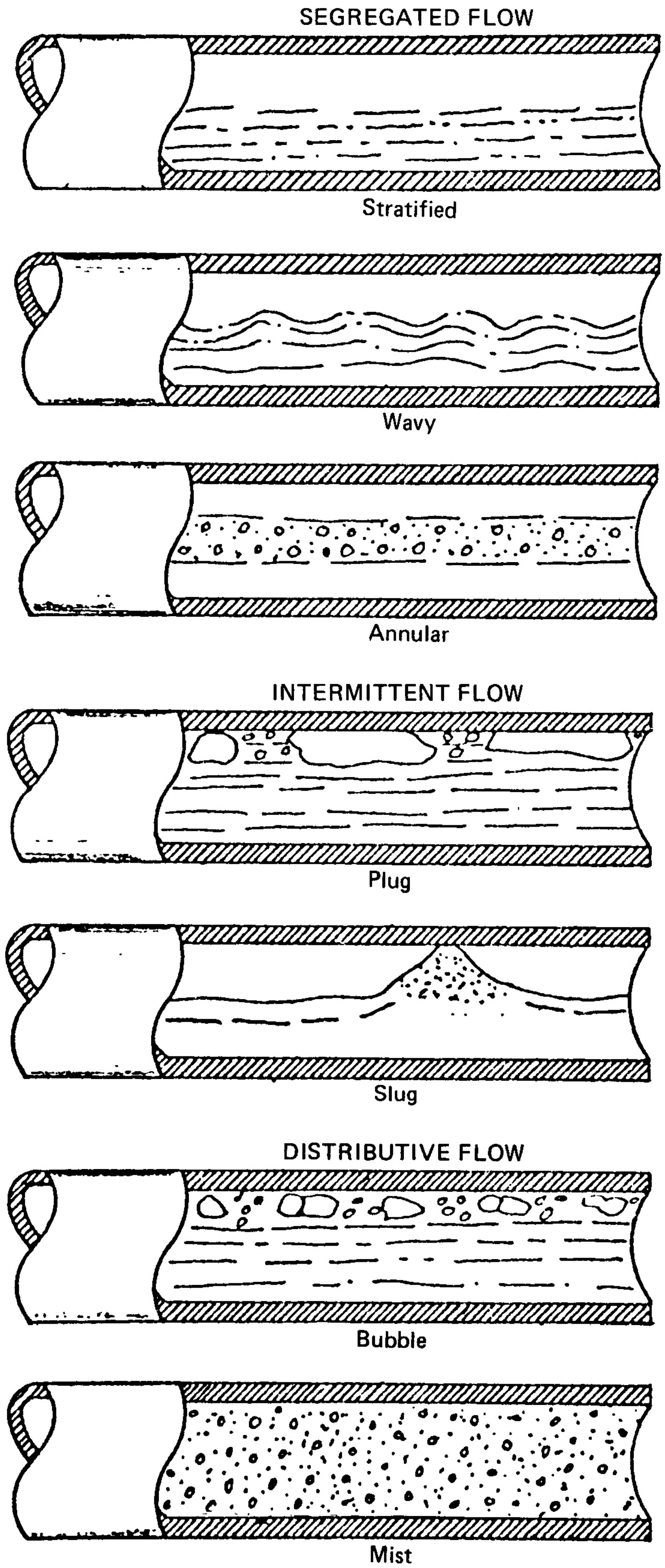


Figure A-1. Two-phase flow regimes in vacuum sewers.

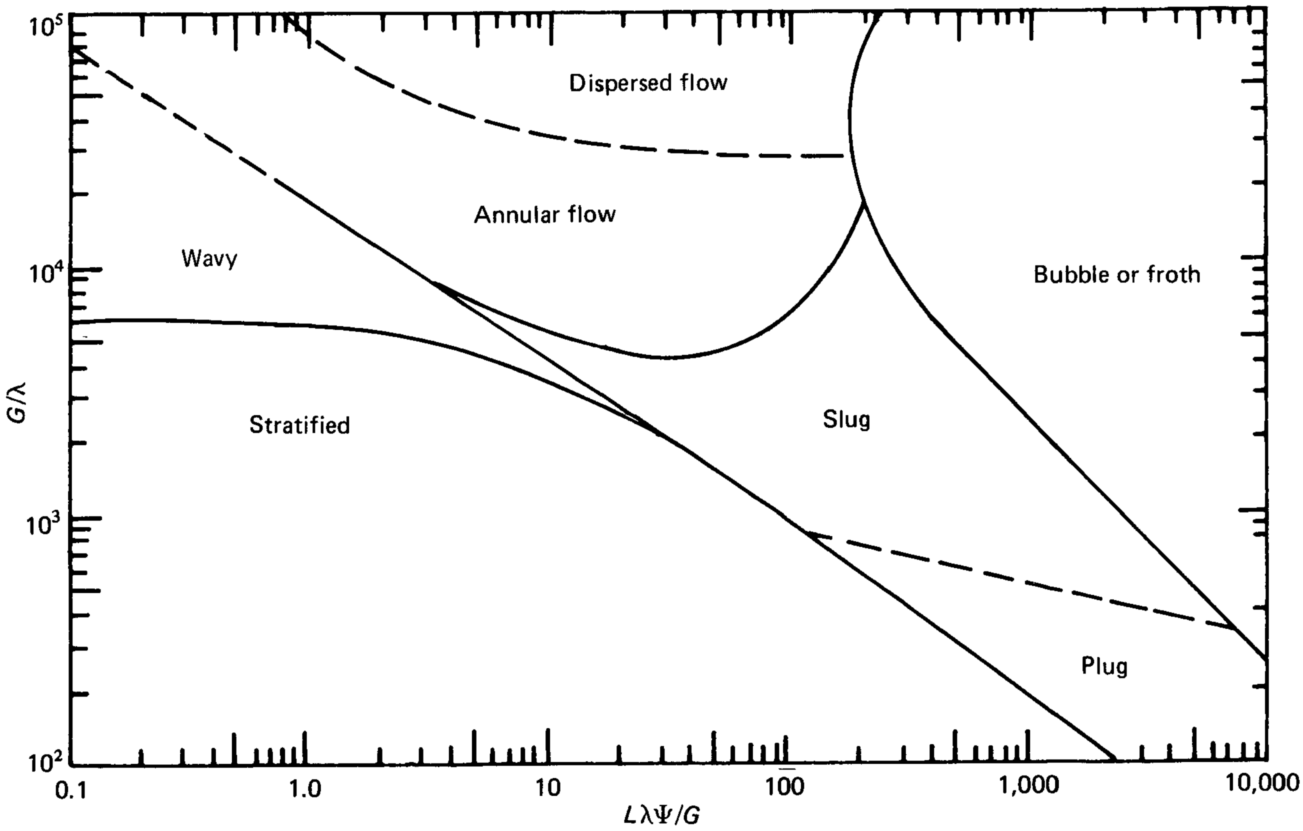


Figure A-2. Flow pattern region according to Baker.[1]

