Design of Small-Diameter Variable-Grade Gravity Sewers
ABSTRACT


An Alabama community uses modified septic tanks to produce clarified effluent that is transported in small plastic lines to a lagoon. Variable-grade gravity sewers as small as 2 inches in diameter transport effluent almost 1,000 feet without problems. The sewers were installed following the profile of the land, without manholes and for less than $2 per linear foot. Cost per family for the complete system (tanks, sewers, and lagoon) was comparable to the cost of conventional septic tank/drainfield systems. This report describes a successful application of the concept of variable-grade gravity sewers and provides engineering guidance for the design and installation of similar systems.

KEYWORDS: VGS, PVC pipe, STEP, septic-tank-effluent-pumping system, sewage-disposal system, variable-grade gravity sewer, Mt. Andrew system

CONTENTS

Introduction.................... 1
The VGS concept.................. 1
Description of system used at Mt. Andrew, Ala............... 1
Operational characteristics of the Mt. Andrew system.... 3
Calculations to lower the hydraulic gradient................. 11
Summary.......................... 12
References......................... 13

Design examples.................... 7
Example 1......................... 7
Example 2......................... 9
Recommended design consider-
ations for VGS systems........... 4


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DESIGN OF SMALL-DIAMETER, VARIABLE-GRADE GRAVITY SEWERS

by John D.* Simmons and Jerry O. Newman1

INTRODUCTION

Low-pressure, septic-tank-effluent pumping (STEP) systems have been in operation for years (Bowne 1977, Goldstein and Moberg 1973, Office of Appropriate Technology 1977). These systems have an economic advantage over large-diameter gravity sewers because small plastic pipe can be laid independent of grade at a fraction of the cost of conventional sewers on uniform grade. The cost of STEP systems, however, could be reduced if the expense of the pump and wet well following each septic tank were eliminated. Another consideration may be increased reliability by eliminating electric pumps and float switches. In some situations, septic tank effluent can be collected and transported using small-diameter, variable-grade gravity sewer (VGS) systems.

The U.S. Department of Agriculture, Agricultural Research Service's Rural Housing Research Unit (RHRU), Tuskegee Institute, and the Farmers Home Administration (FmHA) worked cooperatively on a project in Alabama to develop and test an innovative sewage-disposal system. Using concepts from the FmHA, the RHRU designed and installed a rural community sewage system at Mt. Andrew, Ala. The subdivision (which was sponsored by Tuskegee Institute) contained 31 houses financed by the FmHA. The project was initiated to test several basic theories, but the most intriguing was the possibility of using a small-diameter, variable-grade gravity system in conjunction with slightly modified septic tanks. This publication explains the VGS concept, describes a successful application of the concept, and provides engineering guidance for similar installations.

THE VGS CONCEPT

A variable-grade gravity sewer operates on the principle of a sink trap. Imagine a series of sink traps stretched out over a long distance. If there is positive net fall from inlet to outlet, any amount of liquid put in the upper end will eventually reach the lower end. The VGS line is laid at relatively constant depth regardless of grade and, therefore, has a profile showing many "uphill" and "downhill" sections. Overall, the outlet is lower than the inlet, and in fact, the outlet is lower than any house served by the sewer. The draining process will involve delays, surcharging, and many transitions from full to partial pipe flow. After initial filling of the low sections, however, all the effluent put in the inlet will reach the outlet. It is easy to see that some sections will remain full at all times.

DESCRIPTION OF SYSTEM USED AT MT. ANDREW, ALA.