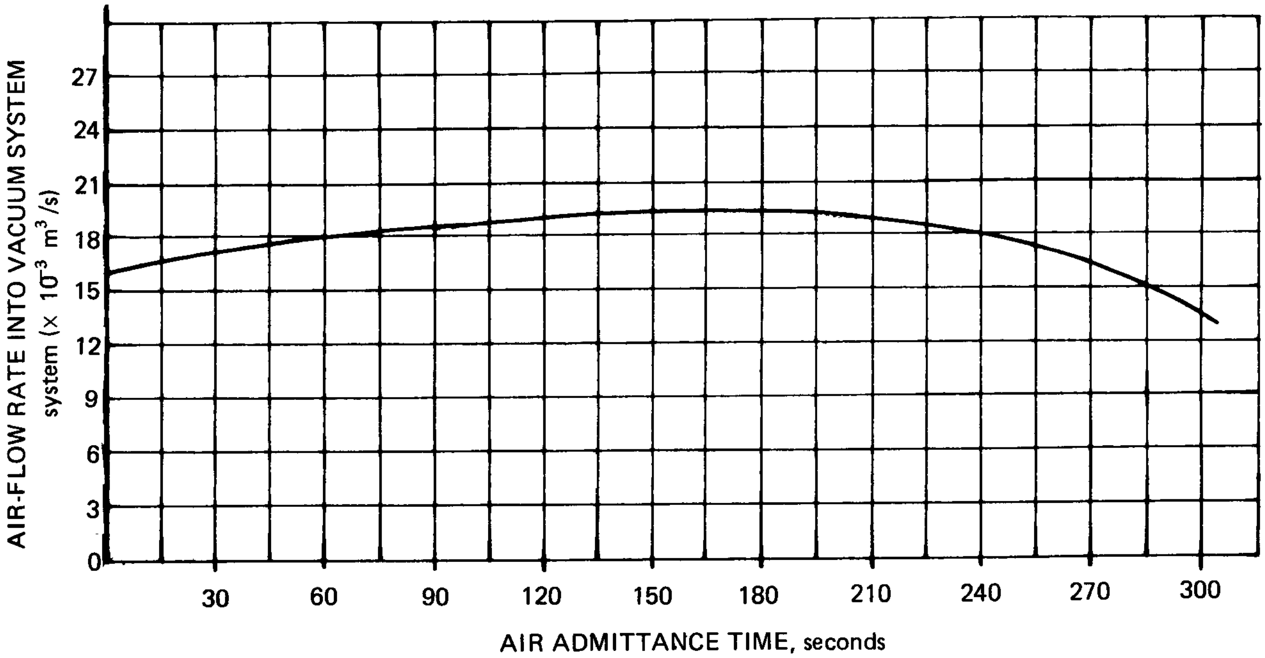


Figure A-3. Air-flow rate into vacuum sewer system after sewage discharge for various air admittance times.[4]

flowing faster than the liquid. The gaseous phase seems to slip by the liquid phase. The slip velocity is the difference in the phase velocities, or

$$V_s = V_l - V_g \tag{2}$$

The above analysis agrees with previous Navy research[6] that found liquid velocity in the range of 1.83-3.05 m/s (6-10 ft/s) and gas velocity of 9-15 m/s (30 ft/s). Solids, generally neglected in these analyses, flowed at 1.22 m/s (4 ft/s).

FRICTION HEADLOSS

Early vacuum sewers may have accounted for friction loss via an equalizing two-phase friction factor, ϕ_{tp}. Averill and Heinke[7] developed a ϕ_{tp} based on a homogeneous model, where

$$-(\partial \rho / \partial z)\, f_{tp} = \phi_{tp}[-(d\rho/dz) f_{\text{full pipe}}] \tag{3}$$

or the two-phase flow friction factor is some safety factor greater than unity times the full pipe friction factor.

AIRVAC

AIRVAC presents an even simpler model of headloss by multiplying full pipe flow by a derived 2.75 safety factor. This number is obtained from an assumed 2:1 air to liquid ratio in the pipe. The liquid, therefore, is flowing at three times the velocity as full pipe flow to deliver the same liquid flow rate. The Hazen-Williams formula raises the velocity to the 1.85 power, whereas the Darcy-Weisbach formula squares the velocity. The average of the velocities raised to the respective powers is 7.5 and 9. Since only one-third of the pipe diameter is said to be wetted, 7.5 + 9/2 x 1/3 = 2.75 is the average two-phase friction factor applied to the full pipe flow friction factor.

Mechanical Energy Balance

A more elegant and reliable analysis of friction headloss in vacuum systems was developed by Dukler,[8] using a similarity analysis verified by other researchers' experimental data.[2]

The general equation for pressure gradient in constant slip, two-phase flow is:

$$\partial \rho / \partial z = -[(\tau_f)_{cs} + \alpha \rho_{cs}(g/g_c)]/(1 - AC_{cs}) \tag{4}$$

The pressure gradient equation for dynamic headloss in vacuum sewers is a function of three distinct terms.

- Friction term, $(\tau_f)_{cs}$
- Inclined flow term, $\alpha \rho_{cs}(g_c/g)$ where α is valid from +10° to -10°
- Acceleration term, $(1 - AC_{cs})$

Friction. The friction term $(\tau_f)_{cs}$ is evaluated by:

$$(\tau_f)_{cs} = f_{cs} V_{ns}^2 \rho_{cs}/(2 g_c D) \tag{5}$$

with

$$\mathrm{Re}_{cs} = \rho_{cs} D V_{ns}/\mu_{ns} \tag{6}$$

$$f_o = \{2 \log [\mathrm{Re}_{cs}/(4.5223 \log \mathrm{Re}_{cs} - 3.8215)]\}^{-2} \tag{7}$$

$$\rho_{cs} = \rho_l(\lambda^2/R_l) + \rho_g(1-\lambda)^2/(1-R_l) \tag{8}$$

$$V_{cs} = V_{sl} + V_{sg} \tag{9}$$

$$\mu_{cs} = \mu_l\lambda + \mu_g(1-\lambda) \tag{10}$$

$$f_{cs} = \alpha(\lambda) f_o \tag{11}$$

$$\alpha(\lambda) = 1 - (\ln \lambda/\xi) \tag{12}$$

$$\xi = 1.281 + 0.4781 (\ln \lambda) + 0.444 (\ln \lambda)^2 + 0.094 (\ln \lambda)^3 + 0.00843 (\ln \lambda)^4 \tag{13}$$

The friction factor, f_{cs}, can be evaluated from the constant slip Reynolds number using the smooth tube friction factor ($C > 150$) and a standard Moody Diagram.

If alternative friction factors are desired, a correction can be applied as follows:

$$1/f_o^{1/2} = -2 \log [(\in/3.7D) + 2.51/(\mathrm{Re}_{cs} f_o^{1/2})] \tag{14}$$

The friction term is then a function of viscosity; density; Reynolds number; Euler number; flowing volume holdup, λ; and the in situ volumetric holdup, R_l.

The following volume holdup, λ, is the ratio of liquid volumetric flow rate to the total volumetric flow, or the ratio of the liquid superficial velocity to the total superficial velocity. The superficial velocity of either phase is calculated by assuming the pipe is occupied by only one phase and dividing that phase's flow by the pipe's cross-sectional area.

$$\lambda = Q_l/(Q_l + Q_g) = V_{sl}/(V_{sl} + V_{sg}) \tag{15}$$

The in situ volumetric holdup, R_l, is a more difficult concept to grasp. While in homogeneous distribution flow, $R_l = \lambda$, intermittent and segregated flow regimes, as seen in vacuum sewers, do not result in this equality and a separate estimate of R_l is necessary. The in situ volumetric holdup is a key variable in the analysis of two-phase flow and is termed R_l for liquid and R_g for gas (which equals $1 - R_l$). R_l is the fraction of a pipe element that is occupied by the liquid for some pipe length. R_l is then the average over both length and cross-section in slip flow, as opposed to λ varying only with cross-sectional area.

Hughmark[9] developed a holdup correlation, as presented by DeGance and Atherton,[2] by solving the following equation for R_l:

$$F = R_l - 1 + K(1-\lambda) = 0 \tag{16}$$

where K is a function of δ.

If

$$C_1 = 0.642\ V_{ns}{}^{0.5} G_t{}^{0.1667} D^{0.0417} / V_{sl}{}^{0.25} \tag{17}$$

then

$$\delta = C_1 / [R_l(\mu_l - \mu_g) + \mu_g]^{0.1667} \tag{18}$$

By taking the function derivative

$$\partial F/\partial R_l = 1 + (1 - \lambda)(\partial K/\partial \delta)(\partial \delta/\partial R_l) \tag{19}$$

$$\partial \delta/\partial R_l = C_1(\mu_l - \mu_g)/[R_l(\mu_l - \mu_g) + \mu_g]^{0.1667} \tag{20}$$

For $\delta < 10$:

$$K = -0.16367 + 0.31037\delta - 0.03525\delta^2 + 0.001366\delta^2 \tag{21}$$

and for $\delta > 10$:

$$K = 0.75545 + 0.00035.85\delta - (0.1436 \times 10^{-4})\delta^2 \tag{22}$$

For the derivative, if $\delta < 10$:

$$\partial K/\partial \delta = 0.31037 - 0.07050\delta + 0.00410\delta^2 \tag{23}$$

and for $\delta > 10$:

$$\partial K/\partial \delta = -0.003585 + (0.2872 \times 10^{-4})\delta \tag{24}$$

Because

$$\partial \delta/\partial R_l = C_1(\mu_l - \mu_g)/[R_l(\mu_l - \mu_g) + \mu_g]^{0.1667} \tag{25}$$

and

$$\partial F/\partial R_l = 1 + (1 - \lambda)(\partial K/\partial \delta)(\partial \delta/\partial R_l) \tag{26}$$

then

$$R_{l_{i+1}} = 1 - F(\partial F/\partial R_{l_i}) \tag{27}$$

Successive iterations are necessary to obtain a more accurate estimate of the holdup correlation, R_l. Successive iterations yielding R_{l_i} and $R_{l_{i+1}}$ to two significant figures are satisfactory.

Inclined Flow. The inclined flow term, $\alpha\rho_{cs}(g_c/g)$, relates the angle of incline and the constant-slip flow density. Little, if any, decrease in friction is experienced when dealing with vacuum sewers having minimum slopes laid in flat terrain. At slopes from 1° to 10°, progressively greater theoretical effects are experienced. At slopes exceeding 10° inclined or declined, the accuracy of this predictor diminishes rapidly. Because most lift in gravity sewers is installed at 45°, headloss here must be empirical and is counted solely as elevation loss as frictional distances are small.

Acceleration. The acceleration term, $1 - AC_{cs}$, is most evident during mist flow regimes, described by high values of R_e and low values of R_l. As can be seen from the design examples, the acceleration terms in the range of application to vacuum sewers is negligible.

The headloss from the acceleration term reflects changes in velocity because of slugs breaking down and falling to a lower elevation trap; and liquid level in the trap building up, finally reaching the crest of the pipe, and then being hurtled along as the differential pressure acts across a full face of liquid. The acceleration term loss is a calculation of a change in two-phase kinetic energy.

Depending on initial assumptions, two expressions for acceleration can be used. The first equation is the simpler of the two, yet equation 29 is generally thought to be more accurate based on the nature of holdup.

$$AC_{cs} = G_g{}^2/[\rho_g g_c \rho(1 - R_l)] \tag{28}$$

or

$$AC_{cs} = -|(G_l V_{sl}/R_l) + (G_g/V_{sg}/R_g)(1 - R_l/R_g)/g_c\rho \tag{29}$$

In design problems, both equations are usually found to have virtually identical effects on headloss.