The system consists of modified septic tanks, small-diameter PVC transport lines, and a lagoon for final treatment (fig. 1). Sewage is liquefied in modified septic tanks called interceptor tanks (Rose 1971). The interceptor tanks (fig. 2) have two compartments, an initial primary treatment compartment, followed by liquid storage. Six 2-in.-PVC "clarifier tubes" are installed between the two compartments. The parallel tubes are installed in the separating wall

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It was intended that a low-velocity upward flow through the tubes would permit solids to settle from the liquid and slide back into the primary treatment chamber. The tubes were also intended to keep any scum and objects of near neutral buoyancy from getting into the liquid storage compartment of the tank. Effluent from the tanks is transported to a lagoon by (1) a 3-in.-PVC-gravity line, (2) a 2-in.-PVC-gravity line, or (3) a 3-in.-PVC-pressure/gravity line. Of the 31 houses in the subdivision, 13 use the 3-in.-gravity line and 10 the 2-in.-gravity line. The interceptor outlets of the remaining 8 houses are located below the level of the sewer and effluent is pumped to the 3-in.-pressure/gravity line. If several pumps are running simultaneously, this line may be a pressure line to the lagoon. The lines were laid along the existing grade, independent of the elevation, and cost about $2 per linear foot. The system does not include manholes or cleanouts as they are not needed. The grades of the lines are variable, but the houses and the interceptor tanks are all located above the lagoon. The 2-in. gravity sewer is of particular interest (fig. 3). This sewer is 970 feet long and its...
grade changes from positive to negative slope at several points before reaching the lagoon. It is one of the first small-diameter, variable-grade gravity sewers in the United States.

The interceptor tanks draining into this sewer have a unique type of outlet consisting of vertical standpipes 12 inches from the bottom (fig. 4). This configuration allows for surge storage above the outlet while providing a vent for the sewer, thereby promoting proper drainage from the tank. A depth of 12 inches is not necessary; we recommended that the unusable storage depth be kept to a minimum since any standing water takes away from the volume of surge storage. Suggestions for improvements on interceptor tank design are discussed further in the text.

Figure 4.—Interceptor tank used with the 2-in. sewer.

OPERATIONAL CHARACTERISTICS OF THE MT. ANDREW SYSTEM

After several years of operation, this system has given good service with little maintenance. The interceptor tanks functioned without major problems, although sludge accumulated quickly because of the small volume of the primary treatment compartment. Several tanks were subjected to abnormal loading due to rather large families occupying the small houses. As a result, several tanks required pumping after being in service a little more than a year. In the first 4 years of operation, two pumps failed. One pump failed because of a sticking float switch and the other was unexplained.

After the system was used for 18 months, the RHRU initiated an experiment that required sampling from the tanks, lines, and lagoon. Before the sampling began, the tanks were pumped and about one-third of them were inspected. There was no evidence of clogging of the clarifier tubes and no appreciable accumulation of sludge in the liquid-storage section.

Sampling revealed that the interceptor did not provide superior treatment for suspended solids and BOD when compared to a conventional septic tank. The average concentrations of suspended solids discharged from the interceptor tanks were 102 ppm and concentrations of BOD were 229 ppm. Findings of a water/sewage use survey conducted in the subdivision, however, showed that most of the tanks were heavily loaded. The septic tanks were sized according to recommendations for two-bedroom houses in Alabama. Because of compartmentation, only 508 gallons of usable volume were left in the primary treatment compartment. Large families occupying these small houses caused the tanks to receive almost twice the wastewater as would be expected from two-bedroom rural houses. Samples taken from properly loaded tanks had 68 ppm average concentrations of suspended solids and 206 ppm of BOD. Samples taken from heavily loaded tanks averaged 104 ppm concentrations of suspended solids and 356 ppm of BOD. While it may have been desirable to have an effluent quality superior to that of a conventional septic tank, it is not necessary. This is because the VGS lines can transport effluent with solids concentrations.
in the range normally discharged from septic tanks. However (from sampling effluent lines), it was determined that all floating matter (sludge, scum, and large particles that may cause problems in the small plastic lines) was removed from the effluent.

The transport lines have served the community without problems, and no tank has failed to drain because of trouble with these lines. After 18 months of operation, the 2-in. line was unearthed at several locations and inspected. At low points, sections of the pipe were removed and inspected for solids buildup. The line was coated on the inside with a thin, grayish residue, but no heavy solids had accumulated. This finding is especially significant as the quality of the effluent was no better than that of the average septic tank. In fact, quite often the effluent from the overloaded septic tanks had higher concentrations of suspended solids than values reported as being normally discharged from septic tanks (Bond, Straub, and Prober 1974 and Hutzler, Otis, and Boyle 1974).