NOMENCLATURE

AC	Acceleration term, defined by equations 28, 29
Ap	Cross-sectional area of conduit, ft^2
C_1	Parameter, equation 17
D	Inside diameter of conduit, ft
Eu_{tp}	Two-phase Euler number
f	Friction factor, one-phase flow
f_{cs}	Friction factor, two-phase constant slip
f_o	Friction factor, defined by equation 7
F	Parameter, equation 16
Fr	Froude number, $V_{ns}{}^2/g_c D$
f_{tp}	Friction factor, two-phase flow
g	Local acceleration, ft/(s)(s)
g_c	Gravitational constant, 32.174 (lb.) mass x (ft)/(lb) force (s)(s)
G_g	Gas mass flux, lb/(ft^2)(s)
G_l	Liquid mass flux, lb/(ft^2)(s)
G_t	Total superficial mass flux, lb/(ft^2)(s); (Wt/Ap)
i	Any given point in a conduit
K	Parameter: if $\delta < 10, K = 0.1637 - 0.31037\delta + 0.3525\delta^2 - 0.001366\delta^3$ if $\delta > 10, K = 0.75545 - 0.00358\delta + 0.1436 \times 10^{-4}\delta^2$ where $\delta = Re^{1/6}Fr^{1/8}/\lambda^{1/4}$
Q_l, Q_g	Volumetric flow rate of liquid or gas
Re_{tp}	Two-phase Reynolds number, $DG_t/[R_l\mu_l + (1 - R_l)\mu_g]$
R	Universal gas constant, 1,545 x (lb) force (ft)/(lb/mole)(°R)

R_g	Inplace gas holdup
R_l	Inplace liquid holdup
T	Temperature
t	Time
V	Volume
V_g	Velocity of gas, ft/s
V_l	Velocity of liquid, ft/s
V_s	Slip velocity, ft/s
V_{sl}	Superficial liquid velocity, ft/s
V_{sg}	Superficial gas velocity, ft/s
W_l	Liquid mass flow rate, lb/hr
W_g	Gas mass flow rate, lb/hr
W_t	Total mass flow rate, lb/hr
Z	Conduit length, ft
$\partial\rho/\partial z$	Pressure gradient (total pressure gradient), lb/(ft^3)/ft

Greek Letters

α	Conduit slope, Sine θ
$\alpha(\lambda)$	Defined in equation 12
δ	Parameter, equation 18
$\in$	Absolute roughness, equation 14
λ	Flowing volume-holdup of liquid
ξ	Parameter, defined in equation 13
μ	Viscosity (lb) (s)/ft^2
ρ	Density, lb/ft^3
ϕ	AIRVAC's two-phase friction factor
σ	Surface tension, dyn/cm^2
Ψ	Correlating parameter, defined in equation 1
τ_f	Partial derivative of pressure with respect to Z of frictional contributions

Subscripts

c	Cycle
cs	Constant slip
CT	Collection tank
DP	Discharge pump
f	Frictional
g	Gas
l	Liquid
ns	No slip
RT	Reserve tank
Sg	Superficial gas
S	Superficial liquid
t	Total
tp	Two phase
VP	Vacuum pump

REFERENCES

[1]O. Baker, "Experiences with Two-Phase Pipelines," presented at the joint meeting of the Canadian Natural Gas Processing Association and the Natural Gasoline Association of America, Calgary, Canada, Sept. 15, 1960.

[2] A. E. DeGance and R. W. Atherton, "Chemical Engineering Aspects of Two-Phase Flow," *Chem. Eng.*, 8-part series, 1970-71.

[3] J. W. Rezek and I. A. Cooper, "Preliminary Report on Vacuum Sewer System at Lake of the Woods, Va., Volume I: Hydraulic Field Test Program," report prepared for Lake of the Woods Service Company and Utilities, Inc., Rezek, Henry, Meisenheimer and Gende, Inc., Libertyville, Ill., Nov. 1974.

[4] C. R. Brockmeier, C. R. Brockmeier Consulting Engineers, Los Angeles, Calif., unpublished test data, 1975.

[5] E. P. Skillman, Civil Engineering Laboratory, Naval Construction Battalion Center, Port Hueneme, Calif., unpublished test data, 1975.

[6] E. P. Skillman, "Characteristics of Vacuum Wastewater Transfer Systems," presented at the American Society of Mechanical Engineers Conference on Environmental Systems, July 1976.

[7] D. W. Averil and G. W. Heinke, "Vacuum Sewer Systems," report prepared for the Northern Science Group of the Canadian Department of Indian Affairs and Northern Development, Jan. 1973.

[8] A. E. Dukler et al., "Frictional Pressure Drop in Two-Phase Flow: (A) Comparison of Existing Correlations for Pressure Loss and Holdup; (B) An Approach Through Similarity Analysis," *Am. Inst. Chem. Eng., J., 10*, 38-43, (A); 44-51, (B).

[9] G. A. Hughmark, "Holdup in Gas Liquid Flow," *Chem. Eng. Prog.*, *58*, 62-65, 1962.

Appendix B

DESIGN EXAMPLE

This design example is based on serving a small rural town located in a flat sandy area with consistently high ground water (fig. B-1). Lines are initially laid out to serve the town from a central location, one line east (line A) and the other west (line B). Line sizes are chosen based on serving the peak period. Two homes' gravity lateral will use one valve that will connect to the vacuum main. Traps are placed at a maximum of every 300 feet to reform slugs or where line elevation drops 1 foot below the invert after the previous trap. Because the trap drops 6 inches before being raised 1.5 feet, at a 45° angle, each trap requires 1.5 feet of lift to gain 1 foot of elevation. This assumption is conservative, based on Electrolux's[1] data indicating only 1/2-metre headloss is experienced by each 1 metre static lift. Their recovery of 1/2 metre may be caused by a partial mixing of sewage and air, lowering average density and, therefore, energy required to lift the liquid.

The lines are sloped from trap to trap, based on flowing 0.61 m/s (2 ft/s) at 0.7 full pipe flow. The velocity chosen allows for suspension of solids, while the 0.7 full pipe flow at maximum flow periods still allows sufficient void volume across the top of the liquid to allow for transfer of air. The transfer of air is the mechanism for reestablishing the local vacuum gradient along the pipeline.

After traps are laid out, the line is arbitrarily divided into segments. Headloss for each segment is calculated, first via the mechanical energy balance, then by AIRVAC's method as a checking procedure. Significant variations should be investigated.

Collection station design considerations for size of collection tank, reserve vacuum tank, vacuum pumps, and sewage force main pumps are presented. Formulas derived by AIRVAC[2] appear workable and are used.

BASE CONDITIONS

Line length = 1,000 ft (2 lines)
40 homes, 3.5 persons per home, 75 gallons per capita per day
Design at peak flow, four times average
Divide line into four segments
Assumes entire system under 1/2 atm vacuum (15 inches)

Segment	Length	Homes	Maximum Q	Pipe size	Slope
A	250	40	29	4 inches	0.0030
B	250	30	22	3 inches	.0055
C	250	20	15	3 inches	.0055
D	250	10	8	3 inches	.0055

PIPING PROFILE

Traps are every 300 feet or drop of 1-foot elevation in level terrain. Starting at end of Segment D (10 + 00):

Trap 1 at station 10 + 00 = 1 ft/0.0055 ft/ft = station 8 + 18

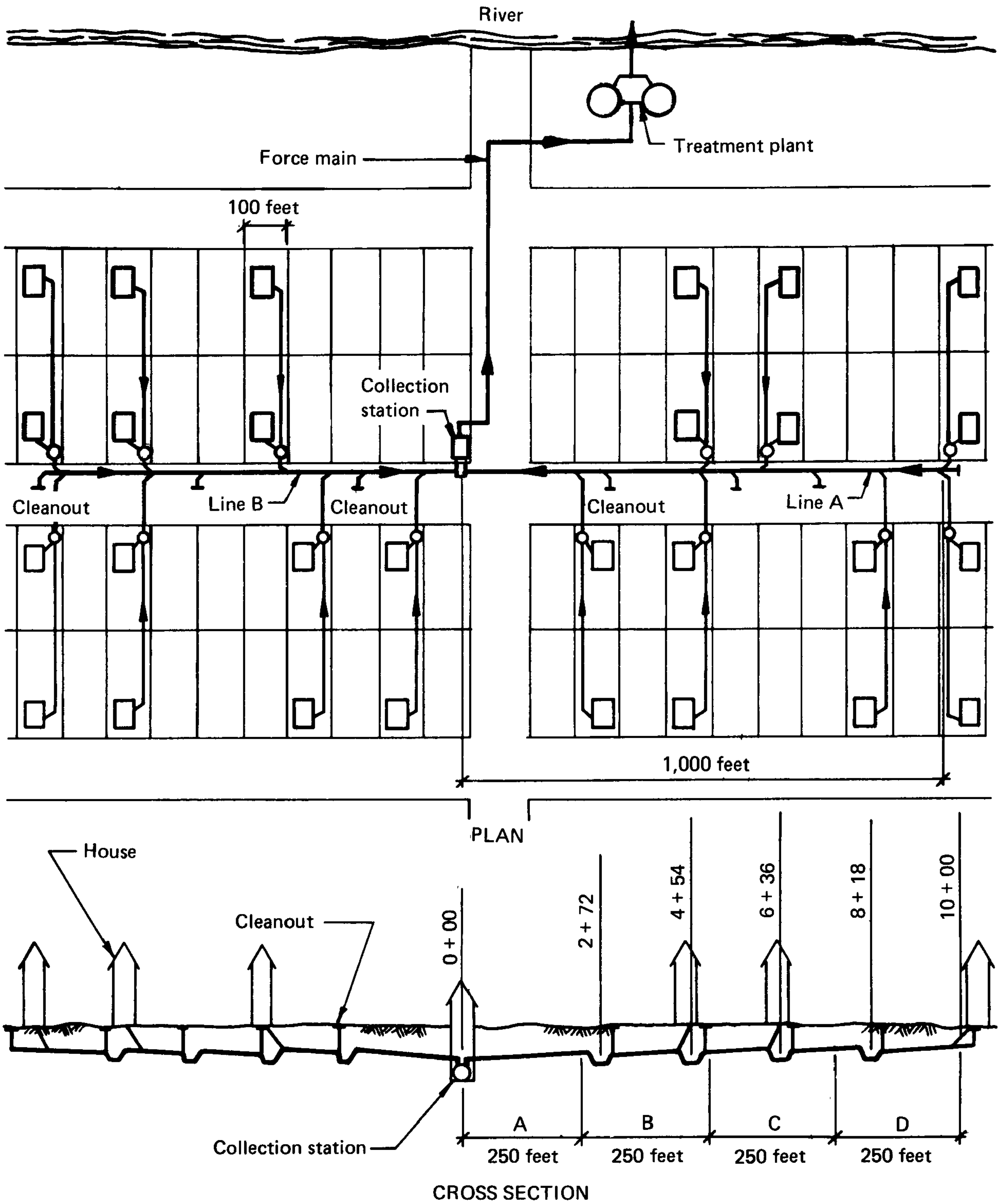


Figure B-1. Vacuum sewer system design example.

Trap 2 at station 8 + 18 = 1 ft/0.0055 ft/ft = station 6 + 36

Trap 3 at station 6 + 36 = 1 ft/0.0055 ft/ft = station 4 + 54

Trap 4 at station 4 + 54 = 1 ft/0.0055 ft/ft = station 2 + 72

Trap 5 at (slope changes at 2 + 50 to 0.0030) drop in (2 + 72 - 2 + 50) = 22 ft; 0.0055 × 22 = 0.12; 1 - 0.12 = 0.88

$$2 + 50 - 0.88/0.0030 = 250 - 293.3 = 43 \text{ feet}$$

Therefore, trap 4 is last trap before collection tank.

Elevation loss per trap = 1.5 ft

4 traps at 1.5 ft	= 6.0
Final trap lifts 1.0 ft	= 1.0
Total static h_{LS}	= 7.0 ft

FLOW CONDITIONS

Parameter	A	B	C	D
D, ft	0.33	0.25	0.25	0.25
Ap, ft^2	.0873	.0491	.0491	.0491
T, °F	68.0	68.0	68.0	68.0
P, lb/ft^2	1,058.4	1,058.4	1,058.4	1,058.4
α, sin θ	0.003	.0055	.0055	.0055
Q_l, gal/min	29.2	21.9	14.6	7.3
Q_g, gal/min	291.7	218.8	145.8	72.9
V_{sl}, ft/s	.76	.99	.66	.33
V_{sg}, ft/s	7.61	9.94	6.63	3.31
V_{ns}	8.37	10.93	7.29	3.64
ρ_l, lb/ft^3	62.4	62.4	62.4	62.4
ρ_g, lb/ft^3	.0752	.0752	.0752	.0752
μ_l, cp	1.008	1.008	1.008	1.008
μ_g, cp	.01794	.01794	.01794	.01794
W_l, lb/hr	14,612	10,959	7,306	3,653
W_g, lb/hr	176.0	132.0	88.0	44.0
λ	.0907	.0907	.0907	.0907
G_t, lb/ft^2/s	48.04	62.75	41.83	20.92
G_g, lb/ft^2/s	.57	.75	.50	.25
G_l, lb/ft^2/s	47.47	62.00	41.33	20.67