ESTIMATING WASTEWATER FLOWS FROM RURAL COMMUNITIES

Based on the current estimates of water use and peak demand flow rates, many sewer systems are being overdesigned. Estimates from 50 to 100 gallons per person of daily discharge from single-family homes are common (Water Pollution Control Federation 1969). Many State regulatory agencies, however, have set 400 gallons per day per capita for laterals and 250 gallons per day per capita for trunk sanitary sewers as minimum acceptable design flow rates where no actual measurements or other pertinent data are available (Water Pollution Control Federation 1969). The use of large, inaccurate flow rates can result in overdesigned systems and excessive installation costs. Data obtained by the FmHA in rural areas show average domestic per capita water use to be less than 50 gallons per day. Therefore, FmHA engineers recommend 150 gallons per day per design tap for domestic water, and since VGS systems should have no infiltration, sewage flows should be similar to incoming water flows. Recent Department studies have shown that the average sewage discharge rate from rural families is 4,500 gallons per month, and with sufficient onsite surge storage, collection lines could then be designed for 0.1 gal/min per residential connection. In practical applications, assuming one day's storage of 150 gallons is sufficient, an amount equal to 4 times 0.1 gal/min or 0.4 gal/min per connection for the design flow rate is reasonable (Rose 1980). Regardless of the procedure used, it should be apparent that the design flow rate is one of the most important numbers chosen for the design of sewer systems.

RECOMMENDED DESIGN CONSIDERATIONS FOR VGS SYSTEMS

While a design procedure may involve any number of steps, the basic steps to be followed in the design of these systems are:

1. Plot the ground profile along the proposed sewer.
2. Note on the profile the anticipated discharge elevation of the interceptor or septic tanks.
3. Make a careful, well-thought-out estimation of the maximum number of houses to be served by the proposed sewer. Give thought to any possible expansion.
4. Calculate the flow rate to be expected in the sewer.
   a. For VGS systems with onsite surge storage:
\[ Q = 0.4 \text{ gal/min} \times N \quad (1) \]

where \( Q \) = total flow in gal/min, and
N = the number of design residential-sized connections.

The VGS system with onsite surge storage is the configuration that is most recommended. The arrangement requires approximately 150 gallons of onsite liquid storage. Storage may be forced by using a 2-in. standpipe on the outlet (fig. 5).

![Figure 5.—Interceptor tank showing recommended improvements including a standpipe for forced storage and vent from liquid storage.](image)

Liquid storage must be vented and a vent through the separating wall near the top should be sufficient since the primary chamber is vented back through the stack on the roof of the house. The liquid storage was not vented as such at Mt. Andrew, but we suspect that loosely fitting lids provided a sufficient vent between the two chambers.

We suggest making an allowance for the flow from an additional 10 houses at the upstream end of the sewer for possible expansion.

b. For VGS systems without onsite forced storage (that is, existing septic tanks):

\[ Q = 0.6 \text{ gal/min} \times N + 10 \text{ gal/min} \quad (2) \]

where \( Q \) = total flow in gal/min, and
N = the number of design residential-sized connections.

This flow equation is similar to equation (1) but accounts for higher expected flow rates due to lack of hydraulically controlled forced storage. While some form of modified septic tank with extra surge storage is preferred, sometimes conventional septic tanks may be used. If a VGS collection system were used to replace a series of failing drainfields, in all probability conventional septic tanks would have been a minimum because standing water in this chamber serves no useful purpose.
already in place. In this case, depending on the hydraulic nature of the VGS system, the surge storage of the septic tanks may be used to control the hydraulic gradient. If septic tank surge storage proves inadequate in controlling the hydraulic gradient, individual forced-storage must be provided. It is conceivable that a single forced-storage tank could serve several septic tanks and discharge into a VGS line. The VGS concept lends itself to solving a variety of problems and any single design solution to a problem is not absolutely rigid.

c. For systems comprised of combinations of VGS lines and strategically placed pumping units or STEP systems:

No simple equation can describe expected flow from every possible variation of these hybrid systems. While VGS lines may serve most homes in a community, some can only be served by pumping effluent up to the sewer. In these instances (and in lieu of accurate flow estimates), we suggest that the effluent be pumped into a forced-storage surge tank located near (and above) the VGS line. This surge tank (with an orificed standpipe) would then gravity discharge into a VGS line similar to the forced storage of the interceptor tank liquid section. The simplest version of this hybrid system would be a single house with its own septic tank draining into a single wet well which is pumped up to a single surge tank near the VGS sewer. There could be instances, however, where several houses use a common wet well, or several wet wells pump up to a common surge tank. These recommendations are not rigid and many options are available to the design engineer. If an accurate flow determination could be produced through rigorous engineering calculations, the local forced-storage surge tanks could be eliminated.

5. Estimate pipe size for sewers and service lines based on the calculated flow rates and the Hazen-Williams formula using a "C" factor of 150 for plastic pipe (AWWA Standards Committee on Plastic Pipe 1971, Plastics Pipe Institute 1971, Uni-Bell Plastic Pipe Corporation 1982). Two-inch pipe should be the minimum-sized sewer serving several houses. Service lines from tanks to the sewer, however, should be a minimum of 1-1/2 inches.

6. Plot the preliminary hydraulic gradient for the proposed line on the ground profile (see design examples in next section).

7. Adjust pipe size and/or depth of cut as necessary to control the maximum hydraulic gradient and to minimize the cost of the system (see VGS design examples in next section).

8. Establish the need for special equipment such as check valves. The maximum hydraulic gradient may need to be adjusted to avoid backflow into an individual tank or a group of tanks.
If deeper burial or increasing the pipe size causes large increases in system cost, a backwater valve on the tank outlet may be the solution. In these situations, a backwater valve is preferred over a conventional check valve because this type of valve remains open when not in use and provides a vent for the line. Interceptor tanks (septic tanks) with outlet elevations less than 6 in. above the main line elevation or the static water level in the main line dips should be designed with pumping units. All pump discharge lines should have check valves to prevent drainback and backwater entry from the sewer. To avoid back pressure and siphoning slugs, all air in the VGS lines should be at atmospheric pressure. Where house connections do not provide adequate venting, air vents should be provided upstream and downstream of inverted siphon sections. Mechanical air relief valves are not recommended. Air vents made from standpipes topped with tees or double 90° elbows would be sufficient and maintenance free.

The most important design consideration is sizing the sewer line to accommodate the required number of homes. In contrast to large-diameter gravity sewers, slope is of little concern since VGS lines follow the contour of the terrain. The entire system, though, still requires a net positive grade. Results from the study at Mt. Andrew indicate that particle buildup will not be a problem. Researchers have reported that even well-designed conventional septic tanks do not discharge troublesome solid matter. Excessive velocity is of little concern. Since VGS lines follow onsite solids separation, abrasive grit should not be present to scour those sections of lines that may handle effluent at high velocity. Manholes or cleanouts are undesirable because they increase the cost of the system as well as the potential for inflow and sediment entry.

DESIGN EXAMPLES

Example 1

While there may be many design steps to consider, determining the expected maximum flow, sizing the pipe, and establishing the maximum expected hydraulic gradient is at the heart of the procedure. Consider the 2-in. sewer serving the Mt. Andrew community. Figure 6 shows the hydraulic gradient resulting from the maximum expected flow (when determined using \( Q = 0.4N \)) in the line as it is presently being used. The 2-in. pipe diameter was shown to be more than adequate for the expected flow from the 10 houses. In fact, at maximum flow the hydraulic gradient has a slope of less than 0.1 ft per 100 ft in the low section of the pipe. The dynamic hydraulic gradient at design flow, as shown in figure 6, is almost identical to the hydrostatic (no flow)
gradient that appears horizontal.