Segment A

R_l (in place holdup):

Assume $R_l = 1.0$, then

$$C_1 = 0.642\, V_{ns}^{0.5} G_t^{0.1667} D^{0.0417} / V_{sl}^{0.25}$$

$$C_1 = 3.6219$$

$$\delta = C_1/[R_l(\mu_l - \mu_g) + \mu_g]^{0.1667} = 3.6171$$

δ	< 10
K	$= -0.16367 + 0.31037\delta - 0.03525\delta^2 + 0.001366\delta^3$
K	$= 0.5624$
$\partial K/\partial\delta$	$= 0.31037 - 0.07050\delta + 0.0041\delta^2 = 0.1090$
F	$= R_l - 1 + K(1-\lambda) = 1 - 1 + 0.5624\ (1 - 0.0907) = 0.5114$
$\partial\delta/\partial R_l$	$= C_1(\mu_l - \mu_g)/[R_l(\mu_l - \mu_g) + \mu_g]^{0.1667} = 3.374$
$\partial F/\partial R_l$	$= 1 + (1-\lambda)(\partial K/\partial\delta)(\partial\delta/\partial R_l) = 1.3344$
$R_{l_{i+1}}$	$= 1 - F/(\partial F/\partial R_{l_i}) = 1 - 0.5114/(1.3344) = 0.6168$

Successive iterations of the above process are repeated using R_{i+i} until R_{l_i} and $R_{l_{i+1}}$ are sufficiently close at two significant figures.

Iteration	R_l	F	$\partial F/\partial R_l$
1	1.0	0.5114	1.3344
2	.6168	.1559	1.3428
3	.8339	.4025	1.2053
4	.6916	.2244	1.3492
5	.8337	.3557	1.3509
6	.7367	.2650	1.3478
7	.8028	.3270	1.3500
8	.7578	.2853	1.3486
9	.7884	.3136	1.3494
10	.7676	.2944	1.3488
11	.7817		

Iterations 10 and 11 are sufficiently close to use a two-significant figure R_l correlation number of 0.77.

The dynamic pressure loss for segment A can now be calculated as follows:

$$(\partial\rho/\partial z)f_{tp} = -[(\tau_f)_{cs} + \alpha\rho_{cs}(g/g_c)]/(1 - AC_{cs})$$

where:

$\rho_{cs} = \rho_l(\lambda^2/R_l) + \rho_g(1-\lambda)^2/(1-R_l)$

$\rho_{cs} = 62.4\ (0.0907^2/0.77) + 0.0752\ (1 - 0.0907)^2/(1 - 0.77)$

$\rho_{cs} = 0.9370$

$\mu_{ns} = \mu_{cs} = \mu_l\lambda + \mu_g\ (1-\lambda) = 1.008\ (0.0907) + 0.0752\ (1 - 0.0907) = \mu_{cs} = 0.1598$

$Re_{cs} = \rho_{cs} D V_{ns}/\mu_{ns}$

$Re_{cs} = 1{,}488\ (0.9370)(0.33)(8.37)/(0.1590) = 2.41 \times 10^4$

$$f_o = \{2 \log [\mathrm{Re}_{cs}/(4.5223 \log \mathrm{Re}_{cs} - 3.8215)]\}^{-2}$$
$$= \{2 \log [2.4 \times 10^4/(4.5223 \log 2.41 \times 10^4 - 3.8215)]\}^{-2} = 0.0248$$

$$\alpha(\lambda) = 1 - \{\ln \lambda/\xi\}$$

$$\xi = 1.281 + 0.4781 (\ln \lambda) + 0.444 (\ln \lambda)^2 + 0.094 (\ln \lambda)^3 + 0.00843 (\ln \lambda)^4$$

$$\xi = 1.6713$$

$$\alpha(\lambda) = 1 - \ln 0.0907/1.6713 = 2.4361$$

$$f_{cs} = \alpha(\lambda) f_o = 2.4361(0.0248) = 0.0603$$

$$(\tau_f)_{cs} = f_{cs} V_{ns}^2 \rho_{cs}/2 g_c D$$
$$= (0.0603)(8.37)^2 (0.9370)/2(32.174)(0.33) = 0.1864$$

$$\mathrm{AC}_{cs} = [(G_l V_s/R_l) + (G_g V_{sg}/R_g)(1 - R_l/R_g)]/g_c \rho$$

$$\mathrm{AC}_{cs} = (47.47)0.76/0.77 + (0.57)(7.61)/(0.23)(1 - 0.77/0.23)/(32.174 \times 10584)$$

$$\mathrm{AC}_{cs} = 0.000076$$

However, in this range a more reflective acceleration term would be:

$$\mathrm{AC}_{cs} = G_g^2/\rho_g g_c \mathrm{P} R_g$$

$$\mathrm{AC}_{cs} = (0.57)^2/[(0.0752)(32.174)(1058.4)(1 - 0.77)]$$

$$\mathrm{AC}_{cs} = 0.00016$$

Then,

$$(\partial \rho/\partial z) = -[0.1864 - 0.003 (0.9730)]/(1 - 0.00016)$$

$$(\partial \rho/\partial z) = -0.1835 \text{ lb/ft}^2\text{/ft} = -2.94 \text{ ft/1,000 ft}$$

Segment B

$$C_1 = 0.642(10.93)^{0.5} (62.75)^{0.1667} (0.25)^{0.0417}/(0.99)^{0.25}$$

$$C_1 = 4.004$$

$$\delta = 3.9933$$

$$K = 0.6006$$

$$\partial K/\partial \delta = 0.0942$$

$$F = 0.5461$$

$$\partial \delta/\partial R_l = 3.9537$$

$\partial F/\partial R_l$ = 1.3387

$R_{l_{i+1}}$ = 0.59, after iterations, use R_l = 0.75

ρ_{cs} = 0.9332

μ_{cs} = 0.1598

Re_{cs} = 2.31×10^4

f_o = 0.0250

ξ = 1.6713

$\alpha(\lambda)$ = 2.4361

f_{cs} = 0.0607

$(\tau_f)_{cs}$ = $0.0609(10.93)^2\ (0.9332)/2(32.174)(0.25) = 0.4220$

AC_{cs} = $G_g{}^2/\rho_g g_c \rho R_g = (0.75)^2/(0.752)(32.174)(1058.4)(0.25) = 0.0009$