The calculations used to determine the hydraulic gradient for the 2-in. sewer are shown in table 1. Design data similar to that shown in table 1 can be produced by using the procedures outlined in WPCF Manual of Practice No. 9, Design and Construction of Sanitary and Storm Sewers, Chapter 2 (1969). Determining the design flow rate is the major departure from the procedure outlined in the WPCF manual. Table 1 was constructed as follows:

Columns 1 and 2, line-section stations (from station 1 to station 2), are constructed using information from the profile plot. Sections chosen should provide a segment of relatively uniform grade and flow rate. The upper station may be selected on the basis of change in grade or change of flow rate or to provide a convenient section length for calculations.

Column 3, elevation of station 2, is obtained from the profile plot. For the initial design, use the top of the pipe as this will normally be established by minimum cover considerations and will not change with the size of pipe.

Column 4, section length in 100 ft, is obtained by subtracting column 1 from column 2 and dividing by 100.

Column 5, elevation difference, is obtained by subtracting the elevation of station 1 from the elevation of station 2 (col. 3).

Column 6, section slope, is obtained by dividing the elevation difference (col. 5) by the section length (col. 4).

Table 1.—Suggested form (with calculations) for design of variable-grade, gravity sewers (example 1)

<table>
<thead>
<tr>
<th>Line section stations</th>
<th>Elev. of station 1</th>
<th>Elev. of station 2</th>
<th>Design Pipe Full Flow Rate</th>
<th>Fric. Head</th>
<th>Fric. Flow</th>
<th>Elevation Difference</th>
<th>Section Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>From</td>
<td>To</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(%)</td>
<td>(gal/ min)</td>
<td>(gal/ min)</td>
</tr>
<tr>
<td>0+00</td>
<td>0</td>
<td>0</td>
<td>5.4</td>
<td>0.70</td>
<td>5.4</td>
<td>7.71</td>
<td>8.0</td>
</tr>
<tr>
<td>0+70</td>
<td>0+97</td>
<td>5.8</td>
<td>0.27</td>
<td>0.4</td>
<td>1.48</td>
<td>8.0</td>
<td>2</td>
</tr>
<tr>
<td>0+97</td>
<td>1+76</td>
<td>5.8</td>
<td>0.79</td>
<td>0</td>
<td>7.6</td>
<td>2</td>
<td>03</td>
</tr>
<tr>
<td>1+76</td>
<td>2+06</td>
<td>6.4</td>
<td>0.30</td>
<td>0.6</td>
<td>2.00</td>
<td>7.6</td>
<td>2</td>
</tr>
<tr>
<td>2+06</td>
<td>4+18</td>
<td>8.6</td>
<td>2.12</td>
<td>2.2</td>
<td>1.04</td>
<td>7.2</td>
<td>2</td>
</tr>
<tr>
<td>4+18</td>
<td>6+58</td>
<td>8.6</td>
<td>2.40</td>
<td>0</td>
<td>6.4</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>6+58</td>
<td>7+71</td>
<td>10.6</td>
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<td>2</td>
<td>1.77</td>
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<td>2</td>
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<tr>
<td>7+71</td>
<td>9+12</td>
<td>16.2</td>
<td>1.41</td>
<td>5.6</td>
<td>3.97</td>
<td>5.2</td>
<td>2</td>
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<tr>
<td>9+12</td>
<td>9+70</td>
<td>15.3</td>
<td>0.58</td>
<td>-0.9</td>
<td>-1.55</td>
<td>4.4</td>
<td>2</td>
</tr>
</tbody>
</table>

1G = gravity head
2PF = partial flow
3Mathematically, zero slope produces zero flow.
Column 7, design flow rate, is obtained from the flow rate formulas given in the previous section. For any given line section, the design flow rate should be the flow rate at station 1. For this particular example, note the addition of 4 gal/min at the upstream end which is the flow allowance for 10 more houses.