$\partial \rho/\partial z$ = - 0.4172 lb/ft²/ft = - 6.69 ft/1,000 ft

Segment C

C_1 = 3.3823

δ = 3.3778

K = 0.5352

$\partial K/\partial \delta$ = 0.1190

F = 0.4867

$\partial \delta/\partial R_l$ = 3.3442

$\partial F/\partial R_l$ = 1.3619

$R_{l_{i+1}}$ = 0.64, after iterations, use 0.78

ρ_{cs} = 0.7748

μ_{cs} = 0.1598

Re_{cs} = 1.46×10^4

f_o = 0.0280

ξ = 1.6713

$\alpha(\lambda)$ = 2.4361

f_{cs} = 0.0682

$(\tau_f)_{cs}$ = 0.2196

AC_{cs} = 9.9×10^{-5}

$\partial \rho / \partial z$ = - 0.2142 $lb/ft^2/ft$ = - 3.44 ft/1,000 ft

Segment D

C_1 = $0.642(3.64)^{0.5}(20.92)^{0.1667}(0.25)^{0.0417}/(0.33)^{0.25}$

C_1 = 2.5321

δ = 2.5288

K = 0.4179

$\partial K / \partial \delta$ = 0.1583

F = 0.3800

$\partial \delta / \partial R_l$ = 2.5036

$\partial F / \partial R_l$ = 1.3604

$R_{l_{i+1}}$ = 0.7207, after iterations use 0.83

ρ_{cs} = 0.9842

μ_{cs} = 0.1598

Re_{cs} = 8.33×10^3

f_o = 0.0324

ξ = 1.6713

$\alpha(\lambda)$ = 2.4361

f_{cs} = 0.789

$(\tau_f)_{cs}$ = 0.0640

AC_{cs} = 0.0003

$\partial \rho / \partial z$ = 0.0586 $lb/ft^2/ft$ = 0.94 ft/1,000 ft

DYNAMIC HEADLOSS (h_{LD})

By AIRVAC calculation:

Segment	Maximum Q	Pipe size	Pipe h_{LD}		Two-phase factor		Two-phase h_{LD}, ft/1,000 ft
A	29.2	4-in	0.62	x	2.75	=	1.71
B	21.9	3-in	1.86	x	2.75	=	5.12
C	13.6	3-in	1.00	x	2.75	=	2.75
D	7.3	3-in	0.39	x	2.75	=	1.07

By mechanical energy balance:

Segment	h_{LD}, 1,000 ft 1,000 ft[a]
A	2.94 x 0.280 = 0.82
B	6.69 x 0.310 = 2.05
C	3.44 x 0.280 = .96
D	0.94 x 0.280 = .26
	Total Dynamic h_{LD} 4.09 ft

[a]250 ft per segment + 30 ft for each trap.

By AIRVAC (h_{LD} = 2.75 x full pipe h_{LD}):

Segment	h_{LD}, 1,000 ft 1,000 ft[a]
A	1.71 x 0.280 = 0.48
B	5.12 x 0.310 = 1.59
C	2.75 x 0.280 = 0.77
D	1.07 x 0.280 = 0.30
	Total dynamic h_{LD} 3.14 ft

[a]250 ft per segment + 30 ft for each trap.

Total Headloss (h_{LT})

$$\text{Total } h_{LT} = \underset{\text{(static)}}{h_{LS}} - \underset{\text{(dynamic)}}{h_{LD}} + \underset{\text{(valve)}}{h_{LV}}, \text{ not to exceed 18 ft} \qquad (30)$$

(h_{LV} = 5 ft reserved for valve operation)

Mechanical Energy Balance

$$h_{LT} = 7.0 + 4.09 + 5.0 = 16.09 \text{ ft}$$

$$\text{AIRVAC } h_{LT} = 7.0 + 3.14 + 5.0 = 15.14 \text{ ft}$$

Therefore, the vacuum piping system is functional because both the AIRVAC and mechanical energy balance equations yield a headloss of less than 18.0 feet.

DISCHARGE PUMP

Discharge pumps shall be sized to handle 120 percent of the design peak of the maximum sewage flow with the largest pump out of service and with a minimum size of 80 gal/min each. Assume each pump would be the same size. Treatment plant sizing should reflect discharge pump sizing requirements.

$$Q_{DP} = 1.2 \times Q_{max} = 1.2 \times 29.1 = 69.8 \qquad (31)$$

Thus, 69.8 gal/min for two pumps would require that each pump be sized at 80 gal/min each.

AIRVAC suggests 25 feet of head should be added to the design point to account for collection tank vacuum. Sufficient net positive suction head must also be available, as described earlier.

Vacuum Pump Capacity

Vacuum pump sizing allows for withdrawal of the volume of air in the mains, based on peak flow conditions with a 100 percent safety factor with the largest unit out of service. An allowance for valve sensor leakage in the AIRVAC system of 0.25 f^3/min per valve must be added.

Vacuum pump sizing should also consider the length of pump running time. Research in this area addressed by Skillman[3] showed optimum performance is affected by both vacuum pump size and vacuum reserve. A compromise should be reached between a large-sized pump cycling frequently and a minimum-sized pump that may not be capable of maintaining a satisfactory system vacuum during high-flow periods. A reasonable design equation is presented below:

$$Q_{VP} = 2(Q_l + Q_g) \times 1\ ft^3/7.48\ gal + 0.25 \times \text{No. valves} \qquad (32)$$

Because two lines (lines A and B) enter the station, the sum of the air flows must be considered:

$$Q_{VP} = 2 \times 2(291.7 + 29.2) \times 1/7.48 + 0.25 \times 40 = 181.6\ ft^3/min$$

Because 181.6 ft^3/min is required with the largest pump out of service, use three pumps rated at 91 ft^3/min at 45.7 cm (18 inches mercury). Pump curves should be analyzed for shutoff vacuum and free-flow conditions.

Collection Tank Volume

While most pump station design manuals suggest the minimum time between cycles should be as low as 10 minutes,[4] AIRVAC suggests 30 minutes at half the average daily flow.

The minimum time between cycles, or 30 minutes, is the sum of the filling time plus the pumping time, or

$$\text{cycle time} = \text{filling time} + \text{pumping time} \qquad (33)$$

Because the collection tank's operating volume is a maximum of 65 percent of the total collection tank volume, the following calculation shows the required size, with a minimum of 400 gallons:

$$V_{CT} = tc/[0.65(1/Q_{min} + 1/Q_{DP} - Q_{min})] \qquad (34)$$

$$V_{CT} = 30/[0.65(1/7.3 - 1/80 - 7.3)] = 307.6 \text{ gal}$$

Therefore, use a 400-gallon tank.

Reserve Tank Volume

AIRVAC calculates reserve tank volume from two equations. Their first equation relates a total volume necessary to bring the vacuum up from 16 to 20 inches in t minutes, with t usually 1 or 1.5, and the previously calculated vacuum pump capacity, Q_{VP}:

$$V_t = 3Q_{VP} \times t \qquad (35)$$

or

$$V_t = 3 \times 181.6 \times 1 = 544.80$$

The second equation determines the reserve tank volume, with a minimum size of 400 gallons. For this equation it is assumed that one-third of the piping is occupied by the gaseous phase:

$$V_{RT} = V_t - 1/3\ V \text{ piping} - 0.35(V_{CT}) \qquad (36)$$

$$V_{RT} = 544.80 - 1/3(877.4) - 0.35(400) = 112.3 \text{ gal}$$

The minimum-sized tank recommended is 400 gallons; the size of the reserve tank in this example, therefore, is 400 gallons.

Auxiliary Power

In order to size standby generator sets, an analysis of the continuity of electrical service should be undertaken. If outage times are significant and frequent, full load power should be recommended. If infrequent, short outages occur, generator set sizing should be sized to operate one vacuum pump and one discharge pump. If local standards dictate more conservative requirements, they should be followed.

REFERENCES

[1]"Electrolux Vacu-Flow System for Nash Sewerage," report prepared for the Borough of Nash, England, by Electrolux Corporation, Stockholm, Sweden, Feb. 1976.

[2]*Design Criteria Manual*, AIRVAC, The Vacuum Sewer Systems, Rochester, Ind., May 1976.

[3]E. P. Skillman, "Characteristics of Vacuum Wastewater Transfer Systems," presented at the American Society of Mechanical Engineers Conference on Environmental Systems, July 1976.

[4]"Lift Stations Engineering Manual," Clow Corporation, Waste Treatment Division, Florence, Ky., undated.

METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Length	meter	m	*Basic SI unit*	39 37 m = 3 281 ft = 1 094 yd
	kilometer	km		0 6214 mi
	millimeter	mm		0 03937 in
	micrometer or micron	μm or μ		3 937 X 10^{5} in = 1 X 10^{4} Å
Area	square meter	m^2		10 76 sq ft = 1 196 sq yd
	square kilometer	km^2		0 3861 sq mi = 247 1 acres
	square millimeter	mm^2		0 001550 sq in
	hectare	ha	The hectare (10,000 m^2) is a recognized multiple unit and will remain in international use	2 471 acres
Volume	cubic meter	m^3		35 31 cu ft = 1 308 cu yd
	litre	l		1 057 qt = 0 2642 gal = 0 8107 X 10^{-4} acre ft
Mass	kilogram	kg	*Basic SI unit*	2 205 lb
	gram	g		0 03527 oz = 15 43 gr
	milligram	mg		0 01543 gr
	tonne	t	1 tonne = 1,000 kg	0 9842 ton (long) = 1 102 ton (short)
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 in a mass of 1 kg	0 2248 lb = 7 233 poundals
Moment or torque	newton meter	N m	The meter is measured perpendicular to the line of action of the force N Not a joule	0 7375 lb ft 23 73 poundal ft
Flow (volumetric)	cubic meter per second	m^3/s		15 850 gpm = 2,119 cfm
	liter per second	l/s		15 85 gpm

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Velocity linear	meter per second	m/s		3 281 fps
	millimeter per second	mm/s		0 003281 fps
	kilometers per second	km/s		2,237 mph
angular	radians per second	rad/s		9 549 rpm
Viscosity	pascal second	Pa s		0 6722 poundal(s)/sq ft
	centipoise	Z		1 450 X 10^{7} Reyn (μ)
Pressure or stress	newton per square meter or pascal	N/m^2 or Pa		0 0001450 lb/sq in
	kilonewton per square meter or kilopascal	kN/m^2 or kPa		0 14507 lb/sq in
	bar	bar		14 50 lb/sq in
Temperature	Celsius (centigrade)	°C		(°F−32)/1 8
	Kelvin (abs)	°K		°C + 273 2
Work, energy, quantity of heat	joule	J	1 joule = 1 N m where meters are measured along the line of action of force N.	2 778 X 10^{7} kw hr = 3 725 X 10^{7} hp hr = 0 7376 ft-lb = 9 478 X 10^{-4} Btu
	kilojoule	kJ		2 778 X 10^{-4} kw hr
Power	watt	W	1 watt = 1 J/s	44 25 ft lbs/min
	kilowatt	kW		1 341 hp
	joule per second	J/s		3 412 Btu/hr

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Precipitation, run-off, evaporation	millimeter	mm	For meteorological purposes, it may be convenient to measure precipitation in terms of mass/unit area (kg/m^2) 1 mm of rain = 1 kg/m^2	
Flow	cubic meter per second	m^3/s		35 31 cfs
	liter per second	l/s		15 85 gpm
Discharges or abstractions, yields	cubic meter per day	m^3/d	1 l/s = 86 4 m^3/d	0 1835 gpm
	cubic meter per year	m^3/year		264 2 gal/year
Usage of water	liter per person per day	l/person/day		0 2642 gcpd

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Density	kilogram per cubic meter	kg/m^3	The density of water under standard conditions is 1,000 kg/m^3 or 1,000 g/l or 1 g/ml	0 06242 lb/cu ft
Concentration	milligram per liter (water)	mg/l		1 ppm
BOD loading	kilogram per cubic meter per day	$kg/m^3/d$		0 06242 lb/cu ft/day
Hydraulic load per unit area, e g., filtration rates	cubic meter per square meter per day	$m^3/m^2/d$	If this is converted to a velocity, it should be expressed in mm/s (1mm/s = 86 4 m^3/m^2/day)	3 281 cu ft/sq ft/day
Air supply	cubic meter or liter of free air per second	m^3/s l/s		
Optical units	lumen per square meter	lumen/m^2		0 09294 ft candle/sq ft

*Miles are U S statute, qt and gal are U S liquid, and oz and lb are avoirdupois

☆U S GOVERNMENT PRINTING OFFICE 1977— 757-140/6603