Column 8, pipe size, is the nominal pipe size and the result of choices based on the slope (col. 6) and the design flow rate (col. 7). For columns 9 to 12, calculations should be based on minimum inside diameter (I.D.)

Column 9, full pipe flow, is computed using the Hazen-Williams formula with a "C" factor of 150. This calculation is used to help establish the flow condition, full or partial. The friction head per 100 ft is equal to the section slope in percent (col. 6). This calculation can be made simpler by rearranging the Hazen-Williams formula, combining the constant terms, and solving for flow:

\[ Q_{fp} = \left( \frac{\text{Slope}}{r} \right)^{0.54} \]  

Where \( Q_{fp} \) = full pipe flow in gal/min, Slope = section slope (col. 6) expressed as a percent, and \( r \) = a constant of proportionality according to size and type of pipe (table 2).

Column 10, friction head at \( Q_d \) (the design flow), is determined using the design flow (\( Q_d \) from col. 7) and the Hazen-Williams formula with \( C = 150 \). This calculation can be made simpler by rearranging the equation used in the previous step and solving for the friction head:

\[ h_f = r \times Q_d^{1.85} \]  

Where \( h_f \) = friction head at design flow,

\( r \) = a constant of proportionality used in the previous step, and \( Q_d \) = design flow (col. 7).

Column 11, friction loss in section, is calculated by multiplying the friction head (col. 10) times the section length in 100 ft (col. 4). Where partial pipe flow exists, friction loss equals the gravity head (col. 5).

Column 12, elevation of hydraulic gradient, is determined by adding the friction loss in the section (col. 11) to the hydraulic gradient elevation of station 1. The elevation of the hydraulic gradient of station 1 is the previous value in column 12. The result of the calculation for column 12 is the elevation of the hydraulic gradient at station 2.

Where partial flow exists, the water level within the pipe is the elevation of the hydraulic gradient. Where full pipe flow exists, however, and the friction head (col. 10) is greater than the section slope (col. 6), the hydraulic gradient will be above the top of the pipe. If any section, some distance from the outfall, were to be analyzed separately, the calculations could present an unclear picture of the true situation and indicate partial flow in a section surcharged from a flow friction situation existing downstream. By starting at station 0+00 and working toward the upstream end of the pipe, the elevation of the hydraulic gradient can be properly plotted.

Example 2

Thus far, the design example has dealt with a more than adequate sewer serving a small number of houses. As an example of problem solving, consider the same profile with a higher design flow from an additional 22 gal/min (design flow from 55 houses) at the upstream
Table 2.—Constants of proportionality for various sizes and types of PVC pipe

<table>
<thead>
<tr>
<th>Type of pipe</th>
<th>Nominal size of pipe (in.)</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDR 26</td>
<td>2.0</td>
<td>2.23x10^{-3}</td>
</tr>
<tr>
<td>SDR 26</td>
<td>2.5</td>
<td>8.75x10^{-4}</td>
</tr>
<tr>
<td>SDR 26</td>
<td>3.0</td>
<td>3.35x10^{-4}</td>
</tr>
<tr>
<td>SDR 26</td>
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<td>1.74x10^{-4}</td>
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<td>SDR 26</td>
<td>4.0</td>
<td>9.79x10^{-5}</td>
</tr>
<tr>
<td>SDR 26</td>
<td>6.0</td>
<td>1.10x10^{-5}</td>
</tr>
</tbody>
</table>

end (station 9 + 70) of the 2-in. line. The design calculations of this additional flow (in a 2-in. SDR 26 line) are shown in table 3. The maximum hydraulic gradient resulting from the additional flow is shown in figure 7. The maximum hydraulic gradient, which became substantially elevated, is plotted above the pipe profile using the dashed line. Three houses, at 600, 700, and 800 ft from the lagoon, have their tank outlets below or near the hydraulic gradient. This situation presents the possibility of excessive backflow into these tanks and causes the need for redesign.

The choice would be either to deepen the cut of trench or to increase the diameter of the pipe. Deepening the cut any appreciable amount may involve changing the installation procedure or even changing some trenching equipment. Remember, one of the most attractive features of this type of sewer is that it can be put in at a relatively constant depth, independent of grade, minimizing trenching costs. Any time trenching machine operators do anything other than dig at constant depth, they must check their operations to comply with special instructions. To do this involves extra time and probably extra labor. If it is absolutely necessary, changing the trench depth is an option and the job is easier to

Figure 7. Sewer profile showing hydraulic gradient resulting from increased flow.
Table 3.—Preliminary calculations with 2-in. sewer subjected to an increase in flow rate of 22 gal/min (example 2)

<table>
<thead>
<tr>
<th>Line section</th>
<th>Elev. of sta.</th>
<th>Sec. in ft</th>
<th>Elev. of next sta.</th>
<th>Sec. in ft</th>
<th>Design flow rate Qf (gal/min)</th>
<th>Full pipe size Qfp (gal/min)</th>
<th>Fric. Elevation Qd (ft)</th>
<th>Fric. Elevation Qfp (ft)</th>
</tr>
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<tbody>
<tr>
<td>0+00</td>
<td>0</td>
<td>30.0</td>
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<td></td>
</tr>
<tr>
<td>0+00</td>
<td>0+70</td>
<td>5.4</td>
<td>0.70</td>
<td>5.4</td>
<td>7.71</td>
<td>30.0</td>
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<td>1.20 G</td>
</tr>
<tr>
<td>0+70</td>
<td>0+97</td>
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<td>1.77</td>
<td>27.6</td>
<td>2</td>
<td>1.16 13.30</td>
</tr>
<tr>
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<td>9+12</td>
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<td>1.41</td>
<td>5.6</td>
<td>3.97</td>
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<td>1.01 G</td>
</tr>
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<td>9+12</td>
<td>9+70</td>
<td>15.3</td>
<td>0.58</td>
<td>-0.9</td>
<td>-1.55</td>
<td>26.4</td>
<td>2</td>
<td>0.95 0.55 16.75</td>
</tr>
</tbody>
</table>

In this case lowering the hydraulic gradient by cutting deeper would involve deeper cuts for long distances. By adding relatively short sections of larger pipes, however, the problem can be solved. The hydraulic gradient can be lowered most effectively by increasing the size of pipe in the inverted siphon sections because these sections are full at all times (see next section on calculations to lower the hydraulic gradient). Changing the diameter of the sewer from 2 to 2-1/2 in. from station 0+97 to station 1+76 and from station 4+18 to station 6+58 lowered the hydraulic gradient sufficiently.

The dot-dash line above the pipe profile in figure 7 represents the new hydraulic gradient. Table 4 presents a set of design calculations with those columns affected by the change.

CALCULATIONS TO LOWER THE HYDRAULIC GRADIENT

In the examples given, lengths of larger pipe were chosen arbitrarily between existing stations to simplify repeated trials of design solutions. In actual practice, adding an extra several hundred feet (or several thousand feet) just to get to a convenient pre-existing station may be poor economics. A shorter length ending somewhere between existing stations may be needed. This may require creating a new station and its length can be calculated as follows:

When, in a given length, L (in 100 ft), friction loss \( \Delta h_{FL} \) causes the hydraulic gradient to be too high by an amount \( \Delta h_{FL} \), the hydraulic gradient can be lowered by inserting a length, \( X \), of larger pipe. The length, \( X \), can be determined from:

\[
X = \frac{\Delta h_{FL}}{h_{fL} - h_{f2}}. \tag{5}
\]

In the equation, \( h_{fL} \) is the friction loss per 100 ft at design flow, and subscripts 1 and 2 refer to the original pipe size and the larger pipe, respectively.
Table 4.—Final calculations showing the effect of the adjusting size of pipe to lower the hydraulic gradient of the overloaded sewer (example 2)

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>From To</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(%)</td>
<td>(gal/min)</td>
<td>(gal/min)</td>
<td>(ft/100ft)</td>
<td>(ft)</td>
<td>(ft)</td>
</tr>
<tr>
<td>0+00</td>
<td>0</td>
<td>0.70</td>
<td>5.4</td>
<td>7.71</td>
<td>30.0</td>
<td>2.0</td>
<td>81.5</td>
<td>1.20</td>
</tr>
<tr>
<td>0+70</td>
<td>0+97</td>
<td>5.8</td>
<td>0.27</td>
<td>0.4</td>
<td>1.48</td>
<td>30.0</td>
<td>2.0</td>
<td>33.4</td>
</tr>
<tr>
<td>0+97</td>
<td>1+76</td>
<td>5.8</td>
<td>0.79</td>
<td>0</td>
<td>0.0</td>
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<td>2.5</td>
<td>0.46</td>
</tr>
<tr>
<td>1+76</td>
<td>2+06</td>
<td>6.4</td>
<td>0.30</td>
<td>0.6</td>
<td>2.00</td>
<td>29.6</td>
<td>2.0</td>
<td>39.3</td>
</tr>
<tr>
<td>2+06</td>
<td>4+18</td>
<td>8.6</td>
<td>2.12</td>
<td>2.2</td>
<td>1.04</td>
<td>29.2</td>
<td>2.0</td>
<td>27.6</td>
</tr>
<tr>
<td>4+18</td>
<td>6+58</td>
<td>8.6</td>
<td>2.40</td>
<td>0</td>
<td>0.0</td>
<td>28.4</td>
<td>2.5</td>
<td>0.43</td>
</tr>
<tr>
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<td>7+71</td>
<td>10.6</td>
<td>1.13</td>
<td>2.0</td>
<td>1.77</td>
<td>27.6</td>
<td>2.0</td>
<td>36.8</td>
</tr>
<tr>
<td>7+71</td>
<td>9+12</td>
<td>16.2</td>
<td>1.41</td>
<td>5.6</td>
<td>3.97</td>
<td>27.2</td>
<td>2.0</td>
<td>56.9</td>
</tr>
<tr>
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<td>15.3</td>
<td>0.58</td>
<td>-0.9</td>
<td>-1.55</td>
<td>26.4</td>
<td>2.0</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Friction loss in the length, L, is then:

\[ \Delta h_f = (L-X)h_f + Xh_{f2} \]  \hspace{1cm} (6) \]

If X exceeds L, the solution is not valid. Calculations producing invalid results can be avoided by selecting a pipe size for which \( h_{f2} \) is less than the desired average hydraulic gradient.

SUMMARY

Adequate sewage collection and disposal is still a major problem in many sections of the United States. Onsite pretreatment with septic tanks is no problem, but in many rural areas, where soils do not meet local requirements for disposal systems, inexpensive collection and transport systems are needed to convey liquefied sewage to central treatment facilities. A system similar to the installation at Mt. Andrew is an economic solution to the problem.

The Mt. Andrew system has been operating satisfactorily since its installation in 1975. This installation demonstrates that the variable-grade gravity system, with approximately one day's storage on site, has worked satisfactorily with small-diameter plastic sewers. A well-designed septic tank can be used in place of interceptor tanks used at Mt. Andrew, but a wet well (pump sump) still would be needed for those tanks requiring effluent pumps. Using standard procedures in sewer design as well as procedures outlined in this publication, a workable small-diameter, variable-grade gravity sewer can be properly designed. Since the design process is a series of trial and error calculations, the process lends itself very well to the use of an interactive program in a desk top computer. A program (written in Fortran) that can calculate the "design matrix" can be obtained from the authors.
REFERENCES


Water Pollution Control Federation. 1969. WPCF Manual of Practice No. 9 - Design and Construction of Sanitary and Storm Sewers. Water Pollution Control Federation, Washington, D.C.